Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings

with Additional Commentary for ASCE/SEI 41-17

FEMA P-2006 / June 2018

Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings with Additional Commentary for ASCE/SEI 41-17

Prepared by

APPLIED TECHNOLOGY COUNCIL
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Prepared for

FEDERAL EMERGENCY MANAGEMENT AGENCY
Michael Mahoney, Project Officer
Andrew Herseth, Project Monitor
William T. Holmes, Subject Matter Expert
Washington, D.C.

ATC MANAGEMENT AND OVERSIGHT
Jon A. Heintz, Program Executive, Program Manager
Ayse Hortacsu, Project Manager
Veronica Cedillos, Associate Project Manager

PROJECT TECHNICAL COMMITTEE
Bret Lizundia (Project Technical Director)
Michael Braund
Jim Collins
Ronald W. LaPlante
Brian McDonald
Mark Moore

PROJECT REVIEW PANEL
David Biggs
Anthony B. Court
Roy Lobo
James Parker
Robert Pekelnicky
Peter Somers
Williston L. Warren (ATC Board Contact)

FOCUS GROUP
Marshall Carman
Joseph R. Jones
Valerie Martin
Ryan McDaniel
Meghann Riedner

WORKING GROUP
Ryan Bogart
Lawrence Burkett
Casey Champion
Alex Chu
Jie Luo
Steve Patton
Chris Tokas
Kylin Vail

SEAOC REVIEWERS
Russell Berkowitz (Program Manager)
Jonathan Buckalew
Lachezar Handzhiyski
Kari Klaboe
Jeremiah Legrue
Dion Marriott
Rose McClure
Ben Mohr
Brian Olson
Jim Reber
Andrew Shuck
Fred Turner

2018

Notice

Any opinions, findings, conclusions, or recommendations expressed in this publication do not necessarily reflect the views of the Applied Technology Council (ATC), the Department of Homeland Security (DHS), or the Federal Emergency Management Agency (FEMA). Additionally, neither ATC, DHS, FEMA, nor any of their employees, makes any warranty, expressed or implied, nor assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, product, or process included in this publication. Users of information from this publication assume all liability arising from such use.

Cover photograph – Seismic retrofit with shear wall infill at University of California, Berkeley Campus (courtesy of Bret Lizundia, Rutherford + Chekene, San Francisco, California).

Preface

For over 30 years, the Federal Emergency Management Agency (FEMA) has had an extensive and long-term program to address the seismic safety of existing buildings. This program has led to the development of guidelines and standards for existing buildings that form the basis of current seismic evaluation and performance-based design of seismic retrofits in the United States.

In 2014, the Applied Technology Council (ATC), with funding from FEMA under Task Order Contract HSFE60-12-D-0242, commenced a series of projects (ATC-124, ATC-124-1, and ATC-124-2) to develop a document that would present design examples for seismic retrofit and evaluation of buildings using the consensus standard ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings*, published by the American Society of Civil Engineers. This *Example Application Guide* and its design examples provide helpful guidance on the interpretation and use of ASCE/SEI 41-13, and the *Guide* is intended to benefit both practicing engineers and building officials who have limited or no experience with ASCE/SEI 41, as well as those engineers and building officials who have used ASCE/SEI 41 in the past but have specific questions.

Following the 2014 publication of ASCE/SEI 41-13, the ASCE Standards Committee on Seismic Rehabilitation initiated work on updating the standard. This update coincided with the development of this *Example Application Guide,* and coordination between the two groups was necessary to ensure that the guidance provided is current and consistent with the ASCE 41 standard and to stay apprised of issues under consideration by the committee that might affect the design examples during the development of the *Example Application Guide*. In December 2017, ASCE/SEI 41-17 was published. The examples in this *Example Application Guide* use ASCE/SEI 41-13 as the basis for the provisions, but noteworthy revisions in ASCE/SEI 41-17 are highlighted.

ATC is indebted to the leadership of Bret Lizundia, Project Technical Director, and to the members of the ATC-124 Project Teams for their efforts in developing this *Guide*. The Project Technical Committee, consisting of Michael Braund, Jim Collins, Ron LaPlante, Brian McDonald, and Mark

Moore, managed and performed the technical development efforts. Ryan Bogart, Lawrence Burkett, Casey Champion, Alex Chu, Jie Luo, Steve Patton, and Kylin Vail provided assistance in the development of the design examples as members of the Project Working Group. Chris Tokas developed many of the figures in the document. Collaboration with the Structural Engineering Association of California (SEAOC) was ensured through the participation of the Existing Buildings and Seismology Committees. Russ Berkowitz served as the SEAOC Program Manager and coordinated the review effort. During the formation of the project, input from the Project Focus Group who provided advice on presentation, organization, and ease of use of the *Guide* was invaluable. The Project Review Panel, consisting of David Biggs, Tony Court, Roy Lobo, James Parker, Bob Pekelnicky, Peter Somers, and Bill Warren provided technical review, advice, and consultation at key stages of the work. The names and affiliations of all who contributed to this report are provided in the list of Project Participants.

ATC also gratefully acknowledges Michael Mahoney (FEMA Project Officer), Drew Herseth (FEMA Project Monitor), and Bill Holmes (FEMA Subject Matter Expert) for their input and guidance in the preparation of this document. Carrie Perna (ATC) provided report production services, Veronica Cedillos (ATC) assisted in project management.

Ayse Hortacsu	Jon A. Heintz
ATC Director of Projects	ATC Executive Director

Table of Contents

List of Figures

List of Tables

Location of ASCE 41-13 Provisions

The table below lists the ASCE 41-13 chapters and sections discussed in this *Example Application Guide*. The treatment of provisions within the *Guide* varies based on the subject matter of the example presented. The list below is not exhaustive, the reader is referred to the Table of Contents and Chapter 1 for an overview of the organization and scope of the *Guide.*

ASCE 41-13 Chapter/ Section No.	ASCE 41-13 Title	Example Application Guide Chapter/Section No.
Chapter 2	Performance Objectives and Seismic Hazards	
§ 2.2	Performance Objectives	3.2; 6.4; 7.1; 10.2
§ 2.3	Target Building Performance Levels	3.2
§ 2.4	Seismic Hazard	3.3; 8.3; 10.2
§ 2.5	Level of Seismicity	3.4; 10.2
Chapter 4	Tier 1 Screening	
§ 4.3	Benchmark Buildings	12.6
§ 4.4	Selection and Use of Checklists	4.2; 6.2; 6.5; 10.5; 12.6
§ 4.5	Tier 1 Analysis	7.4; 8.3; 9.5; 12.6
Chapter 5	Tier 2 Deficiency-Based Evaluation and Retrofit	
§ 5.5	Procedures for Seismic-Force-Resisting Systems	4.2; 6.6
Chapter 6	Tier 3 Systematic Evaluation and Retrofit	
§ 6.2	Data Collection Requirements	3.5; 9.6; 10.3
Chapter 7	Analysis Procedures and Acceptance Criteria	
§ 7.2	General Analysis Requirements	4.6; 4.7; 7.5; 10.4; 10.5; 10.6; 10.7; 10.8; 11.3
§ 7.3	Analysis Procedure Selection	4.2; 7.7; 10.4; 11.3; 13.6
§ 7.4	Analysis Procedures	4.3; 7.5; 7.7; 9.7; 10.4; 11.3; 13.6
§ 7.5	Acceptance Criteria	4.4; 4.5; 7.5; 10.5; 10.9; 11.4

ASCE 41-13 Chapter/ Section No.	ASCE 41-13 Title	Example Application Guide Chapter/Section No.
Chapter 8	Foundations and Geologic Site Hazards	
§ 8.4	Foundation Strength and Stiffness	5.6; 9.8; 11.6
§ 8.5	Kinematic Interaction and Radiation Damping Soil-Structure Interaction Effects	11.7
Chapter 9	Steel	
§ 9.2	Material Properties and Condition Assessment	9.6
§ 9.4	Steel Moment Frames	8.4; 8.5
§ 9.5	Steel Braced Frames	9.7; 9.8
Chapter 10	Concrete	
§ 10.2	Material Properties and Condition Assessment	9.6; 10.3
§ 10.4	Concrete Moment Frames	10.9; 11.5
§ 10.7	Concrete Shear Walls	10.4; 10.5; 11.4
Chapter 11	Masonry	
§ 11.2	Condition Assessment and Material Properties	13.3; 13.4; 13.5; 13.6; 13.8
§ 11.3	Masonry Walls	13.6; 13.8
Chapter 12	Wood and Cold-Formed Steel Light Frame	
§ 12.2	Material Properties and Condition Assessment	7.5
§ 12.3	General Assumptions and Requirements	7.7
§ 12.4	Wood and CFS Light-Frame Shear Walls	7.5; 7.6
§ 12.5	Wood Diaphragms	7.7
Chapter 13	Architectural, Mechanical, and Electrical Components	4.8
Chapter 15	System-Specific Performance Procedures	
§ 15.2	Special Procedure for Unreinforced Masonry	12.7; 12.8; 12.9; 12.10; 12.11; 12.12; 12.13; 12.14; 12.16
Appendix A	Guidelines for Deficiency-Based Procedures	
§ A.2	Procedures for Building Systems	6.6; 8.3
§ A.3	Procedures for Seismic Force-Resisting Systems	6.6; 8.3

Chapter 1
Introduction

1.1 Purpose

ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings*, is the consensus national standard for the seismic evaluation and retrofit of existing buildings. This standard was first published in 2007 (ASCE, 2007) and then updated in 2014 (ASCE, 2014), and again in 2017 (ASCE, 2017b). Although ASCE/SEI 41 has been adopted by various jurisdictions, its implementation can be challenging for those unfamiliar with the provisions because its methods are different in many ways from those used in the design of new buildings. This *Example Application Guide* provides helpful guidance on the interpretation and the use of ASCE/SEI 41-13 (referred to in this document as ASCE 41-13) through a set of examples that address key selected topics. The *Guide* covers topics that commonly occur where guidance is believed to be beneficial, with topics effectively organized and presented such that information is easy to find. Commentary accompanies the examples to provide context, rationale, and advice, including discussion of revisions to the standard made in the 2017 publication of ASCE 41-17.

1.2 Target Audience

The target audience for this *Guide* is both practicing engineers and building officials who have limited or no experience with ASCE 41-06 or ASCE 41-13 and those engineers and building officials who have used these documents in the past, but have specific questions. It is assumed that the user has seismic design experience and a working knowledge of seismic design concepts. The document includes guidance and examples from locations representing higher and lower seismic hazard levels.

1.3 Background

In 2014, the ASCE/SEI Standards Committee on Seismic Rehabilitation completed a three-year process of combining ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003), and ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings* (including Supplement No. 1) (ASCE, 2007). These two preceding standards are based on methodologies set forth in a series of documents.

ASCE 31-03 was an updated version of FEMA 310, *Handbook for Seismic Evaluation of Buildings - A Prestandard* (FEMA, 1998c), which in turn was an update of the original FEMA 178 report, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* (BSSC, 1992), which was based on ATC-14, *Evaluating the Seismic Resistance of Existing Buildings* (ATC, 1987).

ASCE 41-06 began as an updated version of FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA, 2000g), which was in turn an update of FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997a), and FEMA 274, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997b).

A timeline of development of the standards is shown in Figure 1-1.

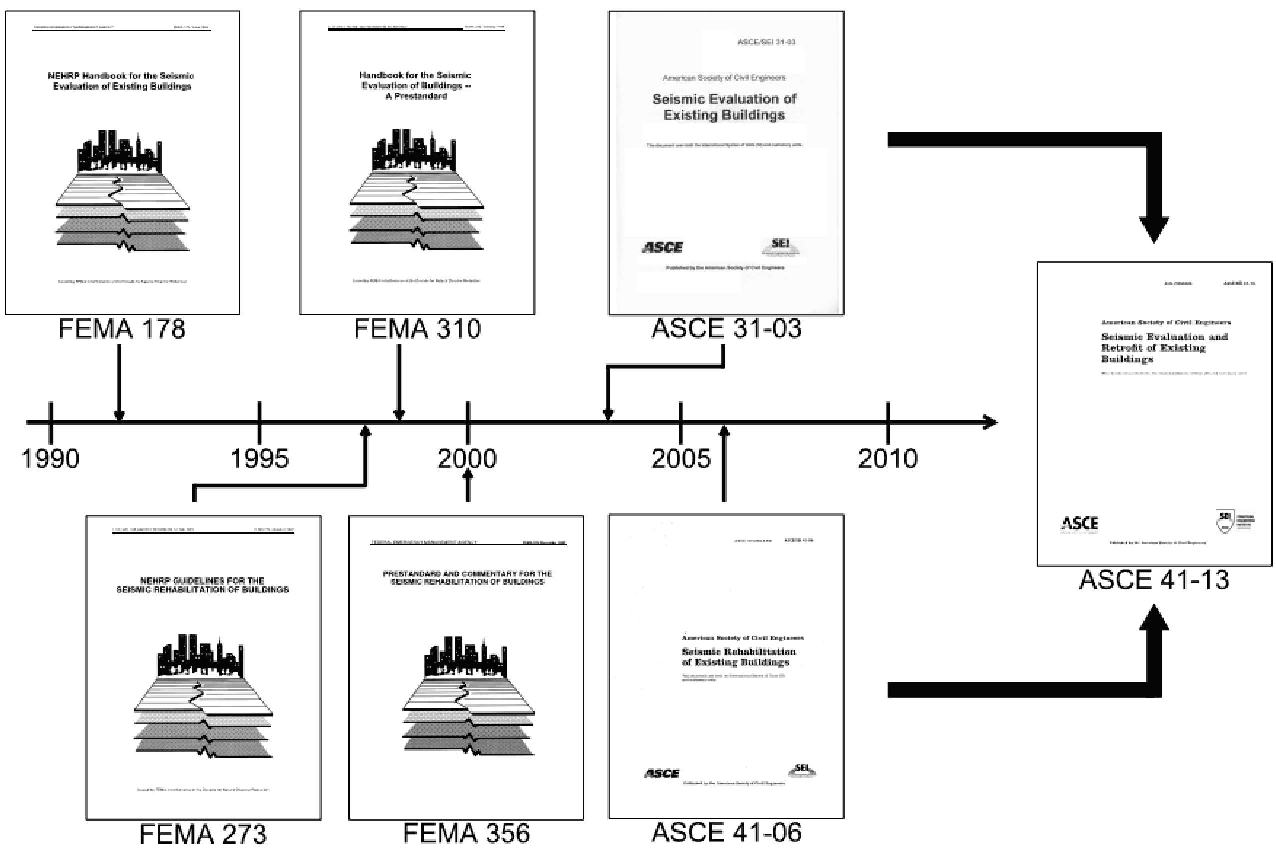


Figure 1-1 Development of seismic evaluation and retrofit procedures.

ASCE 31-03 and ASCE 41-06 were used widely in the profession, especially in California. Regulatory agencies, such as the California Office of Statewide Health Planning and Development (OSHPD) for hospitals and the California Division of the State Architect (DSA) for schools, and public building owners, such as the U.S. General Service Administration and the

Department of Veterans Affairs, have directly referenced or permitted the use of these documents to evaluate and retrofit existing buildings (Pekelnicky and Poland, 2012). In 2009, when the standards committee initiated the update cycle for ASCE 31 and ASCE 41, it was decided to combine the standards into one document and to coordinate the evaluation and retrofit procedures. The combined standard, ASCE 41-13, retains the three-tiered approach found in ASCE 31-03, while relying on the technical provisions in ASCE 41-06 as the basis for all the analytical procedures. Appendix B provides a summary of key changes from the ASCE 41-06 edition that were made for ASCE 41-13.

ASCE 41-13 was developed by the ASCE Standards Committee on Seismic Rehabilitation following a consensus standard process that has been accredited by the American National Standards Institute (ANSI). This development process required significant balloting of both technical and organizational changes in the standard through a diverse committee of structural engineers, academic professors and researchers, and industry representatives. Subcommittees and technical issue teams were formed to focus on specific topics and technical updates and to investigate potential inconsistencies when combining the ASCE 31-03 and ASCE 41-06 standards. The efforts of the ASCE Standards Committee on Seismic Rehabilitation spanned more than a three-year period and included a public review and commenting process following the Committee's approval of the standard.

All ASCE standards are typically updated or reaffirmed by the consensus standards development process at intervals of approximately five years. The ASCE Standards Committee on Seismic Rehabilitation published ASCE 41-17 in 2017. Substantial changes in this most recent edition of the standard are discussed in Section 2.4 of this *Guide*.

1.4 Basic Principles of ASCE 41-13

The basic principles and philosophical approach of ASCE 41-13 differ from those used for seismic design and detailing requirements of building codes for new structures.

Design of the seismic force-resisting system for new buildings in standards, as outlined in ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), is based on prescriptive design and detailing provisions for components. Note that the ASCE 7-16 update to ASCE 7-10 was recently published in 2017 (ASCE, 2017a), but since ASCE 41-13 references ASCE 7-10, this *Guide* also uses ASCE 7-10. For economic reasons, structures designed to this standard are not designed with sufficient strength to remain elastic and undamaged under design seismic loading, but instead are detailed to be able to sustain seismic damage while continuing to withstand vertical and lateral forces. Structural components are thus designed for seismic demands that are modified by the code-specified system coefficients. These factors are used across the structure, in recognition of a relatively high level of confidence in material properties and of well-controlled detailing requirements.

Based on these provisions, the code for new buildings allows for a uniform application of system coefficients and also provides a margin of safety to account for uncertainty in earthquake hazards, building response, and the design and construction processes.

Like new buildings designed per ASCE 7-10, existing buildings typically do not have sufficient lateral capacity to remain elastic and undamaged in the design seismic event. Unlike new buildings, many existing structures have archaic or irregular lateral systems or lack the ductile detailing required by more recent building codes. Thus, the inelastic behavior of components is not expected to be consistent throughout an existing structure, and component ductility capacities must be evaluated individually.

ASCE 41-13 includes provisions for linear and nonlinear analysis, both of which capture the effect of ductility and yielding through individual modifiers rather than global ones. In both linear and nonlinear analyses, global seismic demands applied to the analysis model are unreduced from the elastic level. Linear analysis accounts for the ductility of individual components by applying component-specific *m*-factors, which allow the calculated elastic seismic demands on individual elements to reach several times the elements' expected capacities. In nonlinear analysis, component ductility is modeled directly through component-specific nonlinear yielding curves, so that ductile elements can deform and soften when overloaded.

ASCE 41-13 more explicitly targets levels of performance than ASCE 7-10 by establishing performance objectives that pair levels of shaking with expected performance levels. Section 2.1 of this *Guide* provides a more detailed technical overview of ASCE 41-13 and performance objectives. Section 2.2 of this *Guide* provides a more detailed technical discussion of the ASCE 41-13 philosophy and compares it with that of the ASCE 7-10 provisions.

1.5 Scope of the *Example Application Guide*

This *Guide* illustrates provisions set forth in ASCE 41-13. Brief commentary is provided regarding relevant provisions from ASCE 41-17.

Writing a practical guide that addresses all aspects of a document as complex as ASCE 41-13 is simply not possible. Therefore, one of the critical tasks for the project team was to select those topics and building types that would most benefit from additional guidance or worked examples. The selection was informed by a survey of current ASCE 41 users, who identified issues that would most benefit from a design guide.

1.5.1 Organization of Guide

The *Guide* presents a combination of short topic examples in the earlier chapters and more detailed building and material specific examples in the later chapters. When possible, specific building examples were sourced from previously published design example documents, and adapted for the needs of the specific example.

The *Guide* is organized to follow the order of ASCE 41-13, where possible.

- **Chapter 2** provides an overview of ASCE 41-13, discusses how ASCE 41-13 fits in the overall process of seismic evaluation and retrofit design, reviews changes from the previous ASCE 41-06 version, summarizes key changes in the recently published ASCE 41-17 edition, and finishes with general words of advice and tips on successful strategies for using ASCE 41-13.

- **Chapter 3 and Chapter 4** provide a series of specific examples related to a number of topics including: Performance Objectives and Target Building Performance Levels; Seismic Hazard Levels; the Levels of Seismicity used in ASCE 41-13; data collection, material testing, and knowledge factors; analysis procedures; determination of forces and target displacements; primary versus secondary elements; force-controlled and deformation-controlled actions; overturning; out-of-plane wall strength; and nonstructural components. These key topics are drawn from ASCE 41-13 Chapters 2, 3, 6, 7, and 13.

- **Chapter 5** addresses general foundation evaluation strategies and issues. It covers bounding analyses, load-deformation curves, useful soil-structure interaction techniques, and important collaboration needs with geotechnical engineers. These key topics are drawn from ASCE 41-13 Chapter 8.

- **Chapter 6** reviews Tier 1 screening and Tier 2 deficiency-based evaluation and retrofit. Several examples are provided including a detailed example using a tilt-up building. These key topics are drawn from ASCE 41-13 Chapter 4 and Chapter 5, as well as Chapter 7 and Chapter 12.

- **Chapter 7 through Chapter 13** provide design examples for a wood tuck-under building, older steel moment frame, steel braced frame, older concrete shear wall, and an unreinforced masonry bearing wall building. These are more complete examples where a relatively comprehensive evaluation and retrofit are shown. Examples cover a range of seismicity levels, Performance Objectives, and analysis procedures.

- **Appendix A** provides a list of other documents that present design examples that may also be useful.

- **Appendix B** provides a summary of important changes made in ASCE 41-13 from the provisions in ASCE 41-06.

Table 1-1 shows a summary of the system-specific example applications presented in this *Guide.* The structure types are identified in terms of common building types, as defined in ASCE 41-13 Table 3-1. Figure 1-2 shows an image of each design example building.

A list of symbols defining key notation, and a list of references cited are provided at the end of this *Guide*.

1.5.2 What is Not Covered in the Guide

The *Guide* does not provide retrofit cost information or detailed information about retrofit techniques. Additionally, the following outlines the topics not included in this *Guide*. Appendix A provides a matrix of other design guides that address many of these topics.

- **Seismic Design Concepts:** Although the target audience for the *Guide* may include those with limited experience with ASCE 41, it is assumed that they have seismic design experience and a working knowledge of seismic design concepts. Thus, the *Guide* does not provide detailed discussion of such concepts.

- **Construction Documents:** No guidance is provided on the preparation of construction documents and describing the retrofit work to a contractor. FEMA 547, *Techniques for the Seismic Rehabilitation of Buildings* (FEMA, 2006), is a helpful source for such information.

Table 1-1 **Summary of Examples Covered in the *Guide***

Chapter No.	FEMA Building Type	Risk Category	Location	Level of Seismicity	Performance Objective	Analysis Procedure	Evaluation/ Retrofit Procedure
6	PC1	II	Anaheim, CA	High	BPOE	LSP	Tier 1 and Tier 2
7	W1a	III	San Jose, CA	High	BPOE and Partial Retrofit	LSP	Tier 1 and Tier 3
8	S1	II	San Francisco Bay Area, CA	High	BPOE	LSP, LDP, NSP	Tier 1, 2, 3
9	S2	III	Charlotte, NC	Moderate	Immediate Occupancy at BSE-1N	LSP, NSP	Tier 1 and Tier 3
10	C2	II	Seattle, WA	High	BPOE	LSP	Tier 3
11	C2	II	Seattle, WA	High	BPOE	NSP	Tier 3
12	URM	II	Los Angeles, CA	High	Reduced	Special Procedure	Special Procedure
13	URM	II	Los Angeles, CA	High	BPOE	LSP	Tier 3

Notes: PC1 = Precast or tilt-up concrete shear walls with flexible diaphragm
W1a = Wood multi-story, multi-unit residential (tuck-under)
S1 = Steel moment frame with stiff diaphragm
S2 = Steel braced frame with stiff diaphragm
C2 = Concrete shear wall with stiff diaphragm
URM = Unreinforced masonry bearing wall
LSP = Linear Static Procedure
LDP = Linear Dynamic Procedure
NSP = Nonlinear Static Procedure
BPOE = Basic Performance Objective for Existing Buildings (see Chapter 2 of this *Guide*)
BSE-1E = Seismic Hazard Level (see Chapter 2 of this *Guide*)
BSE-1N = Seismic Hazard Level (see Chapter 2 of this *Guide*)

- **Performance Criteria:** Only general discussion is provided on the selection of performance criteria that may be mandated by the existing building code adopted by the governing jurisdiction, local ordinance, or is otherwise addressed in ASCE 41-13. The requirements of specific jurisdictions are not discussed.
- **Computer Modeling and Software:** No guidance is provided on the use of specific analysis software or complete computer models other than tips and general guidance on specific modeling issues that should be considered.
- **Geotechnical Topics:** Limited guidance is provided on geotechnical issues that would otherwise be performed by a geotechnical engineer.

The focus is on the scope and tasks performed by a structural engineer as they relate to foundation design and geohazard mitigation.

- **Nonstructural Components:** Nonstructural components in buildings should be considered in all seismic strengthening projects; however, this *Guide* only shows a simple example on the application of the nonstructural provisions of ASCE 41-13. The lack of numerous examples in this document is not to deemphasize the importance of

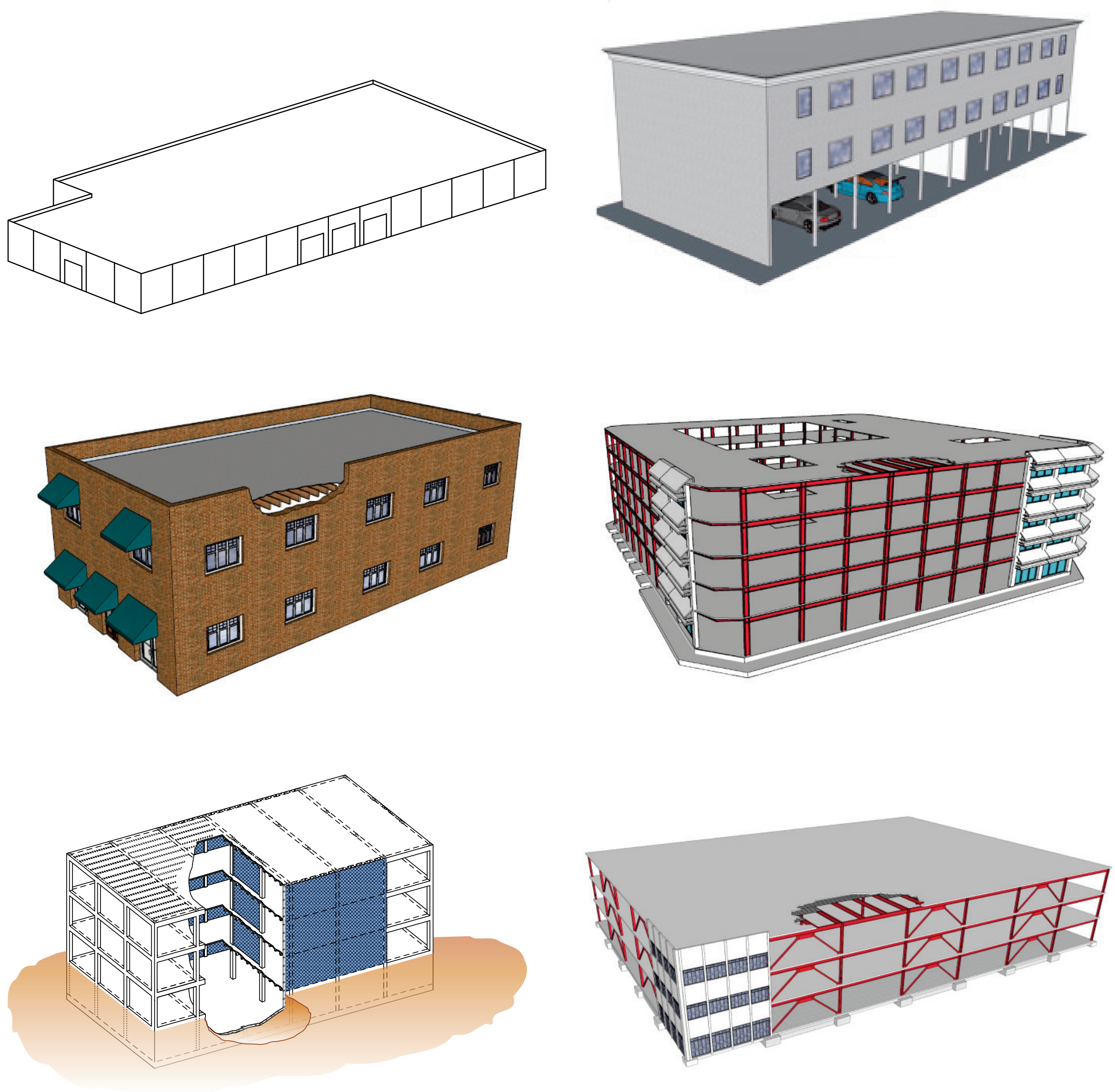


Figure 1-2 Illustration of selected example buildings in this *Guide* (clockwise from upper left: PC1, W1a, S1, S2, C2, URM).

addressing these nonstructural systems, but rather, as often done on major seismic strengthening projects, the nonstructural components are typically replaced or strengthened to comply with those provisions applicable to new buildings in ASCE 7. Furthermore, the 2015 *International Existing Building Code* (IEBC) does not require nonstructural components to be strengthened when mandatory seismic strengthening is triggered.

- **Seismic Isolation and Energy Dissipation:** No examples are provided for seismic isolation or energy dissipation systems as these are used infrequently; the topics are highly specialized, and there are other publications that provide guidance.
- **Other Seismic Rehabilitation Documents:** The design examples in this *Guide* are focused on ASCE 41-13 and do not address other seismic rehabilitation documents and special procedures, such as the IEBC Appendix Chapters (ICC, 2015). However, where such overlap exists, such as in the unreinforced masonry bearing wall design example, some reference to similar procedures contained in other seismic rehabilitation documents is provided.
- **Retrofit Techniques:** Although examples in this *Guide* will show analysis of selected retrofits, detailed information about retrofit techniques is not provided. A document such as FEMA 547 (FEMA, 2006) provides extensive information on retrofit techniques and should be referenced.

1.6 How to Use this Guide

ASCE 41-13 is a standard developed through a consensus-based process. On the other hand, this *Example Application Guide* and the examples within it have been developed by the FEMA-funded ATC project team, with input from the Structural Engineers Association of California's Existing Buildings and Seismology Committees.

The examples in the *Guide* do not necessarily illustrate the only appropriate methods of design and analysis using ASCE 41-13. Proper engineering judgment should always be exercised when applying these examples to real projects. The *Example Application Guide* is not meant to establish a minimum standard of care but, instead, presents reasonable approaches to solving practical engineering problems using the ASCE 41-13 methodology.

Margin boxes like the one shown here are used to highlight useful tips and key terms, offer commentary on key issues and alternative approaches,

Useful Tip
Throughout the *Example Application Guide*, blue margin boxes are used to draw attention to key issues, short cuts, alternatives, and other issues.

summarize design example features, and identify provisions that changed in ASCE 41-17.

Several other helpful features are also provided. These include the following:

- A matrix in the section "Location of ASCE 41-13 Provisions" shows the location in the *Guide* for specific ASCE 41-13 sections.
- Flowcharts summarize the steps in more complicated analysis procedures.
- Graphics illustrate building geometry, key components, and free-body diagrams in calculations.

In developing the *Guide*, several strategies and conventions were adopted in the design example presentations. These include the following.

- Where there are a series of similar components that would be evaluated by the same calculation procedure, a worked out example of the calculations is typically shown in detail only once. Summary tables then show the results for the other similar components.
- Significant figures are taken to a reasonable level for engineering presentation that is generally consistent within the example. Summary tables often are based on calculation spreadsheets that have more significant figures, so the final value of a calculation or compilations in tables that add values can have small roundoff differences.
- The focus is on key selected items in each example to keep the document size manageable. Not all necessary items that would need to be checked or designed are shown. In many cases, a list of these additional items is provided.
- Computer output is shown in some design examples. Neither FEMA, nor the authors and project participants, endorse any particular computer software program or vendor.
- For brevity, the convention "ASCE 41-13 § 7.13" is used when referring to that section within the standard. For other standards, the section symbol "§" is not used, and the convention is ACI 318-11 Section 21.13.4. To avoid confusion between a section in a standard and one within this *Example Application Guide*, a convention such as "Section 8.4 of this *Guide*" has been established.
- Terminology in the *Guide* is intended to match that given in ASCE 41-13, including capitalization. A glossary is provided for convenience.

Chapter 2

Guidance on Use of ASCE 41-13

2.1 ASCE 41-13 Overview

The ASCE 41-13 standard establishes a performance-based design methodology that differs from seismic design procedures for new buildings, and is a combination of ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003), and ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings* (ASCE, 2007). The provisions are wide-ranging to accommodate buildings of different types and eras, and address a variety of structural and nonstructural systems. Where structural materials or components are not explicitly addressed in ASCE 41-13, guidelines are also provided to utilize the standard for these cases.

Performance-based design concepts are implemented through a selection of one or more targeted building *Performance Objectives* consisting of pairings of *Building Performance Levels* and *Seismic Hazard Levels*. A Building Performance Level is a combination of the performance of both the structural and nonstructural components and is expressed as a discrete damage state: Immediate Occupancy (IO), Damage Control, Life Safety (LS), Limited Safety, and Collapse Prevention (CP) for Structural Performance Levels; and Operational, Position Retention, Life Safety, and Not Considered for Nonstructural Performance Levels. There is also the Enhanced Safety Structural Performance Range that covers the damage states between the Immediate Occupancy and Life Safety Structural Performance Levels and the Reduced Safety Structural Performance Range that covers the damage states between the Life Safety and Collapse Prevention Structural Performance Levels. Seismic Hazard Levels are defined as their probability of exceedance in a specified time period and may include, for example, a ground motion with a 10% probability of exceedance in 50 years or 50% probability of exceedance in 50 years. ASCE 41-13 contains multiple predefined Seismic Hazard Levels including Basic Safety Earthquake (BSE) levels BSE-1E and BSE-2E for use with the Basic Performance Objective for Existing Buildings, and BSE-1N and BSE-2N for use with the Basic Performance Objective Equivalent to New Building Standards. The BPOE and BPON Performance Objective definitions are given in ASCE 41-13

Key Terms

Building Performance Level: What happens to the building in the earthquake?

Seismic Hazard Level: How severe is the shaking?

Performance Objective: How much damage is acceptable at a given intensity of shaking?

Immediate Occupancy (IO): Building is safe to occupy soon after an earthquake

Life Safety (LS): Structure is damaged but retains a margin against the onset of collapse

Collapse Prevention (CP): Structure is damaged and maintains gravity support but retains no margin against collapse

BSE-2N: Ground motion consistent with that used in ASCE 7-10 for new buildings and defined as the Risk-Targeted Maximum Considered Earthquake (MCE_R)

BSE-1N: Ground motion defined as 2/3 of the BSE-2N

BSE-2E: Ground motion defined as 5% probability of exceedance in 50 years, but not greater than BSE-2N

BSE-1E: Ground motion defined as a 20% probability of exceedance in 50 years, but not greater than BSE-1N

Table 2-1 and Table 2-2, and they depend on the Risk Category for the building.

ASCE 41-13 provides a three-tiered evaluation and retrofit approach that can be used as an efficient means for identifying and mitigating potential seismic deficiencies in an existing building. There are limitations on building types and heights, as well as vertical and horizontal combinations of seismic-force-resisting systems, for which the Tier 1 and Tier 2 procedures are permitted. For eligible buildings, the *Tier 1 Screening* Procedure is used to assess primary components and connections in the seismic force-resisting system, as well as nonstructural systems, through the use of standardized checklists and simplified structural calculations (called Quick Checks). The checklist screening is general in nature and intended to familiarize the design professional with the building characteristics and components and to identify potential seismic deficiencies that may preclude the building from meeting the given Performance Objective. If the building conforms to a model building type and no potential deficiencies are identified in the Tier 1 screening, there is sufficient confidence that the building will meet the given Performance Objective and further engineering evaluation is not required by the standard. If a Tier 1 screening does not demonstrate compliance with the Performance Objective, then a *Tier 2 Deficiency-Based Evaluation* may be performed utilizing linear analysis procedures to review those items identified as potential deficiencies. Tier 1 and 2 procedures are intended to assess and reduce seismic risk efficiently by using simplified procedures for specific building types and are permitted to demonstrate compliance with only Immediate Occupancy, Damage Control or Life Safety Structural Performance Levels depending on the Risk Category of the structure.

For structures not meeting the Tier 1 and Tier 2 criteria and limitations, or where the design professional decides to perform a more detailed evaluation for any Structural or Nonstructural Performance Level, the procedures for the *Tier 3 Systematic Evaluation* Procedure should be used to assess all of the building components. The Tier 3 analysis may confirm deficiencies that were identified as potential deficiencies in the Tier 1 evaluation. The Tier 3 procedure utilizing nonlinear analysis may also lead to a more economical retrofit solution in cases when the simplified Tier 1 and 2 procedures provide conservative results. Because the Tier 2 and Tier 3 procedures reference the same linear analysis provisions, a Tier 3 linear analysis will produce similar results to a Tier 2 approach.

Four types of analysis procedures are permitted in ASCE 41-13 for the evaluation of building performance:

- Linear Static Procedure (LSP)
- Linear Dynamic Procedure (LDP)
- Nonlinear Static Procedure (NSP)
- Nonlinear Dynamic Procedure (NDP)

The LSP and LDP are the only procedures permitted for Tier 2 analyses, while any of the four methods may be used for Tier 3. For the LSP and LDP, the analysis is expected to produce displacements that approximate maximum displacements expected for the selected Seismic Hazard Level, but will produce component forces that exceed those that would occur in a yielding building. Component actions are defined as force-controlled (remains elastic) or deformation-controlled (allowed to exceed the yield capacity of the element). The acceptance criteria for deformation-controlled actions include component modification factors (*m*-factors) to account for anticipated inelastic response demands and capacities.

Key Terms

LSP: Linear static procedure

LDP: Linear dynamic procedure

NSP: Nonlinear static procedure

NDP: Nonlinear dynamic procedure

Backbone curve: Force versus deformation curve for a specific component action (e.g. shear, axial, etc.)

For the nonlinear procedures, considerable judgment and experience are required to model the necessary portions of the building as a nonlinear system with the structural components (e.g., beams, columns, connections, and foundations) represented using the modeling parameters (backbone curves) provided in the standard. Component demands are evaluated using acceptance limits for the associated component action and structural performance level. These nonlinear modeling parameters and acceptance criteria can also be used for the design of a new building when the Performance Objective is shown to provide equivalency to that of new building codes and standards.

For additional information on the development of the methodology, refer to FEMA 274, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997b).

2.2 Comparison of ASCE 41-13 and ASCE 7-10 Design Principles

This section provides a detailed summary and comparison of the basic principles and philosophical approach of ASCE 41-13 for existing buildings and ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), for new structures.

2.2.1 New Building Seismic Design Principles

Design of the seismic force-resisting system for new buildings in standards such as ASCE 7-10 is based on prescriptive design and detailing provisions for components. For economic reasons, the structure is designed and detailed to sustain damage with specific members expected to yield under the design seismic loading (SEAOC, 2008b). The structural system is also designed to meet an established set of conditions and configurations so that the building can be analyzed with a uniform set of system coefficients (R, Ω_0, and C_d) and global design requirements, such as base shear strength, inter-story drift, and torsion.

Special detailing provisions are intended to allow yielding in predetermined zones, which are designed to sustain cyclic, inelastic action during an earthquake. Since the gravity system undergoes the same lateral displacement as the vertical seismic force-resisting system, components that support gravity loads are also designed with special detailing provisions to help ensure displacement compatibility and that global stability is not compromised.

The anticipated inelastic behavior is incorporated into the design with the response modification factor, R. The R-factor is dependent on the seismic force-resisting system and results in a design base shear (V in Figure 2-1) that is a fraction of the elastic seismic force (V_E in Figure 2-1). Since the design and detailing requirements are controlled, the nonlinear behavior is assumed to be consistent throughout the structural system, and a single R-factor is utilized in the design of all components.

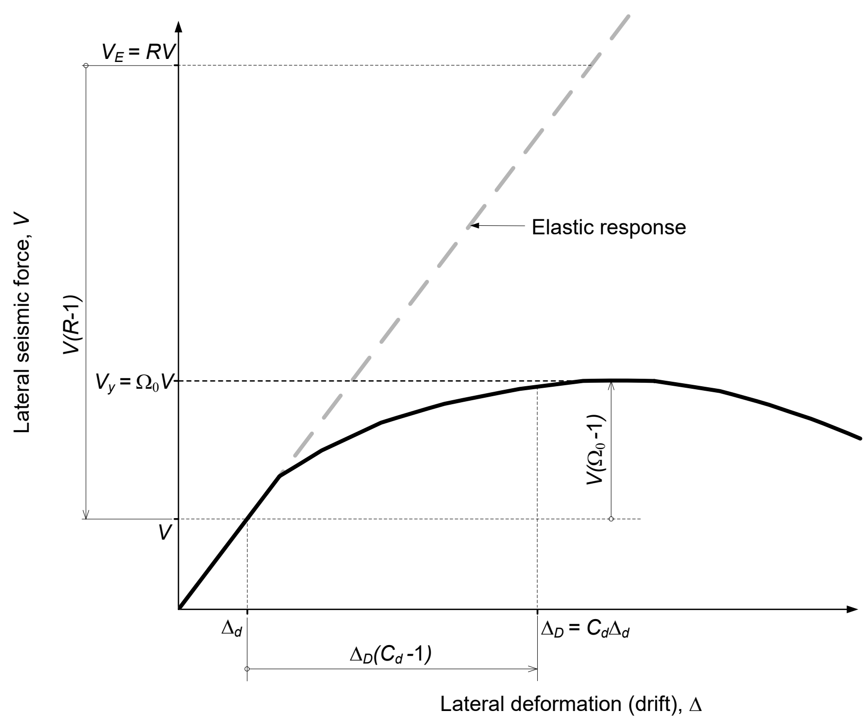


Figure 2-1 System force-deformation relationships and terminology used in new building design procedures of ASCE 7-10.

The system overstrength factor, Ω_0, is dependent on the material and the seismic force-resisting system. The Ω_0 factor is intended to address the potential for increased forces, V_y, due to actual, higher material strengths and any post-yield strength increase for the structural system. This factor is only utilized for particular components and connections to ensure a strength hierarchy and to control the inelastic behavior in specific zones. Similarly, the deflection amplification factor, C_d, is dependent on the material and structural system and is used to approximate the inelastic deformation, Δ_D, beyond the design displacement, Δ_d, which results from designing the structure with an R-factor. For more information on the derivation of these system coefficients, refer to the ASCE 7-10 Commentary.

Figure 2-1 shows an example force-displacement relationship along with the aforementioned system coefficients and design parameters for new building seismic design.

For new building design, a relatively high level of confidence in component capacities is achieved with material testing requirements and the application of a strength reduction factor, ϕ.

Through the use of these provisions, codes and standards for new buildings allow for a uniform application of system design coefficients and also provide a margin of safety to account for variability in building response and the design and construction processes, as well as uncertainty in earthquake hazards.

2.2.2 ASCE 41-13 Seismic Evaluation and Retrofit Design Principles

The provisions in ASCE 41-13 for the evaluation and retrofit of existing buildings are based upon component-level assessments, as opposed to the system-level approach for new buildings. Existing buildings have a wide range of seismic force-resisting system types, varying from de-facto systems of archaic materials (unreinforced masonry) to systems that are similar to those used in new building construction (concrete frames). Most of these systems typically do not meet the detailing requirements required by more recent building codes for the seismic force-resisting system and gravity load-carrying system. Therefore, the inelastic behavior of components may not be consistent throughout a structure, and the components must be evaluated on an individual basis to assess the seismic performance. The component-level assessments are in turn utilized to evaluate the likely global performance of the structure.

The ASCE 41-13 provisions apply a displacement-based approach for the various analysis procedures. A fundamental difference between ASCE 41-13 and ASCE 7-10, which reduces the demand with a system *R*-factor, is that ASCE 41-13 linear procedures capture yielding and ductility through modifiers (*m*-factors) that increase the elastic capacity of component actions to determine adequacy compared to an unreduced seismic demand; these modifiers vary by material and target building performance, and are specific to each component action. The ASCE 41-13 linear analysis procedures are intended to provide a conservative and approximate estimate of building response and a reliable performance. Nonlinear analysis procedures typically provide a more accurate assessment of building response and performance by explicitly incorporating yielding in all of the components in a structural model. Accordingly, nonlinear procedures utilize less conservative acceptance criteria than those used for linear procedures for the same target Performance Objective.

A pseudo seismic force is calculated in the ASCE 41-13 linear static procedure (LSP) similar to the base shear used for new buildings. However, the pseudo seismic force is unreduced (no *R*-factor) and includes modification factors (C_1, C_2, and C_m) to account for expected inelastic displacements, strength degradation, and higher mode effects. The unreduced seismic force is applied to the building, and the resulting demand on each component is assessed. Component yielding and ductility, where expected to occur, are accounted for with modification factors applied to the component capacity. Specifically, ductile component actions, such as flexure in a moment frame beam, are evaluated as deformation-controlled by using expected material capacities and *m*-factors in proportion to the acceptable level of ductility. Because ASCE 41-13 addresses ductility at a component level rather than a global level like ASCE 7-10, the seismic demand forces for the LSP are typically significantly higher than that determined using new building code design, since the seismic demand is not reduced by a global *R*-factor. The component capacities, which include expected material strengths and are multiplied by component-specific *m*-factors, are correspondingly higher as well. Select components, including those deemed to be critical to maintaining gravity support, are evaluated as force-controlled components, which are required to remain essentially elastic and their capacities are not increased by ductility factors. The linear dynamic procedure (LDP) is a response-spectrum-based modal or linear response history analysis procedure that is utilized when the distribution of seismic forces within a structure cannot be adequately assessed with the LSP.

Figure 2-2 shows a typical force-displacement relationship and some of the corresponding terminology used for linear procedures in ASCE 41-13. The underlying concept is that when the amplified pseudo seismic force is applied to a linearly elastic model of the building, then the resulting displacement amplitude approximates the maximum displacement expected during the selected Seismic Hazard Level.

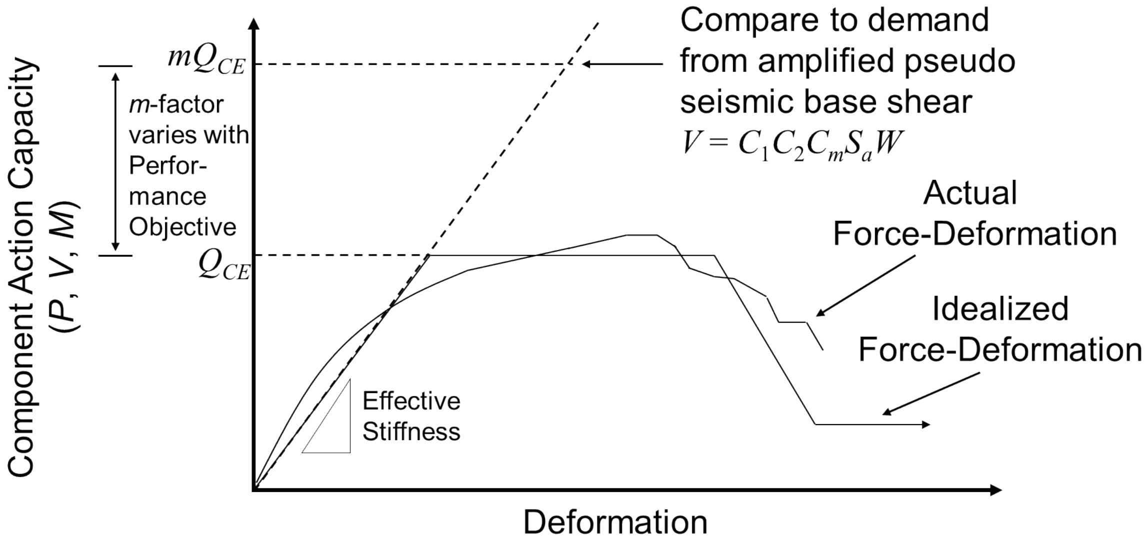


Figure 2-2 Deformation-controlled component force-displacement relationships and terminology used in ASCE 41-13.

The nonlinear static procedure (NSP) utilizes a nonlinear mathematical model with the structural components' strength, stiffness, and yielding characteristics explicitly modeled. The building is subjected to increasing lateral deflection while the displacement demand on seismic force-resisting elements is evaluated. This process captures yielding of individual components and any redistribution of forces, resulting in an idealized lateral force versus displacement relationship for each direction under consideration. A target (demand) displacement is approximated based on the spectral acceleration and modification factors that are similar to those used in the LSP. The components of the seismic force-resisting system and gravity system are evaluated at the target displacement using the appropriate component action acceptance criteria. The nonlinear dynamic procedure (NDP) utilizes a similar mathematical model subjected to ground motion acceleration histories to obtain forces and displacements, which are also compared to component acceptance criteria.

The performance-based design approach of ASCE 41-13 is intended to evaluate the building for the actual expected behavior during an earthquake. Therefore, the expected component strength, Q_{CE}, is used with a strength reduction factor of $\phi = 1.0$. Existing buildings often have no inspection or materials testing records, so destructive investigation may be used to determine material properties. Depending on the level of information available, default material strengths may be utilized, and a knowledge factor may be applied to reduce the material strength where appropriate.

2.2.3 ASCE 7-10 and ASCE 41-13 Design Examples

2.2.3.1 Overview

In order to illustrate the differences between the ASCE 7 and ASCE 41 approaches, the following design examples include calculations for the concrete building described in Chapter 10 of this *Guide* using both the ASCE 7-10 and ASCE 41-13 standards to derive the following:

- Design base shear values
- Design forces and capacities for a concrete shear wall

The building is a three-story office building located in Seattle, Washington comprised of concrete shear walls with concrete frames. For the purposes of comparison, the BSE-2E seismic hazard level is used for both analyses. (For more information on Seismic Hazard Levels and determination, see Section 3.3 of this *Guide*.)

2.2.3.2 ASCE 7-10 Example

In accordance with ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2011), Section 21.13.4, the gravity frame is detailed to produce a ductile flexural response and is not relied upon to resist any seismic forces. Therefore, the shear walls are designed to resist all of the seismic forces.

The design spectral response acceleration for the BSE-2E seismic hazard level is S_{DS} = 1.08g, as shown in Section 10.2.3 of this *Guide*.

The structural system is assumed to be an ordinary concrete shear wall building frame system because of the lack of reinforcing detailing in the existing wall. Therefore, the seismic design parameters are $R = 5$ and $\Omega_0 = 2.5$ (ASCE 7-10 Table 12.2-1). Note that this seismic force-resisting system would not be permitted for new building construction at this location.

The base shear is:

$$\begin{aligned} V &= C_s W = S_{DS}/(R/I_e)W \qquad \text{(ASCE 7-10 Eq. 12.8-1)} \\ &= 1.08/(5/1.0)W = 0.216W \end{aligned}$$

Given a seismic weight of W = 2,880 kips, the base shear is V= 622 kips.

The shear demand in Wall D is given in Table 10-9 of this *Guide* as 23% of the total base shear, so the resultant Wall D shear force is V_u = 0.23 × 622 kips = 143 kips.

The material strengths of the concrete and reinforcement are given as:

f'_c = 2,500 psi

f_y = 40,000 psi

Note that special concrete shear walls would be required for new construction and ACI 318-11 would not permit a concrete strength, f'_c, of less than 3,000 psi or a reinforcing ratio, ρ, less than 0.0025; however, these values are used for the sake of this comparison.

For Wall D, the shear wall strength is given by ACI 318-11 Equation 21-7:

$$\begin{aligned} V_n &= A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y) \\ &= 1{,}920\left[2.0(1.0)\sqrt{2{,}500} + 0.0016(40{,}000)\right] \times 10^{-3} = 315 \text{ kips} \end{aligned}$$

Per the example in Chapter 10 of this *Guide*, the wall is flexure-controlled, so the shear strength is:

ϕV_n = 0.75(315 kips) = 236 kips

The ratio of $V_u/\phi V_n$ is:

$V_u/\phi V_n$ = (143 kips)/(236 kips) = 0.61

Therefore, the wall has adequate capacity to resist the demand.

2.2.3.3 ASCE 41-13 Example

The relative rigidity and distribution of forces to the shear wall and frame are unknown, so both would be evaluated in the ASCE 41-13 analysis. For this example, only the shear wall evaluation is performed, and the shear wall is assumed to resist all of the seismic forces.

The linear static procedure is utilized for this example with the pseudo seismic force (base shear) calculated for the transverse direction per Section 10.4.7 of this *Guide*. As in the previous ASCE 7-10 example, the BSE-2E seismic hazard level is selected with S_a = 1.08g. The Life Safety Performance Level is used in this example to be consistent with the underlying performance assumed by ASCE 7-10.

$$\begin{aligned} V &= C_1 C_2 C_m S_a W && \text{(ASCE 41-13 Eq. 7-21)} \\ &= 1.17(1.01)(0.8)(1.08)W = 1.02(2{,}880) = 2{,}940 \text{ kips} \end{aligned}$$

C_1, C_2, and C_m are modification factors calculated in Section 10.4.6.1 of this *Guide*.

The resultant shear demand in Wall D is given in Table 10-9 of this *Guide* as:

Q_{UD} = 672 kips

Note that the ASCE 41-13 shear demand is significantly higher than in the ASCE 7-10 analysis.

The capacity of Wall D is determined using expected material strengths (see Section 10.3 of this *Guide*):

f'_{ce} = 2,500 (1.5) = 3,750 psi

f_{ye} = 40,000 (1.25) = 50,000 psi

Commentary

In traditional engineering practice, the term demand-capacity ratio or DCR represents the code demand divided by the capacity. For shear in a shear wall at the factored level, this would be $V_u/\phi V_n$.

ASCE 41-13 has a different and very specific definition of DCR. ASCE 41-13 Equation 7-16 defines DCR as Q_{UD}/Q_{CE}. The capacity does not include the *m*-factor or knowledge factor, κ. The DCR is thus a measure of required component ductility.

In the *Example Application Guide*, the term "acceptance ratio" is defined as $Q_{UD}/m\kappa Q_{CE}$ as this represents the traditional concept where a component with an acceptance ratio equal to or less than one would have "acceptable" or sufficient capacity.

The expected shear strength of the wall is calculated similarly to the previous section with ϕ = 1.0 and is reported in Section 10.4.4 of this *Guide* as:

Q_{CE} = 388 kips

As shown in Section 10.5.1 of this *Guide*, the wall is treated as deformation-controlled. An *m*-factor of m = 2.33 is calculated for the Life Safety Performance Level from ASCE 41-13 Table 10-21, and the knowledge factor is taken as κ = 0.9.

The resultant acceptable shear wall loading is:

$m\kappa Q_{CE}$ = 2.33(0.9)(388 kips) = 814 kips

Note the expected elastic shear wall capacity determined using ASCE 41-13 procedures is also significantly higher than that calculated with ASCE 7-10.

Therefore, the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ is:

$Q_{UD}/m\kappa Q_{CE}$ = (672 kips)/(814 kips) = 0.83

Again, the wall has adequate capacity to resist the demand.

2.2.3.4 Summary and Comparison

The demand and capacity for the shear wall are shown in Table 2-1 for the two different standards. As previously noted, the ASCE 41-13 approach provides higher demands and capacities relative to the ASCE 7 procedure.

For new building design using ASCE 7-10, extensive detailing provisions would be followed to complete the design of the shear wall (e.g., boundary elements), as well as the gravity frame and other structural components.

Table 2-1 Summary of Shear Wall Demands, Capacities, and Acceptance Ratios

Standard	Demand	Capacity	Acceptance Ratio $V_u/\phi V_n$ or $Q_{UD}/m\kappa Q_{CE}$
ASCE 7-10	V_u = 143 kips	ϕV_n = 236 kips	0.61
ASCE 41-13	Q_{UD} = 672 kips	$m\kappa Q_{CE}$ = 814 kips	0.83

Useful Tip

Because ASCE 41-13 effectively uses unreduced demands, the force levels are significantly higher than those used in ASCE 7 which has an *R*-factor. This is accounted for by applying the *m*-factor to capacities.

For an existing building evaluation using ASCE 41-13, components would be analyzed based on the as-built detailing and resultant *m*-factors, and then compared to the unreduced seismic demand. Where acceptance criteria are not satisfied, the structural system may be augmented to reduce individual component demand, or the specific component strength and/or ductility may be increased.

Although there is no direct comparison between the ASCE 7-10 and ASCE 41-13 approaches, the philosophies of new building design and existing building evaluation and retrofit are similar when considered from a purely mathematical standpoint. The *R*-factors reduce the demand and are generally constant for the building, whereas *m*-factors increase the capacity and vary depending on the expected ductility of the component action. Although not directly equivalent, comparison of the system *R*-factor and component *m*-factors, as well as the Ω_0 factor and the amplified seismic demands of ASCE 41-13, can provide context to assist with engineering decisions.

2.3 When Should ASCE 41-13 be Used?

The selection of ASCE 41-13 for the evaluation or retrofit of a building and the determination of the design criteria are dependent on a number of factors. Table 2-2 provides a list of common evaluation methods as they relate to ASCE 41-13. Note that ASCE 41-13 is for use only with undamaged buildings. The column in Table 2-2 for earthquake-damaged buildings is to provide comparative context.

ASCE 41-13 is predominantly used for the evaluation and retrofit of existing buildings. The standard is intended for a range of audiences including engineers, building officials, building owners, government agencies, and policy makers. The provisions are applicable to all types of building structures and enable design professionals to develop a practical and effective approach to assess the seismic performance of a building. An assessment or retrofit utilizing ASCE 41-13 provisions may be performed on a voluntary basis, as the result of a state mandate or local ordinance, or possibly as a requirement for a financial transaction. One can also arrive at the ASCE 41-13 standard through building code regulations or seismic triggers for modifications, alterations, or repairs to an existing building. Lastly, in the context of new buildings, there is very little guidance in current building codes and standards on the use of nonlinear analysis procedures for design. As a result, ASCE 41-13 may be used by practitioners as the basis for new building designs that employ nonlinear analysis methods, subject to the approval of the Authority Having Jurisdiction (AHJ). The recently published ASCE 7-16 standard explicitly references ASCE 41-13 for deformation-controlled acceptance criteria and, by inference, nonlinear modeling parameters.

Table 2-2 Comparison of Seismic Evaluation Methods

Evaluation Method	Undamaged Buildings	Earthquake-Damaged Buildings
Rapid Evaluation	FEMA P-154[1]	ATC-20[4] Rapid
Quick Evaluation	ASCE 41-13 Tier 1	ATC-20 Detailed
Intermediate Evaluation	ASCE 41-13 Tier 2	FEMA 352[5] ATC-52-4[6]
Detailed Evaluation	ASCE 41-13 Tier 3 FEMA P-807[2] FEMA P-58[3]	FEMA 306[7] ATC-52-4

[1] FEMA P-154, *Rapid Visual Screening of Buildings for Seismic Hazards: A Handbook* (FEMA, 2015a)

[2] FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories* (FEMA, 2012b)

[3] FEMA P-58, *Seismic Performance Assessment of Buildings, Volume 1 – The Methodology* (FEMA, 2012c)

[4] ATC-20-1, *Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC, 2005)

[5] FEMA 352, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* (FEMA, 2000c)

[6] ATC-52-4, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Post-Earthquake Repair and Retrofit Requirements* (ATC, 2011)

[7] FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual* (FEMA 1998a)

Where evaluations are performed on a voluntary basis, the building owner, with the engineer's guidance, is typically permitted to select the Performance Objective and evaluation procedure (i.e., Tier 1, Tier 2, and/or Tier 3) that are most suitable for the subject building and circumstances. In most model building codes, voluntary seismic improvements are permitted, provided an engineering analysis is performed to demonstrate that the retrofitted structure (and nonstructural components, where applicable) is no less conforming with the building code provisions than prior to the retrofit. For scenarios involving policy mandates or seismic triggers, the required Performance Objectives are stipulated by the AHJ and pertinent building codes.

ASCE 41-13 accommodates a spectrum of Performance Objectives by allowing users to select a range of Performance Levels and Seismic Hazard

Levels. Recommendations for the selection of an appropriate Performance Objective are beyond the scope of this document and can vary substantially depending on the circumstances. However, for many seismic evaluations and retrofits, the Basic Performance Objective (referred to as the Basic Safety Objective (BSO) in ASCE 41-06) is often used as reference point. ASCE 41-13 provides both a Basic Performance Objective for Existing Buildings (BPOE) and a Basic Performance Objective Equivalent to New Building Standards (BPON). Each of these Performance Objectives consists of a selected target Structural Performance Level in combination with a specific Seismic Hazard Level that varies with the designated building Risk Category, which is determined in accordance with the governing building code. The Performance Objective is dictated by the Risk Category.

Key Terms

BPOE: Basic Performance Objective for Existing Buildings accepts a lower level of safety and higher risk of collapse than would be provided by standards for new buildings.

BPON: Basic Performance Objective Equivalent to New Building Standards is meant to provide performance equivalent new building standards.

The BPOE accepts a lower level of safety and a higher risk of collapse than that which would be provided by standards for new buildings. Buildings that satisfy the BPOE requirements are expected to experience little damage from relatively frequent, moderate earthquakes, but the potential exists for significant damage and economic loss from the most severe and infrequent earthquakes. The following are three overarching, historical reasons for accepting greater risk in existing buildings:

- Recently constructed buildings are not rendered deficient with subsequent code changes
- Existing buildings are expected to have a shorter remaining life than the 50-year life often assumed for new buildings
- The cost of achieving a higher level of certainty in performance for existing buildings is often disproportionate to the additional benefit

This philosophy has been included in previous standards, including ASCE 31-03, as well as the *International Existing Building Code* (ICC, 2015), which allow design seismic forces for existing buildings to be 75% of seismic forces required for new buildings. Reduced seismic forces for existing buildings are also often permitted by local jurisdictions.

The BPON is meant to provide performance equivalent to that of new buildings designed to ASCE 7-10 by using the Seismic Hazard Levels specified therein. Nonetheless, the structural systems of an existing building are generally not as robust as those of a new building due to the lack of prescriptive requirements, and there is typically a higher level of uncertainty in performance when compared to new buildings. The acceptance criteria in ASCE 41-13 have not been directly calibrated to the performance provided by new building codes and standards. However, the National Institute of

Standards and Technology (NIST) has released a series of technical notes with the results of assessments of code-compliant buildings using the ASCE 41-13 methodology for different building systems (NIST, 2015a; 2015b; 2015c).

The AHJ may have specific requirements for the Performance Objective, so it is recommended to discuss and confirm the proposed design criteria with the building official and the owner before proceeding with the retrofit design. If Performance Objectives and design criteria are not stipulated by the building official or owner, the Basic Performance Objective for Existing Buildings may be adopted, and enhanced or lowered at the discretion of the owner.

Key questions to consider when determining the applicability and implementation of the ASCE 41-13 standard are shown in Figure 2-3.

2.4 What is New in ASCE 41-17?

Following the completion of ASCE 41-13, the ASCE 41 Standards Committee on Seismic Rehabilitation initiated the development of ASCE 41-17 including proposed changes to ASCE 41-13. Subsequently, ASCE 41-17 was published in December 2017. This section highlights significant technical changes made in ASCE 41-17. For a more detailed discussion of changes, see Pekelnicky et al. (2017). As noted in Chapter 1 of this *Guide*, margin boxes in the *Guide* highlight provisions with noteworthy changes in ASCE 41-17

2.4.1 Chapter 1 General Requirements

No significant changes were made to ASCE 41-17 Chapter 1. This chapter serves as an introduction to the standard, including discussion of the typical seismic evaluation and retrofit processes, as well as key definitions and notations used throughout the standard.

2.4.2 Chapter 2 Performance Objectives and Seismic Hazards

The scope of this chapter consists of defining the various Performance Objectives in the standard and the associated Seismic Hazard Levels and target Building Performance Levels. Significant changes in ASCE 41-17 Chapter 2 include requirements related to the Basic Performance Objective for Existing Buildings (BPOE) and the Hazards Reduced Nonstructural Performance Level. Other modifications include direct references to the ASCE 7-16 standard for development of seismic hazard design parameters and ground motion selection and scaling.

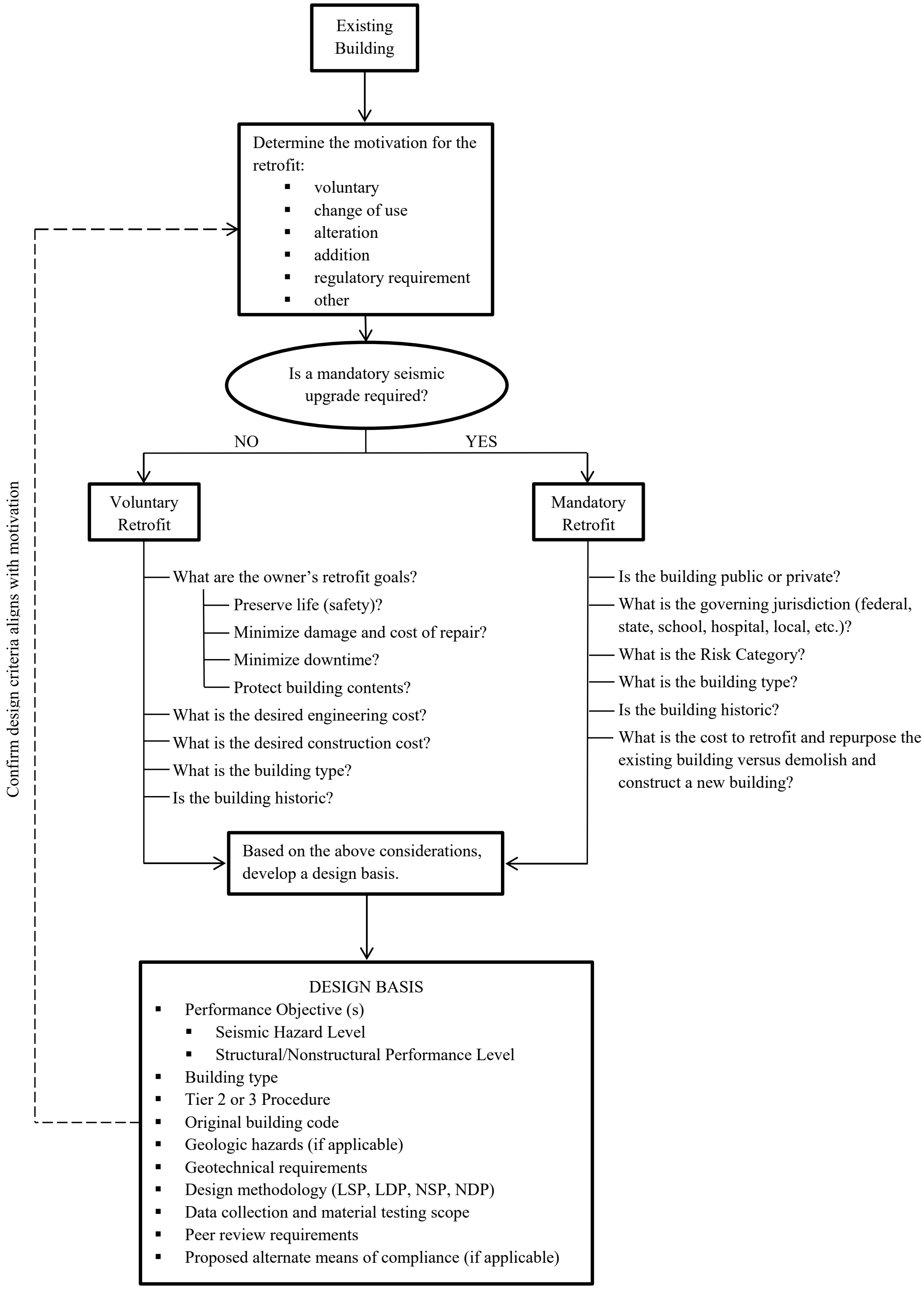


Figure 2-3 Retrofit design flowchart.

2.4.2.1 Basic Performance Objective for Existing Buildings

Concerns were raised that meeting Life Safety in the 225-year hazard in ASCE 41-13 might not provide Collapse Prevention performance in the 975-year hazard, especially in regions where there is a significant difference in intensity between the two hazards. ASCE 41-17 changes the BPOE for Tier 1 and 2 evaluations to require consideration of the Collapse Prevention Performance Level at the BSE-2E Seismic Hazard Level, rather than the Life Safety Performance Level at the BSE-1E Seismic Hazard Level, for Risk Category I, II, and III buildings. For Risk Category IV buildings, performance must be evaluated in Tier 1 and Tier 2 evaluations at both the BSE-1E and BSE-2E Seismic Hazard Levels.

2.4.2.2 Hazards Reduced Nonstructural Performance Level

A small subset on nonstructural components was identified to represent as much a risk to the building occupants as a partial or total collapse of a building. The ASCE 41 committee decided that such hazards should have a significant margin of safety beyond the BSE-1E hazard and introduced a new Hazards Reduced Nonstructural Performance Level (N-D) to encompass mitigating only the most significant nonstructural hazards. ASCE 41-06 and its predecessor FEMA documents had a Hazards Reduced Nonstructural Performance Level that attempted to accomplish a similar objective. The following items were incorporated into the Hazards Reduced Nonstructural Performance Level:

- Release of hazardous materials
- Failure of heavy cladding over sidewalks where many people congregate
- Failure of heavy ceilings in assembly spaces
- Failure of large architectural appendages and marquees
- Failure of heavy interior partitions and veneers

ASCE 41-17 includes a note that permits components identified above to be excluded from the Hazards Reduced Nonstructural Performance Level if it can be demonstrated that the component does not pose a threat of serious injury to many people due to falling or failing under the Seismic Hazard Level being considered.

2.4.3 Chapter 3 Evaluation and Retrofit Requirements

Chapter 3 of ASCE 41-13 provides general evaluation and retrofit requirements, detailed descriptions of model or Common Building Types, requirements for investigation of existing conditions, and limitations on use of Tier 1 and Tier 2 provisions. Revisions to ASCE 41-17 Chapter 3 were fairly minimal. Provisions for Benchmark Buildings from ASCE 41-13 Chapter 4 were relocated to this chapter to emphasize applicability to all buildings, and not just those eligible for Tier 1 procedures. Cold-formed steel systems were added the Common Building Type descriptions and the list of Benchmark Buildings.

2.4.4 Chapter 4 Tier 1 Screening

Chapter 4 of ASCE 41-13 covers Tier 1 screening. The most significant changes to the Tier 1 Screening provisions (and Tier 2 Evaluation provisions in Chapter 5) were made in response to the change in the BPOE, requiring evaluation at the BSE-2E Seismic Hazard Level. In order to evaluate structural performance at the BSE-2E, the checklists and Quick Check procedures had to be revised to accommodate screening for the Collapse Prevention and Limited Safety structural performance levels, in addition to Life Safety. In researching the checklist development, the committee found that all the items identified in the Life Safety structural checklists affect the collapse probability of the building. Therefore, the structural checklists could be retitled as Collapse Prevention with little change.

Other changes to Chapter 4 include minor revisions to the system modification factors, M_S, for the Quick Checks and miscellaneous updates to the default yield strengths provided for steel, which are commonly used for Tier 1 screenings.

2.4.5 Chapter 5 Tier 2 Deficiency-Based Evaluation and Retrofit

Chapter 5 of ASCE 41-13 outlines the requirements for a Tier 2 analysis that focuses on the deficient components identified using the Tier 1 procedures. The majority of the changes to the Tier 2 procedure were in clarifying the appropriate level of analysis required and what needs to be evaluated based on the checklist statement that is found noncompliant. The bigger change to the Tier 2 procedure comes from the BPOE change, which in ASCE 41-17 requires explicit consideration of the BSE-2E Seismic Hazard Level.

2.4.6 Chapter 6 Tier 3 Systematic Evaluation and Retrofit

The provisions in Chapter 6 of ASCE 41-13 are fairly short and define the basic requirements for seismic evaluations and retrofits using the Tier 3 systematic approach. One key component of this chapter is the various descriptions and conditions for data collection and the associated material knowledge factor, κ, used in the analysis calculations. Revisions in ASCE 41-17 are fairly minor; they aim to clarify the data collection requirements in

Table 6-1 and the intended applications for the minimum, usual, and comprehensive levels of knowledge. In addition, sources and requirements for as-built information and the content of the existing design drawings used are now more explicitly described in the provisions and commentary.

2.4.7 Chapter 7 Analysis Procedures and Acceptance Criteria

Chapter 7 of ASCE 41-13 covers an array of topics regarding analysis procedures, including selection of the appropriate analysis method, mathematical modeling, soil-structure interaction, and acceptance criteria for deformation-controlled and force-controlled actions. Provisions for soil-structure interaction and incorporation of foundation flexibility were revised for clarification purposes and to limit the total amount of force reduction that can be accounted for due to this behavior. The most noteworthy changes to Chapter 7 affect force-controlled actions in linear analysis and nonlinear analysis.

2.4.7.1 Force-Controlled Action in Linear Analysis

The only significant change to the linear analysis procedure is related to the treatment of force-controlled actions. Not adjusting the force-controlled evaluation for Performance Levels creates a situation that is in conflict with the definition of the Life Safety Performance Level providing a margin against collapse and being distinct from the Collapse Prevention Performance Level.

The ASCE 41 committee decided that in order to provide a margin of safety against collapse that is called for in the definition of Life Safety, there should be some margin against failure of a force-controlled action built in to the provisions. To accomplish this, the equation to evaluate force-controlled actions was changed as follows:

$$Q_{UF} = Q_G \pm \frac{Q_E \chi}{C_1 C_2 J}$$

The χ factor is 1.3 for Life Safety and Immediate Occupancy Performance Levels and 1.0 for the Collapse Prevention Performance Level. The χ factor only applies when the demand is calculated using the pseudo-lateral force, Q_E, and not when the demand is calculated based on a capacity-based design. If the shear demand in a concrete column is based on the formation of a plastic moment at each end, then no χ factor amplification is required. However, if the demand is calculated based on the force reported from the analysis model divided by C_1C_2 and a J-factor equal to the lesser DCR of the column bending moments or 2, then the χ factor would apply.

2.4.7.2 Nonlinear Analysis

The 2015 NEHRP Provisions update included a complete re-write of the nonlinear response history analysis provisions found in ASCE 7. Those updates were then passed on to the ASCE 7 committee, which further refined them for incorporation into ASCE 7-16. Haselton et al. (2017a and b), Jarret et al. (2017), and Zimmerman et al. (2017) provide detailed discussion of the updates. Many of those changes were applied to the nonlinear analysis procedures in ASCE 41-17.

2.4.8 Chapter 8 Foundations and Geologic Site Hazards

Chapter 8 in ASCE 41-13 covers foundation and geologic site hazard requirements. The ASCE 41-13 edition included significant updates to the shallow foundation provisions in Chapter 8, and the ASCE 41-17 edition builds upon and improves these methodologies based on user feedback and numerous case studies. Important updates include more relaxed linear acceptance criteria (*m*-factors) for analyzing foundation overturning actions using either a fixed based or flexible base modeling assumption. In addition, the derivation of the expected vertical load at the soil-footing interface was revised for a more appropriate method of determining the moment capacity of a rigid shallow foundation, consistent with the new procedures.

The damping and kinematic soil-structure interaction procedures in ASCE 41-17 Sections 8.5.1 and 8.5.2 were updated based on information in NIST GCR 12-917-21 (NIST, 2012). The changes consist of updated equations and definitions of variables, as well as prohibiting the use of these provisions for deep foundations since that was not the original intent or basis of the provided equations.

2.4.9 Chapter 9 Steel and Iron

Significant updates were made to the steel provisions regarding the modeling and acceptance criteria for steel columns. The column provisions in ASCE 41-13 and previous editions require the ductility of a column be reduced from that of a beam once the axial force including both gravity loads and seismic forces exceeds 20% of the expected axial buckling capacity in the direction of bending ($P_{UF}/P_{CL,x} > 0.2$). The columns then become force-controlled when the axial force ratio increases to more than 50% of the expected axial buckling capacity in the direction of bending ($P_{UF}/P_{CL,x} > 0.5$). A subcommittee reviewed a number of different research reports on the performance of steel columns under combined axial load and bending. That led to the change of the column axial load ratio from the maximum axial load divided by the expected capacity to the gravity load divided by the yield capacity. Additional research indicated that the axial buckling capacity could be replaced by the yield capacity in the denominator. A regression analysis of the data from the papers showed that the ductility of the column could be expressed as a function of the gravity axial load ratio, the web and flange compactness ratios (h/t_w and $b/2t_f$), and the length divided by the weak-axis radius of gyration (L/r_y). Both the *m*-factors and the nonlinear modeling and acceptance parameters were updated based on this research and will yield less conservative assessments of columns in steel buildings.

A few other more minor revisions were made to Chapter 9 in ASCE 41-17. The governing equations and acceptance criteria for link beams in eccentrically braced frames were updated for better alignment with AISC 341. Several updates regarding material strengths were incorporated into Chapter 9, including the addition of HSS and pipe sections to Table 9-1 and Table 9-3, and different properties specified for cast and wrought iron components in Section 9.10.

Along with revisions to the acceptance criteria for steel frame members, provisions for cold-formed steel systems were relocated from Chapter 12 to Chapter 9 and further developed to include detailed procedures for shear wall systems, special moment frames, and strap braced systems. New modeling parameters and acceptance criteria were provided for these systems, and new checklists and procedures were included for Tier 1 and Tier 2 analyses.

2.4.10 Chapter 10 Concrete

The primary changes to the concrete provisions in ASCE 41-17 involve testing of existing anchors, updates to modeling parameters and acceptance criteria for concrete columns, updates to wall stiffness provisions, and clarifications regarding the evaluation of concrete elements with net tension.

The addition of testing requirements for existing concrete anchors was one of the most critical changes for the concrete chapter. In many existing concrete buildings, there are existing cast-in-place and post-installed connections of structural and nonstructural components necessary for transferring seismic forces or anchoring falling hazards (e.g., out-of-plane wall anchorage and anchorage of heavy equipment in an evacuation route). Until more recent building codes, these anchors were not designed and installed per well-defined design procedures and quality control requirements, and there were typically no testing requirements, especially for post-installed mechanical and adhesive anchors. The committee decided to add minimum testing requirements for usual and comprehensive data collections for existing cast-in-place and post-installed anchors.

Another major revision in the concrete provisions involved concrete columns. Ghannoum and Matamoros (2014) summarize much of the work that led to column modeling changes, resulting in column parameters in the form of equations rather than the past table form. The new equation format makes it easier to calculate modeling parameters for different conditions and removes the need for triple interpolation required in previous editions of ASCE 41.

In addition to the anchor testing and column provisions, other technical changes to concrete provisions improved the evaluation of structural walls and elements with net axial tension, which also lead to more consistency between linear and nonlinear procedures. The change for elements in net tension was another change with notable consequences for linear procedures. Axial demands in tension were thus clarified to be analyzed as deformation-controlled actions. This change, along with the changes for wall stiffness, are expected to provide more consistency between ASCE 41-13 linear and nonlinear procedures.

2.4.11 Chapter 11 Masonry

The masonry provisions underwent a number of significant updates. There were updates to the assessment of out-of-plane actions in unreinforced masonry walls based on research. The Collapse Prevention Level evaluation can still be carried out using the table that provides maximum h/t ratios. For the Life Safety Performance Level, an assessment of the wall for dynamic stability based on Penner and Elwood (2016) has been added. The assessment is for walls with $h/t > 8$ and compares the 1.0-second acceleration parameter against a series of coefficients multiplied together. The coefficients account for the wall aspect ratio, the diaphragm flexibility, the height of the walls in the building, and the axial force on the walls. For the Immediate Occupancy Performance Level, the walls must not experience any overstress in the flexural tension strength of the mortar under out-of-plane loading. Provisions for unreinforced masonry spandrel beams have been added. Equations are provided to determine the shear and flexural capacity of a spandrel beam, which are based whether there is a lintel or a masonry arch supporting the spandrel beam. Shear and flexure can be considered deformation-controlled actions.

Revisions were also made to bed-joint sliding equation requirements for unreinforced masonry shear strength and refined commentary was provided regarding pier height assumptions when there are openings of different height on each side of the pier.

The provisions for steel and concrete frames with masonry infill were completely rewritten and provide an easier method to model masonry as a compression strut within the frame. The panels are classified as strong or weak and flexible or stiff with respect to the frame. There are different modeling parameters and capacities if the frame is nonductile concrete or either ductile concrete or steel framing encased in concrete. The acceptance criteria for the infill panels is based on the ratio of the strength of the frame without the panels to the strength of the infill panels and the aspect ratio of the panels. The criteria for out-of-plane actions when considering arching action have also been updated.

2.4.12 Chapter 12 Wood Light Frame

As noted above, cold-formed steel light frame provisions were moved in ASCE 41-17 from Chapter 12 to the Chapter 9 covering steel. Chapter 12 updates for wood members were relatively minimal with some clarifications provided for reducing diaphragm and shear wall strengths of older, non-conforming systems. Where nominal 2-inch framing is present at panel edges in lieu of 3-inch framing, expected strengths for diaphragms and shear walls should be reduced by 10%-20%.

2.4.13 Chapter 13 Architectural, Mechanical, and Electrical Components

Chapter 13 of ASCE 41-13 covers anchor and bracing of nonstructural components. Revisions in ASCE 41-17 include the Hazards Reduced Nonstructural Performance Level requirements, the addition of testing requirements for existing anchors, and the addition of requirements for rooftop solar photovoltaic arrays.

2.4.14 Chapter 14 Seismic Isolation and Chapter 15 Design Requirements for Structures with Supplemental Energy Dissipation

The evaluation and retrofit procedures for buildings using seismic isolation and energy dissipation devices are contained in Chapter 14 of ASCE 41-13. For the ASCE 41-17 standard, these provisions were split into two separate chapters: Chapter 14 for seismic isolation systems and Chapter 15 for energy dissipation devices (with all subsequent chapters renumbered accordingly). Both chapters included updates that are intended to better align the ASCE 41-13 standard with all the recent efforts and updates for the ASCE 7 standard.

2.4.15 Chapter 16 System-Specific Performance Procedures

The only system-specific performance procedure is the Special Procedure for Unreinforced Masonry, which is used for qualifying unreinforced masonry bearing wall buildings with flexible diaphragms. A number of detailed revisions were made in ASCE 41-17 to make the Special Procedure more compatible with the 2015 International Existing Building Code (ICC, 2015).

2.5 Tips for Using ASCE 41-13

Based on experience with using ASCE 41-06 and ASCE 41-13, the following general advice, tips, and guidance are offered.

- When utilizing ASCE 41-13 for an evaluation or retrofit, it is important to understand the requirements of the Authority Having Jurisdiction, and any special review requirements.
- ASCE 41-13 is not always organized in a sequential way, nor were the provisions holistically developed (with the exception of the Special procedure for Unreinforced Masonry). An evaluation is performed on a component-by-component basis, which often requires jumping between chapters for analysis provisions, component strengths, and acceptance criteria. In the examples of this *Guide*, the starting point in ASCE 41-13 and reference sections related to the next steps are indicated.
- Before following the procedures in the standard, ASCE 41-13 Chapter 1 through Chapter 3 including commentaries should be reviewed.
- It is important to read all associated text and table footnotes in the associated chapter in ASCE 41-13 rather than simply applying the equations. For example, there are many instances where the text and footnotes significantly alter *m*-factors or when certain equations are not applicable.
- ASCE 41-13 uses displacement-based design. Thus, the inelastic response of a building is primarily about deformation compatibility and ductility on a component level. Section 2.2 of this *Guide* discusses this in more detail.
- Understanding component behavior and whether an element is classified as force-controlled or deformation-controlled are essential.
- Obtaining demand-capacity ratios (DCRs) provides an indication of the magnitude and distribution of inelastic demands and is necessary in understanding governing behavior modes for components and systems.
- For nonlinear procedures, reclassification of certain force-controlled actions to deformation-controlled actions is permitted in some cases.

- Boundary conditions in models can make a significant difference in resulting behavior mechanisms and analysis results. Consideration should be given to foundation connections and conditions, as well as soil-structure interaction, when developing models.
- In a two-level evaluation or design, it may be helpful to check component acceptance criteria for one Structural Performance Level and Seismic Hazard Level and then spot compare with the other Structural Performance Levels and Seismic Hazard Levels under consideration to determine if any can be ruled out by inspection using relative mathematical ratios.
- Even though they may appear straightforward, some equations actually require detailed iteration and parallel calculations to complete. The determination of the target displacement is an example. It requires determination of element DCRs.
- When using the nonlinear analysis procedures, it is not necessary to model everything as a nonlinear element—it is time consuming and misleading. It is worthwhile to develop an initial understanding of the likely elements that will experience nonlinear behavior based on comparative strength and only model them as nonlinear elements. Other elements can be modeled as linear elements. The assumptions or calculations can be revised after review of initial results.
- One gravity column-beam bay (with the entire gravity load assigned to it) should be modeled for investigating deflection compatibility checks.
- The application of ASCE 41-13 to light-frame wood construction can be challenging as the methodology requires the determination of the various failure limit states of connections, connection hardware, and the multiple mechanisms in the load path, which are not typically required when designing a new wood structure. Furthermore, ASCE 41-13 requires metal straps and hold-downs to be evaluated as force-controlled actions which require them to remain essentially elastic, whereas for new structures, these components are typically not designed with the overstrength factor, Ω_0, and are permitted to yield and deform. As a result, these components may not satisfy the ASCE 41-13 requirements without significant investigation into other failure mechanisms in the load path that may further reduce the demand to these components.
- ASCE 41-13 has limitations on the use of linear procedures and the Nonlinear Static Procedure that depend on the extent of nonlinearity, building irregularities, and higher mode effects. However, typically, it is not possible to determine in advance if the limitation applies, and significant analysis is needed to evaluate the limitation requirements.

Chapter 3
Performance Objectives and Seismic Hazards

3.1 Overview

This chapter provides discussion and example applications for a set of specific topics that affect a number of different building types. Topics are taken from ASCE 41-13 Chapters 2, 3, and 6 (ASCE, 2014). These include selecting Performance Objectives and Target Building Performance Levels, developing appropriate seismic demands and Levels of Seismicity, and addressing data collection requirements and the associated knowledge factor.

3.2 Performance Objectives and Target Building Performance Levels (ASCE 41-13 § 2.2 and § 2.3)

3.2.1 Introduction

The criteria of the building evaluation or retrofit are defined by the owner, engineer, and/or regulator in terms of Performance Objectives. A Performance Objective is defined by an expected damage state (Building Performance Level) for a given shaking intensity (Seismic Hazard Level). In essence, by choosing a Performance Objective, one defines how much damage is acceptable at a given intensity of ground shaking. A building retrofit can target more than one Performance Objective. For instance, one might design a retrofit such that only minor damage would be expected in moderate ground shaking, but collapse would be prevented in a rare event. This section provides a brief discussion on the most commonly used Performance Objectives; subsequent sections provide guidance on defining the Seismic Hazard Level and associated Building Performance Levels.

ASCE 41-13 is written for very broad application so that the user has many options for choosing a Seismic Hazard Level and target Building Performance Level to define a Performance Objective. However, ASCE 41-13 evaluations and retrofits are typically based on predefined hazard levels. For most buildings and sites, these hazard levels are defined using the General Procedure as described in ASCE 41-13 § 2.4.1 as follows:

- BSE-1E: 20%/50 year hazard (capped at BSE-1N)
- BSE-2E: 5%/50 year hazard (capped at BSE-2N)

- BSE-1N: matches ASCE 7-10 Design Earthquake
- BSE-2N: matches ASCE 7-10 Risk Targeted Maximum Considered Earthquake (MCE_R)

In a manner very similar to ASCE 7, the General Procedure in ASCE 41-13 provides a method for determining the short period (0.2-second) and 1-second spectral accelerations, and prescribes formulae to define the full design spectrum based on those two values and site soil conditions. Online seismic hazard maps and other tools make it feasible for the structural engineer to readily define the design spectra associated with these predefined hazards. In unusual circumstances, one might choose a Seismic Hazard Level other than those predefined (BSE-1E, BSE-2E, BSE-1N, and BSE-2N) to define a Performance Objective. This will require some knowledge of seismic hazard functions and their various adjustments, and the requirements for the Site-Specific Procedure of ASCE 41-13 § 2.4.2 might prove helpful.

The ASCE 41-13 commentary has explicitly defined 16 discrete Performance Objectives ("a" through "p" in ASCE 41-13 Table C2-2). Four primary building Performance Objectives are defined as combinations of these discrete objectives, and include the Basic Performance Objective for Existing Buildings (BPOE, ASCE 41-13 § 2.2.1), Enhanced Performance Objective (ASCE 41-13 §2.2.2), Basic Performance Objective Equivalent to New Building Standards (BPON, ASCE 41-13 § 2.2.4), and Limited Performance Objectives (ASCE 41-13 § 2.2.3). These Performance Objectives apply to structural as well as nonstructural elements.

3.2.2 Basic Performance Objective for Existing Buildings (BPOE) (ASCE 41-13 § 2.2.1)

Traditionally, existing buildings have been evaluated or retrofit to lower seismic demands than those used for the design of new buildings. One approach has been to require retrofits to be designed to 75% of the base shear for new buildings. Higher risks are accepted to avoid immediately rendering buildings deficient with code changes, to address the shorter remaining design life associated with existing buildings, and to recognize that the cost to retrofit can be much higher than the associated cost to improve performance at the time of new design. The BPOE for Risk Category I and II structures is defined in ASCE 41-13 as the combination of two discrete Performance Objectives: (1) Life Safety Performance Level at a Seismic Hazard Level defined by 20% probability of exceedance in 50 years; and (2) Collapse Prevention Performance Level at a Seismic Hazard Level defined by 5% probability of exceedance in 50 years. The BPOE is intended to replicate the reduced performance criteria traditionally allowed for existing buildings compared to new construction. Tier 1 and Tier 2 evaluations are based on the BPOE and cannot be done using the BPON, while the BPOE can also be used for Tier 3.

The BPOE is the most commonly used Performance Objective, and is applicable to all building types. Alternate Performance Objectives might be chosen in those instances where higher performance is desired, or where a lower performance is tolerable based on available retrofit resources.

3.2.3 Enhanced Performance Objective (ASCE 41-13 § 2.2.2)

Enhanced Performance Objectives are those that exceed the performance associated with the BPOE. This can be achieved by:

- Using higher target Structural and/or Nonstructural Performance Levels than specified for the BPOE Seismic Hazard Level given the building's Risk Category
- Using a higher Seismic Hazard Level than the BSE-1E and/or the BSE-2E Seismic Hazard Levels, given the building's Risk Category
- Using a higher Risk Category than the building would normally be assigned to, in order to select target Structural and/or Nonstructural Performance Levels

The use of an Enhanced Performance Objective is not common, but it could be used for buildings for which better seismic performance is desired, such as police and fire stations, communication centers, hospitals, schools, or where the building owner intends to keep the building operational during or soon after a significant seismic event.

3.2.4 Limited Performance Objective (ASCE 41-13 § 2.2.3)

Limited Performance Objectives are those that fall below the performance associated with BPOE. Limited Performance Objective might include voluntary retrofits where resources are limited or mandatory retrofits intended to correct known deficiencies. There are two types: a Reduced Performance Objective or a Partial Retrofit Objective.

A building retrofit with a Reduced Performance Objective is comprehensive and addresses the entire building, but uses a lower Seismic Hazard Level or a lower target Building Performance Level than required by the BPOE. For example, the Special Procedure for unreinforced masonry bearing wall buildings in ASCE 41-13 § 15.2 uses a Limited Performance Objective because it only has a one-level check that, per the commentary of ASCE 4-13 § 15.2.1, is assumed to achieve the equivalent of Collapse Prevention

Performance Level at the BSE-1E Seismic Hazard Level. Chapter 12 of this *Guide* provides an example.

A Partial Retrofit Objective is used when only a subset of the building's deficiencies are to be addressed, usually focusing on a key vulnerability (e.g., a weak story, tilt-up walls anchored with ledgers loaded in cross grain bending, or pre-Northridge moment frame connections). The key vulnerability might come to light following a Tier 1 or Tier 2 evaluation, or might be identified by a jurisdiction for mandatory retrofit.

3.2.5 Target Building Performance Levels (ASCE 41-13 § 2.3)

Key Terms

Structural Performance Levels:

S-1: Immediate Occupancy

S-2: Damage Control

S-3: Life Safety

S-4: Reduced Safety

S-5: Collapse Prevention

Nonstructural Performance Levels:

N-A: Operational

N-B: Position Retention

N-C: Life Safety

N-D: Not Considered

Target Building Performance Level refers to a combination of the intended performance levels of structural and nonstructural components, and is designated by a number to indicate the structural performance and a letter to indicate the nonstructural performance. Structural Performance Levels range from S-1 to S-5, corresponding to Immediate Occupancy, Damage Control, Life Safety, Reduced Safety, and Collapse Prevention, respectively. Nonstructural Performance Levels range from N-A to N-C for Operational, Position Retention, and Life Safety, respectively, or to N-D if nonstructural performance is not considered. Thus, a target Building Performance Level for both structural and nonstructural life safety would be designated (3-C), while a target of Immediate Occupancy and Operational would be designated (1-A). The focus is on the post-earthquake disposition of the building, the ability to resume normal functions, and the ability to protect life safety of the occupants. ASCE 41-13 Tables C2-4 through C2-7 provide illustrative examples of what the damage (or lack thereof) to structural and nonstructural components might be at each of the intended Performance Levels. It is important to appreciate that, as noted in ASCE 41-13, because of inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected. Compliance with the ASCE 41-13 standard should not be considered a guarantee of performance. Information on the reliability of achieving various performance levels can be found in Chapter 2 of FEMA 274 (FEMA, 1997b).

Some considerations in picking a target Building Performance Level for a building include life expectancy, criticality of its function to the owner or community, and the type and/or number of occupants. Anchorage of cladding and veneer along with partition walls are examples where the target Nonstructural Performance Level can significantly affect the operation of the building and life safety of the occupants as they exit the building. Hospitals, emergency communication centers, fire stations, and police stations would be examples where a target Operational Nonstructural Performance Level would often be considered.

3.3 Seismic Hazard (ASCE 41-13 § 2.4)

In most cases, the Seismic Hazard Level will be determined using the General Procedure described in ASCE 41-13 § 2.4.1. In this *Guide*, the calculation of seismic hazard using the General Procedure will first be demonstrated via hand calculations, and then verified using available online tools. Very significant structures, local ground hazards, or the need to do response history analyses might warrant development of site-specific ground motion hazards of ASCE 41-13 § 2.4.2. Application of ASCE 41-13 § 2.4.2 is typically done by (sub)consultants with specialized experience and training, and is outside the scope of this *Guide*. However, the structural engineer should carefully review any site-specific hazard study prepared by an outside consultant, and Section 3.3.4 of this *Guide* discusses some of the important aspects of that review.

It is often necessary to determine more than one hazard level for a project. For instance, for Tier 3, the BPOE requires Life Safety checks at the BSE-1E Seismic Hazard Level and Collapse Prevention checks at the BSE-2E Seismic Hazard Level. In addition, if a Tier 1 screening is to be performed, then selection of the appropriate checklist from ASCE 41-13 Appendix C requires that the Seismic Hazard Level (Very Low, Low, Moderate, or High) be defined per ASCE 41-13 § 2.5 based on BSE-1N, and therefore the spectral accelerations associated with that hazard would also need to be determined. Also, ASCE 41-13 § 4.1.2 prescribes BSE-1E as the Seismic Hazard Level for Tier 1 screening and Quick Checks (ASCE 41-13 § 4.5.3), and therefore the spectral accelerations associated with that hazard would need to be determined if Tier 1 Quick Checks are part of the evaluation.

3.3.1 Example of the General Procedure for Hazard Caused by Ground Shaking (ASCE 41-13 § 2.4.1)

This example demonstrates how to calculate the parameters S_{XS} and S_{X1} using the General Procedure described in ASCE 41-13 § 2.4.1. These parameters will need to be determined for both the BSE-1E and BSE-2E Seismic Hazard Levels. The USGS-mapped short period and 1-second rock accelerations for each Seismic Hazard Level are obtained from online sources available at the time of this writing, such as the ATC Site-Specific Hazard Map website (available at: https://hazards.atcouncil.org) and the SEAOC Seismic Design Map website (available at: https://www.seismicmaps.org/). Next, these values are adjusted for Site Class and checked to verify that they do not exceed the associated BSE-1N and BSE-2N values. All of the spectra of this example are associated with 5% critical damping.

This example presumes the building is located in San Jose, California (33.3306°N, 121.87636°W) in an area classified as Site Class D. The values calculated in this section are utilized in the Chapter 7 example of a wood tuck-under structure.

BSE-2E: 5%/50-Year Spectral Response Acceleration (ASCE 41 § 2.4.1.3), Capped at BSE-2N

> **Commentary**
>
> Some online tools provide values in terms of the gravitational constant, g, for spectral acceleration parameters. ASCE 41-13 and ASCE 7-10 do not apply the term g to spectral acceleration parameters. The ASCE 41-13 approach is used in this document.

Design short-period spectral response acceleration parameter, $S_{XS,BSE\text{-}2E}$, is determined as $F_{a,5/50} \times S_{S,5/50}$, but need not be greater than $S_{XS,BSE\text{-}2N}$.

The short-period spectral response acceleration, S_S, is determined through an online tool as:

$S_{S,5/50}$ = 1.588

$S_{S,BSE\text{-}2N}$ = 1.50

Note that $S_{S,5/50}$ exceeds $S_{S,BSE\text{-}2N}$, which is nominally based on a more rare (2%/50) hazard. This is because $S_{S,BSE\text{-}2N}$ is the ASCE 7-10 risk-adjusted MCE value, which in some areas is limited by a deterministic value, while $S_{S,5/50}$ is from an unadjusted uniform hazard spectrum. ASCE 41-13 eliminates the possibility of the BSE-2E Seismic Hazard Level exceeding the corresponding BSE-2N Seismic Hazard Level by capping the BSE-2E Seismic Hazard Level.

The factor to adjust the short-period spectral response acceleration for site class, F_a, is determined per ASCE 41-13 § 2.4.1.6:

$F_{a,5/50}$ = 1.0 for $S_s > 1.25$, Site Class D (ASCE 41-13 Table 2-3)

$F_{a,BSE\text{-}2N}$ = 1.0 (ASCE 41-13 Table 2-3)

$S_{XS,BSE\text{-}2N} = F_{a,BSE\text{-}2N} \times S_{S,BSE\text{-}2N} = 1.0 \times 1.50 = 1.50$

$S_{XS,BSE\text{-}2E} = F_{a,5/50} \times S_{S,5/50} = 1.0 \times 1.588 = 1.588$

Since $S_{XS,BSE\text{-}2E} > S_{XS,BSE\text{-}2N}$, the BSE-2E Seismic Hazard Level at this site is capped by the BSE-2N Seismic Hazard Level.

Therefore, $S_{XS,BSE\text{-}2E} = 1.50$

$S_{X1,BSE\text{-}2E} = F_{v,5/50}\, S_{1,5/50}$, but need not be greater than $S_{X1,BSE\text{-}2N}$

The mapped 1-second period spectral response acceleration, S_1, is determined as:

$S_{1,5/50}$ = 0.555

$S_{1,BSE\text{-}2N}$ = 0.60

The factor to adjust the 1-second period spectral response acceleration for site class, F_v, is determined per ASCE 41 § 2.4.1.6:

$F_{v,5/50}$ = 1.5 for $S_1 > 0.5$, Site Class D (ASCE 41-13 Table 2-4)

$F_{v,BSE\text{-}2N}$ = 1.5 (ASCE 41-13 Table 2-4)

$S_{X1,BSE\text{-}2N} = F_{v,BSE\text{-}2N} \times S_{1,BSE\text{-}2N} = 1.50 \times 0.60 = 0.90$

$S_{X1,BSE\text{-}2E} = F_{v,5/50} \times S_{1,5/50} = 1.50 \times .555 = 0.832$

Since $S_{X1,BSE\text{-}2E} < S_{X1,BSE\text{-}2N}$, the BSE-2E Seismic Hazard Level at this site is *not* capped by the BSE-2N Seismic Hazard Level.

Therefore, $S_{X1,BSE\text{-}2E} = 0.832$

BSE-1E: 20%/50-Year Spectral Response Acceleration (ASCE 41 § 2.4.1.4), Capped at BSE-1N

$S_{XS,BSE\text{-}1E} = F_a \times S_{S,20/50}$, but need not be greater than $S_{XS,BSE\text{-}1N}$

The mapped short-period spectral response acceleration, S_S, is determined as:

$S_{S,20/50} = 0.978$

The factor to adjust the short-period spectral response acceleration for site class, F_a, is determined per ASCE 41-13 § 2.4.1.6 for Site Class D. Since the mapped S_S is between the S_S values in the columns in Table 2-3 of ASCE 41-13, linear interpolation is necessary.

$F_{a,20/50} = 1.1 + 0.1 \times ((1.0 - 0.978) / (1.0 - 0.75)) = 1.109$

The BSE-1N short-period spectral response acceleration is calculated per ASCE 41 § 2.4.1.2.

$S_{XS,BSE\text{-}1N} = 2/3\ S_{XS,BSE\text{-}2N} = (2/3)(1.5) = 1.0$

$S_{XS,BSE\text{-}1E} = F_{a,20/50} \times S_{S,20/50} = 1.109 \times 0.987 = 1.084$

Since $S_{XS,BSE\text{-}1E} > S_{XS,BSE\text{-}1N}$, the BSE-1E Seismic Hazard Level at this site is capped by the BSE-1N Seismic Hazard Level.

Therefore, $S_{XS,BSE\text{-}1E} = 1.0$

$S_{X1,BSE\text{-}1E} = F_v \times S_{1,20/50}$, but need not be greater than $S_{X1,BSE\text{-}1N}$

The mapped 1-second period spectral response acceleration, S_1, is determined as:

$S_{1,20/50} = 0.327$

The factor to adjust the 1-second period spectral response acceleration for site class, F_v, is determined per ASCE 41-13 § 2.4.1.6 for Site Class D.

Since the mapped S_1 is between the S_1 values in the columns in ASCE 41-13 Table 2-4, linear interpolation is necessary.

$$F_{v,20/50} = 1.6 + 0.2 \times ((0.4 - 0.327) / (0.4 - 0.3)) = 1.746$$

The BSE-1N 1-second period spectral response acceleration is calculated per ASCE 41-13 § 2.4.1.2.

$$S_{X1,BSE\text{-}1N} = 2/3\ S_{X1,BSE\text{-}2N} = (2/3)(0.90) = 0.60$$

$$S_{X1,BSE\text{-}1E} = F_{v,20/50} \times S_{1,20/50} = 1.746 \times 0.237 = 0.571$$

Since $S_{X1,BSE\text{-}1} < S_{X1,BSE\text{-}1N}$, the BSE-1E Seismic Hazard Level at this site is *not* capped by the BSE-1N Seismic Hazard Level.

Therefore, $S_{X1,BSE\text{-}1E} = 0.571$

A summary of the spectral response acceleration parameters for this site is provided in Table 3-1. The values for T_S are from ASCE 41-13 Equation 2-9, which is calculated as $T_S = S_{S1}/S_{XS}$. The values for T_0 are from ASCE 41-13 Equation 2-10, which is $T_0 = 0.2T_S$.

Table 3-1 Spectral Accelerations for Site in San Jose, CA, Site Class D

	ASCE 41-13 Section 2.4.1 Spectral Ordinates				Uncapped BSE-2E & 1E	
Value	BSE-2N	BSE-1N	BSE-2E	BSE-1E	5% in 50yr	20% in 50yr
S_{XS} (g)	1.50	1.00	1.50	1.00	1.59	1.08
S_{X1} (g)	0.90	0.60	0.83	0.57	0.83	0.57
T_0 (sec)	0.12	0.12	0.11	0.11	0.10	0.11
T_s(sec)	0.60	0.60	0.56	0.57	0.52	0.53

Design spectra are constructed using the S_{XS} and S_{X1} values as described in ASCE 41-13 § 2.4.1.7. It is often informative to show all ASCE 41-13 design spectra on a single plot. As further discussed in the following section, the differences between design base shears corresponding to each of the ASCE 41-13 Seismic Hazard Levels could be a factor in selecting the most appropriate Performance Objective. In addition, more than one spectrum might be required for a full investigation. The BSE-1E spectra would be used, for instance, to calculate the Tier 1 pseudo seismic forces in ASCE 41-13 § 4.5.2.1. If using the Tier 1 checklists, one would also query the BSE-1N spectral accelerations and determine the Level of Seismicity per ASCE 41-13 § 2.5.

The response spectra associated with the various Seismic Hazard Levels considered in ASCE 41-13 are shown for the San Jose example site in Figure 3-1. The thin black solid line is the BSE-2N spectrum, and the thin dash-dot line is the 5% in 50-year spectrum. By definition, the BSE-2E spectrum follows the 5%/50 spectrum as seen in the longer periods. However, since the 5%/50 spectrum exceeds the BSE-2N spectrum in the short period range, the BSE-2E spectrum is capped at the BSE-2N Seismic Hazard Level, as called-out in the figure with a red circle. A similar capping of the BSE-1E spectrum is seen in the lower set of curves.

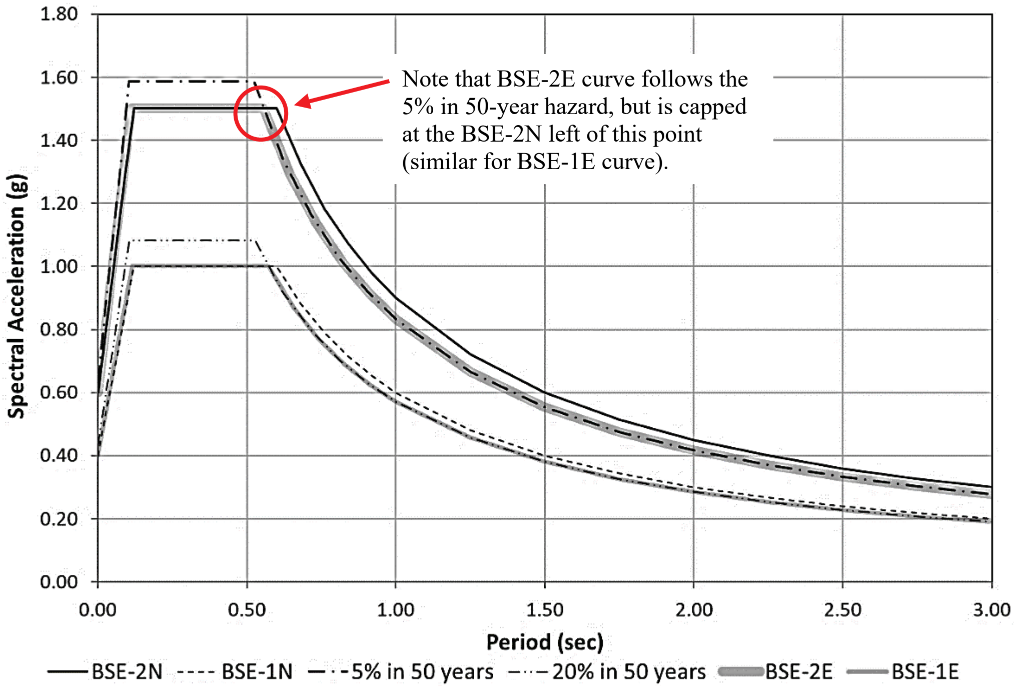


Figure 3-1 Input horizontal spectra for a San Jose, California site.

3.3.2 Seismic Design Spectra Web Tools

Online tools available from ATC (https://hazards.atcouncil.org) and SEAOC (https://www.seismicmaps.org/) eliminate much of the effort (and potential for error) associated with defining seismic hazards for the conterminous United States, Alaska, Hawaii, Puerto Rico and the U.S. Virgin Islands.

This section demonstrates how to use these tools to find the BSE-2E spectral acceleration values for the same San Jose, California location shown in the previous section.

The following information must be provided by the user using the screen input options:

- Design Standard or Reference: ASCE 41-13
- Seismic Hazard Level: Select from BSE-1E, BSE-2E, BSE-1N, and BSE-2N. Some tools allow for selection of a "custom" hazard level

- Soil Site Classification: Select from A, B, C, D or E per ASCE 41-13 § 2.4.1.6.1
- Latitude and longitude or site address (entered in the search box on the map)

With these values defined, the tool "looks up" the spectral accelerations based on the USGS mapping for rock sites, and makes the necessary adjustments for deterministic caps, site soils, and design scale factors. However, at the time of writing, the ATC and SEAOC tools do not cap the BSE-1E and BSE-2E values at BSE-1N and BSE-2N values, respectively, and this needs to be done manually after comparing the values returned by the online tools. Both tools perform the calculations specified in ASCE 41-13 § 2.4.1 required to completely define the design input spectra for use in ASCE 41-13 evaluation or retrofit. If the tool presents the capability to generate a detailed report documenting each step described in ASCE 41-13 § 2.4.1, this is the recommended version for inclusion in the project file.

Figure 3-2 and Figure 3-3 present summary reports and BSE-2E acceleration spectra generated by the ATC and SEAOC tools, respectively, for the Site Class D example in San Jose, California.

3.3.3 Comparison of BSE-1E, BSE-2E, and ASCE 7-10 Design Levels

Traditionally, design loads for seismic retrofit have been prescribed as some portion (i.e., 75%) of the design loading for a new structure of similar type and occupancy. In an effort to provide a probabilistic basis for the design loading, ASCE 41-13 has incorporated risk-targeted hazard mapping of ASCE 7-10 for the BSE-2N/1N hazards, and uniform seismic hazard mapping for the BSE-2E/1E hazards. As a result, the ratio of new construction to retrofit loading can vary significantly with seismic region.

Figures 3-4 and Figure 3-5 compare the spectral accelerations associated with BPOE and BPON Performance Objectives in several cities throughout the United States. In Central and Eastern United States, where earthquakes are less frequent, the retrofit design spectral accelerations for ASCE 41-13 BSE-1E and BSE-2E can be significantly lower than those required for BSE-2N or BSE-1N (i.e., by ASCE 7-10). ASCE 41-13 users should be aware that loads associated with BSE-1E and BSE-2E can be well below those traditionally associated with evaluation or retrofit of existing buildings.

ATC Hazards by Location

Search Information

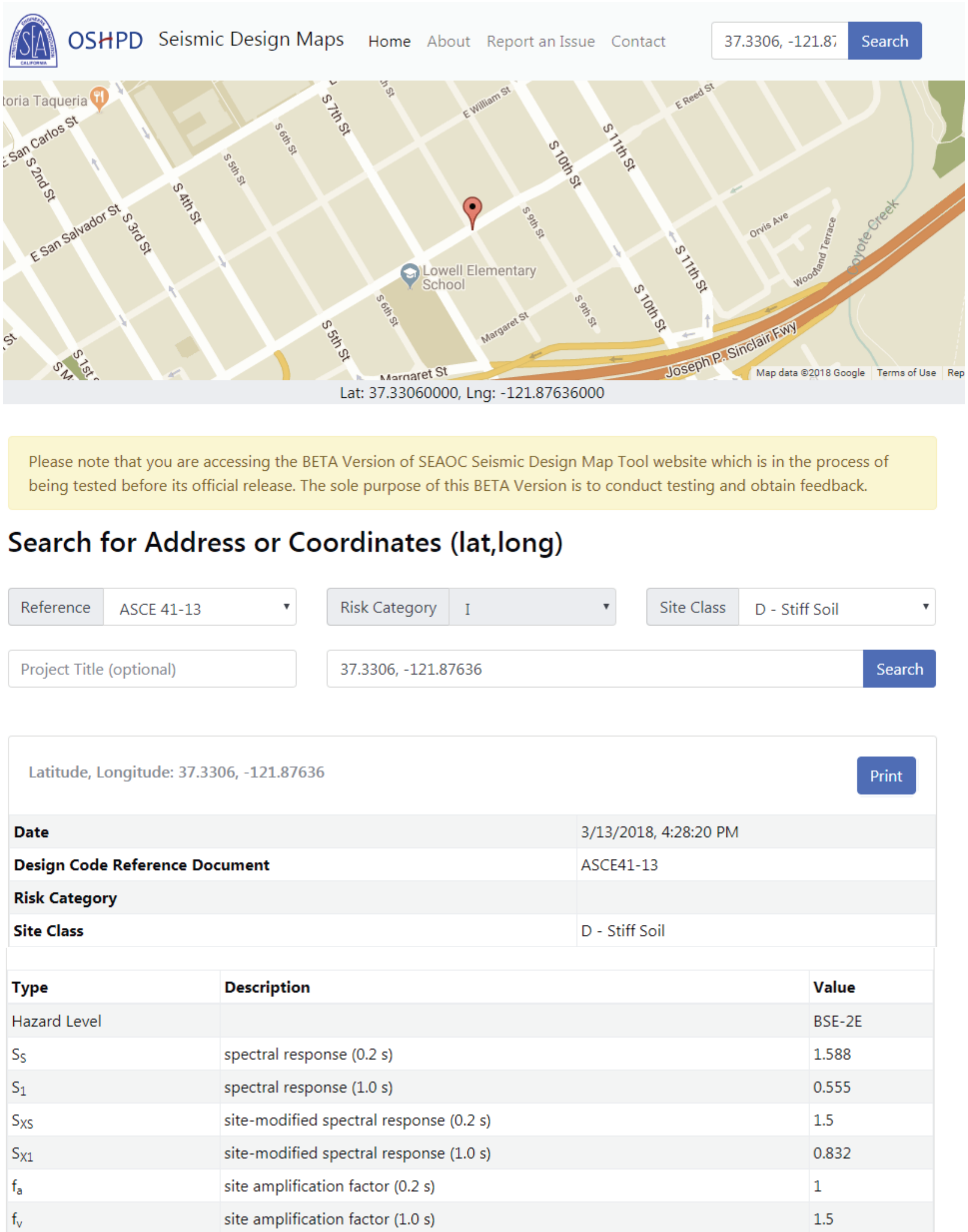
Coordinates:	37.330637, -121.876309
Timestamp:	2018-05-29T00:18:38.518Z
Hazard Type:	Seismic
Reference Document:	ASCE41-13
Site Class:	D
Report Title:	Not specified

Map Results

Hazard Level BSE-2E

Name	Value	Description
S_S	1.588	MCE_R ground motion (period=0.2s)
F_a	1	Site amplification factor at 0.2s
S_{XS}	1.5	Site modified spectral response (0.2s)
S_1	0.555	MCE_R ground motion (period=1.0s)
F_v	1.5	Site amplification factor at 1.0s
S_{X1}	0.832	Site modified spectral response (1.0s)

Figure 3-2 Summary report from ATC Hazards by Location website for site in San Jose, California.



Please note that you are accessing the BETA Version of SEAOC Seismic Design Map Tool website which is in the process of being tested before its official release. The sole purpose of this BETA Version is to conduct testing and obtain feedback.

Search for Address or Coordinates (lat,long)

Reference: ASCE 41-13 | Risk Category: I | Site Class: D - Stiff Soil

Project Title (optional) | 37.3306, -121.87636 | Search

Latitude, Longitude: 37.3306, -121.87636

Print

Date	3/13/2018, 4:28:20 PM
Design Code Reference Document	ASCE41-13
Risk Category	
Site Class	D - Stiff Soil

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	1.588
S_1	spectral response (1.0 s)	0.555
S_{XS}	site-modified spectral response (0.2 s)	1.5
S_{X1}	site-modified spectral response (1.0 s)	0.832
f_a	site amplification factor (0.2 s)	1
f_v	site amplification factor (1.0 s)	1.5

Figure 3-3 Summary report from SEAOC Seismic Design Maps website for site in San Jose, California.

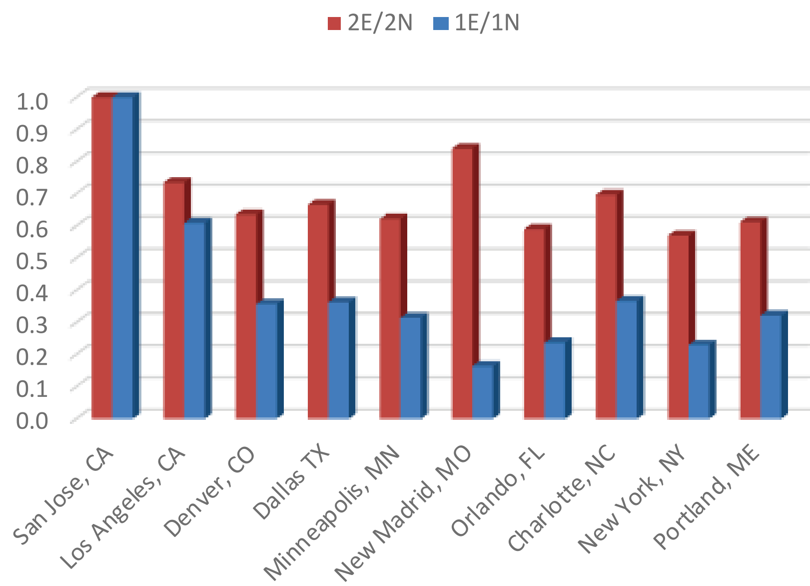


Figure 3-4 Ratios of BSE-2E to BSE-2N and BSE-1E to BSE-1N for short period spectral acceleration at various cities assuming Site Class D.

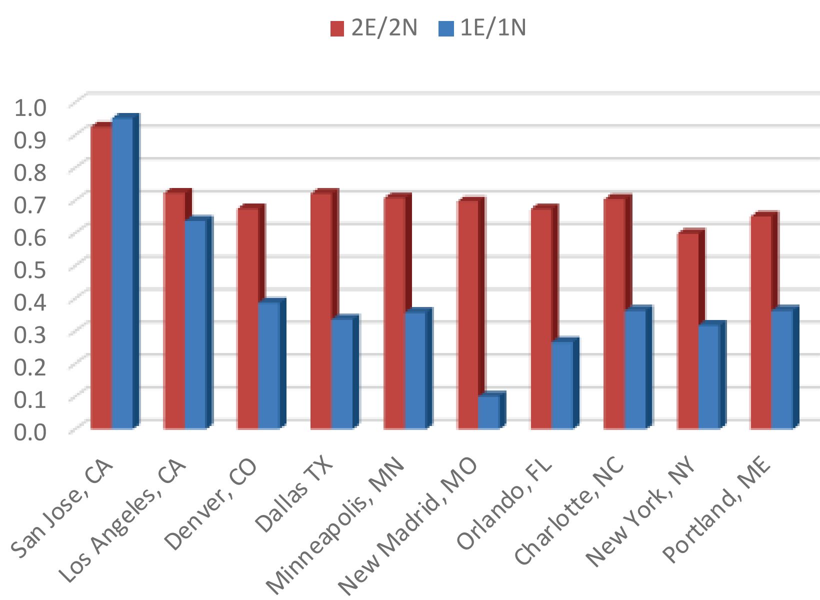


Figure 3-5 Ratios of BSE-2E to BSE-2N and BSE-1E to BSE-1N for 1-second spectral acceleration at various cities assuming Site Class D.

ASCE 41-17 Revision

Revisions have been made to the Tier 1 and Tier 2 procedures in ASCE 41-17 to address concerns of some professionals that the ratio of the BSE-1E to BSE-1N was so small that evaluations conducted only using the BSE-1E Seismic Hazard Level would be significantly less conservative than those done using ASCE 31-03 and not provide the commensurate performance at the BSE-2E Seismic Hazard Level as the BPOE indicates.

For example, in New Madrid, Missouri, for Site Class D and short period spectral acceleration, where the BSE-1E to BSE-1N ratio is 0.16, the BSE-2E to BSE-1E ratio is nearly 8, while the ratio of Collapse Prevention to Life Safety *m*-factors is 1.3. This is addressed in ASCE 41-17 by requiring the use of the BSE-2E Seismic Hazard Level for the BPOE in the Tier 1 and Tier 2 procedures.

3.3.4 Site-Specific Procedure for Hazards Caused by Ground Shaking (ASCE 41-13 § 2.4.2)

ASCE 41-13 requires site-specific hazard procedures for Site Class F and near-fault Site Class E. However, for very significant structures, where there is some concern about local hazards, or when acceleration time histories are needed for nonlinear dynamic analysis, a more comprehensive site-specific hazard determination of ASCE 41 § 2.4.2 may be appropriate. The Site-Specific Procedure can be used to either define the design response spectra, or, if nonlinear dynamic analysis is contemplated, to select and scale acceleration time histories. Site-specific hazard definition requires specialized expertise in seismic source characterization, attenuation relationships, and site soil effects. It is not typically done by the design structural engineer but by a consultant with the appropriate training and experience. Because of its potential complexity, site-specific probabilistic hazard analyses are often subject to formal peer review. This section provides a discussion of some of the important aspects of a site-specific hazard assessment of which the structural engineer should be aware when reviewing reports from specialty consultants.

Detailed information on the selection and scaling of response histories is contained in NIST GCR 11-917-15 report, *Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses* (NIST, 2011). For advice and guidance on performing nonlinear analyses, NIST GCR 10-917-5 report, *NEHRP Seismic Design Technical Brief No. 4: Nonlinear Structural Analysis for Seismic Design: A Guide for Practicing Engineers* (NIST, 2010), is a valuable resource. In addition, NIST 12-917-21, *Soil-Structure Interaction for Building Structures* (NIST, 2012), provides useful checklists for the information that the structural engineer may need to be provided from the specialty consultant and/or the geotechnical engineer for various analytical procedures, as well as the information the structural engineer should provide to the specialty consultant and/or the geotechnical engineer.

3.3.4.1 Ground Motion Scaling

ASCE 41-17 Revision

The ground motion scaling and selection requirements have been extensively modified in ASCE 41-17. The new standard essentially adopts the requirements of ASCE 7-16 with some modifications reflecting application to existing buildings.

A seismic response history analysis is performed by subjecting a computer building model to earthquake shaking, which is represented by ground motion records consisting of two horizontal components (plus vertical, if it is being considered). Ground motion acceleration time histories used for the structural analysis should be selected to have source mechanisms (fault types), magnitudes, source-to-site distances, and local soil conditions that are consistent with those that control the seismic hazard at the subject site. Although the allowable range of magnitudes, distances, and site conditions can be somewhat relaxed so that a sufficient number of ground motions with appropriate spectral shapes are available, in order to simulate large earthquakes that will cause severe damage to buildings, the recorded ground motions will often need to be scaled up in intensity.

Simply scaling the motion amplitude to match/exceed design spectral acceleration may not address other important ground motion characteristics. For instance, strong motions generated by large magnitude events will create motions with different spectral shapes and longer durations. Ground motion scaling must be done with care to generate motions with characteristics representative of the actual demands. It is typical to select motions such that the necessary scale factor is limited, and an allowable scale factor range of approximately 0.25 to 4 is common (ASCE 7-16 Section C16.2); scale factors outside this range might be appropriate for the specific circumstances, but should be carefully considered.

3.3.4.2 Maximum Motions from a Single Event

Seismic events produce ground motions with unique signatures, and taking too many representative ground motions from a single event might bias the design suite. According to ASCE 41-13 § 2.4.2.2, the minimum number of events used to sample ground motions is three. If the specialty consultant has used multiple motions from a single event, the selection criteria should be discussed with the consultant.

3.3.4.3 Minimum Number of Records

The minimum number of ground motion sets (two horizontal, one vertical if considered) is three as specified in ASCE 41-13 § 2.4.2.2. ASCE 41-13 Table 7-1 provides more detail, and establishes requirements that depend on the proximity of the building to the fault, whether the mean or maximum of the results from the suite of records is needed, and what Performance Objective is used. The table presents how many records will be used, and whether rotation of the records will be used. For example, for a near-field site (one that is at 5 km or less from the fault) being evaluated or designed to BPON, seven or more horizontal pairs are required when the average of results is used or between three and six horizontal pairs are required if the maximum values are used. In this case, rotation of the horizontal components is required, so the number of runs doubles.

ASCE 41-17 Revision

One major change in ASCE 41-17 is an increase in the minimum number of ground motion suites from 3 to 11. The associated acceptance criteria are simplified, and ASCE 41-13 Table 7-1 has been eliminated.

The designer and specialty consultant should discuss the minimum number of records required by ASCE 41-13 for the approach that is planned and whether additional confidence that can be obtained by running additional records is desired.

3.3.4.4 Near-Fault Building Sites

As the name implies, near-fault sites are located in close proximity to the causative fault for an earthquake, as measured by distance from site to source. In the near-fault region, it has been recognized that ground motions sometimes exhibit a large pulse near the beginning of the record. These types of ground motions can have significantly different characteristics than those recorded at larger distances, and may induce large displacement and strength demands in structures that increase the risk of earthquake-induced collapse. The design engineer should ask the specialty consultant whether the building location qualifies as near-fault, i.e., whether deaggregation indicates controlling earthquakes are within a certain threshold distance, and how the suite of motions was selected. If the building location qualifies as near-fault, ground motions that contain pulse-like and forward directivity effects should be included in the suite and rotated accordingly.

3.4 Levels of Seismicity

Useful Tip

The Levels of Seismicity in ASCE 41-13 have been adjusted to match the Seismic Design Categories in ASCE 7-10.

The Level of Seismicity used in ASCE 41-13 § 2.5 had been adjusted to match ASCE 7-10. The descriptors are very general and intended to reflect the degree of seismic hazard; SDC A: Very Low, SDC B: Low, SDC C: Moderate, and SDC D-F: High. The Level of Seismicity must be determined to define limits of applicability of Tier 1 and Tier 2 procedures (ASCE 41-13 § 3.3.1.1), to define the force delivery reduction factor, J (ASCE 41-13 § 7.5.2.1.2), to select the appropriate Tier 1 and Tier 2 checklist(s) (ASCE 41-13 § 4.4), and to determine the evaluation and mitigation requirements for nonstructural components (ASCE 41-13 § 13.2). The Level of Seismicity is completely defined by S_{DS} and S_{D1} associated with the BSE-2N Seismic Hazard.

The following is an example of how to determine the Level of Seismicity for a building located at the San Jose, California site. The results from the ATC online tool are shown in Figure 3-6 for BSE-2N. It is noted that ASCE Hazards Tool (https://asce7hazardtool.online/) developed for ASCE 7 also returns BSE-2N values.

- Seismic and Site Data:
  - Location: San Jose, California
  - Site Class: D
  - Risk Category: II (office building)

Spectral Response Acceleration Parameters

Determine S_{DS} and S_{D1}:

$S_{DS} = (2/3)F_a S_{S,\text{ BSE-2N}}$ (ASCE 41-13 Eq. 2-12)

$S_{S,\text{ BSE-2N}} = 1.5$ from Figure 3-6

$F_a = 1.0$ with $S_{S,BSE\text{-}2N} \geq 1.25$ (ASCE 41-13 § Table 2-3)

$S_{D1} = (2/3)\ F_v S_{1,\ \mathrm{BSE\text{-}2N}}$ (ASCE 41-13 Eq. 2-13)

$S_{1,BSE\text{-}2N} = 0.6$ from Figure 3-6

$F_v = 1.5$ with $S_{1,\ \mathrm{BSE\text{-}2N}} \geq 0.50$ (ASCE 41-13 § Table 2-4)

$S_{DS} = 2/3(1.0)(1.5) = 1.0 \geq 0.50$

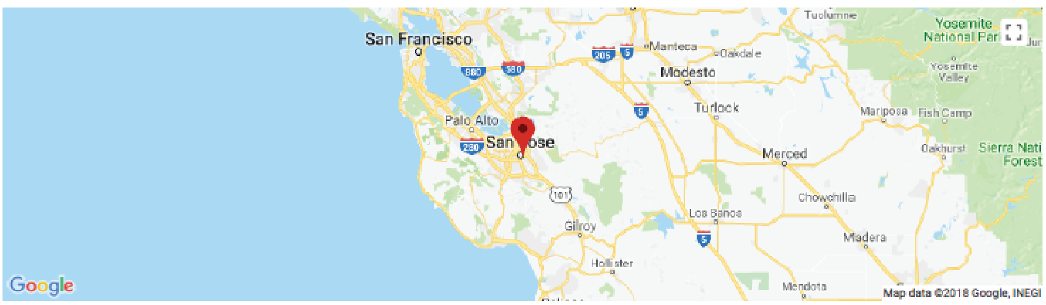
$S_{D1} = 2/3(1.5)(0.6) = 0.6 \geq 0.20$

ATC Hazards by Location

Search Information

Coordinates:	37.330637, -121.876309
Timestamp:	2018-05-29T00:18:38.518Z
Hazard Type:	Seismic
Reference Document:	ASCE41-13
Site Class:	D
Report Title:	Not specified

Map Results



Text Results

Hazard Level BSE-2N

Name	Value	Description
SsUH	2.01	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
CRS	1.13	Coefficient of risk (0.2s)
SsRT	2.271	Probabilistic risk-targeted ground motion (0.2s)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S_S	1.5	MCE_R ground motion (period=0.2s)
F_a	1	Site amplification factor at 0.2s
S_{XS}	1.5	Site modified spectral response (0.2s)
S1UH	0.722	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
CR_1	1.082	Coefficient of risk (1.0s)
S1RT	0.782	Probabilistic risk-targeted ground motion (1.0s)
S1D	0.6	Factored deterministic acceleration value (1.0s)
S_1	0.6	MCE_R ground motion (period=1.0s)
F_v	1.5	Site amplification factor at 1.0s
S_{X1}	0.9	Site modified spectral response (1.0s)

Figure 3-6 Summary report from the ATC Hazards by Location website showing BSE-2N results for site in San Jose, California.

Per ASCE 41-13 § Table 2-5, either S_{DS} greater than 0.50g or S_{D1} greater than 0.20g would define the Level of Seismicity for this office building in San Jose, California as High.

3.5 Data Collection Requirements

Prior to commencing evaluation, the engineer must gather sufficient information of the building and site. General requirements for as-built data collection are described in ASCE 41-13 § 3.2, which points to Chapters 4 (Tier 1), 5 (Tier 2) and 6 (Tier 3) for additional data gathering requirements depending on the specific evaluation procedures to be used.

- Tier 1, ASCE 41-13 § 4.3.2 Field Verification and § 4.3.3 Condition Assessment: For Tier 1 screening, § 4.3.2 requires field verification to confirm that the building was constructed in general conformance with the record drawings and that no modifications have been made that significantly affect the expected performance of the lateral force-resisting system, while § 4.3.3 requires field verification that no significant deterioration of structural components has occurred.
- Tier 2, ASCE 41 § 5.2.2 As-Built Information, § 5.2.3 Condition Assessment, and § 5.2.6 Knowledge Factor: For Tier 2 deficiency-based evaluation, § 5.2.2 notes that additional as-built information may be needed beyond that for Tier 1. § 5.2.3 focuses on condition assessment requirements when Tier 2 procedures are used to evaluate deterioration and damage. § 5.2.6 requires a default knowledge factor, κ, of 0.75 for Tier 2 evaluations. A higher value of κ can be used if justified by the provisions of ASCE 41-13 § 6.2.4.
- Tier 3, ASCE 41 § 6.2 Data Collection Requirements: These provisions prescribe data collection requirements for Tier 3 evaluation. The extent of data collection will define a graded *level of knowledge* as either minimum, usual, or comprehensive. A knowledge factor, κ, is selected from ASCE 41 Table 6-1 depending on the performance requirements, analysis procedure and level of knowledge. Increasing values of κ represent higher confidence that the structural properties are sufficiently accurate. For linear procedures, a minimum level of knowledge is permitted, but nonlinear procedures require either usual or comprehensive, as defined in later chapters.

Often the seismic evaluation is started before the knowledge factor is determined. In such cases it is acceptable to assume a value of κ, but that value must be justified per the procedures of ASCE 41-13 § 6.2 and Table 6-1 prior to finalization of the analyses.

The foundation and material chapters of ASCE 41-13 have related data collection requirements that accompany those in ASCE 41-13 § 6.2 and Table 6-1. ASCE 41-13 § 8.2 covers foundation condition assessment. Steel condition assessment is addressed in ASCE 41-13 § 9.2, concrete in § 10.2, masonry in § 11.2 (and § 15.2 for the Special Procedure for URM buildings), and wood and cold-formed steel light frame in § 12.2. The detailed design examples in Chapter 5 through Chapter 13 of this *Guide* provide detailed information about data collection requirements specific to foundations and different materials.

ASCE 41-17 Revision

Revisions were made in ASCE 41-17 § 6.2 and Table 6-1 as well as in the foundation and material chapter data collection requirements to better clarify the data collection requirements.

Chapter 4
Analysis Procedures and Acceptance Criteria

4.1 Overview

This chapter provides discussion and example applications for a set of specific topics that affect a number of different building types. They were identified as needing clarification for new users of the ASCE 41-13 standard (ASCE, 2014). Topics are taken from ASCE 41-13 Chapter 3, Chapter 6, Chapter 7, and Chapter 13. These include the following:

- Selection of analysis procedures
- Determination of forces and deformations to use in analysis
- Categorization of primary and secondary elements
- Categorization of force-controlled and deformation-controlled elements
- Overturning
- Out-of-plane wall strength
- Equipment anchorage

4.2 Selection of Analysis Procedure (ASCE 41-13 § 7.3)

The type of analysis required for a seismic evaluation is dependent on many factors, such as building type, building height, structural configuration, and the level of evaluation. These attributes and the associated analysis limitations are discussed in ASCE 41-13 Chapter 3 through Chapter 7, and guidance is provided below with section references for the various levels (tiers) of evaluation. Note that the use of Tier 1 and Tier 2 evaluation procedures is limited to specific building types and heights with certain Performance Levels, in accordance with ASCE 41-13 § 3.3.1, whereas the Tier 3 systematic evaluation procedure is permitted for all buildings and all Performance Levels.

4.2.1 Tier 1 Screening

The majority of the Tier 1 Screening procedure consists of qualitative checklist statements. However, for most building types and Levels of Seismicity, there are simplified analysis calculations included as "Quick

Checks," triggered by evaluation statements from the qualitative checklist. These Quick Checks are performed using a pseudo seismic analysis in accordance with ASCE 41-13 § 4.5.2.

4.2.2 Tier 2 Deficiency-Based Evaluation and Retrofit

For the Tier 2 Deficiency-Based Evaluation, the analysis method is limited to either the linear static procedure (LSP) or the linear dynamic procedure (LDP) per ASCE 41-13 § 5.2.4; nonlinear analysis is not permitted. Use of the LSP is subject to the five limitations outlined in ASCE 41-13 § 7.3.1.2, which primarily pertain to long-period structures, buildings with geometric or stiffness irregularities, or non-orthogonal seismic force-resisting systems. For these scenarios, the LDP must be used to better predict the potentially complex distribution of seismic forces throughout the building. As noted in ASCE 41-13 § 5.2.4, the additional limitations on linear procedures specified in ASCE 41-13 § 7.3.1.1 do not apply for a Tier 2 evaluation. Specifically, the presence of in-plane and out-of-plane discontinuities, as well as weak story and torsional strength irregularities, does not preclude the use of Tier 2 linear procedures. Where the LDP is the selected analysis method or is required due to the aforementioned LSP restrictions, either the response spectrum method or the response history method is permissible.

4.2.3 Tier 3 Systematic Evaluation and Retrofit

Tier 3 Systematic Evaluation is permitted for any type of building or structural configuration, but the analysis procedure used for the evaluation is subject to all of the limitations in ASCE 41-13 § 7.3. These limitations and the underlying rationale are summarized in the following sections for linear and nonlinear analysis procedures.

4.2.3.1 Tier 3 Linear Procedures

- For regular buildings with uninterrupted load paths, linear procedures are considered sufficiently accurate to predict the distribution of seismic demands, and the corresponding acceptance criteria have built-in margins of safety that are appropriate for this level of accuracy.
- Use of the LSP for a Tier 3 evaluation, however, is limited to regular low-rise and mid-rise buildings with orthogonal seismic force-resisting systems. Specific quantitative criteria are provided in ASCE 41-13 § 7.3.1.2 and are consistent with the restrictions imposed on a Tier 2 evaluation.
- Both the LSP and the LDP are also subject to the limiting provisions in ASCE 41-13 § 7.3.1.1. Four different types of irregularities are defined: in-plane discontinuity, out-of-plane discontinuity, weak story, or

torsional strength. Linear procedures are not permitted for structures with in-plane and out-of-plane discontinuities due to potential inaccuracies stemming from a linear analysis. For structures with either a weak story irregularity or torsional strength irregularity (or both), linear procedures may be utilized only if it can be demonstrated that the building has limited nonlinear response for the Seismic Hazard Level under consideration. This demonstration consists of calculating the demand-capacity ratio (DCR) for all primary components per ASCE 41-13 Equation 7-16 and comparing it to the lesser of 3.0 or the associated *m*-factor for the component action. This DCR calculation signifies the magnitude of inelastic response for a particular component and is only intended for determining the applicability of the linear analysis procedures. The linear procedures have limits on the DCR since the elastic analysis does not include force redistribution caused by some members yielding at lower forces relative to other components in the system. Where the DCR evaluation indicates this condition, nonlinear procedures are required to more accurately determine the structural behavior.

4.2.3.2 Tier 3 Nonlinear Procedures

- Tier 3 evaluations require the use of nonlinear analysis methods when a building has one or more of the irregularities listed in ASCE 41-13 § 7.3.1.1, with the DCR exception discussed in the previous section for a weak story or torsional strength irregularity.

- Nonlinear analysis procedures are intended to provide a more accurate determination of the building response. Therefore, these procedures require a more robust understanding of the material properties and building configuration, which can significantly affect the nonlinear behavior of the structure. Data collection requirements are specified in ASCE 41-13 § 6.2.4.3, which calls for either the "usual" or "comprehensive" data collection requirements defined in ASCE 41-13 Table 6-1.

- For complex structural systems or configurations, it may be more efficient to proceed directly with nonlinear analysis. As discussed in Section 4.2 of this *Guide*, the effort involved in creating an intricate linear model in order to evaluate whether linear procedures are permissible may be greater than that required to generate a simple nonlinear model.

- The nonlinear static procedure (NSP) is limited to buildings with two distinct characteristics in accordance with ASCE 41-13 § 7.3.2.1:

- The structure must have a strength ratio, $\mu_{strength}$, that is less than the maximum strength ratio, μ_{max}. These ratios are calculated in accordance with ASCE 41-13 Equations 7-31 and 7-32, respectively. These calculations are essentially a comparison of the degree of nonlinearity in the primary system to the degradation level that could lead to dynamic instability (for the Seismic Hazard Level under consideration).
- The structural response must also not have significant higher mode effects, unless additional linear procedures are performed. The relative contribution of higher modes is determined by comparing the base shear from the first mode response to the base shear from all modes combined, using a sufficient number of modes to capture 90% mass participation. However, the NSP is permitted if the nonlinear analysis is supplemented with an LDP analysis. For this scenario, the structural components must meet the acceptance criteria for both analysis procedures, except the linear *m*-factors are permitted to be increased by a factor of 1.33 for the LDP.

- The nonlinear dynamic procedure (NDP) is permitted for all building types and configurations.

Figure 4-1 provides a summary of the analysis procedure selection process for each level of evaluation.

4.2.4 Examples

The following examples illustrate some common scenarios for selection the appropriate analysis procedure for Tiers 2 and 3. Section 6.4.3 of this *Guide* presents a complete Tier 1 screening example.

4.2.4.1 Tier 2 Deficiency-Based Evaluation and Retrofit Example

A voluntary seismic evaluation and retrofit is to be performed for a five-story, pre-Northridge steel moment frame office building over a reinforced concrete podium. The building is rectangular, and the first floor contains an open lobby, and therefore the upper story moment frames are discontinuous. The owner has elected to adopt the Basic Performance Objective for Existing Buildings (BPOE) for the evaluation and retrofit. This qualitative example shows the steps to determine the applicable analysis procedures.

- Building information
  - Steel moment frame (Type S1) and concrete shear wall (Type C2)
  - Office use: Risk Category II

- Six stories total
- Constructed in 1981

- Seismic Hazard Level and Building Performance Level (ASCE 41-13 Table 2-1)
  - Seismic Hazard Level: BSE-1E
  - Structural Performance Level: Life Safety (S-3)
- Building site characteristics
  - $S_{XS} = 1.00$
  - Seismicity = High (ASCE 41-13 Table 2-5)

Tier 1 Screening

Analysis Procedures:	Linear Static Analysis
Analysis Limitations:	For use with Quick Checks
Analysis Exceptions:	None
Section Reference(s):	§ 4.5

If the Tier 1 evaluation finds deficiencies, move to a Tier 2 Deficiency-Based Evaluation.

Tier 2 Deficiency-Based Evaluation and Retrofit

Analysis Procedures:	LSP or LDP
Analysis Limitations:	Use of LSP (§ 7.3.1.2)
Analysis Exceptions:	None
Section Reference(s):	§5.2.4, §7.4.1, § 7.4.2

If the Tier 2 evaluation does not apply or deficiencies are found, mitigate or evaluate with a Tier 3 linear evaluation.

Tier 3 Systematic Evaluation and Retrofit – Linear

Analysis Procedures:	LSP or LDP
Analysis Limitations:	Use of LSP & LDP (§ 7.3.1.1) Use of LSP (§ 7.3.1.2)
Analysis Exceptions:	Weak story or torsional strength irregularity with DCR<3.0 or *m*-factor
Section Reference(s):	§ 7.4.1, § 7.4.2

If a Tier 3 linear evaluation is not permitted, perform a Tier 3 nonlinear evaluation.

Tier 3 Systematic Evaluation and Retrofit – Nonlinear

Analysis Procedures:	NSP or NDP
Analysis Limitations:	Use of NSP (§ 7.3.2.1)
Analysis Exceptions:	NSP supplemented by LDP for buildings with higher mode effects
Section Reference(s):	§ 7.4.3, § 7.4.4

Figure 4-1 Summary of the analysis procedure selection process with reference to sections in ASCE 41-13 for each level of evaluation.

First, the applicability of a Tier 2 Deficiency-Based Evaluation is assessed.

- Benchmark Buildings (ASCE 41-13 § 4.3)
  - Because the building was constructed before 1994, the building does not meet the benchmark requirements. Therefore, further evaluation is required.
- Limitations on the use of Tier 2 Procedures (ASCE 41-13 § 3.3.1)
  - The Life Safety (S-3) Structural Performance Level is permitted for Tier 2 procedures.
- Vertical combinations (ASCE 41-13 § 3.3.1.2.2)
  - The building meets the Life Safety Structural Performance Level requirement, as well as the limitations of ASCE 41-13 Table 3-1 and Table 3-2. If the building had a horizontal combination of seismic force-resisting system, the requirements of ASCE 41-13 § 3.3.1.2.2.1 would also be assessed.

Since the requirements of ASCE 41-13 § 3.3.1 are satisfied, Tier 1 and Tier 2 procedures are permitted for this building. A Tier 1 screening is then performed to identify deficiencies, and the Tier 2 procedure will be used to evaluate the deficient items. The following steps are performed to determine the allowable linear procedures for the Tier 2 evaluation.

- Limitations on the use of the LSP are determined in accordance with ASCE 41-13 § 7.3.1.2 as referenced in ASCE 41-13 § 5.2.4. Note that the limitations of ASCE 41-13 § 7.3.1.1 do not apply to the Tier 2 procedure, so the presence of a vertical discontinuity does not preclude the use of Tier 2 linear procedures.
- An elastic mathematical model of the building is developed per ASCE 41-13 § 7.2.3 along with the material specific Chapter 9 and Chapter 10 of ASCE 41-13 with sufficient detail to evaluate the LSP limitations as described below.
  - A modal analysis is performed that demonstrates that the fundamental period is less than 3.5 times T_S, where T_S is the characteristic period of the response spectrum at which the constant acceleration segment of the response spectrum transitions to the constant velocity segment. It is defined per ASCE 41-13 Equation 2-9 as S_{X1} / S_{XS}.
  - There are no vertical offsets in the building, so the ratio of horizontal dimensions of adjacent stories is 1.0.

- A torsional stiffness irregularity for each story is determined by evaluating if the drift along any side of the structure is more than 150% of the average story drift. The analysis indicates that there are no torsional stiffness irregularities.
- A vertical stiffness irregularity for each story is determined by evaluating if the average story drift is greater than 150% of that of the story above or below. The analysis indicates that there are no vertical stiffness irregularities.
- The building has an orthogonal seismic force-resisting system.

Based on the results of these five limitation checks, the LSP is permitted for the Tier 2 Deficiency-Based Evaluation and Retrofit. If any of the above limitations were not satisfied, an LDP analysis would be required.

4.2.4.2 Tier 3 Systematic Evaluation and Retrofit Example

This example is based on the same building as in the previous example; however, the building is undergoing a change of occupancy to Risk Category IV, which requires compliance with International Building Code-level (IBC) seismic forces. An ASCE 41-13 Tier 3 procedure, as referenced by the 2015 IEBC, *International Existing Building Code* (ICC, 2015b), Section 301.1.4.1, is used to demonstrate compliance. IEBC Table 301.1.4.1 specifies the two-level Performance Objective of Immediate Occupancy with BSE-1N Seismic Hazard Level and Life Safety with the BSE-2N Seismic Hazard Level. Note that this is equivalent to the Basic Performance Objective Equivalent to New Building Standards (BPON) as defined by ASCE 41-13 § 2.2.4 and Table 2-2. This qualitative example evaluates the allowable analysis procedures for this building using the Tier 3 procedure.

The use of linear procedures is evaluated per the requirements of ASCE 41-13 § 7.3.1. Specifically, the four irregularity conditions are evaluated. If all four conditions were satisfied, the applicability of the LSP would be evaluated similar to the previous example. However, the presence of an in-plane discontinuity irregularity prohibits the use of linear analysis procedures. Accordingly, nonlinear procedures must be used for this building, and ASCE 41-13 § 7.3.2.1 is followed to determine if the NSP is permitted.

- A nonlinear mathematical model is created and analyzed to generate an idealized force-displacement curve per ASCE 41-13 § 7.4.3.2.4. The strength ratios $\mu_{strength}$ and μ_{max} are calculated from ASCE 41-13 Equations 7-31 and 7-32 respectively, and $\mu_{strength}$ is found to be less than μ_{max}.

- A modal response spectrum analysis is performed with sufficient modes to produce 90% mass participation. The shear in every story for this analysis is compared to the story shear results for a second modal response analysis considering only the first mode. The results demonstrate that the 90% mass participation story shears do not exceed 130% of the story shear considering only the first mode response.

Therefore, the NSP is permitted for this building. Note that if the modal response spectrum analysis did not meet these requirements, the NSP would still be permitted, but an LDP would also have to be performed. On the other hand, if μ_{strength} exceeds μ_{max}, then an NDP analysis would be required.

4.3 Determination of Forces and Target Displacements (ASCE 41-13 § 7.4)

4.3.1 Introduction

This section illustrates the calculation of pseudo seismic forces for the linear static procedure (LSP), scaling pseudo seismic forces for linear dynamic procedure (LDP), and determining target displacements for nonlinear static design procedure (NSP).

4.3.2 Example of Pseudo Seismic Force Calculations for Linear Static Procedure (ASCE 41-13 § 7.4.1)

Useful Tip

See Section 2.2.2 of this *Example Application Guide* for more discussion on pseudo seismic force.

The pseudo seismic force is the sum of lateral inertial forces applied to the building to produce displacements approximately equal to those the actual structure is expected to undergo during design earthquake loading. The pseudo seismic force, per ASCE 41-13 Equation 7-21, includes C_1 and C_2 factors to modify design displacement to represent those expected for a “yielding” structure. These values are based on analytical and experimental investigation of earthquake response of yielding structures (see FEMA 274 Section C3.3.1.3 (FEMA, 1997b) for more background on the C_1 and C_2 factors). The C_m value in ASCE 41-13 Equation 7-21 is included in the pseudo force to reduce the conservatism of the LSP for buildings where higher mode mass participation reduces seismic forces.

ASCE 41-13 § 7.4.1.3 presents an alternate method for determining pseudo seismic forces where the product of C_1C_2 is selected from ASCE 41-13 Table 7-3, based on the fundamental period and the maximum m-factor (m_{max}) used in the direction under consideration for all primary seismic-force-resisting system elements. This method is used in the following example. Chapter 10 of this *Guide* presents an iterative process whereby a preliminary pseudo seismic force is used to determine preliminary maximum Demand Capacity Ratios (DCRs). Then, based on the DCR_{max} values in each direction, C_1C_2

values are calculated and the final pseudo seismic forces are determined. The C_m value is the same for both methods. Results from both methods are compared in Section 4.3.2.2 of this *Guide*.

4.3.2.1 Seismic Design Parameters and Performance Level

The pseudo seismic force calculations are based on the three-story concrete shear wall building in the Chapter 10 design example of this *Example Application Guide*.

- Building Information
  - Concrete shear wall building (Type C2)
  - Office use: Risk Category II
  - Three stories, 42 feet tall above base at ground floor
- Seismic Hazard Level and Building Performance Level
  - Seismic Hazard Level: BSE-2E
  - Structural Performance Level: Collapse Prevention (CP)
- Building Site Characteristics
  - Site Class D
  - Seattle, Washington
  - Latitude: 47.6143 ° N
  - Longitude: 122.3358 ° W

The following ground motion parameters are obtained for the BSE-2E Seismic Hazard Level using the tools presented in Chapter 3:

S_{XS} = 1.08

S_{X1} = 0.62

4.3.2.2 Alternate Method for Determining Pseudo Seismic Forces

The following illustrates using the alternate method for determining the pseudo seismic force where C_1C_2 and C_m are determined from ASCE 41-13 Table 7-3 and Table 7-4. The pseudo seismic forces are determined for each direction under consideration. This method is applicable in both the transverse and longitudinal directions.

Pseudo seismic force, V, is determined per ASCE 41-13 § 7.4.1.3.1:

$$V = C_1C_2C_mS_aW \qquad \text{(ASCE 41-13 Eq. 7-21)}$$

Building period, T, is determined using the empirical equation:

$T = C_t h_n^{\beta}$, Method 2 (ASCE 41-13 Eq. 7-18)

where:

$C_t = 0.020$

$\beta = 0.75$

$h_n = 42$ ft

$T = 0.020(42)^{0.75} = 0.33$ seconds

Preliminary C_1C_2 is determined in each direction from ASCE 41-13 Table 7-3:

Transverse direction:

In the transverse direction looking at ASCE 41-13 Table 10-22 for concrete shear Walls A and G (see Section 10.5.1.2 and 10.5.1.4 of this *Guide* for parameters in selecting the m-factors for the walls), $m_{max} = 6.0$. With an m_{max} greater than or equal to 6 and $0.3 < T \leq 1.0$ from ASCE 41-13 Table 7-3, then:

$C_1C_2 = 1.2$

Longitudinal direction:

In the longitudinal direction, $m_{max} = 1.0$, based on all the primary concrete shear walls in the longitudinal direction being under-reinforced (less than 0.0015) and evaluated as a force-controlled element. With $m_{max} = 1.0$ and $0.3 < T \leq 1.0$ from ASCE 41-13 Table 7-3, then:

$C_1C_2 = 1.0$

C_m is determined from ASCE 41-13 Table 7-4:

$C_m = 0.80$ (Concrete shear wall building with 3 or more stories)

Response acceleration parameters are determined per ASCE 41-13 § 2.4.1:

$T_0 = 0.2T_S$ (ASCE 41-13 Eq. 2-10)

where:

$T_S = S_{X1}/S_{XS}$ (ASCE 41-13 Eq. 2-9)

$S_{X1} = 0.62$ (given)

$S_{XS} = 1.08$ (given)

$T_S = 0.62/1.08 = 0.574$ seconds

T_0 = 0.2(0.574) = 0.115 seconds

Since $T_0 \leq T \leq T_S$:

S_a = S_{XS}/B_1 (ASCE 41-13 Eq. 2-6)

Determine damping modifier B_1:

B_1 = 4/[5.6-ln(100β)] (ASCE 41-13 Eq. 2-11)

where:

β = 0.05 (effective viscous damping ratio, β = 0.05 corresponds to 5% damping which is typical for concrete office buildings)

B_1 = 4/{5.6-ln[100(0.05)]} = 1.0

S_a = S_{XS}/B_1 = 1.08/1.0 = 1.08

Pseudo seismic force:

Transverse direction:

$$V = C_1C_2C_mS_aW$$
$$= 1.20(0.8)(1.08)W$$
$$= 1.04W$$

Longitudinal direction:

$$V = C_1C_2C_mS_aW$$
$$= 1.0(0.8)(1.08)W$$
$$= 0.86W$$

The pseudo seismic force looks much larger than ASCE 7-10 values where the seismic force is reduced by a system-wide response modification factor, *R*, taking into account ductility and detailing of the building lateral resisting system. ASCE 41-13 addresses ductility and detailing on the capacity side using *m*-factors and expected strength for deformation-controlled members.

Section 10.4.6 of this *Guide* presents values of these values with the iterative process as 1.02W for V_{TRANS} and 0.86W for V_{LONG}. The comparison shows that pseudo seismic forces are similar for both methods.

V_{TRANS} = 1.04W versus 1.02W

V_{LONG} = 0.86W versus 0.86W

4.3.3 Scaling Pseudo Seismic Forces for Linear Dynamic Procedure (ASCE 41-13 § 7.4.2)

Pseudo seismic forces for linear dynamic procedure (LDP) are not required to be scaled as they are in ASCE 7. The LDP in ASCE 41-13 is intended to displace the structure to its anticipated displacement, which is taken as C_1C_2 times the unreduced pseudo seismic forces associated with the Seismic Hazard and Performance Level considered. Therefore, no artificial floor is needed as included in ASCE 7-10 requirements.

Per ASCE 41-13 § 7.4.2.3.2, the diaphragm forces determined by LDP are required to be scaled similar to ASCE 7 and taken as not less than 85% of the static force value.

4.3.4 Determination of Target Displacement

For the nonlinear static procedure (NSP), the target displacement is calculated per ASCE 41-13 § 7.4.3.3. The target displacement represents the expected displacement of the roof level at the Seismic Hazard Level under consideration. Building components are then evaluated at that displacement to determine whether they have sufficient capacity to accommodate the displacement. ASCE 41-13 Equation 7-28 defines the target displacement:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{(ASCE 41-13 Eq. 7-28)}$$

where:

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multi-degree-of-freedom (MDOF) system

C_1 = Modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response

C_m = Effective mass factor

S_a = Response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration

T_e = Effective fundamental period of the building in the direction under consideration, defined as:

$$T_e = T_i (K_i/K_e)^{1/2} \qquad \text{(ASCE 41-13 Eq. 7-27)}$$

where:

T_i = Elastic fundamental period (in seconds) in the direction under consideration

K_i = Elastic lateral stiffness of the building in the direction under consideration

K_e = Effective lateral stiffness of the building in the direction under consideration, as shown in ASCE 41-13 Figure 7-3.

Additional information regarding each coefficient is as follows:

- C_0: There are three methods for determining this modification factor. The first is to use the first mode mass participation factor multiplied by the ordinate of the first mode shape at the control node. The second is the mass participation factor calculated using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by the ordinate of the shape vector at the control node. The values for either of these methods can come from a computer model that calculates the dynamic properties of the building. The third method is to use default values from ASCE 41-13 Table 7-5, which depend on the number of stories, the type of loading pattern used (triangular or uniform), and whether the building is or is not a "shear" building where story drift decreases with increasing height. The third method is the most common.
- C_1: Calculation of C_1 first requires determination of μ_{strength}, which is:

$$\mu_{\text{strength}} = [S_a/(V_y/W)]C_m \qquad \text{(ASCE 41-13 Eq. 7-31)}$$

where:

S_a = As defined above

W = Effective seismic weight in accordance with ASCE 41-13 § 7.4.1.3.1, which provides details regarding storage, partition, equipment, and snow loads.

C_m = Effective mass factor

The value μ_{strength} is the ratio of elastic strength to yield strength. The NSP is not permitted where μ_{strength} exceeds $\mu_{\max}$. $\mu_{\max}$ is defined in ASCE 41-13 Equation 7-32 and involves understanding the post-yield negative slope of the global force-displacement relationship and P-Δ

effects on the slope. After μ_{strength} is calculated, then it can be used for ASCE 41-13 Equation 7-29.

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{aT_e^2} \qquad \text{(ASCE 41-13 Eq. 7-29)}$$

where:

a = Site class factor: 130 for Site Class A or B, 90 for Site Class C, and 60 for Site Class D, E, or F

T_e = Effective fundamental period as defined above

C_1 also has bounds related to the effective period of the building. Where T_e is less than 0.2 seconds, then the C_1 need not be taken as greater than the value calculated at $T = 0.2$ seconds. For T_e greater than 1.0 seconds, then the $C_1 = 1.0$.

- C_2: Calculation of C_2 also depends on μ_{strength} and T_e. It is defined as:

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{\text{strength}} - 1}{T_e}\right)^2 \qquad \text{(ASCE 41-13 Eq. 7-30)}$$

  For T_e greater than 0.7 seconds, then $C_2 = 1.0$.

- C_m: There are two methods for calculating C_m. The first is to use the effective mass factor in ASCE 41-13 Table 7-4, which depends on the seismic force-resisting system and the number of stories. The second approach is to use the effective modal mass participation factor calculated for the fundamental mode using an eigenvalue analysis. For T_e greater than 1.0 seconds, then $C_m = 1.0$.

As can be seen, even though ASCE 41-13 Equation 7-28 appears relatively straightforward, calculation of the coefficients themselves is not. It involves developing the model of the building, displacing it to get a global force-displacement curve, idealizing the curve, calculating effective stiffness properties and effective periods, determining the extent of anticipate strength beyond the yield strength, and understanding the negative post-yield portion of the global force-displacement curve. In addition, the entire process is iterative. A preliminary target displacement is calculated, and then based on the displacement, the idealized global force-displacement curve is refined, and the various coefficients are recalculated. The process is repeated until reasonable convergence is achieved.

A detailed example of calculating the target displacement for the NSP is shown in Section 11.3 of this *Guide* for a concrete building, including all the necessary steps. For more information on the technical basis of the target displacement method and its coefficients, see FEMA 440 (2005).

4.4 Primary vs. Secondary Elements (ASCE 41-13 § 7.2.3.3 and ASCE 41-13 § 7.5.1.1)

Elements of a building that provide gravity or seismic load resistance as part of a continuous load path to the foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections may be designated as primary or secondary.

Primary elements are traditional lateral force-resisting elements, such as shear walls, braced frames, and moment frames, and must be included in the analysis model. Primary elements can be existing elements or new elements that are part of a seismic retrofit. Secondary elements are typically existing elements that unintentionally participate in the seismic response and typically provide unreliable resistance to earthquake loads. Secondary elements are usually either gravity load-carrying elements (gravity beams and columns) or nonstructural elements that are not isolated from the structural system. Table 4-1 provides some common secondary elements that may be part of the listed common model building types.

Table 4-1 Common Secondary Elements

Primary System	Examples of Secondary Elements
Wood (W1, W1a, W2) and flexible diaphragms in non-light frame (S1a, S2a, S5a, C2a, C3a, PC1, RM1, URM)	Gravity beams and columns, and participating nonstructural elements such as gypsum walls, lath and plaster walls, and stucco walls
Moment frame (S1, S3, C1, PC2a)	Gravity beams and columns, concrete slab to column, stair framing, cladding and partition walls rigidity attached
Shear wall (S4, S5, C2, C3, PC1a, PC2, RM2, URMa)	Concrete slab to concrete columns, gravity columns
Braced frame (S2)	Gravity beams and columns

The concept of redundancy is an important part of designing structures for seismic resistance. In many structures, nearly all elements and components of the building participate to some degree in the structure's seismic force-resisting system. ASCE 41-13 encourages including all primary and secondary elements in the analysis to better understand the building behavior. If only primary seismic force-resisting elements are included in the evaluation, ASCE 41-13 may appear to be conservative especially when more advanced analysis is performed. Below is commentary from FEMA 274 (FEMA, 1997b) that discusses the origin of primary and secondary elements:

Key Terms

Primary Component: An element that is required to resist the seismic forces and accommodate seismic deformations in order for the structure to achieve the selected Performance Level.

Secondary Component: An element that accommodates seismic deformations but is not required to resist the seismic forces it may attract in order for the structure to achieve the selected Performance Level. The total stiffness of secondary components cannot exceed 25% of the stiffness of primary components.

Key Terms

Structural Component: An element of a building that provides load resistance as part of a load path; designated as primary or secondary.

Nonstructural Component: An architectural, mechanical, or electrical element of a building that is permanently installed in, or is an integral part of, a building system.

The component is required to be classified as structural if stiffness or strength exceed 10% of the structural components.

"If a structure has sufficient redundancy, it may be permissible to allow failure of some elements, as long as this does not result in loss of gravity load-carrying capacity or overall lateral stability. FEMA 273 introduced the concept of "primary" and "secondary" elements in order to allow designers to take advantage of the inherent redundancy in some structures, and to permit a few selected elements to experience significant damage rather than requiring massive rehabilitation programs to prevent such damage."

If secondary elements are not supporting gravity loads and the consequence of failure is relatively insignificant, many in practice will consider it sufficient to model those elements with deformation-controlled Type 3 force-displacement relationship (Figure 4-6). This reclassification is permissible only if the requirements of ASCE 41-13 § 7.5.1.2 are met. These include:

- The component actually exhibits the Type 3 deformation-controlled behavior depicted in Figure 4-6
- The gravity load-carrying load path is not altered or an adequate alternative path is provided
- The total gravity load supported by reclassified components does not exceed 5% of the total at that story
- All remaining components meet their acceptance criteria
- If the reclassification results in a change in the expected mechanism of the building, then the Type 3 components strength shall be increased by the ratio of Q_{CE}/Q_y, the analysis rerun, and all components rechecked.

For linear analysis, if the total initial stiffness of secondary elements exceeds 25% of the total initial lateral stiffness of primary elements, some secondary elements must be reclassified as primary and included in the analysis model to reduce the relative stiffness of secondary elements to be less than 25% per ASCE 41-13 § 7.2.3.3. All remaining secondary elements that are not explicitly modeled are checked for the earthquake-induced deformations in combination with gravity loads. To make this confirmation, it is sometimes helpful to make a second analysis model with deformations induced into the secondary elements in combination with gravity loads.

Nonstructural elements are also required to be considered and required to be classified as a structural element if their initial lateral stiffness or strength exceeds 10% of the total initial lateral stiffness or expected strength of a story, respectively. All elements should be included if their participation affects any irregularities. An example of this could be torsional response from rigidly attached stairs on one side of a moment frame building or cladding systems with different stiffness characteristics on opposing sides of the building. The process of determining which elements need to be included in the model can be iterative and requires a good understanding of how the secondary and nonstructural elements are detailed. If their detailing does not allow for isolation from seismic deformations, then they should be included in the initial model to understand their relative stiffness to determine if they are required to be treated as a primary.

For nonlinear analysis, all elements (primary and secondary) should be included in the model with their respective strength and stiffness, including degradation, for more accurate results. Nonlinear analysis treats the acceptance criteria for primary and secondary elements the same since strength and stiffness degradation and force redistribution are explicitly captured in the analysis model. Linear analysis cannot account for this degradation; therefore, only primary elements are required to be included in the model. Figure 4-2 through Figure 4-4 show examples of primary and secondary elements.

Figure 4-4 shows that for a relatively stiff system with concrete shear walls, designation of primary and secondary requires judgment and understanding of the system behavior. In that example, the primary shear wall and short pier are the primary system since they are both likely to have more than the 25% stiffness limitation of secondary. The spandrels adjacent to the wall or pier should also be modeled since their geometry directly affects the primary elements stiffness.

Figure 4-5 is the plan of a building where the secondary system provides significant unintended lateral stiffness relative to the primary system. The primary system is a perimeter concrete moment frame, and the secondary system is the gravity system consisting of a concrete flat plate slab spanning to interior concrete columns. The slab-column gravity connections resist 72% of the base shear in north-south transverse direction due to the long diaphragm span and reduced number of moment frame bays in that direction. Since this is greater than the 25% limit in ASCE 41-13 § 7.2.3.3, these secondary elements are required to be reclassified as primary and included in analysis model.

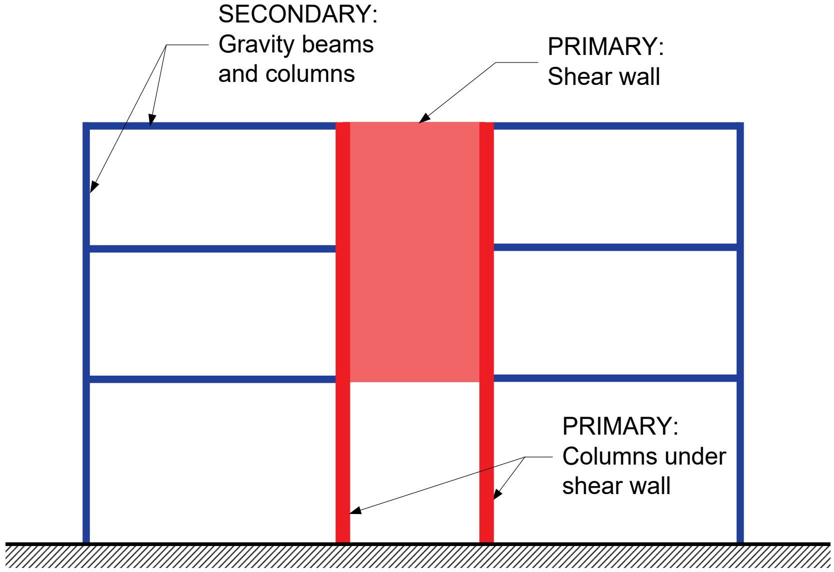


Figure 4-2 Shear wall building illustrating primary and secondary components.

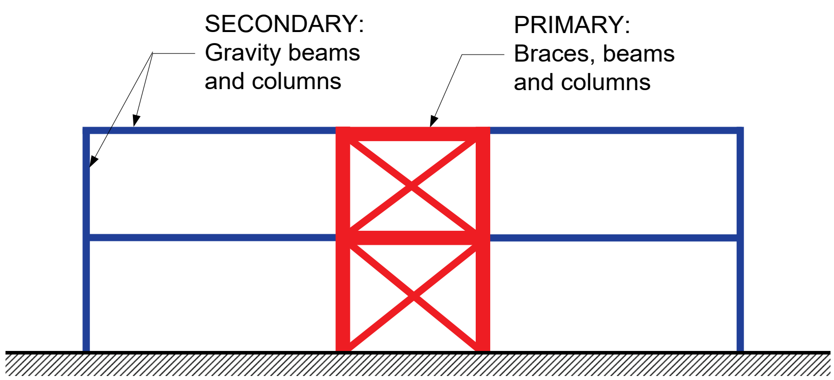


Figure 4-3 Braced frame building illustrating primary and secondary components.

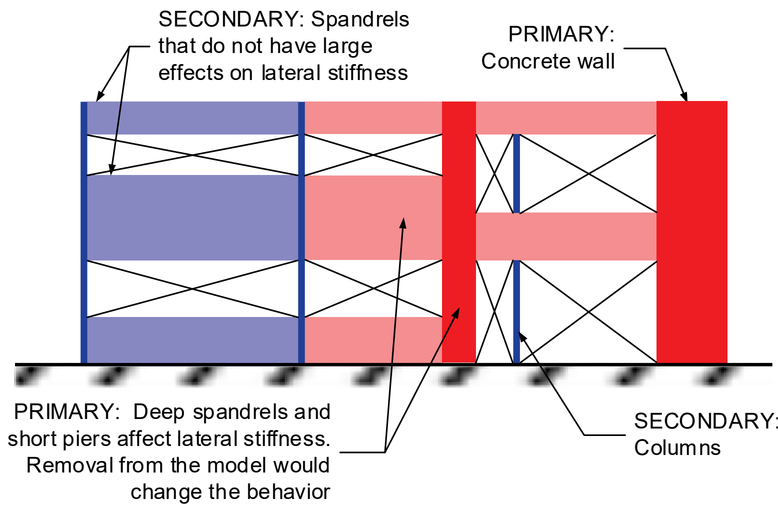


Figure 4-4 Perforated concrete shear wall building illustrating primary and secondary components.

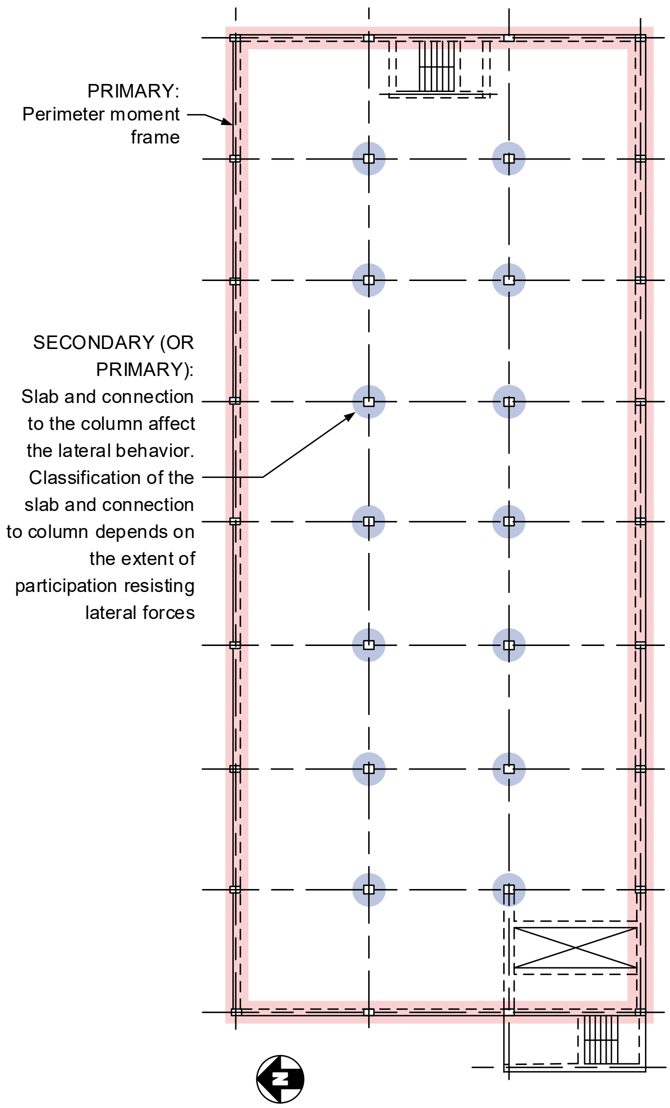
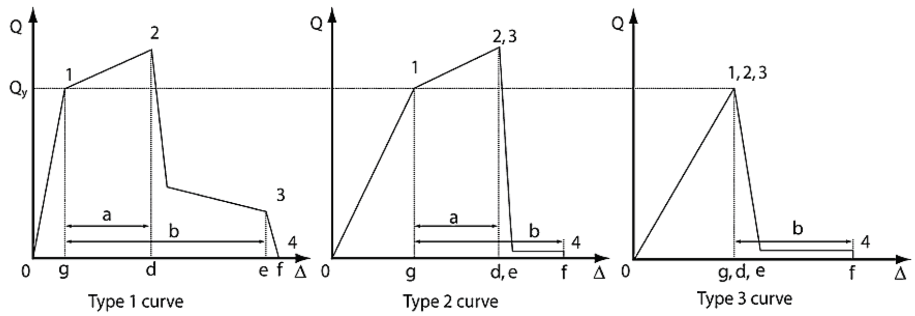


Figure 4-5 Concrete moment frame building plan illustrating primary and secondary components.

4.5 Force-Controlled and Deformation-Controlled Actions (ASCE 41-13 § 7.5.1.2)

Force-controlled actions (FCA) are expected to have sudden loss of strength after failure, and are not allowed to exceed the nominal strength of the element being evaluated. On the other hand, deformation-controlled actions (DCA) are permitted to deform beyond yield as approximated by their component modification factor (*m*-factor). Ideally, a building system has an identifiable seismic force-resisting system, consisting of horizontal and vertical systems. Within each of these, some component actions are expected to be capable of performing after initial yielding (deformation-controlled actions), whereas some actions are required to transmit forces elastically due to their limited ductility (forced-controlled actions). This concept is somewhat similar to how new buildings are designed with special load combinations (those containing overstrength, Ω_0, factor) applied to components that are intended to remain essentially elastic, and code-level forces (with a reduction factor, *R*) are used for the elements of the lateral force-resisting system that are expected to yield. New buildings also have requirements to ensure intended behavior, such as strong column/weak beam behavior in moment frames. Figure 4-6 (which is ASCE 41-13 Figure 7-4) illustrates the component behavior curves that define if an element is deformation-controlled (Type 1) or force-controlled (Type 3). Behavior in between the Type 1 and Type 3 (Type 2) depends on the amount of plastic deformation before strength loss, to determine whether it is considered deformation-controlled or force-controlled.



Notes:
1. Only secondary component actions permitted between points 2 and 4;
2. The force, Q, after point 3 diminishes to approximately zero.

Figure 4-6 Force-displacement curves (ASCE 41-13 Figure 7-4). Printed with permission from ASCE.

Figure 4-7 through Figure 4-9 illustrate some common examples of DCA and FCA for two selected different building systems and a brace connection. It is noted that for each building, component action designations could be different based on other factors, such as high axial load.

To address the frequently occurring scenario where a small number of elements fail to meet the acceptance criteria, particularly in stiff force-controlled elements, ASCE 41-13 § 7.5.1.2 and ASCE 41-13 Figure 7-4 permit the reclassification of some force-controlled element to allow strength loss, provided vertical load-carrying capability remains and dynamic instability does not occur. A common example of this would be a localized loss of a coupling beam that is not supporting gravity loads beyond its self-weight that fails early due to its stiffness. If the element is able to be further damaged without resulting in loss of gravity load capacity, this element could be considered a deformation-controlled element with a Type 3 curve per ASCE 41-13 § 7.5.1.2.

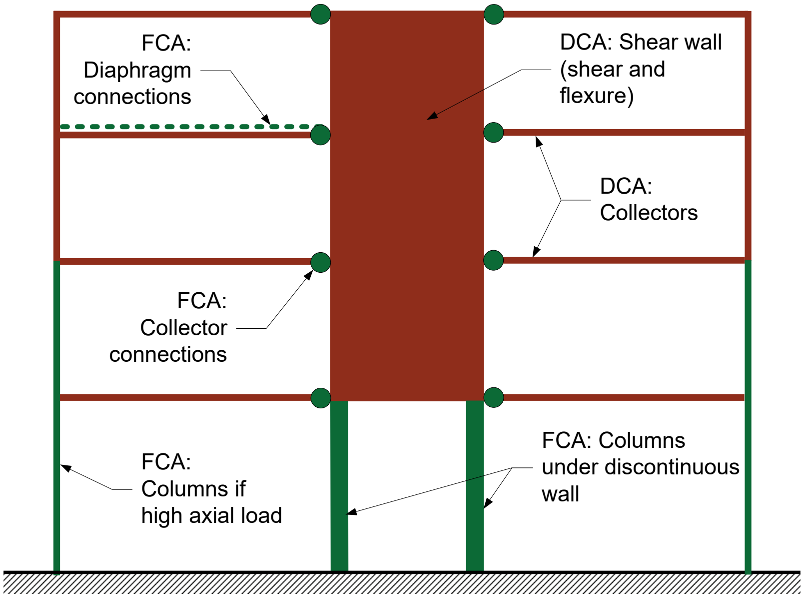


Figure 4-7 Shear wall building illustrating force- and deformation-controlled actions.

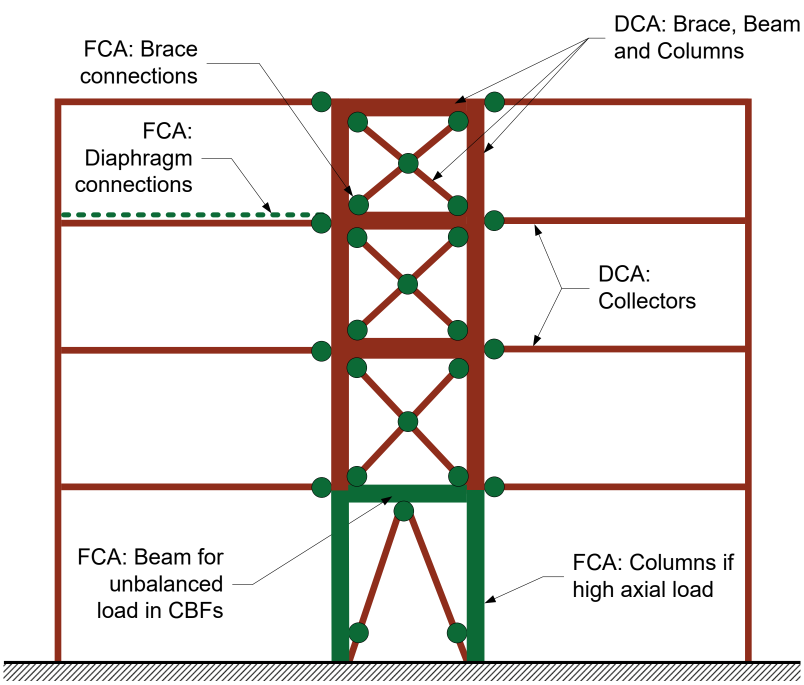


Figure 4-8 Braced frame building illustrating force- and deformation-controlled actions.

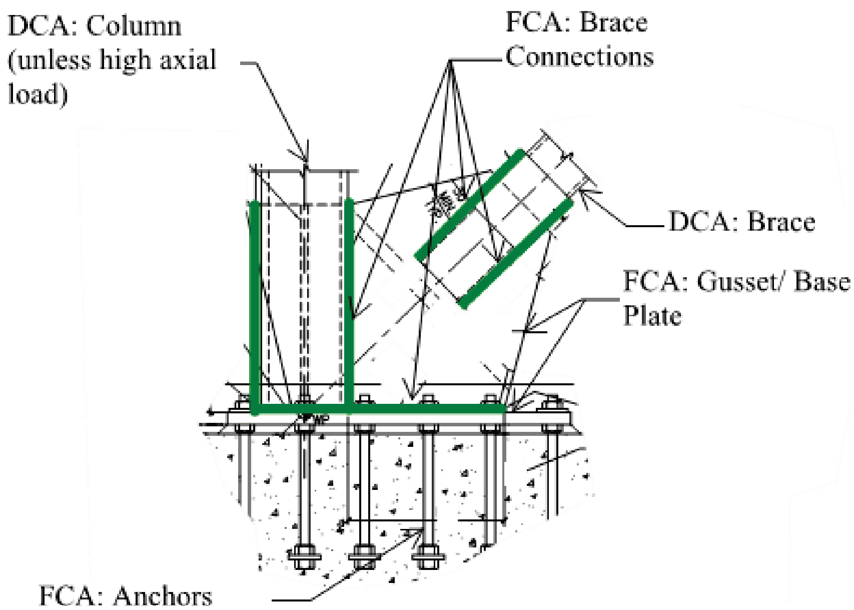


Figure 4-9 Braced frame connection illustrating force- and deformation-controlled actions.

Table 4-2 lists common elements that would be designated force-controlled for different structural systems and the reasons for that designation.

Table 4-2 Examples of Force-Controlled Elements

	Force-Controlled Component	Trigger	ASCE 41-13 Reference
Tier 2	Captive columns	Tier 2: Required if columns are shear controlled	§ 5.5.2.3.3
	Precast concrete moment frame elements or connections	Tier 2: If Tier 1 check fails	§ 5.5.2.4
	Tilt-up panel to panel connection	Tier 2: If connection is being used to transfer overturning	§ 5.5.3.3.3
General	Diaphragm ties	Always required by reference section	§ 7.2.9.5
	Continuity	Always required by reference section	§ 7.2.10
	Out-of-plane connections	Always required by ASCE 41-13 § 7.2.11	§ 7.2.11.1
	Out-of-plane structural walls	Always required by ASCE 41-13 § 7.2.11	§ 7.2.11.2, § 11.3.5.3
	Transfer diaphragm forces	Discontinuous vertical elements	§ 7.4.1.3.4, § 7.4.2.3.2
	Foundations reactions	Fixed base assumption	§ 8.4.5.2.1
	Seismic earth pressure on retaining walls	Always required by reference section	§ 8.6

Table 4-2 Examples of Force-Controlled Elements (continued)

	Force-Controlled Component	Trigger	ASCE 41-13 Reference
Steel	Anchorage to concrete	If controlled by concrete failure	§ 9.3.2.4
	Steel columns	Axial demand/capacity over 50%	§ 9.4.2.4.2, § 9.4.2.4.3, Table 9-5 Note 12, Table 9-6 Note 6
	Fully-restrained (FR) connections	Yielding of beam remote of connection	§ 9.4.2.4.2, § 9.4.2.4.3
	Partially-restrained (PR) frame connections	Limits state not listed in Table 9-5	§ 9.4.3.4.2
	Braced frame connections	Always required unless connection is explicitly modeled and ductility is justified	§ 9.5.2.4.1, § 9.5.4.4.1
	Steel beams in V-type of inverted V-type braced frames	Unbalanced load effects in concentrically braced frames (CBF)	§ 9.5.2.4.2
	Eccentrically braced frame (EBF) braces and connections	Always required by reference section	§ 9.5.3.4.1
	Connections of metal deck diaphragms	Always required by reference section	§ 9.8.1.4, § 9.8.2.4, § 9.8.3.4
	Archaic diaphragms	Always required by reference section	§ 9.8.5.4
	Connections of steel piles to pile caps	Always required by reference section	§ 9.9.4
	Cast and wrought iron	Always required by reference section	§ 9.10.3
	Fully-restrained connections - welded flange plates	Where limits other than plate net section govern	Table 9-5, Table 9-6
Concrete	Concrete actions not listed tables	Unless component testing is performed	§ 10.3.2.1
	Concrete columns - axial load	When also in bi-axial bending	§ 10.3.3
	Anchorage to concrete	Always required by reference section	§ 10.3.6.1, § 10.3.6.2
	Slab-column moment frames	FCA are all actions not listed in reference section	§ 10.4.4.4.1
	Concrete frame with masonry infills	FCA are all actions not listed in reference section	§ 10.6.2.4
	Concrete shear walls or wall segment	Transverse reinforcing ratio less than 0.0015	§ 10.7.2.3
	Concrete shear walls, wall segments, and coupling beams	Actions other than flexure or shear	§ 10.7.2.4.1
	Concrete braced frames	FCA are all actions not listed in reference section	§ 10.9.5.1

Table 4-2 Examples of Force-Controlled Elements (continued)

	Force-Controlled Component	Trigger	ASCE 41-13 Reference
Concrete (cont'd)	Concrete diaphragm connections	Always required by reference section	§ 10.10.2.4
	Un-topped precast diaphragms	Always required by reference section	§ 10.11.2
	Existing foundation systems	Always required by reference section	§ 10.12.3
	Concrete slab-columns	Slabs controlled by rebar development of splices, or high shear demand without continuity reinforcing.	Table 10-15 Note 5, Table 10-16 Note 5
Masonry	Concrete shear wall controlled by shear	High axial demand	Table 10-20 Note 1, Table 10-22 Note 1
	URM toe crushing, diagonal tension, and axial compression	Always required by reference section	§ 11.3.2, § 11.3.2.3
	Reinforced masonry walls - axial compression	Always required by reference section	§ 11.3.4.3
	Reinforced masonry walls controlled by shear	Walls with high axial demand	Table 11-6 Note 2, Table 11-7 Note 2
	Anchorage to masonry	Always required by reference section	11.5.2
	Existing masonry foundations	Always required by reference section	11.6.2
Light Frame	Light-frame components supporting discontinuous shear walls	Flexure and shear on beams and axial compression on columns	12.3.4.1, 12.3.4.2
	Light-frame bodies of connections	Always required by reference section	12.3.3.1, 12.3.3.2
	Axial compression and connections between steel rods and wood members	Always required by reference section	12.7.13
	Light-frame components subject to axial compression	Always required by reference section	Table 12-3
	Wood connection	Actions on connection not listed in table	Table 12-3 Note 4, Table 12-4 Note 4

4.5.1 J-Factor (ASCE 41-13 § 7.5.2.1.2)

The linear procedures are based on amplified elastic force demands to approximate expected nonlinear deformations. The structural system may have some level of component action yielding; hence, the actual forces present on an element will be less than those computed in the analysis. For force-controlled elements, the methodology seeks to maintain those as essentially elastic to avoid brittle failure and therefore require comparison of their capacity to the expected demand. The derivation of this demand is obtained by removing the amplification factors, C_1C_2, and introducing a force reduction factor, J, to account for the yielding occurring in the deformation-controlled elements. The product of C_1C_2 is divided out since they are modifications to the pseudo lateral force to better capture inelastic demands in yielding elements, as follows:

$$Q_{UF} = Q_G + Q_E/(C_1C_2J) \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

ASCE 41-17 Revision

In ASCE 41-17, Equation 7-35 was revised to add a 1.3 factor on Q_E for Life Safety Performance Level checks to better provide the margin of safety against collapse implied by the Life Safety Performance Level definition.

Since force-controlled elements are to remain elastic, these modifications from inelastic yielding would overpredict the demand. Figure 4-10 modifies a figure from FEMA 274 (FEMA, 1997b) to illustrate the axial force-displacement behavior of a column. Many times, columns in the primary system are required to be force-controlled as they support significant gravity load. Under only gravity load, there is no displacement which defines point b on the vertical axis. As seismic load is added to the system, there is increased lateral displacement and yielding occurring in the primary system which will cause the earthquake demands in the force-controlled elements to reduce from the elastic response demands (point c) due to yielding elements in the load path. The value of J is to be taken as the smallest demand-to-capacity ratio of elements in the load path of the force-controlled element representing the most demand that could be delivered to the forced-controlled element. Alternatively, ASCE 41-13 § 7.5.2.1.2 allows J to be taken between 1.0 and 2.0, depending on the Level of Seismicity as defined by ASCE 41-13 § 2.5.

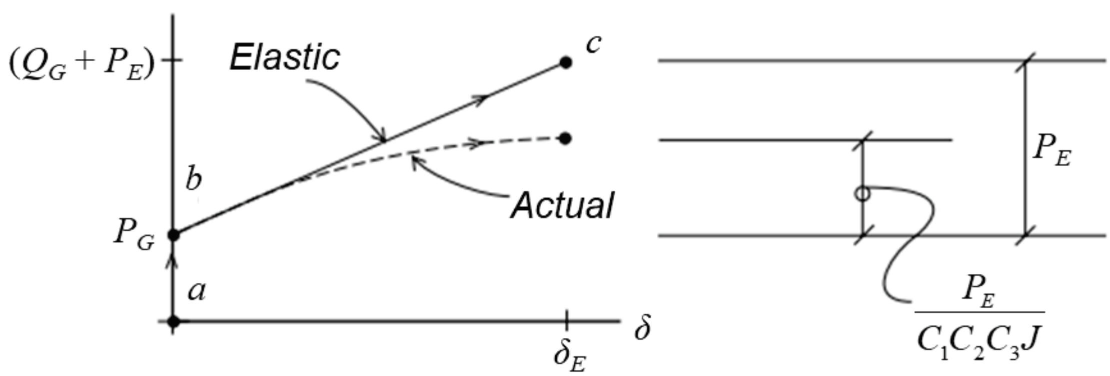


Figure 4-10 *J*-factor example at column (from FEMA, 1997b).

Where elements delivering demands to the force-controlled elements either remain elastic or are required to satisfy Immediate Occupancy performance level, J is required to be taken as 1.0. Refer to example in Section 4.6.4 of this *Guide* for application of J-factor examples.

4.6 Overturning—Wood Shear Wall Example (ASCE 41-13 § 7.2.8)

The overturning effects caused by seismic forces are evaluated in this section for a two-story light-frame wood shear wall to determine if the seismic forces can be resisted by the stabilizing effects of gravity load or if the positive attachments provided at each story are required and adequate for the anticipated loading. The overturning effects at the foundation-soil interface are also evaluated per ASCE 41-13 Chapter 8.

4.6.1 Overview

Example Summary

Structure Type: Two-story wood-frame shear wall

Performance Objective: BPOE

Risk Category: II

Location: Sacramento, California

Level of Seismicity: High

Reference Documents:

NDS-2012

SPDWS-08

AISI S100-07/S2-10

AISI S213-07/S1-09

This example illustrates the procedure and calculations to evaluate the overturning effects at each level of a two-story wood-frame shear wall in accordance with ASCE 41-13 § 7.2.8 using the linear static procedure. This section of ASCE 41-13 addresses the overturning stability and connections at each story, but does not include an evaluation of the foundation or overturning at the foundation-soil interface, which is addressed in ASCE 41-13 Chapter 8. The following calculations are illustrated:

- Calculation of pseudo seismic force (ASCE 41-13 § 7.4.1.3)
- Evaluation of overturning and strap at the second floor (ASCE 41-13 § 7.2.8, § 12.2, and § 12.3)
- Evaluation of overturning and hold-down at the first floor (ASCE 41-13 § 7.2.8, § 12.2, and § 12.3)
- Evaluation of the hold-down anchor-to-footing connection (ASCE 41-13 § 10.3.6)
- Evaluation of overturning at the foundation-soil interface (ASCE 41-13 § 8.4.2.3)

The Basic Performance Objective for Existing Buildings (BPOE) is the targeted performance. The building is assumed to be a Risk Category II building. This example illustrates the Tier 3 procedure that requires a two-level assessment: per ASCE 41-13 Table 2-1, the Structural Performance Levels required to be evaluated for the Tier 3 procedure are Life Safety at the BSE-1E Seismic Hazard Level and Collapse Prevention at the BSE-2E Seismic Hazard Level. It is noted that this building would qualify for the Tier 2 procedure and require only a single-level assessment (Life Safety at the BSE-1E Seismic Hazard Level). The following sections first check the Collapse Prevention Performance Level for the BSE-2E seismic hazard as that typically governs; then a brief comparison will be made for the Life

Safety Performance Level for the BSE-1E seismic hazard at the end of each section.

Figure 4-11 illustrates the configuration, geometry, and loading for the shear wall segment being evaluated.

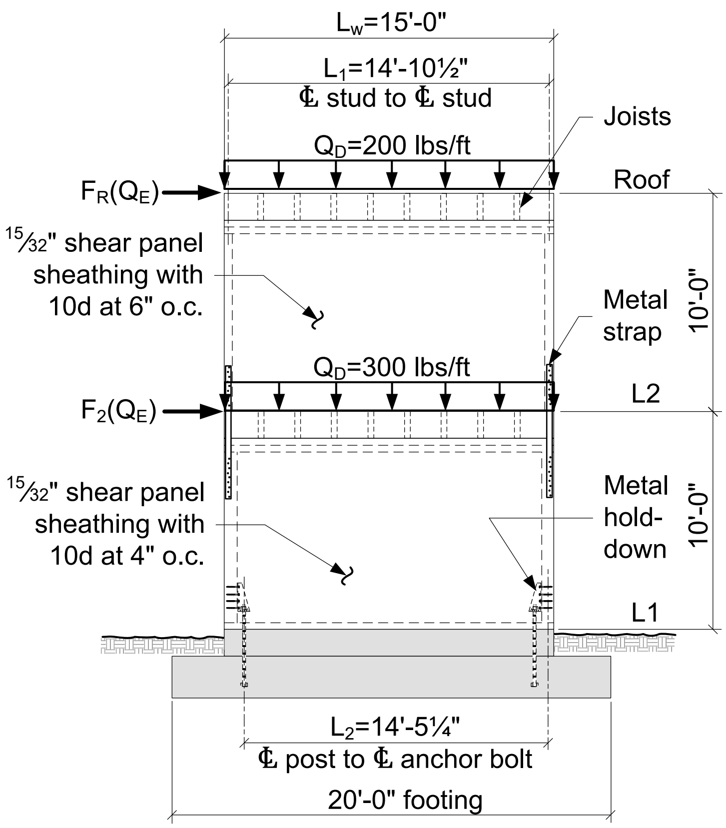


Figure 4-11 Elevation of two-story wood frame shear wall.

The following building information is given:

- Location: Sacramento, California
- Site Class D
- Two-story, 10 foot story heights
- 15/32-inch wood structural panel (Structural I) shear wall, 15 feet in length, with 10d at 6 inches on center edge nailing at second story and 10d at 4 inches on center edge nailing at first story.
- Building seismic weight at roof and floor level of 140 kips and 180 kips, respectively. It is assumed that 25% of seismic load is resisted by example shear wall as a full building analysis is not included in this example.
- Douglas Fir-Larch framing

- Existing construction documents and material test reports provide the following material strengths:
  - Light-gauge metal strap yield and tensile strength, f_y = 50,000 psi and f_u = 65,000 psi (per manufacturer's evaluation report)
  - Anchor rod yield strength, f_y = 36,000 psi
  - Allowable soil bearing pressure (D+L) = 2,500 psf

4.6.1.1 Application of ASCE 41-13 Methodology to Wood-Framed Shear Walls

After reviewing this example, it will become evident that the level of effort customarily needed for the equivalent new wood shear wall design in accordance with ASCE 7-10 is significantly less than that to perform an evaluation of an existing shear wall in accordance with ASCE 41-13. The primary reason for the difference in effort between the standards is in the overall methodology, which is discussed in detail in Chapter 2 of this *Guide*. ASCE 41-13 requires the user to fully understand the behavior of the building and the controlling mechanisms in the entire load path in order to properly evaluate each component, whereas ASCE 7-10 contains prescriptive load and detailing requirements for each component. This results in significantly more detailed analysis for ASCE 41-13 to follow the mechanisms in each component along the entire load path.

In order to assess each mechanism using ASCE 41-13 provisions, the material capacity of each component that makes up a connection assembly must be determined, but that information is not always readily available from manufacturer's published catalogs and evaluation reports for wood connection hardware. The load capacities listed in these documents are based on numerous load and deflection test criteria, and only the governing values are typically listed. In order to not make conservative assumptions in ASCE 41-13 and to best estimate the various component strengths, it is necessary to know the values of each test criteria. If manufacturers of connection hardware can provide these values, it will help simplify the rigorous analysis and minimize any conservative assumptions.

There is also a disconnect between the evaluation of components that are intended to remain essentially elastic in ASCE 41-13 as compared to the design of those elements in ASCE 7-10. For wood frame shear wall construction, ASCE 7-10 does not require any components, such as connection hardware, to be designed for an amplified design load using an overstrength factor (i.e., force-controlled) except for beam or column elements supporting discontinuous shear walls. ASCE 41-13 diverges from this approach and requires connection hardware (among other components) to be treated as force-controlled. As a result, connection hardware designed in accordance with ASCE 7-10 without an amplified design load may fail the ASCE 41-13 force-controlled component evaluation.

4.6.2 Spectral Response Acceleration Parameters

The overturning calculation in ASCE 41-13 § 7.2.8.1 requires the pseudo seismic forces to be calculated in accordance with ASCE 41-13 § 7.4.1.3.1 and requires the parameters, S_{XS} and S_{X1}, to be determined for both the BSE-1E and BSE-2E Seismic Hazard Levels. These parameters can be obtained using the tools presented in Chapter 3 of this *Example Application Guide* and are as follows for this site:

$S_{XS,BSE\text{-}2E} = 0.651\text{g}$

$S_{X1,BSE\text{-}2E} = 0.400\text{g}$

$S_{XS,BSE\text{-}1E} = 0.429\text{g}$

$S_{X1,BSE\text{-}1E} = 0.284\text{g}$

The Level of Seismicity is determined in accordance with ASCE 41-13 Table 2-5 and is always based on the BSE-1N spectral response parameters as calculated below.

$S_{DS,BSE\text{-}1N} = (2/3)\,F_aS_{S,BSE\text{-}2N} = (2/3)(1.26)(0.675\text{g}) = 0.567\text{g} > 0.5\text{g}$

$S_{D1,BSE\text{-}1N} = (2/3)\,F_vS_{1,BSE\text{-}2N} = (2/3)(1.813)(0.293\text{g}) = 0.354\text{g} > 0.2\text{g}$

Per ASCE 41-13 Table 2-5, the Level of Seismicity is High.

4.6.3 Pseudo Seismic Force on the Wall and Seismic Force at Each Level

The linear static procedure utilizes the pseudo seismic force, V, in ASCE 41-13 § 7.4.1.3 to calculate the seismic demands on elements. See Section 4.3.2 of this *Guide* for a more detailed example of determining the pseudo seismic force.

Pseudo seismic forces will be computed for both the Life Safety and Collapse Prevention Performance Levels for the BSE-1E and BSE-2E Seismic Hazard Levels, respectively.

$V = C_1C_2C_mS_aW$ (ASCE 41-13 Eq. 7-21)

where:

C_1 and C_2:

In this example, the simplified alternate method will be used to determine the combined factors C_1C_2 per ASCE 41-13 Table 7-3. This table requires the building fundamental period, T, to be calculated in accordance with ASCE 41-13 § 7.4.1.2. For this example, the empirical period formulation in Method 2 in ASCE 41-13 § 7.4.1.2.2 will be used as follows:

$$T = C_t h_n^{\beta} = (0.020)(20 \text{ ft})^{0.75} = 0.19 \text{ seconds}$$

where:

C_t = 0.020 for all other framing systems

h_n = height above the base to the roof level = 20 ft

β = 0.75 for all other framing systems

The selection of the combined factors C_1C_2 per ASCE 41-13 Table 7-3 also requires the determination of $m_{\max}$, which is the largest m-factor for all primary elements of the building in the direction under consideration. Since the primary mechanism is yielding of the plywood shear wall, $m_{\max}$ will most likely be the m-factor for the wood structural panel sheathing per ASCE 41-13 Table 12-3 as follows:

$m_{\max LS}$ = 3.8 for Life Safety Performance Objective

$m_{\max CP}$ = 4.5 for Collapse Prevention Performance Objective

The value for combined factors C_1C_2 per ASCE 41-13 Table 7-3 for a fundamental period of 0.19 seconds and $2 \le m_{\max} \le 6$ is as follows:

C_1C_2 = 1.4

This example only focuses on the evaluation of overturning on one shear wall in a larger structure. If a full building assessment were performed, validation of the C_1 and C_2 factors should be done in accordance with ASCE 41-13 § 7.4.1.3.1 considering the actual demand-capacity ratios of the controlling components through an iterative process.

C_m:

C_m is obtained per Table 7-4 of ASCE 41-13.

C_m = 1.0 for all other systems and two stories in height

S_a:

S_a is determined per ASCE 41-13 § 2.4. In accordance with ASCE 41-13 § 7.2.3.6, the example is based on 5% damping; however, 10% damping would be permitted if qualifying cross walls were present. Since this structure is a shear wall building and short in height, the period of the building most likely occurs on the constant acceleration portion of the response spectrum.

$S_{a,BSE\text{-}2E}$:

$$T_{S,BSE\text{-}2E} = \frac{S_{X1,BSE-2E}}{S_{XS,BSE-2E}} \qquad \text{(ASCE 41-13 Eq. 2-9)}$$

$$= \frac{0.400\text{g}}{0.651\text{g}} = 0.61 \text{ seconds}$$

$$T_{0,BSE\text{-}2E} = 0.2T_{S,BSE\text{-}2E} \qquad \text{(ASCE 41-13 Eq. 2-10)}$$

$$= 0.2\ (0.61 \text{ seconds}) = 0.12 \text{ seconds}$$

Since $T_{0,BSE\text{-}2E} < T < T_{S,BSE\text{-}2E}$ = 0.12 < 0.19 < 0.61 seconds, then:

$$S_{a,BES\text{-}2E} = \frac{S_{XS,BSE-2E}}{B_1} = \frac{0.651\text{g}}{1.0} = 0.651\text{g} \qquad \text{(ASCE 41-13 Eq. 2-6)}$$

$$B_1 = \frac{4}{\left[5.6 - ln\left(100\beta\right)\right]} \qquad \text{(ASCE 41-13 Eq. 2-11)}$$

$$= \frac{4}{\left\{5.6 - ln\left[100\left(0.05\right)\right]\right\}} = 1.0$$

$S_{a,BSE\text{-}1E}$:

$$S_{a,BES\text{-}1E} = \frac{0.429\text{g}}{1.0} = 0.429\text{g}$$

W:

W is the effective seismic weight of the building as given in Section 4.6.1 above.

$$W = W_R + W_2 = 140 \text{ kips} + 180 \text{ kips} = 320 \text{ kips}$$

Therefore,

$$V_{BSE\text{-}2E} = C_1C_2C_mS_{a,BSE\text{-}2E}W = 1.4(1.0)(0.651\text{g})W = 0.911W = 292 \text{ kips}$$

$$V_{BSE\text{-}1E} = C_1C_2C_mS_{a,BSE\text{-}1E}W = 1.4(1.0)(0.429\text{g})W = 0.601W = 192 \text{ kips}$$

Pseudo Seismic Force to Each Level

The pseudo seismic force is vertically distributed to each level in accordance with ASCE 41-13 § 7.4.1.3.2:

$$F_x = C_{vx} V \qquad \text{(ASCE 41-13 Eq. 7-24)}$$

where:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(ASCE 41-13 Eq. 7-25)}$$

w = effective seismic weight at the level

h = height of level above the base

k = 1.0 since $T \leq 0.5$ seconds

The resulting pseudo seismic force at each level and the 25% of the seismic forces resisted by the wall being evaluated, as given in the problem statement, are summarized in Table 4-3 and Table 4-4.

Table 4-3 BSE-2E Seismic Forces, Given *V* = 292 kips

Level	w_x (kips)	h_x (feet)	$w_x h_x$ (k-ft)	C_{vx}	F_x (kips)	$0.25F_x$ (kips) *
Roof	140	20	2,800	0.609	178	44.5
Second	180	10	1,800	0.391	114	28.5
Total	320	-	4,600	1.000	292	73.0

* Per problem statement in Section 4.6.1, 25% of the seismic forces are resisted by the wall being evaluated.

Table 4-4 BSE-1E Seismic Forces, Given *V* = 192 kips

Level	w_x (kips)	h_x (feet)	$w_x h_x$ (k-ft)	C_{vx}	F_x (kips)	$0.25F_x$ (kips) *
Roof	140	20	2,800	0.609	117	29.2
Second	180	10	1,800	0.391	75	18.8
Total	320	-	4,600	1.000	192	48.0

* Per problem statement in Section 4.6.1, 25% of the seismic forces are resisted by the wall being evaluated.

Evaluate Shear Wall Strength at Each Level

This example focuses solely on evaluating the overturning on a shear wall; however, to provide a more complete overview of an evaluation, this section will also evaluate the shear wall shear strength at each level. The shear walls are sheathed with plywood sheathing on one face and gypsum board on the other. In accordance with ASCE 41-13 § 12.4.1, where dissimilar wall sheathing may exist on opposite sides of a wall, the weaker sheathing is ignored, and the shear is analyzed based solely on the sheathing with the greater capacity. In this example, only the plywood sheathing is permitted to be considered.

The shear wall shear strengths are evaluated as deformation-controlled actions in accordance with ASCE 41-13 § 12.4.4.6.3. The acceptance criteria for components being evaluated for deformation-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.2.1, as follows:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

where:

m = component capacity modification factor from ASCE 41-13 Table 12-3 for the entry “Wood structural panel sheathing or siding” for Collapse Prevention Performance Level = 4.5 and for Life Safety Performance Level = 3.8.

κ = knowledge factor in accordance with ASCE 41-13 § 6.2.4, which for this example, will be assumed equal to 1.0 as the material strengths were determined based on existing drawings and material test reports as indicated in the problem statement.

Q_{CE} = The expected shear strength of the shear wall is determined in accordance with ASCE 41-13 § 12.4.4.6.2 based on 1.5 times the load and resistance factor design shear wall capacities in accordance with SDPWS-2008, *Special Design Provisions for Wind and Seismic Standard with Commentary* (AWC, 2008), Table 4.3A with a resistance factor, ϕ, equal to 1.0. Per Figure 4-11 and Section 4.6.1, the wall is sheathed with 15/32-inch wood structural panels, Structural I, with 10d at 6 inches on center edge nailing at the second level and 10d at 4 inches on center edge nailing at the first level. The nominal unit shear capacity per SDPWS-2008 Table 4.3A is 680 lb/ft for edge nailing at 6 inches on center and 1,020 lb/ft for edge nailing at 4 inches on center. The shear wall length is 15 feet. The expected shear strength of the shear walls are as follows:

$$Q_{CE\,\mathrm{Level2}} = 1.5(680\text{ lb/ft})(15\text{ ft})/1{,}000\text{ lb/kip} = 15.3\text{ kips}$$

$$Q_{CE\,\mathrm{Level1}} = 1.5(1{,}020\text{ lb/ft})(15\text{ ft})/1{,}000\text{ lb/kip} = 23.0\text{ kips}$$

Q_{UD} = Q_E = story shears per Table 4-3

$$Q_{E\,\mathrm{Level2}} = V_{\mathrm{Level2}} = 44.5\text{ kips}$$

$$Q_{E\,\mathrm{Level1}} = V_{\mathrm{Level1}} = 73.0\text{ kips}$$

Check the shear strength of the shear walls for deformation-controlled action:

$$m\kappa Q_{CE} > Q_{UD}$$

$m\kappa Q_{CE\ \text{Level2}} = 4.5(1.0)(15.3\text{ kips}) = 68.9\text{ kips}$

68.9 kips > 44.5 kips, therefore, the shear at Level 2 is satisfactory

$m\kappa Q_{CE\ \text{Level1}} = 4.5(1.0)(23.0\text{ kips}) = 103.5\text{ kips}$

103.5 kips > 73.0 kips, therefore, the shear at Level 1 is satisfactory

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the reduction in the Level 2 shear from the BSE-2E to the BSE-1E level is 66% (29.2/44.5), which is much less than the reduction of the m-factor from Collapse Prevention to Life Safety of 84% (3.8/4.5).

4.6.4 Evaluation of Overturning and Strap at the Second Floor

The shear wall is evaluated to resist overturning effects caused by seismic loads using the linear static procedure in accordance with ASCE 41-13 § 7.2.8.1. The equations in this section evaluate the stabilizing effects due to gravity loads alone to resist overturning. If gravity loads alone cannot resist the overturning demands to satisfy these equations, then positive attachment shall be provided and the attachment components evaluated as deformation-controlled or force-controlled elements. In the case of wood connection and connectors, in accordance with ASCE 41-13 § 12.3.3.1, connectors that link wood-to-wood or wood-to-metal (i.e., nails) are considered deformation-controlled elements, and the body of connection hardware (i.e., strap) is considered a force-controlled element.

The example below will first evaluate the resistance to overturning by the stabilizing effects of gravity loads alone, then if not satisfied, evaluate the straps that provide overturning resistance at each end of the wall. The BSE-2E Seismic Hazard Level and Collapse Prevention Performance Level will be evaluated first as that typically governs the evaluation. The BSE-1E Seismic Hazard Level and Life Safety Performance Level will be checked at the end of each subsection.

4.6.4.1 Overturning Resisted by the Stabilizing Effects of Gravity Loads Alone at the Second Floor

> **Useful Tip**
>
> When evaluating overturning resisted by the stabilizing effects of gravity loads alone per ASCE 41-13 § 7.2.8.1, ASCE 41-13 Equation 7-6 will typically control and result in less overturning demand at each level than ASCE 41-13 Equation 7-5.

ASCE 41-13 § 7.2.8.1 permits two separate approaches to evaluate overturning, a standard approach and an alternate approach. The standard approach evaluates overturning as a force-controlled action and attempts to limit uplift and inelastic deformation by reducing the overturning demand with the *J*-factor and is defined in ASCE 41-13 Equation 7-5. The alternate approach evaluates overturning similar to a deformation-controlled action by reducing the overturning demand with a ductility factor, μ_{OT}, which is typically much larger than the J-factor and may result in significant uplift and inelastic deformation. The alternate method is defined in ASCE 41-13 Equation 7-6. Where the alternate method is used, it may result is significant deformation and uplift between levels and the evaluation should consider the deformation compatibility of the elements at that interface. The alternate approach is comparable to the method used in ASCE 7-10 for new building design.

The overturning resisted by the stabilizing effects of gravity loads alone are evaluated per ASCE 41-13 § 7.2.8.1 with the standard approach, where the seismic overturning moment, M_{OT}, is resisted by the stabilizing moment due to gravity loads, M_{ST}. Note that ASCE 41-13 Equation 7-5 does not require a 0.9 factor to be applied to M_{ST}, whereas, ASCE 41-13 Equation 7-6 does.

$$M_{ST} > M_{OT}/(C_1C_2J) \qquad \text{(ASCE 41-13 Eq. 7-5)}$$

M_{OT} $= F_{\text{roof}}\,h_{\text{roof}}$ = (44.5 kips)(10 ft) = 445 kip-ft

C_1C_2 = 1.4 per Section 4.6.3 of this *Guide*

J = 2.0, per alternate method in ASCE 41-13 § 7.5.2.1.2(2) for the High Seismicity Level. Utilizing a $J = 2.0$ requires verification that the forces being delivered to the force-controlled components are coming from yielding elements (e.g., yielding shear wall or yielding diaphragm) and not from elastic response of the system; otherwise, the loads to the force-controlled components will be underestimated. Hence, the last paragraph in ASCE 41-13 § 7.5.2.1.2 requires $J = 1.0$ if the components delivering the load will remain elastic. In Sections 4.6.4 and 4.6.5 of this *Guide,* it is demonstrated that the limit-state of shear wall yielding governs, therefore a $J = 2.0$ may be assumed in this section. If a full building assessment were performed, J should be determined using the demand-capacity ratios of the components.

M_{ST} $= Q_D L_w^2/2$ = 200 lb/ft(15 ft)^2/2(1,000 lb/kip)
= 22.5 kip-ft

$M_{OT}/(C_1C_2J)$ = (445 kip-ft)/[(1.4)(2)] = 159 kip-ft

M_{ST} = 22.5 kip-ft < $M_{OT}/(C_1C_2J)$ = 159 kip-ft

Therefore, gravity alone does not resist seismic overturning using the standard approach.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the seismic overturning moment, M_{OT}, is larger for BSE-2E, and the resisting stabilizing moment due to gravity loads, M_{ST}, is the same for both evaluations.

The overturning moment resisted by the stabilizing effects of gravity loads alone is evaluated using the alternate approach in ASCE 41-13 § 7.2.8.1.

$0.9M_{ST} > M_{OT}/(C_1C_2\mu_{OT})$ (ASCE 41-13 Eq. 7-6)

μ_{OT} = 10 for Collapse Prevention per ASCE 41-13 § 7.2.8.1

$0.9M_{ST}$ = 0.9(22.5 kip-ft) = 20.3 kip-ft

$M_{OT}/(C_1C_2\mu_{OT})$ = (445 kip-ft)/[(1.4)(10)] = 31.8 kip-ft

20.3 kip-ft < 31.8 kip-ft

Therefore, gravity alone does not resist seismic overturning using the alternate approach.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the reduction in the roof force, F_r, from the BSE-2E to the BSE-1E level is 66% (29.2/44.5), which is much less than the reduction of the overturning ductility factor, μ_{OT}, from Collapse Prevention to Life Safety of 80% (8/10).

4.6.4.2 Strap Providing Overturning Resistance

Since the requirements for resistance to overturning by the stabilizing effects of gravity loads alone were not satisfied above, the straps that provide overturning resistance at the ends of each wall are evaluated. The straps are 14 gauge × 1-1/4-inch wide with 13-10d nails each end of the strap (26 total nails), see Figure 4-12. As indicated above, in accordance with ASCE 41-13 § 12.3.3.1, the behavior of nail fasteners is considered deformation-controlled actions, and the behavior of a metal strap is considered a force-controlled action.

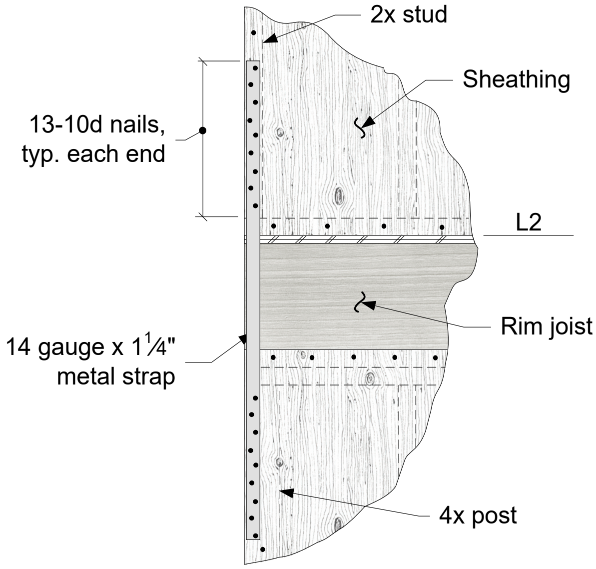


Figure 4-12 Detail of hold-down strap at second floor.

4.6.4.3 Nails in Strap

The acceptance criteria for nails being evaluated for deformation-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.2.1, as follows:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

where:

m = component capacity modification factor from ASCE 41-13 Table 12-3 for the entry "Nails – 8d and larger – metal to wood" for Collapse Prevention Performance Level = 6.0

κ = knowledge factor in accordance with ASCE 41-13 § 6.2.4, which for this example, will be assumed equal to 1.0 as the material strengths were determined based on existing drawings and material test reports as indicated in the problem statement

Q_{CE} = expected component strength per ASCE 41-13 § 12.3.2.2.1 is permitted to be calculated based on 1.5 times the load and resistance factor design procedures in ANSI/AWC NDS-2012, *National Design Specification for Wood Construction* (AWC, 2012) with a resistance factor, ϕ, taken equal to 1.0

Z = 119 lbs per NDS-2012 Table 11P for Douglas Fir-Larch, 14 gauge side plate, and 10d common nail

Z' per NDS-2012 Table 10.3.1: All adjustment factors = 1.0 and K_F = 3.32. Per NDS-2012 Appendix N, λ = 1.0 for load combinations with seismic.

> **Useful Tip**
>
> The load combinations in ASCE 41-13 do not contain a vertical seismic component when evaluating elements, such as $0.2S_{XS}$, commonly used for the design of new buildings within ASCE 7-10, except as specifically required in ASCE 41-13 § 7.2.5.2 for cantilever elements or highly stressed gravity load-carrying elements.

$Z' = ZK_F\phi\lambda$ = 119 lb(3.32)(1.0)(1.0) = 395 lb

Q_{CE}= 1.5Z'(# of nails) = 1.5(395 lb)(13)/1000 lb/kip = 7.70 kips

Q_{UD} is the combination of gravity and seismic loads, as follows:

$$Q_{UD} = Q_G \pm Q_E \qquad \text{(ASCE 41-13 Eq. 7-34)}$$

where, per the load combinations in ASCE 41-13 § 7.2.2:

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad \text{(ASCE 41-13 Eq. 7-1)}$$

and:

$$Q_G = 0.9Q_D \qquad \text{(ASCE 41-13 Eq. 7-2)}$$

$$Q_E = F_{\text{roof}}$$

The governing gravity load case for overturning will be $Q_G = 0.9Q_D$.

Moments are summed about the centerline of the end stud of the shear wall at the second floor level with the center-to-center post spacing of L_1 = 14 ft 10 ½ in. = 14.9 ft and length of wall of L_w = 15 ft per Figure 4-11, as follows:

∑ Moments:

$L_1Q_{UD} + 0.9Q_DL_wL_1/2 - F_{\text{roof}}h_{\text{roof}} = 0$

(14.9 ft)Q_{UD} + 0.9(200 lb/ft)(15 ft)(14.9 ft)/2(1000 lb/kip) – 44.5 kips(10 ft) = 0

Solving for Q_{UD} = 28.5 kips (demand on nails)

The nails in the strap are checked for deformation-controlled action:

$m\kappa Q_{CE} > Q_{UD}$

$m\kappa Q_{CE}$ = 6(1.0)(7.7 kips) = 46.2 kips

46.2 kips > 28.5 kips

Therefore, 13-10d nails at each end of the strap (26 total) are satisfactory.

BSE-1E Commentary: The BSE-1E evaluation has nearly the same result as the BSE-2E evaluation shown above since the reduction in the roof force, F_r, from the BSE-2E to the BSE-1E level is 66% (29.2 kips/44.5 kips), which is essentially the same as the reduction of the m-factor from Collapse Prevention to Life Safety of 67% (4/6).

4.6.4.4 Strap

The acceptance criteria for the straps being evaluated as force-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.2.2, as follows:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = knowledge factor per Section 4.6.4.3 of this *Guide*, equal to 1.0.

Q_{CL} = lower-bound component strength per ASCE 41-13 § 12.3.2.3.2 and ASCE 41-13 § 12.2.2.5.2 is based on lower bound strengths in accordance with ASCE 41-13 § 12.2.2.5 which is equal to 0.85 times the expected strength calculated using load and resistance factor design procedures in accordance with AISI S213-07/S1-09 (AISI, 2009b) and AISI S100-07/S2-10 (AISI, 2009a) with a resistance factor, ϕ, taken equal to 1.0.

Per AISI S100-07/S2-10 Section C2, the strap is checked for yielding on the gross section in accordance with AISI S100-07/S2-10 Equation C2-1 and rupture in net section in accordance with AISI S100-07/S2-10 Equation C2-2.

14 gauge × 1-1/4-inch strap with 2 staggered rows of 10d nail holes, F_y = 50 ksi, F_u = 65 ksi. The strap thickness is 0.0685 inches and the holes in the strap are 5/32-inch (0.156) diameter per the manufacturer's evaluation report.

Strap yielding is checked per AISI S100-07/S2-10 Equation C2-1 with ϕ = 1.0 and multiplying by 0.85 to convert to lower bound strength.

$= 0.85T_n = 0.85A_gF_y = 0.85(0.0685 \text{ in.})(1.25 \text{ in.})(50 \text{ ksi}) = 3.64$ kips

Strap rupture is checked per AISI S100-07/S2-10 Equation C2-2 with ϕ = 1.0 and multiplying by 0.85 to convert to lower bound strength. Since the nail holes are staggered, only one hole is included in the net area.

$= 0.85T_n = 0.85A_nF_u = 0.85(0.0685 \text{ in.})(1.25 \text{ in.} - 0.156 \text{ in.})(65\text{ksi})$

$= 4.14$ kips

Strap yielding governs, therefore Q_{CL} is as follows:

$Q_{CL} = 3.64$ kips

The demand, Q_{UF}, on the strap is determined in accordance with ASCE 41-13 § 7.5.2.1.2 and is the lesser of the following:

1. The demand using a limit-state analysis considering the expected strength of the shear wall per ASCE 41-13 § 7.5.2.1.2(1).
2. The demand using the alternate method per ASCE 41-13 § 7.5.2.1.2(2) in accordance with ASCE 41-13 Equation 7-35.
3. The demand using a limit-state analysis considering the expected strength of the nails in the strap per ASCE 41-13 § 7.5.2.1.2(1).

The expected strength of the shear wall at the second story was determined in Section 4.6.3 of this *Guide*.

$$Q_{CE\text{ Level2}} = 1.5(680 \text{ lb/ft})(15 \text{ ft})/1000 \text{ lb/kip} = 15.3 \text{ kips}$$

Compare the expected strength of the shear to the seismic force on the wall using the alternate method in accordance with ASCE 41-13 § 7.5.2.1.2(2) per ASCE 41-13 Equation 7-35. For this comparison, it is only necessary to calculate the last term of this equation, as follows:

$$\frac{Q_E}{C_1C_2J} = \frac{44.5 \text{ kips}}{(1.4)(2)} = 15.9 \text{ kips} > 15.3 \text{ kips}$$

$Q_E = F_{\text{roof}} = 44.5$ kips

$C_1C_2 = 1.4$, per Section 4.6.3

$J = 2.0$, per ASCE 41-13 § 7.5.2.1.2(2) for High Level of Seismicity

The expected strength of the wall based on the procedure in ASCE 41-13 § 12.4.4.6.2 is less than that computed with the alternate method; therefore, the example will use the expected strength of the wall. Applying the load combinations in ASCE 41-13 § 7.2.2 for gravity loads, the most critical load combination for overturning is:

$$Q_G = 0.9Q_D \qquad \text{(ASCE 41-13 Eq. 7-2)}$$

Summing moments about the corner of the shear wall at the second floor level and solving for the strap force, Q_{UF}, is as follows:

$\sum$ Moments:

$$L_1Q_{UF} + 0.9Q_DL_wL_1/2 - Q_{CE,\text{wall}}h_{\text{roof}} = 0$$

$$(14.9 \text{ ft})Q_{UF} + 0.9(200 \text{ lb/ft})(15 \text{ ft})(14.9 \text{ ft})/2(1000 \text{ lb/kip}) - 15.3 \text{ kips}(10 \text{ ft}) = 0$$

Solving for $Q_{UF} = 8.9$ kips (demand on strap due to limit state of the expected strength of the shear wall)

The limit state based on the expected strength of the nails in the strap was determined previously as Q_{CE} = 7.7 kips. This is less than the demand based on the limit state of the expected strength of the shear wall, and therefore governs.

$$Q_{UF} = 7.7 \text{ kips}$$

The strap is checked for force-controlled action:

$$\kappa Q_{CL} > Q_{UF}$$

$$\kappa Q_{CL} = (1.0)(3.6 \text{ kips}) = 3.6 \text{ kips}$$

$$3.6 \text{ kips} < 7.7 \text{ kips}$$

Therefore, the strap is inadequate.

BSE-1E Commentary: The BSE-1E evaluation would have the same results as the BSE-2E evaluation shown above since the governing demand on the strap would be unchanged since it was the limit state based on the expected strength of the nails in the strap and the capacity of the strap would also be unchanged as it is based on the lower bound yield strength of the strap.

This example illustrates a key difference between the evaluation of existing buildings in ASCE 41-13 and the design provisions for new buildings in ASCE 7-10 as they relate to light-frame wood construction. ASCE 7-10 does not require overturning straps in wood light-frame shear walls to be designed for amplified seismic loads using the overstrength factor, Ω_0, which can result in the straps yielding and deforming. This would be similar to treating the straps as deformation controlled in ASCE 41-13. However, ASCE 41-13 instead requires tensile loads on metal straps to be evaluated as a force-controlled action, which requires them to remain essentially elastic. This discrepancy between the two standards explains why straps and hold-down devices in light-frame shear walls may often fail the ASCE 41-13 evaluation.

4.6.5 Overturning and Hold-Down at the First Floor

The shear wall will be evaluated to resist overturning effects caused by seismic loads using the same method outlined in Section 4.6.4 above. The connectors that link wood-to-wood or wood-to-metal are considered deformation-controlled actions, and the body of connection hardware is considered a force-controlled action.

The example below will first evaluate the resistance to overturning by the stabilizing effects of gravity loads alone, then if not satisfied, evaluate the hold-down and post that provide overturning resistance at each end of the wall. The BSE-2E Seismic Hazard Level and Collapse Prevention Performance Level will be evaluated first as that typically governs the evaluation. The BSE-1E Seismic Hazard Level and Life Safety Performance Level will be checked at the end of the section.

4.6.5.1 Overturning Resisted by the Stabilizing Effects of Gravity Loads Alone at the First Floor

The overturning resisted by the stabilizing effects of gravity loads alone are evaluated per ASCE 41-13 § 7.2.8.1 using the standard approach, where the seismic overturning moment, M_{OT}, is resisted by the stabilizing moment due to gravity loads, M_{ST}. See the Section 4.6.4.1 of this *Guide* for a discussion on the two approaches permitted in ASCE 41-13 § 7.2.8.1. For the purposes of this example, only the weight of the superimposed floor and roof dead loads are used to resist overturning; however, other sources of overturning resistance, such as the weight or vertical shear capacity of return walls, may be used.

$$M_{ST} > M_{OT}/(C_1C_2J) \qquad \text{(ASCE 41-13 Eq. 7-5)}$$

where:

M_{OT} $= F_{\text{roof}}(h_{\text{roof}} + h_2) + F_2\,h_2 = (44.5 \text{ kips})(10 \text{ ft} + 10 \text{ ft}) + (28.5 \text{ kips})(10 \text{ ft})$
$= 1{,}180$ kip-ft

C_1C_2 $= 1.4$ per Section 4.6.3 of this *Guide*

J $= 2$, per alternate method in ASCE 41-13 § 7.5.2.1.2(2) for High Seismicity (see Section 4.6.4 of this *Guide* for determination of Level of Seismicity)

$M_{ST} = Q_DL_w^2/2 = (200 \text{ lb/ft} + 300 \text{ lb/ft})(15 \text{ ft})^2/2(1000 \text{ lb/kip})$
$= 56.3$ kip-ft

$M_{OT}/(C_1C_2J) = (1{,}180 \text{ kip-ft})/[(1.4)(2)] = 421$ kip-ft

56.3 kip-ft < 421 kip-ft

Therefore, gravity alone does not resist seismic overturning using the standard approach.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the seismic overturning moment, M_{OT}, is larger for the BSE-2E level and the resisting stabilizing moment due to gravity loads, M_{ST}, is the same for both evaluations.

The overturning resisted by the stabilizing effects of gravity loads alone are evaluated with the alternate approach in ASCE 41-13 § 7.2.8.1.

$0.9M_{ST} > M_{OT}/(C_1C_2\mu_{OT})$ (ASCE 41-13 Eq. 7-6)

μ_{OT} = 10 for Collapse Prevention per ASCE 41-13 § 7.2.8.1

$0.9M_{ST}$ = 0.9(56.3 kip-ft) = 50.7 kip-ft

$M_{OT}/(C_1C_2\mu_{OT})$ = (1,180 kip-ft)/[(1.4)(10)] = 84.3 kip-ft

50.7 kip-ft < 84.3 kip-ft

Therefore, gravity alone does not resist seismic overturning using the alternate approach.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the reduction in the pseudo seismic force, V, from the BSE-2E to the BSE-1E level is 66% (192/292), which is much less than the reduction of the overturning ductility factor, μ_{OT}, from Collapse Prevention to Life Safety of 80% (8/10).

4.6.5.2 Hold-Down Providing Overturning Resistance

Since the requirements for the resistance to overturning by the stabilizing effects of gravity loads alone were not satisfied, the hold-down and post that provide overturning resistance at the ends of each wall are evaluated. The hold-downs are 7 gauge with three 7/8-inch diameter stud bolts and a 7/8-inch anchor bolt, as shown in Figure 4-13. The post is a 4×6 Douglas Fir-Larch in a 2×6 wall.

As indicated above, in accordance with ASCE 41-13 § 12.3.3.1, the bolt fasteners to the wood post are considered deformation-controlled actions and the hold-down body is considered a force-controlled action. As discussed in ASCE 41-13 § C12.3.3, net section fracture of the hold-down post and tear-out of the bolt group from the post are considered force-controlled actions.

4.6.5.3 Bolts in Hold-Down

The acceptance criteria for bolts being evaluated as deformation-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.2.1.

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

where:

m = Component capacity modification factor from ASCE 41-13 Table 12-3 for the entry "Machine Bolts—metal to wood" for Collapse Prevention Performance Level = 3.3

κ = Knowledge factor per Section 4.6.4.3 is equal to 1.0

Q_{CE} = Expected component strength as discussed in Section 4.6.4.3

= 1.5Z' (# of bolts)

Z' = $Z_{ll} C_g K_F \phi \lambda$

Z_{ll} = 1.93 kips per NDS-2012 online calculation tool for Douglas Fir-Larch, 7-gauge side plate, and $^7/_8$-inch diameter bolt (http:// www.awc.org/codes -standards/calculators-software/connection calc)

All adjustment factors are 1.0 except C_g = 0.98 per NDS-2012 Table 10.3.6C and K_F = 3.32. Per NDS-2012 Appendix N, λ = 1.0 for load combinations with seismic.

Z' = 1.93 kips(0.98)(3.32)(1.0)(1.0) = 6.28 kips

= 1.5Z' (# of bolts) = 1.5(6.28 kips)(3) = 28.3 kips

Q_{UD} is the combination of gravity and seismic loads per Section 4.6.4.3.

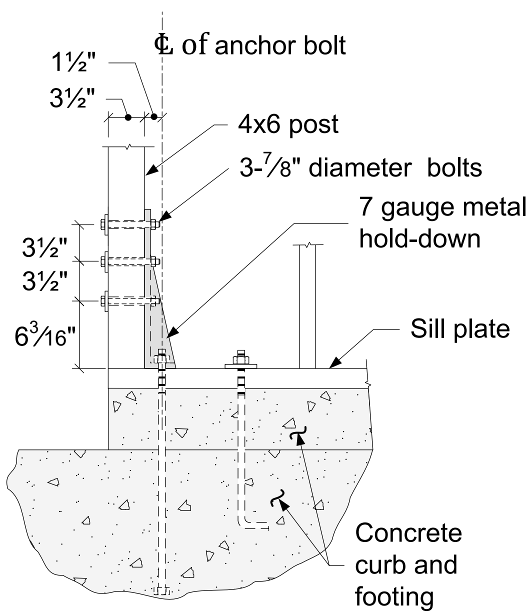


Figure 4-13 Detail of hold-down to post connection at foundation.

Moments are summed about the centerline of the end 4x post of the shear wall at the first floor level with L_2 equal to the distance between the centerline of the end 4× post to the center of the holdown anchor bolt per Figure 4-11 as follows:

$$L_2 = 15\,\text{ft} - \frac{3.5\,\text{in.}}{2(12\,\text{in./ft})} - \frac{3.5\,\text{in.}}{12\,\text{in./ft}} - \frac{1.5\,\text{in.}}{12\,\text{in./ft}} = 14\text{ ft }5\text{-}1/4\text{ in.} = 14.4\text{ ft}$$

$L_2 Q_{UD} + 0.9 Q_D L_w L/2 - F_{\text{roof}}(h_{\text{roof}} + h_2) - F_2 h_2 = 0$

$$(14.4\text{ ft})Q_{UD} + \frac{0.9\left(200\text{ lb/ft} + 300\text{ lb/ft}\right)\left(15\text{ ft}\right)\left(15\text{ ft} - \dfrac{3.5\text{ in.}}{12\text{ in./ft}}\right)}{2\left(1000\text{ lb/kip}\right)}$$

$- 44.5\text{ kips}(10\text{ ft} + 10\text{ ft}) - (28.5\text{ kips})(10\text{ ft}) = 0$

Solving yields Q_{UD} = 78.1 kips (demand on hold-down).

The bolts attaching hold-down to post are checked for deformation-controlled action:

$m\kappa Q_{CE} > Q_{UD}$

$m\kappa Q_{CE} = 3.3(1.0)(28.3\text{ kips}) = 93.4\text{ kips}$

93.4 kips > 78.1 kips

Therefore, 3-7/8-inch diameter bolts are satisfactory.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the reduction in the pseudo seismic force, V, from the BSE-2E to the BSE-1E level is 66% (192 kips/292 kips), which is much less than the reduction of the m-factor from Collapse Prevention to Life Safety of 85% (2.8/3.3).

4.6.5.4 Force-Controlled Components of the Hold-Down

The following components and connection checks for force-controlled actions are evaluated using linear analysis procedures as outlined in ASCE 41-13 § 7.5.2.2.2:

- Body of the hold-down per ASCE 41-13 § 12.3.3.1
- Net section on the hold-down post per ASCE 41-13 § C12.3.3
- Tear-out of bolt group per ASCE 41-13 § C12.3.3

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

κ = Knowledge factor per Section 4.6.4.3 of this *Guide*, equal to 1.0

Q_{CL} = Lower-bound strength per ASCE 41-13 § 12.2.2.5 is equal to 0.85 times the expected strength

4.6.5.5 Lower-Bound Strength of Hold-Down Body

The lower-bound strength of the body of prefabricated hold-downs can be determined a number of ways depending on the availability of manufacturer's test data of the device. The following are three common approaches used:

- **"Average ultimate test load" listed in an evaluation report or catalog:** When the "average ultimate test load" is listed for the hold-down device, this value is typically derived from testing the hold-down device on a steel jig. ASCE 41-13 § 12.2.2.5 permits the "average ultimate test load" to equal the expected strength and requires that it be further modified by multiplying by 0.85 to obtain the lower-bound strength.

- **"Allowable steel strength" listed in an evaluation report performed in accordance with ICC-ES AC 155, *Acceptance Criteria for Hold-downs (Tie-downs) Attached to Wood Members*, (ICC-ES, 2015b):** Current hold-down devices are typically evaluated in accordance with ICC-ES AC 155, which requires that hold-down devices be independently tested to report the strength of the steel hold-down separately from the assembly test that includes the wood post and fasteners. The allowable steel strength value listed in the report is based on allowable stress design. This value is derived from the ultimate test value divided by a factor of safety of 2.5. If the allowable value is reported, and the safety factor is known, then the allowable value is multiplied by the factor of safety to determine expected strength. Then, per ASCE 41-13 § 12.2.2.5, the expected strength is multiplied by 0.85 to obtain the lower-bound strength.

- **Only "allowable tension loads" listed in an evaluation report or manufacture's literature:** When the allowable tension load of the steel body of the hold-down cannot be isolated from the other limit states that determine the allowable tension capacity of a hold-down assembly, such as wood fastener failure or deflection limits, then a different approach must be taken as it is unknown what the governing limit state is for the listed allowable tension load. In this case, when only the allowable tension load is known for hold-downs and they employ fasteners in accordance with the NDS-2012, then an approach is taken based on conversion of published allowable values to load and resistance factor design (LRFD) values in accordance with NDS-2012. The published 133% or 160% allowable tension values for the hold-down are divided by 1.33 or 1.6, respectively, then multiplied by 3.32 (the format conversion factor, K_F, applicable for connections) to obtain the load and resistance factor design (LRFD) value with a $\phi = 1.0$, which is the expected strength. Then, per ASCE 41-13 § 12.2.2.5, the expected strength is multiplied by 0.85 to obtain the lower-bound strength.

When it can be shown that the hold-down published allowable tension load is based on the allowable design values per NDS-2012 for fasteners attaching the hold-down body to the wood post, then the hold-down body expected strength is equal to or greater than that provided by the fasteners and the expected strength of the hold-down body can be determined by multiplying the expected strength determined above by an additional 1.5 in accordance with ASCE 41-13 § 12.3.2.2.1.

For this example problem, the product is listed in a catalog with the "average ultimate test load" indicated, so the default expected strength value will be taken as the average ultimate test load per ASCE 41-13 § 12.2.2.5. The average ultimate test value is indicated as 28.7 kips in the product catalog for the 7 gauge × 3-7/8-inch diameter bolt hold-down.

Accordingly, Q_{CL} = 0.85(28.7 kips) = 24.4 kips (hold-down body).

4.6.5.6 Lower-Bound Strength of Net Section on the Hold-Down Post

The lower-bound strength per ASCE 41-13 § 12.3.2.3.1 and § 12.2.2.5 is equal to 0.85 of the expected strength, which is calculated using load and resistance factor design procedures in accordance with NDS-2012 with a resistance factor, ϕ, taken equal to 1.0. Net section of the hold-down post is determined in accordance with NDS-2012 § E.2 at the hold-down bolts.

The hold-down has 3-7/8-inch diameter bolts spaced at 3-1/2 inches on center with 6-3/16-inch end distance. The post is a 4×6 post with the hold-down bolted on the wide face and the hole diameter for the bolts is 15/16 (0.938) inch. The post is Douglas Fir Larch No. 1.

The net section on the post is checked per NDS-2012 Equation E.2-1 with ϕ = 1.0 and by multiplying by 0.85 to convert to lower bound strength.

$$Q_{CL} = 0.85Z_{NT}'$$

where:

$$Z'_{NT} = F_t' A_{\text{net}}$$

where:

$$F_t' = C_F F_t K_F \phi \lambda$$

where:

C_F = 1.2 per NDS-2012 Supplement Table 4A (all other adjustment factors = 1.0)

F_t = 675 psi per NDS-2012 Supplement Table 4A

K_F = 2.70 per NDS-2012 Table 4.3.1

ϕ = 1.0 per NDS-2012 Table 4.3.1

λ = 1.0 per NDS-2012 Table N3

F_t' = 1.2(675 psi)(2.70)(1.0)(1.0) = 2,187 psi

A_{net} = net section at bolt = (3.5 in.)(5.5 in. – 0.938 in.) = 16.0 in.^{2}

$Z'_{NT} = F_t' A_{\text{net}}$ = 2,187 psi(16.0 in.^{2})/1,000 lb/kip = 35.0 kips

Q_{CL} = 0.85(35.0 kips) = 29.8 kips (net section on the hold-down post)

4.6.5.7 Lower-Bound Strength of Tear-Out of Bolt Group

The lower-bound strength for tear-out of the bolt group will be calculated using the same requirements as for the net section on the hold-down post except NDS-2012 Section E.3 will be used.

The tear-out of the single row of bolts attaching the hold-down to the post are checked per NDS-2012 Equation E.3-2 with ϕ = 1.0 and multiplying by 0.85 to convert to lower bound strength.

$Q_{CL} = 0.85 Z_{RTi}'$

where:

$Z'_{RTi} = n_i F_v' t s_{\text{critical}}$

where:

n_i = number of fasteners in row = 3

F_v' = $C_x F_v K_F \phi \lambda$

where:

C_x = 1.0 (all adjustment factors = 1.0)

F_v = 180 psi per NDS-2012 Supplement Table 4A

K_F = 2.88 per NDS-2012 Table 4.3.1

ϕ = 1.0 per NDS-2012 Table 4.3.1

λ = 1.0 per NDS-2012 Table N3

F_v' = $C_x F_v K_F \phi \lambda$ = 1.0(180 psi)(2.88)(1.0)(1.0) = 518 psi

t = thickness of wood post = 3.5 in

s_{critical} = minimum spacing in row of bolts taken as lesser of end distance or bolt spacing = 3.5 in

$Z'_{RTi} = n_i F_v' t s_{\text{critical}}$ = 3(518 psi)(3.5 in.)(3.5 in.)/1000 lb/kip = 19.0 kips

Q_{CL} = 0.85(19.0 kips) = 16.2 kips (tear-out of bolt group)

4.6.5.8 Force-Controlled Components Not Being Evaluated in Hold-Down Example

This example does not evaluate the following items that should be part of a complete evaluation of the hold-down post:

- Crushing (compression perpendicular to grain) of the sill plate at post ends
- Axial compression on the post
- Combined axial tension and moment due to the eccentricity of the hold-down on the post

The evaluation of the effects of eccentric loading on the post due to the hold-down eccentricity has been the subject of past research. The plywood edge nailing to the post causes a stiffening effect that reduces the moment on the post. A detailed study on this stiffening effect with design recommendations for the hold-down post can be found in Pryor (2002).

4.6.5.9 Demand on Force-Controlled Components of the Hold-Down

The demand, Q_{UF}, on the hold-down is determined in accordance with ASCE 41-13 § 7.5.2.1.2 and is the lesser of the following:

- The demand using a limit-state analysis considering the expected strength of the shear wall per ASCE 41-13 § 7.5.2.1.2 Bullet (1).
- The demand using the alternate method per ASCE 41-13 § 7.5.2.1.2 Bullet (2) in accordance with ASCE 41-13 Equation 7-35.
- The demand using a limit-state analysis considering the expected strength of the bolts in the hold-down per ASCE 41-13 § 7.5.2.1.2 Bullet (1).

The expected strength of the shear wall at the first story was determined in Section 4.6.3 of this *Guide*.

$$Q_{CE\ \mathrm{Level1}} = 1.5(1020\ \mathrm{lb/ft})(15\ \mathrm{ft})/1000\ \mathrm{lb/kip} = 23.0\ \mathrm{kips}$$

The expected strength of the shear wall is compared to the seismic force on the wall using the alternate method in accordance with ASCE 41-13 § 7.5.2.1.2 Bullet (2), Equation 7-35. For this comparison, it is only necessary to calculate the last term of this equation, as follows:

$$\frac{Q_E}{C_1 C_2 J} = \frac{73\ \mathrm{kips}}{(1.4)(2)} = 26.1\ \mathrm{kips} > 23.0\ \mathrm{kips}$$

$$Q_E = F_{\mathrm{roof}} + F_2 = 44.5\ \mathrm{kips} + 28.5\ \mathrm{kips} = 73\ \mathrm{kips}$$

C_1C_2 = 1.4, per Section 4.6.3 of this *Guide*

J = 2.0, per ASCE 41-13 § 7.5.2.1.2 Bullet (2) for High Level of Seismicity

The expected strength of the wall is less than that computed with the alternate method; therefore, the example will use the expected strength of the wall. Applying the load combinations in ASCE 41-13 § 7.2.2 for gravity loads, the most critical load combination for overturning is:

$$Q_G = 0.9Q_D \qquad \text{(ASCE 41-13 Eq. 7-2)}$$

The base shear based on the expected strength of the wall will be vertically distributed to the levels depending on the relative demand-capacity ratios of each story. If the demand-capacity ratio of the first story is larger than second story, then the same distribution required by ASCE 41-13 § 7.4.1.3.2 as determined in Section 4.6.3 of this *Guide* will be used. This distribution is conservative for evaluating overturning loads as the default linear static procedure distribution results in an inverted triangular force distribution with higher story forces in the upper levels. However, if the demand-capacity ratio of the first story is less than the second story, then the distribution will be modified to apply the expected second story shear wall strength as the roof story force and the second floor story force will be the difference between the second and first story walls expected strengths.

The story shears per Table 4-3 are 44.5 kips and 73 kips for the second and first stories, respectively. The expected strengths of the walls per Section 4.6.3 of this *Guide* are 15.3 kips and 23 kips for the second and first stories, respectively. The resulting demand-capacity ratios for each story are 2.9 and 3.2 for the second and first stories, respectively. Since the demand-capacity ratio of the first story is larger than second story, then the lower story will yield first, hence the same distribution required by ASCE 41-13 § 7.4.1.3.2 as determined in Section 4.6.3 of this *Guide* will be used, and is as follows. The roof distribution is 0.609 of the base shear, and the second floor distribution is 0.391 of the base shear, where the base shear is the expected strength of the first story shear wall.

The moments are summed about the corner of the shear wall at the first floor level and solved for the hold-down force, Q_{UF}, as follows:

$$L_2Q_{UF} + 0.9Q_DL_wL/2 - 0.609Q_{CE\ \mathrm{Level1}}h_{\mathrm{roof}} - 0.391Q_{CE\ \mathrm{Level1}}h_2 = 0$$

$$(14.4\text{ ft})Q_{UF} + 0.9(200\text{ lb/ft} + 300\text{ lb/ft})(15\text{ ft})(14.7\text{ ft})/2(1000\text{ lb/kip}) - 0.609(23.0\text{ kips})(10\text{ ft} + 10\text{ft}) - 0.391(23.0\text{ kips})(10\text{ ft}) = 0$$

Solving for Q_{UF} yields 22.3 kips (demand on forced-controlled hold-down components due to limit state of expected strength of the shear wall).

The limit state based on the expected strength of the hold-down bolts was determined previously as Q_{CE} = 28.3 kips. This is more than the demand based on the limit state of the expected strength of the shear wall, and therefore does not govern.

Q_{UF}= 22.3 kips

4.6.5.10 Acceptance Criteria Summary of the Hold-Down Components

The acceptance criteria for the force-controlled components of the hold-down assembly are summarized as follows:

$\kappa Q_{CL} > Q_{UF}$ = 22.3 kips (ASCE 41-13 Eq. 7-37)

κQ_{CL} kips = 1.0(24.4 kips) = 24.4 kips (hold-down body)

κQ_{CL} kips = 1.0(29.8 kips) = 29.8 kips (net section on the hold-down post)

κQ_{CL} kips = 1.0(16.2 kips) = 16.2 kips (tear-out of bolt group)

24.4 kips > 22.3 kips, hold-down body is adequate

29.8 kips > 22.3 kips, net section on the hold-down post is adequate

16.2 kips < 22.3 kips, tear-out of bolt group is not adequate

Therefore, the hold-down components are not adequate due to the bolt tear-out.

BSE-1E Commentary: The BSE-1E evaluation would have the same results as the BSE-2E evaluation shown above since the governing demand on the hold-down components would be unchanged since it was the limit state based on the expected strength of the shear wall and the capacity of the hold-down components would also be unchanged as they are based on their lower bound yield strength.

This example illustrated a key difference between the evaluation of existing buildings in ASCE 41-13 and the design provisions for new buildings in ASCE 7-10 as it relates to light-frame wood construction overturning resistance, which is discussed in greater detail in the last paragraph of Section 4.6.4.4 of this *Guide*.

4.6.6 Hold-Down Anchor-to-Footing Connection

The hold-down anchor rod and its connection to the concrete footing are evaluated as force-controlled components per ASCE 41-13 § 10.3.6.1 where the lower bound strength equals the anchor strength in accordance with ACI 318-11, *Building Code Requirements for Structural Concrete* (ACI, 2011), Section D.3.3.4.4 with $\phi = 1.0$.

The evaluation of the anchor and the connection shall satisfy one of the design options stipulated in ACI 318-11 Section D.3.3.4.3. Where the design option in ACI 318-11 Section D.3.3.4.3 Bullet (b) is used to demonstrate a ductile yield mechanism, the applied load, Q_{UF}, is based the maximum action that can be developed with a limit-state analysis using the expected strength of the component delivering the load in accordance with ASCE 41-13 § 7.5.2.1.2(1). Example limit states are: the expected shear wall in-plane strength, foundation rocking, and hold-down-to-post fasteners. When the design option in ACI 318-11 Section D.3.3.4.3 Bullet (d) is used, the maximum design load is taken as Q_{UF} calculated in in accordance with ASCE 41-13 § 7.5.2.1.2, and no further amplification is required as Q_{UF} is considered to be equivalent to $\Omega_0 E$ loads stipulated in this section.

This example does not provide a detailed evaluation of the hold-down anchor and anchor connection in accordance with ACI 318-11. The tilt-up concrete example in Chapter 6 of this *Guide* provides a detailed concrete anchor example in accordance with ASCE 41-13 and ACI 318-11.

4.6.7 Overturning at the Foundation-Soil Interface

The foundation-soil interface will be evaluated to resist overturning effects caused by seismic loads per ASCE 41-13 Chapter 8 as indicated in ASCE 41-13 § 7.2.8. As with most low-rise wood framed buildings, the foundations are shallow spread footings, and the foundation flexibility is not included in the base computer model or seismic force-resisting analysis. Instead, the base of the building is considered fixed, and the foundations are assumed to be rigid. With this common approach, the foundation overturning analysis is performed in accordance with ASCE 41-13 § 8.4.2.3 *Shallow Footings Considered Rigid (Method 1)* and the acceptance criteria are per ASCE 41-13 § 8.4.2.3.2.1 for the fixed base analysis. See Chapter 5 of this *Guide* for a detailed discussion and application of ASCE 41-13 for the different foundation design and analysis methods. This section will only evaluate the overturning at the foundation-soil interface and will not evaluate the strength of the footing or sliding at the foundation-soil interface which would be required for a complete evaluation.

The foundation supporting the shear wall is 20 feet long by 3 feet wide with a 1 foot high curb. The allowable bearing pressure, q_{allow}, indicated on the existing building drawings for dead load plus live load is 2,500 psf. Figure 4-14 shown below illustrates the geometry and loading on the foundation. The Q_E values shown are for the BSE-2E seismic hazard for the Collapse Prevention assessment.

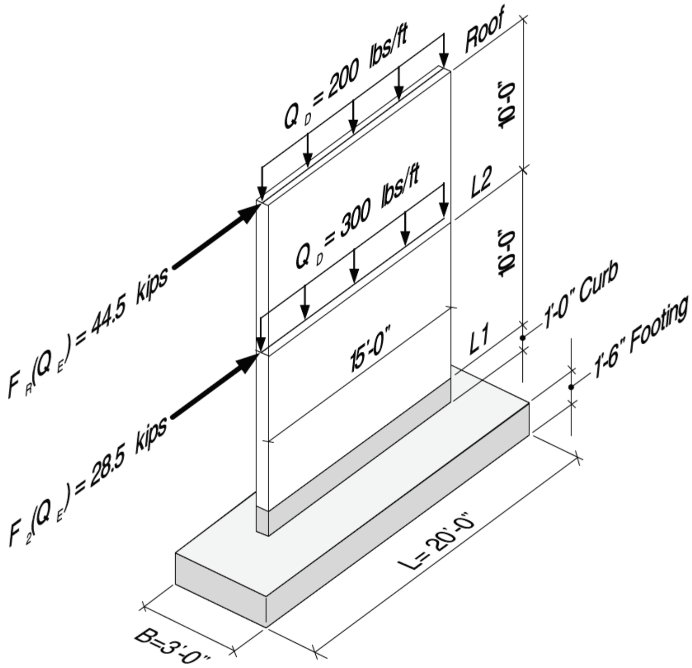


Figure 4-14 Isometric of two-story wood frame shear wall foundation.

Per ASCE § 8.4.2.3.2.1, the foundation soil is classified as deformation controlled and the *m*-factors for Life Safety and Collapse Prevention are 3.0 and 4.0, respectively. This section also permits the use of upper-bound soil capacities, which are equal to two times the expected bearing capacity, q_c, per ASCE 41-13 § 8.4.2, and the expected bearing capacity is equal to three times the allowable bearing pressure, q_{allow}, per ASCE 41-13 § 8.4.1.1 Bullet 1. The allowable bearing pressure, q_{allow}, is that specified for gravity load design (dead plus live loads) and does not include any allowable increase for short-term loading. The acceptance criteria are per ASCE 41-13 § 7.5.2.2. The component forces are calculated in accordance with ASCE 41-13 § 7.5.2.1.1.

Q_{UD} is the combination of gravity and seismic loads, as follows:

$$Q_{UD} = Q_G \pm Q_E \quad \text{(ASCE 41-13 Eq. 7-34)}$$

where, per the load combinations in ASCE 41-13 § 7.2.2, the most critical load combination for overturning is

$$Q_G = 0.9Q_D \qquad \text{(ASCE 41-13 Eq. 7-2)}$$

The vertical load on the soil due to dead load includes the superimposed gravity load at each level, the weight of the foundation and stem wall/curb, and superimposed soil (120 lb/cf) and slab above the top of the foundation (for simplicity, the example will assume 6 inches of soil only above the footing). For the purposes of this example, the weight of the wood frame wall and larger tributary gravity loading on the footing at the ground level from the slab-on-grade have been ignored. Other vertical loads, such as vertical shear capacity of return walls and foundations, may be used to resist overturning, but have not been used in this example.

$$\begin{aligned} P &= Q_D \\ &= [(15\text{ ft})(200\text{ lb/ft} + 300\text{ lb/ft}) + (20\text{ ft})(3\text{ ft})(1.5\text{ ft})(150\text{ lb/ft}^3) \\ &\quad + (15\text{ ft})(0.5\text{ ft})(1\text{ ft})(150\text{ lb/ft}^3) + (20\text{ ft})(3\text{ ft})(0.5\text{ ft})(120\text{ lb/ft}^3)] \\ &\quad /1000\text{ lb/kip} \\ &= 25.7\text{ kips} \end{aligned}$$

Useful Tip

When performing a fixed based rigid foundation analysis, as customarily done for the design of foundations supporting wood-framed structures, the acceptance criteria for evaluating overturning at the footing-soil interface in ASCE 41-13 § 8.4.2.3.2.1 permit the use of upper-bound soil capacities that are two times the expected bearing capacity. With the expected bearing capacity being equal to three times the allowable bearing pressure, this results in the upper-bound soil bearing capacity being equal to six times the allowable bearing pressure.

See Chapter 5 of this *Guide* for further discussion on the evaluation of foundations and the other modeling methods and their respective acceptance criteria.

The overturning moment, M_{OT}, will be taken about the bottom of the footing.

$$\begin{aligned} M_{OT} &= 44.5\text{ kips}(10\text{ ft} + 10\text{ ft} + 1\text{ ft} + 1.5\text{ ft}) + 28.5\text{ kips}(10\text{ ft} + 1\text{ ft} \\ &\quad + 1.5\text{ ft}) = 1{,}358\text{ kip-ft} \end{aligned}$$

The expected bearing capacity, q_c, per ASCE 41-13 § 8.4.1.1 Bullet (1) is:

$$q_c = 3q_{\text{allow}} = 3(2{,}500\text{ psf}/1000\text{ lb/kip}) = 7.5\text{ ksf} \qquad \text{(ASCE 41-13 Eq. 8-1)}$$

The critical contact area, A_c, is determined per ASCE 41-13 § 8.4.2.3. As indicated earlier in this section, the acceptance criteria in ASCE 41-13 § 8.4.2.3.2.1 for a fixed base rigid foundation analysis permit the use of the upper-bound soil capacity which is equal to two times the expected bearing capacity, q_c. The example applies the upper-bound soil capacity, $2q_c$, in the calculation of A_c, as follows:

$$A_c = P/2q_c = (25.7\text{ kips})/(2)(7.5\text{ ksf}) = 1.7\text{ ft}^2$$

The length, L_c, of the critical contact area is:

$$L_c = A_c/B = (1.7\text{ ft}^2)/(3\text{ ft}) = 0.6\text{ ft}$$

The resisting moment, M_R, is derived from the expected moment capacity, M_c, as follows:

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \qquad \text{(ASCE 41-13 Eq. 8-10)}$$

and by substituting the following equations into ASCE 41-13 Eq. 8-10:

$$q = \frac{P}{B_f L_f} = \text{vertical bearing pressure}$$

$$q_c = \frac{P}{L_c B_f} = \text{expected bearing capacity derived from } A_c$$

$P = 0.9P$ since $Q_G = 0.9Q_D$

after the algebraic substitution, the resulting formula is:

$$M_R = 0.9P\left(\frac{L}{2} - \frac{L_c}{2}\right) = 0.9(25.7 \text{ kips})\left(\frac{20 \text{ ft}}{2} - \frac{0.6 \text{ ft}}{2}\right) = 224 \text{ kip-ft}$$

The overturning at the soil-foundation interface is checked for deformation-controlled action per ASCE 41-13 § 7.5.2.2.1. The knowledge factor, κ, equals 1.0 since q_{allow} was taken from existing building drawings.

$m\kappa Q_{CE} > Q_{UD}$, where $Q_{CE} = M_R$ and $Q_{UD} = M_{OT}$

$m\kappa M_R = (4.0)(1.0)(224 \text{ kip-ft}) = 896 \text{ kip-ft}$

896 kip-ft < 1,358 kip-ft

Therefore, foundation size is inadequate for overturning.

BSE-1E Commentary: The BSE-1E evaluation would not govern over the BSE-2E evaluation shown above since the reduction in the pseudo seismic force, V, from the BSE-2E to the BSE-1E level is 66% (192/292), which is less than the reduction of the m-factor from Collapse Prevention to Life Safety of 75% (3/4).

The provisions in ASCE 41-13 for foundation overturning, where foundation flexibility is not included in the analysis of the structure (period, force distribution, etc.), are generally conservative. If the overturning on the foundation-soil interface were reevaluated with loads derived from a model that included foundation flexibility, then the provisions in ASCE 41-13 § 8.4.2.3.2.2 would be permitted with the *m*-factors from ASCE 41-13 Table 8-3. These *m*-factors can be significantly higher than those for the equivalent fixed-based analysis in ASCE 41-13 § 8.4.2.3.2.1. However, modelling of a wood framed building to include foundation flexibility is not commonly done, especially for flexible diaphragm buildings often computed with hand calculations.

Another approach in evaluating the overturning at the foundation-soil interface is to use limit-state analysis to limit the overturning demand. In Section 4.6.5.9 of this *Guide*, the expected strength of the shear wall, Q_{CE}, was determined to be 23 kips. This was distributed to the roof and floor level based on the elastic distribution, which resulted in the roof distribution being 0.609 of Q_{CE}, and the second floor distribution being 0.391 of Q_{CE}.

The revised overturning moment, M_{OT}, based on the expected strength of the wall is as follows:

$$M_{OT} = (0.609)(23 \text{ kips})(10 \text{ ft} + 10 \text{ ft} + 1 \text{ ft} + 1.5 \text{ ft}) + (0.391)(23 \text{ kips})(10 \text{ ft} + 1 \text{ ft} + 1.5 \text{ ft}) = 428 \text{ kip-ft}$$

The resisting moment, M_R, is unchanged and is 224 kip-ft. Since the resisting moment is less than the overturning due to the expected capacity of the wall (224 kip-ft < 428 kip-ft), foundation rocking mechanism will control. Note that an *m*-factor is not applied to the resisting moment in this case since we are assessing which mechanism will control. As indicated above, the foundation size is inadequate for overturning. The foundation would need to be rehabilitated or a more detailed analysis using a flexible base model will need to be performed where higher *m*-factors are permitted.

4.7 Out-of-Plane Strength of Walls (ASCE 41-13 § 7.2.11.2)

Example Summary
Performance Objective: BPOE
Location: Oakland, California
Level of Seismicity: High
Risk Category: II
Reference Document:
TMS 402-11

Walls are required to have adequate out-of-plane strength to resist horizontal seismic inertial loads to span between points of support to prevent walls from becoming unstable. This is most critical for buildings with heavy walls constructed of masonry or concrete since the seismic load will typically govern over the wind loading in areas of high seismicity. The provisions in ASCE 41-13 § 7.2.11.2 require different forces depending on the Structural Performance Level being sought. When evaluating a building for the typical two-level assessment, both Performance Levels and the respective Seismic Hazard Levels are required to be evaluated, and the most stringent result governs.

Although this section focuses on the out-of-plane strength of masonry and concrete walls, the more problematic deficiency in these types of structures is the anchorage of the wall and the development of that anchorage into the supporting diaphragms. This topic is discussed in detail in the tilt-up example in Chapter 6 of this *Guide*.

4.7.1 Overview

This example illustrates the procedure and calculations to check the out-of-plane wall strength of a one-story reinforced concrete masonry bearing wall in accordance with ASCE 41-13 § 7.2.11.2 and ASCE 41-13 § 11.3.5. See Figure 4-15.

The example will also illustrate the procedure and calculations to check the out-of-plane strength of the parapet in accordance with ASCE 41-13 § 13.6.5 treating the parapet as an architectural component.

This example problem analyzes a reinforced masonry wall. For an example application of out-of-plane stability for unreinforced masonry (URM) walls, see the full URM building example in Chapter 12 of this *Guide*.

This example illustrates the Tier 3 procedure that requires a two-level assessment, though this building would qualify for the Tier 2 procedure and require only a single-level assessment to the BSE-1E Seismic Hazard Level. Per ASCE 41-13 Table 2-1 for Risk Category II, for the Tier 3 procedure, the Structural Performance Levels required to be evaluated are Life Safety at the BSE-1E Seismic Hazard Level and Collapse Prevention at the BSE-2E Seismic Hazard Level. This requires a dual evaluation of the wall to be computed for both the Life Safety and Collapse Prevention Performance Levels for the BSE-1E and BSE-2E Seismic Hazard Levels, respectively.

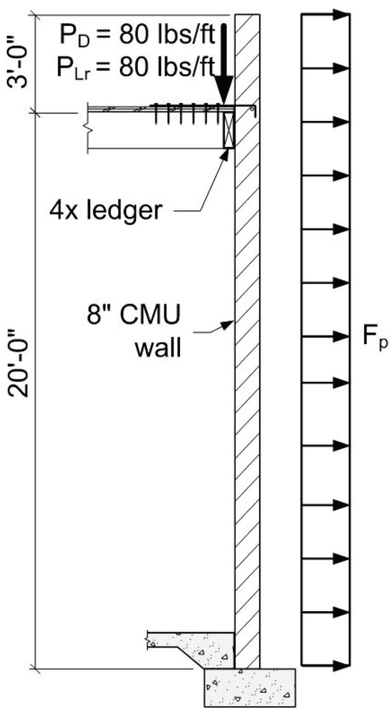


Figure 4-15 Out-of-plane loading on CMU bearing wall.

The analysis of the masonry wall will be performed on a representative 1-foot width of wall length, assuming a uniformly distributed out-of-plane load and no openings in the vicinity of the section being checked.

The following information is given:

- Basic Performance Objective for Existing Buildings (BPOE)
- Risk Category II
- Location: Oakland, California
- Site Class D
- 8-inch nominal concrete masonry wall
- Medium-weight masonry, fully grouted
- Vertical reinforcement = #4 at 16 inches on center at centerline of wall (A_s = 0.15 in.^{2}/ft)
- Existing construction documents and material test reports provide the following nominal strengths which are in turn used as the lower bound material strengths:
    - Masonry compressive strength, f'_{mLB} = 1,500 psi
    - Reinforcing steel yield strength, f_y = 60,000 psi
    - Type S mortar

4.7.2 Determine the Spectral Response Acceleration Parameters

The out-of-plane force calculation in ASCE 41-13 § 7.2.11.2 requires the parameter spectral response acceleration at short periods for the selected Seismic Hazard Level and damping, S_{XS}, to be adjusted for site class. This parameter will need to be determined for both the BSE-1E and BSE-2E Seismic Hazard Levels using the online tools described in Chapter 3 of this *Guide*. It should be noted that at this site, located in Oakland, California, both the BSE-2E and the BSE-1E seismic hazards are capped by the BSE-2N and the BSE-1N seismic hazards, respectively, meaning that the BPOE is the same as the BPON. These parameters are as follows:

$S_{XS,BSE\text{-}2E}$ = 1.699g

$S_{XS,BSE\text{-}1E}$ = 1.133g

4.7.3 Calculate the Out-of-Plane Wall Force per Unit Area, F_p

The provisions for the evaluation of out-of-plane seismic loads for masonry walls are contained in both ASCE 41-13 § 7.2.11.2 and § 11.3.5. The out-of-plane seismic forces are determined in ASCE 41-13 § 7.2.11.2, while the strength and acceptance criteria are contained in ASCE 41-13 § 11.3.5. The out-of-plane seismic loads will be computed for both the Life Safety and Collapse Prevention Performance Levels for the BSE-1E and BSE-2E

Seismic Hazard Levels, respectively, and the most severe loading will govern. The out-of-plane force is determined as follows:

$$F_p = 0.4 S_{XS} \chi W_p \qquad \text{(ASCE 41-13 Eq. 7-13)}$$

F_p shall not be less than $0.1\chi W_p$ (ASCE 41-13 Eq. 7-14); however, this will not govern in areas of High Seismicity.

The unit weight of the wall, W_p, for solid grouted medium weight masonry is 78 psf per Table 3.3.2 of the *2012 Design of Reinforced Masonry Structures* (CMACN, 2012).

The factor, χ, is determined per ASCE 41-13 Table 7-2, depending on the Structural Performance Level.

$$\chi = 1.3 \text{ for Life Safety for BSE-1E}$$

$$\chi = 1.0 \text{ for Collapse Prevention for BSE-2E}$$

$$F_{p,BSE\text{-}1E} = 0.4(1.133\text{g})(1.3)(78 \text{ psf}) = 46 \text{ psf}$$

$$F_{p,BSE\text{-}2E} = 0.4(1.699\text{g})(1.0)(78 \text{ psf}) = 53 \text{ psf (governs)}$$

The BSE-2E Collapse Prevention loading is more severe and governs the out-of-plane wall design at this site. This is the case when the ratio of S_{XS} of the BSE-2E to the BSE-1E seismic hazard levels is larger than the ratio of the χ factor for Life Safety to Collapse Prevention of 1.3.

ASCE 41-17 Revision

The χ factor in ASCE 41-13 Table 7-2 was calibrated to provide force demands in the BSE-1N, which, when checked against the wall or anchor lower-bound capacity, would provide similar results as the out-of-plane anchorage force equation found in ASCE 7 for its Design Earthquake. It was found that the χ factors were providing too conservative demands for the Collapse Prevention limit in the anchorage equation and for all equations for the body of the wall and were recalibrated in ASCE 41-17.

4.7.4 Acceptance Criteria for the Out-of-Plane Masonry Wall Design

The acceptance criteria for the out-of-plane masonry wall design stipulated in ASCE 41-13 § 11.3.5.3 and § 7.2.11 require the walls to be considered force-controlled actions. When evaluating the behavior of force-controlled actions, ASCE 41-13 § 7.5.1.3 requires that the lower-bound component strengths, Q_{CL}, be used. Per ASCE 41-13 § 11.3, the lower-bound strength of masonry walls is permitted to be calculated based on the strength design procedures in TMS 402-11, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries* (TMS, 2011), with a strength reduction factor, ϕ, taken equal to 1.0.

The acceptance criteria for force-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.1.2. This section is intended to be applicable to components that are resisting the pseudo seismic forces from ASCE 41-13 § 7.4.1.3, and is not applicable to F_p forces from ASCE 41-13 § 7.2.11, so the C_1, C_2, or J factors in ASCE 41-13 § 7.5.2.1.2 should not be applied to the F_p forces. The acceptance criteria for force-controlled actions with F_p forces should be as follows per ASCE 41-13 § 7.5.2.2.2:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

κ is the knowledge factor in accordance with ASCE 41-13 § 6.2.4, which for this example, will be assumed equal to 1.0 as the material strengths were determined based on existing drawings and material test reports as indicated in the problem statement.

Q_{UF} is the combination of gravity and seismic loads, as follows:

$$Q_{UF} = Q_G \pm Q_E \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

where, per the load combinations in ASCE 41-13 § 7.2.2:

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad \text{(ASCE 41-13 Eq. 7-1)}$$

and:

$$Q_G = 0.9Q_D \qquad \text{(ASCE 41-13 Eq. 7-2)}$$

$$Q_E = F_p$$

These loads can be combined into two load cases for the out-of-plane evaluation of the masonry wall, as follows:

$$Q_{UF} = 1.1(Q_D + Q_L + Q_S) \pm F_p$$

$$Q_{UF} = 0.9Q_D \pm F_p$$

4.7.5 Calculate the Out-of-Plane Masonry Wall Capacity

The out-of-plane capacity of the masonry wall will be calculated using TMS 402-11. As indicated in the previous section, the design will utilize the strength design method with ϕ equal to 1.0.

Per ASCE 41-13 § 11.3.5.2, second-order moments caused by out-of-plane deflections shall be evaluated since the height-to-thickness ratio of the wall exceeds 20. When determining height-to-thickness ratio, TMS 402-11 Section 3.3.5.3 indicates using the nominal thickness of masonry and not the actual thickness. ASCE 41-13 is not specific, so the TMS 402-11 approach is used.

$$h/t = (20 \text{ ft} \times 12 \text{ in./ft})/8 \text{ in.} = 30 > 20$$

The design below conservatively assumes a simple span from the grade level to the roof and ignores any reduction in the wall moment due to the back-span of the parapet above the roof level or any fixity provided by the foundation. This simplifies the analysis and aligns with the simple span equations in TMS 402-11 Section 3.3.5.

The calculations below will only check the load combination with 90 percent of the dead load as this will be the most critical case for this wall since the eccentric roof loads are small as defined by P_u/A_g and the vast majority of the gravity load is from the wall self-weight. The load combination with 110 percent of the gravity load will result in a stronger wall as the increased gravity load increases the moment capacity of the wall. While this is the case for the wall configuration and loading in this example, the designer should evaluate both load combinations if it is not apparent which will control, especially where there are large superimposed loads at the roof level.

The superimposed dead load on the wall being applied at the ledger is as follows:

$$0.9Q_{D,\mathrm{Roof}} = 0.9(80 \text{ lb/ft}) = 72 \text{ lb/ft}$$

This load is being applied at the face of the ledger with an eccentricity equal to half the wall thickness plus the 4× nominal ledger thickness:

$$e = 3.5 \text{ in.} + 7.63 \text{ in.}/2 = 7.3 \text{ in}$$

The weight of the wall at the mid-height where the point of maximum moment is assumed is as follows:

$$0.9Q_{D,\mathrm{Wall}} = 0.9[78 \text{ sf}(10 \text{ ft} + 3 \text{ ft})] = 913 \text{ lb/ft}$$

$$F_p = 53 \text{ psf (See Section 4.7.3 of this } \textit{Guide}\text{)}$$

The maximum axial load stress in the wall is checked to ensure it is less than the maximum permitted for slender walls per TMS 402-11 Section 3.3.5.3. The h/t ratio of the wall (using nominal wall thickness) does not exceed 30, therefore, the maximum axial stress shall not exceed $0.20f'_m$.

$$P_u/A_g \le 0.20f'_m$$

$$P_u/A_g = (72 \text{ lb} + 913 \text{ lb})/(7.625 \text{ in.})(12 \text{ in./ft}) = 11 \text{ psi}$$

$$0.20f'_m = 0.20(1{,}500 \text{ psi}) = 300 \text{ psi}$$

$$11 \text{ psi} \le 300 \text{ psi, O.K.}$$

The ultimate moment on the wall due to out-of-plane seismic forces and the moment due to eccentric loading at roof are calculated, ignoring P-δ moments using TMS 402-11 Equation 3-26.

$$Q_{UF} = Q_E = M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta \qquad \text{(TMS 402-11 Eq. 3-26)}$$

$$= \frac{(53 \text{ lb/ft})(20 \text{ ft})^2}{8} + 72 \text{ lb}\frac{\frac{7.3 \text{ in.}}{12 \text{ in./ft}}}{2} + (72 \text{ lb} + 913 \text{ lb})(0 \text{ ft})$$

$$= 2650 \text{ lb ft/ft} + 22 \text{ lb ft/ft} + 0 \text{ lb ft/ft} = 2{,}672 \text{ lb ft/ft}$$

In order to calculate the out-of-plane deflection of the wall, it needs to be determined if the wall is cracked or uncracked. The rupture strength of the masonry, f_r, is 163 psi from TMS 402-11 Table 3.1.8.2.

$$M_{cr} = S_n\left(f_r + \frac{P_u}{A_n}\right)$$

$$= \frac{(12\text{ in.})(7.63\text{ in.})^2}{6}\left(163\text{ psi} + \frac{(72\text{ lb} + 913\text{ lb})}{(12\text{ in.})(7.63\text{ in.})}\right)\left(\frac{1\text{ ft}}{12\text{ in.}}\right)$$

= 1,686 lb ft/ft < 2,672 lb ft/ft; therefore, wall is cracked.

The deflection calculation of a slender wall requires both the gross and cracked moment of inertia to be determined per TMS 402-11 Section 3.3.5.5.

$$I_g = \frac{bt^3}{12} = \frac{12\text{ in.}(7.63\text{ in.})^3}{12} = 444\text{ in}^4\text{/ft}$$

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right)(d-c)^2 + \frac{bc^3}{3} \qquad \text{(TMS 402-11 Eq. 3-31)}$$

where:

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000\text{ psi}}{900(1500\text{ psi})} = 21.5$$

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} \qquad \text{(TMS 402-11 Eq. 3-32)}$$

$$= \frac{0.15\text{ in.}^2\text{/ft}(60{,}000\text{ psi}) + (72\text{ lb/ft} + 913\text{ lb/ft})}{0.64(1500\text{ psi})12\text{ in./ft}} = 0.87\text{ in.}$$

$$I_{cr} = 21.5\left[0.15\text{ in.}^2\text{/ft} + \frac{(72\text{ lb/ft} + 913\text{ lb/ft})}{60{,}000\text{ psi}}\frac{7.63\text{ in.}}{2\left(\frac{7.63\text{ in.}}{2}\right)}\right]$$

$$\left[\left(\frac{7.63\text{ in.}}{2}\right) - 0.87\text{ in.}\right]^2 + \frac{12\text{ in.}(0.87\text{ in.})^3}{3}$$

$= 34$ in.^{4}/ft

The deflection calculation process for out-of-plane loading on slender masonry walls is an iterative process using TMS 402-11 Equation 3-30. However, a closed-form direct solution has been developed as shown in Equation 4.5.16 of 2012 *Design of Reinforced Masonry Structures* as follows.

$$\delta_u = \frac{\left(\frac{w_u h^2}{8} + \frac{P_{uf} e}{2}\right) - M_{cr}\left(1 - \frac{I_{cr}}{I_g}\right)}{\frac{48 E_m I_{cr}}{5h^2} - \left(P_{uw} + P_{uf}\right)}$$

$$= \frac{\left(\frac{53 \text{ psf}(20 \text{ ft} \times 12 \text{ in./ft})^2}{8(12 \text{ in./ft})} + \frac{72 \text{ lb/ft}(7.3 \text{ in.})}{2}\right) - 1686 \text{ lb ft/ft}(12 \text{ in./ft})\left(1 - \frac{34 \text{ in.}^4/\text{ft}}{444 \text{ in.}^4/\text{ft}}\right)}{\frac{48(900 \times 1{,}500 \text{ psi})34 \text{ in.}^4/\text{ft}}{5(20 \text{ ft} \times 12 \text{ in./ft})^2} - \left(913 \text{ lb/ft} + 72 \text{lb/ft}\right)}$$

= 2.00 in.

Recalculate the ultimate moment in the wall including the P-δ effects.

$$Q_{UF} = Q_E = M_u = \frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta \qquad \text{(TMS 402-11 Eq. 3-26)}$$

$$= M_u = 2650 \text{ lb ft/ft} + 22 \text{ lb ft/ft} + \left(72 \text{ lb/ft} + 913 \text{ lb/ft}\right)\left(\frac{2.00 \text{ in.}}{12 \text{ in./ft}}\right)$$

= 2650 lb ft/ft + 22 lb ft/ft + 164 lb ft/ft = 2,836 lb ft/ft

Calculate the lower-bound moment strength of the wall.

$$Q_{CL} = M_{n,lb} = \left(A_s f_y + \frac{P_u}{\phi}\right)\left(d - \frac{a}{2}\right)$$

where:

$$a = \frac{A_s f_y + \frac{P_u}{\phi}}{0.8 f'_m b}$$

$$= \frac{0.15 \text{ in.}^2/\text{ft}(60{,}000 \text{ psi}) + \frac{(72 \text{ lb/ft} + 913 \text{ lb/ft})}{1.0}}{0.8\left(1500 \text{ psi}\right)12 \text{ in.}} = 0.69 \text{ in.}$$

$$M_{n,lb} = \left(0.15 \text{ in.}^2/\text{ft}\left(60{,}000 \text{ psi}\right) + \frac{\left(72 \text{ lb/ft} + 913 \text{ lb/ft}\right)}{1.0}\right)\left(\frac{7.63 \text{ in.}}{2} - \frac{0.69 \text{ in.}}{2}\right)\frac{1 \text{ ft}}{12 \text{ in.}}$$

= 2,887 lb ft/ft

The acceptance criteria per Section 4.7.4 of this *Guide* with κ=1.0 is as follows:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

2,887 lb ft/ft > 2,836 lb ft/ft, wall OK.

The maximum reinforcement ratio is determined in accordance with TMS 402-11 Section 3.3.3.5 using the following load combination.

$$P \quad = D + 0.75L + 0.525Q_E = 80 \text{ lb/ft} + 78 \text{ psf } (10 \text{ ft} + 3 \text{ ft}) = 1{,}094 \text{ lb/ft}$$

$$\rho_{max} = \frac{0.64 f'_m}{f_y}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{f_y bd}$$

$$= \frac{0.64(1{,}500 \text{ psi})}{60{,}000 \text{ psi}}\left(\frac{0.0025}{0.0025 + 1.5(0.00207)}\right) - \frac{1094 \text{ lb/ft}}{(60{,}000 \text{ psi})(12 \text{ in./ft})\left(\frac{7.63 \text{ in.}}{2}\right)}$$

$$= 0.0067$$

The wall reinforcement ratio is

$$\rho \quad = \frac{A_s}{bd} = \frac{0.15 \text{ in.}^2/\text{ft}}{(12 \text{ in./ft})\left(\frac{7.63 \text{ in.}}{2}\right)} = 0.0033 < 0.0067, \text{ OK}$$

Note: This example does not check the wall deflection for serviceability, as it is not required for the linear static procedure.

4.7.6 Check the Masonry Parapet for Out-of-Plane Seismic Forces

Reinforced masonry parapets are evaluated as nonstructural components per ASCE 41-13 § 13.6.5. Per ASCE 41-13 Table 2-1 for BPOE, Risk Category II, the Nonstructural Performance Level is the Life Safety Nonstructural Performance Level and need only be evaluated for the BSE-1E Seismic Hazard Level.

Per ASCE 41-13 Table 13-1 for High Seismicity and the Life Safety Nonstructural Performance Level, the parapets are required to be evaluated. ASCE 41-13 § 13.6.5.1 requires reinforced masonry parapets with an aspect ratio greater than 3.0 to be evaluated. The aspect ratio of the parapet in the example is equal to 36 in./7.63 in. = 4.7.

Figure 4-16 illustrates the parapet geometry.

The out-of-plane seismic force on the parapet per ASCE 41-13 Equation 13-1 is shown below. The nonstructural component amplification factor, a_p, and response modification factor, R_p, for cantilever parapets are per ASCE 41-13 Table 13-2 and are 2.5 and 2.5, respectively. $S_{XS,BSE\text{-}1E}$ is 1.133g per Section 4.7.2 above. I_p is equal to 1.0 per ASCE 41-13 § 13.6.5.3.1. Since the parapet is attached at the roof level, the factor x/h equals 1.0. W_p is equal to 78 psf.

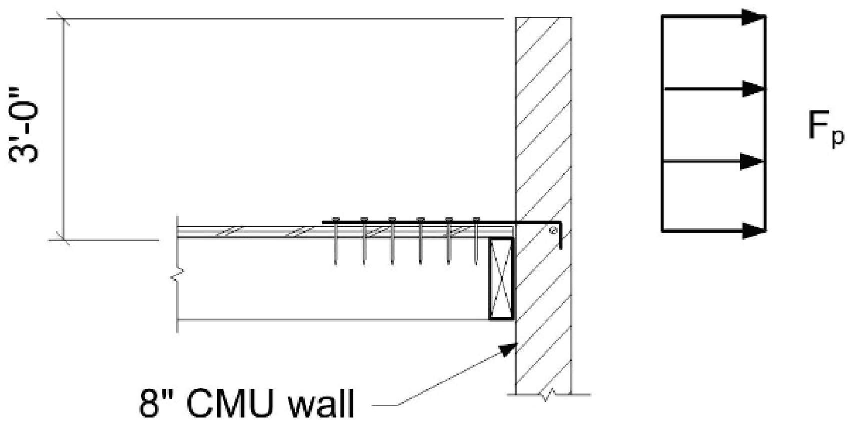


Figure 4-16 Out-of-plane loading on CMU parapet.

$$F_p = \frac{0.4 a_p S_{XS} W_p \left(1 + \dfrac{2x}{h}\right)}{\left(\dfrac{R_p}{I_p}\right)} \qquad \text{(ASCE 41-13 Eq. 13-1)}$$

$$= \frac{0.4(2.5)(1.133\text{g})(78\text{ psf})\left[1+2\left(1.0\right)\right]}{\left(\dfrac{2.5}{1.0}\right)} = 106\text{ psf}$$

F_p maximum and minimum values are calculated per ASCE 41-13 Equations 13-2 and 13-3, respectively, to verify that F_p is bounded by these limits.

F_p (max) $= 1.6 S_{XS} I_p W_p = 1.6(1.133\text{g})(1.0)(78\text{ psf}) = 141\text{ psf}$

F_p (min) $= 0.3 S_{XS} I_p W_p = 0.3(1.133\text{g})(1.0)(78\text{ psf}) = 27\text{ psf}$

F_p (min) $< F_p < F_p$ (max)

27 psf < 106 psf < 141 psf

The F_p seismic out-of-plane force is only applied over the height, h, of the parapet. The moment at the base of the parapet is as follows:

$$M_{uf,Fp} = \frac{wh^2}{2} = \frac{(106\text{ psf})\left(3\text{ ft}\right)^2}{2} = 477\text{ ft lb/ft}$$

Calculate the axial load at the base of the parapet. The load combinations stipulated in ASCE 41-13 § 13.4.3.3 are as follows:

$Q_{UF} = 1.2 W_p + F_{pv} \pm F_p$ (ASCE 41-13 Eq. 13-7a)

$Q_{UF} = 0.9 W_p - F_{pv} \pm F_p$ (ASCE 41-13 Eq. 13-7b)

The vertical seismic force, F_{pv}, per ASCE 41-13 § 13.4.3.2 is only required to be used where specifically required in the acceptance criteria. The acceptance criteria for parapets in ASCE 41-13 § 13.6.5.3.1 do not specifically require vertical seismic forces to be considered (such as is required for appendages in ASCE 41-13 § 13.6.6.3.1).

$$F_{pv} = 0$$

As noted in Section 4.7.5 above, the load combination with the least gravity load will result in the lowest out-of-plane wall strength.

$$Q_{UF} = 0.9W_p - F_{pv} \pm F_p = 0.9W_p + 0 \pm F_p = 0.9W_p \pm F_p$$

$$0.9W_p = 0.9(78 \text{ psf})(3 \text{ ft}) = 211 \text{ lb/ft}$$

ASCE 41-13 § 13.3 states, "Forces on bracing and connections for nonstructural components calculated in accordance with Section 13.4 shall be compared with capacities using strength design procedures." It is assumed that the lower-bound strength should be used in order to be consistent with other sections of ASCE 41-13 that use F_p forces, such as ASCE 41-13 § 7.2.11. The lower-bound strength, Q_{CL}, should be used with a strength reduction factor, ϕ, of 1.0.

The lower-bound moment strength of the wall is calculated as follows:

$$M_{n,lb} = \left(A_s f_y + \frac{P_u}{\phi}\right)\left(d - \frac{a}{2}\right)$$

where:

$$a = \frac{A_s f_y + \dfrac{P_u}{\phi}}{0.8 f'_m b} = \frac{0.15 \text{ in.}^2/\text{ft}(60{,}000 \text{ psi}) + \dfrac{(211 \text{ lb/ft})}{1.0}}{0.8(1500 \text{ psi})12 \text{ in./ft}} = 0.64 \text{in.}$$

$$M_{n,lb} = \begin{array}{l} \left(0.15 \text{ in.}^2/\text{ft}\left(60{,}000 \text{ psi}\right) + \dfrac{\left(211 \text{ lb/ft}\right)}{1.0}\right) \\ \times\left(\dfrac{7.63 \text{ in.}}{2} - \dfrac{0.64 \text{ in.}}{2}\right)\dfrac{1 \text{ ft}}{12 \text{ in.}} \end{array}$$

$$= 2{,}683 \text{ lb ft/ft}$$

The acceptance criteria in ASCE 41-13 § 13.6.5.3.1 state that the parapet shall be capable of resisting the F_p seismic forces. The knowledge factor, κ, in ASCE 41-13 § 6.2.4 is only applicable to component capacities as specified in ASCE 41-13 Chapter 7 and will not be applied when evaluating nonstructural components in ASCE 41-13 Chapter 13.

$$Q_{CL} > Q_{UF}$$

$$2{,}683 \text{ lb ft/ft} > 477 \text{ lb ft/ft},$$

Therefore, the parapet is OK.

4.8 Nonstructural Components (ASCE 41-13 Chapter 13)

4.8.1 Introduction

Evaluation and retrofit of existing nonstructural elements, including equipment anchorage, can be evaluated and, if necessary, retrofitted for different Performance Levels. These include the Life Safety, Position Retention or Operational Performance Levels (see ASCE 41-13 Table C2-5, Table C2-6, and Table C2-7 for specific Performance Levels for nonstructural elements). The requirements are accumulative as the Operational Performance Level includes those that are required for Life Safety and Position Retention.

> **Useful Tip**
> The intent of ASCE 41-13 Chapter 13 is to evaluate existing nonstructural elements. ASCE 7 should be used to address new components.

ASCE 41-13 Chapter 13 goes through specific evaluation and acceptance criteria for the Tier 3, Systematic Evaluation and Retrofit Procedure, for the evaluation and retrofit of existing architectural, mechanical, and electrical components and systems. For the Tier 1 Screening Procedure, nonstructural components are addressed in ASCE 41-13 Chapter 4 and checklists in ASCE 41-13 Chapter 16. The Tier 2 Deficiency-Based Evaluation and Retrofit Procedure is referenced in ASCE 41-13 Chapter 5.

For the Tier 3 systematic evaluation and retrofit procedure, ASCE 41-13 Table 13-1 lists the nonstructural components subject to the Life Safety and Position Retention. Within this table, the requirements for the evaluation and retrofit of these components are noted depending on the Level of Seismicity and evaluation procedure used. For the Operational Performance Level, the key issue for existing mechanical equipment is special certification per ASCE 7-10 § 13.2.2.

This design example will go through the ASCE 41-13 Tier 1 and Tier 2 Evaluation and Deficiency-Based Retrofit and Tier 3 Systematic Evaluation and Retrofit procedures for a rooftop mechanical unit in a Risk Category III building utilizing the Basic Performance Objective for Existing Buildings (BPOE). This example will focus on the Life Safety and Position Retention Performance Levels. At the end of the example, there is a summary table comparing the seismic design criteria per ASCE 7-10, ASCE 41-13, and ASCE 41-17.

> **Useful Tip**
> Operational Performance Level special certification requirements for existing mechanical equipment are located in ASCE 7-10.

4.8.2 Evaluation and Retrofit Procedures

ASCE 41-13 Table 13-1 outlines the evaluation procedure for nonstructural components depending on the nonstructural component, Level of Seismicity, and Performance Level. The evaluation includes force and deformation analysis or prescriptive procedures.

Useful Tip

Vertical seismic forces apply where specifically required per ASCE 41-13 § 13.6, § 13.7, and § 13.8. For Architectural Appendages and Marquees, ASCE 41-13 § 13.6.6.3 is an example where vertical seismic forces are considered.

- Force analysis
  - Horizontal (ASCE 41-13 Equations 13-1, 13-2, and 13-3)
  - Vertical (ASCE 41-13 Equation 13-7)
- Load combination (ASCE 41-13 Equations 13-7a and 13-7b)
- Deformation analysis (ASCE 41-13 Equation 13-8)
- Prescriptive (ASCE 41-13 § 13.4.2)

The force analysis procedure will be illustrated for an HVAC unit in the following example.

4.8.3 Problem Statement

Example Summary

Performance Objective: BPOE

Risk Category: III

Location: Seattle, Washington

Level of Seismicity: High

Reference Document:

AISC Steel Construction Manual 14th Edition

This example illustrates the procedures to evaluate and retrofit the anchorage of a roof mounted HVAC unit to a concrete roof slab. The following procedures and calculations are illustrated:

- Determine Performance Objective for HVAC anchorage (ASCE 41-13 § 2.2.1 and Table 2-1)
- Evaluation requirements of the HVAC unit per ASCE 41-13 Tier 1 Nonstructural Checklist for heavy equipment (ASCE 41-13 § 2.2.1, § 4.4 and §16.17)
- Discuss applicability of ASCE 41-13 Tier 2 evaluation and retrofit procedure for HVAC anchorage (ASCE 41-13 § 13.7.1)
- Tier 3 Systematic Evaluation and Retrofit (ASCE 41-13 § 13.1 and § 13.7)
- Determine applicability of Life Safety and Position Retention requirements and method of analysis (ASCE 41-13 § Table 13-1)
- Condition assessment requirements (ASCE 41-13 §13.2.1 and § 13.2.2)
- Evaluation and retrofit procedures (ASCE 41-13 § 13.3 and § 13.4)
- Acceptance criteria (ASCE 41-13 § 13.3 and § 13.7)

For this example, the Basic Performance Objective for Existing Buildings (BPOE) is the targeted performance. The building is a Risk Category III structure and it is located in an area with a High Level of Seismicity.

Figure 4-17 through Figure 4-19 illustrate the configuration, geometry, weight, and anchorage of the existing HVAC rooftop unit.

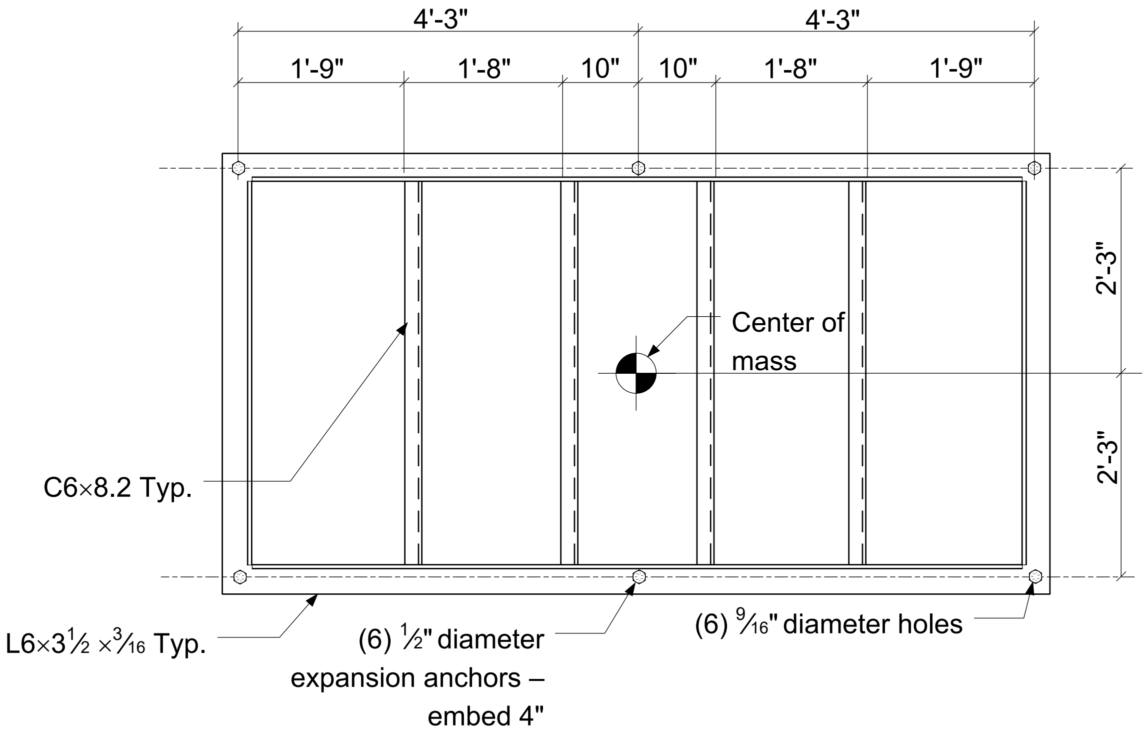


Figure 4-17 HVAC unit plan.

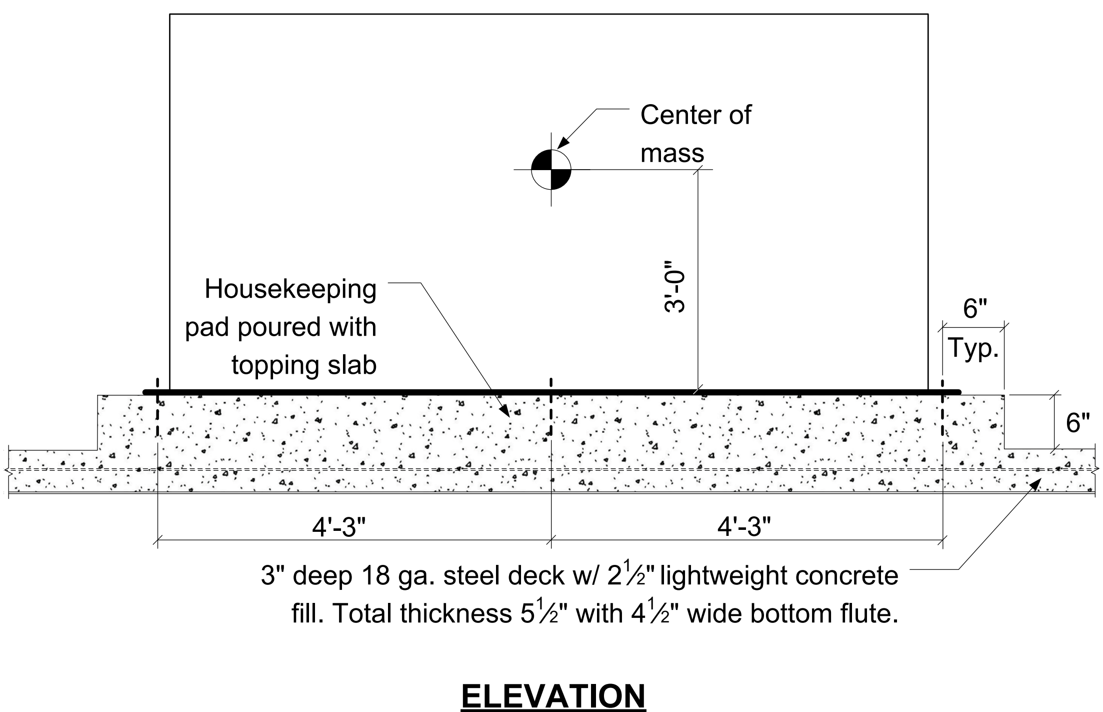


Figure 4-18 HVAC unit elevation.

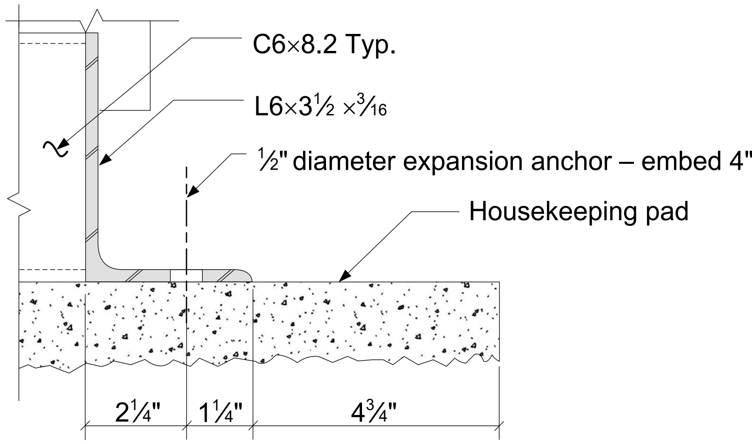


Figure 4-19 HVAC unit anchorage.

The following information is given.

- Location: Seattle, Washington
- Three-story steel structure with composite metal roof decking and light weight concrete topping.
- Site Class D
- Level of Seismicity: High
- $S_{XS} = 0.691$
- Operating weight: 2,700 lbs
- Rooftop installation: Unit installed in 1995
- Anchorage: Six 1/2-in diameter expansion anchors with 4 inch embedment.
- Steel yield strength: $F_y = 36$ ksi
- Light weight concrete compressive strength: 4 ksi
- Internally isolated component. External support anchorage does not require vibration isolation.

4.8.4 Determine Performance Objective and Level of Seismicity

Useful Tip

Risk Category requirements for the Tier 1 Checklist are located in ASCE 41-13 Table 2-1. Risk Category requirements are not specifically addressed in ASCE 41-13 Chapter 3 and Chapter 4.

ASCE 41-13 Table 2-1 defines required Structural and Nonstructural Performance Levels for the BPOE. They vary depending on the building Risk Category and the type of evaluation. For Risk Category III buildings, the Position Retention Nonstructural Performance Level is required for Tier 1 and 2 Evaluations and also for Tier 3 Evaluation and Retrofit Procedures. For nonstructural elements, only one Seismic Hazard Level and Performance Level is required for a Tier 3 Systematic Evaluation and Retrofit.

For this example, the Level of Seismicity is High as determined per ASCE 41-13 § 2.5. The Seismicity and Performance Level will be used in Tier 1 and 2 Evaluations to determine whether the nonstructural element is required to be evaluated. The same is true for Tier 3 Systematic Evaluation and Retrofit.

4.8.5 Tier 1 and 2 Evaluation and Tier 2 Deficiency-Based Retrofit

Per ASCE 41-13 Table 2-1 and Table 4-7 for High Level of Seismicity, the Position Retention Nonstructural Checklist will be used to evaluate the HVAC unit. Note that ASCE 41-13 Table 4-7 does not address Risk Category. Risk Category is addressed in ASCE 41-13 Table 2-1 and for this example Position Retention Performance Level is required for nonstructural components with the building being Risk Category III.

> **Definition**
>
> In the Nonstructural Tier 1 checklists the letters LMH, MH and H follow Life Safety (LS) or Position Retention (PR) in each checklist statement. These indicate applicability of the statement for different levels of seismicity. L = Low, M = Moderate, H = High.

The applicable ASCE 41-13 § 16.17 checklist statement under Mechanical and Electrical Equipment is as follows:

C NC N/A U *LS-not required; PR-H HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lbs is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)*

2,700 lb HVAC equipment anchored with (6) ½-inch expansion anchors embedded 4 inches.

For Life Safety Performance and High Seismicity, the anchorage of mechanical equipment weighing more than 400 pounds is not required to be checked. For this example, with the Position Retention Performance Level, verification that anchorage is present is required. Some engineering judgement is needed here, as there is no specific seismic force specified to evaluate the anchorage. In general, some anchorage is better than none and is considered compliant. The six 1/2-inch diameter expansion anchors are compliant with this statement.

If no anchorage is found, the item is non-compliant. With no anchorage at all there would be no Tier 2 Deficiency Based Evaluation. However, the anchorage of the mechanical unit could be retrofitted using Tier 2 procedures. This would require providing anchorage complying with ASCE 41-13 § 13.7.1. A Tier 3 Systematic Evaluation and Retrofit has the same requirements, as demonstrated later in this example.

4.8.6 Tier 3 Systematic Evaluation and Retrofit

Tier 3 Systematic Evaluation and Retrofit is covered for nonstructural components in ASCE 41-13 Chapter 13. In ASCE 41-13 § 13.2, there is a seven-step procedure outlined that will be followed in this design example.

Step 1: Establish Performance Objective, Seismic Hazard Level, and Level of Seismicity

The following information is provided in Section 4.8.3 of this *Guide*:

- BPOE Performance Objective
- Position Retention Performance Level (PR)
- BSE-1E Hazard Level
- High Seismicity

Step 2: Data Collection and Condition Assessment (ASCE 41-13 § 13.2.1)

ASCE 41-17 Revision

ASCE 41-17 requires testing of existing anchorage of nonstructural elements to concrete and masonry structures.

For this example, there are drawings available of the existing structure, and a detail on the drawings indicates that 1/2-inch diameter expansion anchors were used, as shown in Figure 4-17 and Figure 4-19, to anchor the mechanical unit to a lightweight concrete housekeeping pad. A site visit and field verification are required to verify presence, configuration, and condition of the mechanical unit curb and anchorage. The level of documentation and variation from the documentation will determine the number of nonstructural elements to be observed.

Testing of anchors is not specifically required by ASCE 41-13 § 13.2.1. Testing requirements for anchors in concrete has been added in ASCE 41-17 and should be considered, as the actual embedment of the expansion anchors cannot be determined by visual inspection.

Step 3: Analysis, Evaluation, and Retrofit Requirements (ASCE 41-13 Table 13.1)

In ASCE 41-13 Table 13-1, a "Yes" indicates that a retrofit is required if the component does not meet the acceptance criteria. For our example under the High Seismicity column, with Position Retention for a non-isolated mechanical unit, there is a "Yes," so evaluation of the HVAC anchorage is required. If the HVAC unit were being evaluated for the Life Safety Performance Level or for a lower Level of Seismicity, evaluation or retrofit of the HVAC anchorage would not be required.

Step 4: Interaction between Structural and Nonstructural Component (ASCE 41-13 § 7.2.3.3)

The first step is to determine whether the element is considered structural or nonstructural per ASCE 41-13 § 7.2.3.3. A mechanical unit would be considered a nonstructural element as it has no contribution to the lateral force-resisting system of the building. This step applies more to nonstructural elements such as cladding that in some cases must be considered as secondary structural elements in accordance with ASCE 41-13 § 7.2.3.3. This consideration occurs when the lateral stiffness of the nonstructural element is not negligible compared to that of the structural elements.

Step 5: Classification of Nonstructural Component (ASCE 41-13 § 13.2)

Determine whether the nonstructural component is acceleration sensitive (inertial loading), deformation sensitive (drift or deformation), or both. For this example, per ASCE 41-13 § 13.7.1.2, mechanical equipment is considered acceleration sensitive.

Step 6: Evaluation or Retrofit (ASCE 41-13 § 13.3, 13.4 and Table 13-1)

In ASCE 41-13 Table 13-1, the last column indicates the Evaluation Procedure to use: F (Force), D (Deformation) or P (Prescriptive). For this example, "F" or force evaluation procedure is required.

The equations for determining the seismic forces and load combinations are presented in ASCE 41-13 § 13.4.3 and acceptance criteria are presented in ASCE 41-13 § 13.7.1.1.

HVAC Horizontal Seismic Forces

$$F_p = \frac{0.4 a_p S_{XS} W_p \left(1 + \frac{2x}{h}\right)}{\left(\frac{R_p}{I_p}\right)} \qquad \text{(ASCE 41-13 Eq. 13-1)}$$

$$F_p\,(\max) = 1.6\, S_{XS} I_p W_p \qquad \text{(ASCE 41-13 Eq. 13-2)}$$

$$F_p\,(\min) = 0.3 S_{XS} I_p W_p \qquad \text{(ASCE 41-13 Eq. 13-3)}$$

where:

x = h, HVAC Unit is located on roof

a_p = 2.5 (ASCE 41-13 Table 13.2, internally isolated)

R_p = 2.0 (ASCE 41-13 Table 13.2, internally isolated)

> **Commentary**
>
> ASCE 41-13 Equation 13-1 is basically the same as ASCE 7 with a different seismic hazard. For a comparison of seismic design forces for nonstructural components between ASCE 41-13 and ASCE 7-10, see Section 4.8.7 of this *Guide*.

S_{XS} = 0.691 from Section 4.8.3 of this *Guide*

I_p = 1.0 (ASCE 41-13 § 13.7.1.3.2)

$$F_p = \frac{0.40(2.5)(0.691)(W_p)(3)}{\left(\frac{2.0}{1.0}\right)} \quad \text{(ASCE 41-13 Eq. 13-1)}$$

$= 1.04W_p$

$= (1.04)(2{,}700) = 2800$ lbs

F_p (max) $= 1.6(0.691)(1.0)(W_p)$

$= 1.11W_p$

$= (1.11)(2{,}700) = 2985$ lbs

F_p (min) $= 0.3(0.691)(1.0)(W_p)$

$= 0.21W_p$

$= (0.21)(2{,}700) = 560$ lbs

F_p is greater than F_p(min), 560 lbs and less than F_p(max), 2,985 lbs; therefore, the horizontal seismic force is:

F_p = 2,800 lbs

HVAC Vertical Seismic Forces

ASCE 41-17 Revision

ASCE 41-17 requires applying vertical acceleration to be in line with ASCE 7.

Per ASCE 41-13 § 13.4.3.2, vertical seismic forces are only required where specifically required by ASCE 41-13 § 13.6, § 13.7, and §13.8. For this example, ASCE 41-13 § 13.7 does not specifically require that vertical seismic forces be considered. This deviates from ASCE 7 where vertical acceleration is considered.

Anchorage Seismic Forces

Figure 4-20 presents a schematic of the anchorage seismic forces.

F_p = 2,800 lbs

W_p = 2,700 lbs

H_1 = 36 inches

D = 54 inches

L = 102 inches

For uplift along the long side of the mechanical unit, use load combination ASCE 41-13 Equation 13.7b:

$T = F_p(H_1)/D - 0.9W_p/(2)$

$= (2{,}800 \text{ lbs})(36 \text{ in})/(54 \text{ in}) - 0.9(2{,}700 \text{ lbs})/(2)$

$= 652$ lbs (net uplift)

$T/3$ bolts = 217 lbs per bolt in tension

Uplift along the short side of the mechanical unit:

T = (2,800 lbs)(36 in)/(104 in) – 0.9(2,700 lbs)/(2)

T = -246 lbs (no net uplift)

Dead load is greater than uplift; therefore, side anchors control.

Horizontal Shear per bolt

$F_p/6$ bolts = 2,800 lbs/6 bolts = 467 lbs per bolt

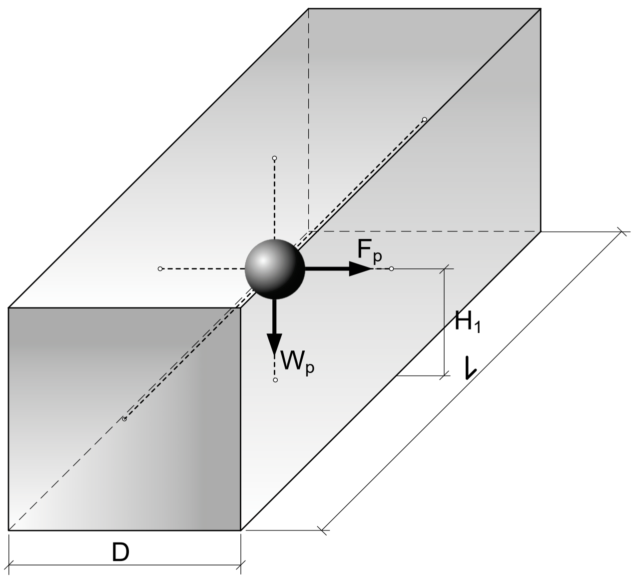


Figure 4-20 Loading diagram for HVAC anchorage.

Prying Action on Angle

Per *2011 AISC Steel Construction Manual* (AISC, 2011), Figure 4-21 shows the prying action on HVAC support angle.

$$t_{\min} = \sqrt{\frac{4Tb'}{\phi p F_u}}$$ (2011 AISC Construction Manual Eq. 9-20a)

T = 0.217 kips (tension load)

t = 3/16 inches for L6×3-1/2×3/16

b = 2.25 – t / 2 = 2.25 – (3/16)/2 = 2.16 inches

d_b = 0.50 inches (bolt diameter)

b' = $b - d_b/2$ (2011 AISC Construction Manual Eq. 9-21)

= 2.16 – 0.5/2 = 1.91 inches

ϕ = 1.0

$p \quad = 2b = 2 \times 2.16 = 4.32$ inches (tributary width)

$F_u \quad = 58$ ksi (minimum tensile strength for ASTM A36 angle

$$t_{\min} = \sqrt{\frac{4(0.217)(1.91)}{(1)(4.32)(58)}}$$

$= 0.081$ inches

$t_{\min} = 0.081$ inches $< t = 0.1875$ inches; therefore, prying action is satisfied with angle and q is considered negligible.

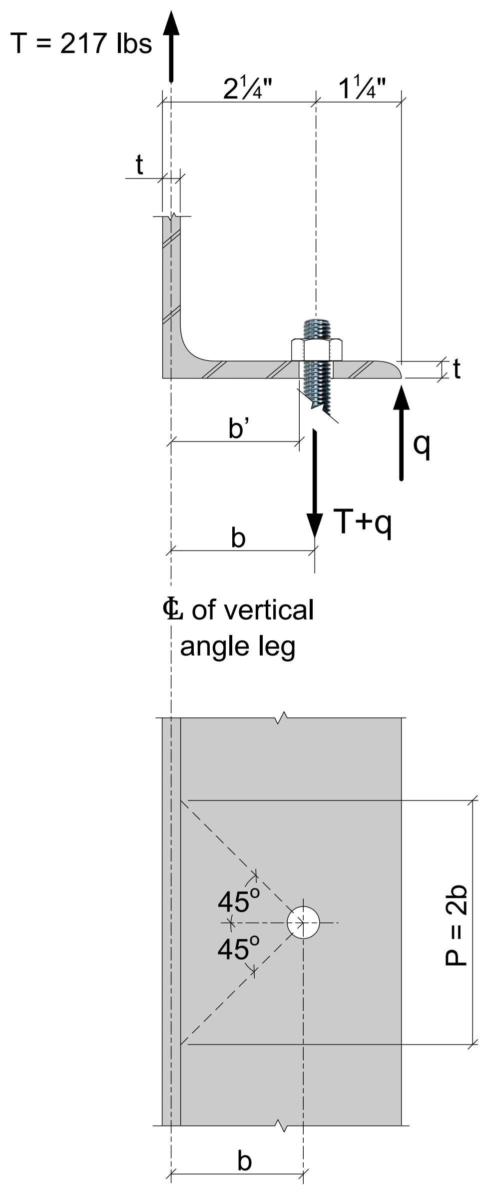


Figure 4-21 Loading diagram for prying action on HVAC support angle.

Anchor Capacity

There is a question, per ASCE 41-13, whether Ω_0 should apply to the pseudo seismic forces for the concrete anchors of nonstructural elements. As background, when ASCE 41-13 was completed, ASCE 7-10, without Supplement 1, was referenced (see ASCE 41-13 § 17.1) which at the time did not include Ω_0 for nonstructural elements. Ω_0 for nonstructural elements was added in the third printing of ASCE 7-10, which added Supplement 1, per Table 16-6.1. So technically, for ASCE 41-13, when ACI 318-11 refers to Ω_0, it is for structural applications, not nonstructural.

> **Useful Tip**
>
> See Section 2.2.2 of this *Example Application Guide* for more discussion on pseudo seismic force.

This has been updated in ASCE 41-17 where Ω_0 is specifically referenced for concrete anchors of nonstructural elements. Based on this, engineering judgement is being used apply Ω_0 to the pseudo seismic forces for this design example to check the acceptability of the concrete anchors. Ω_0 is added to the seismic force, not the net uplift force previously calculated.

Net uplift is determined based on applying Ω_0 to the pseudo seismic force.

From previous calculations.

Ω_0 = 2.5 per ASCE 7-10 Table 16-6.1

F_p = 2,800 lbs

W_p = 2,700 lbs

H_1 = 36 inches

D = 54 inches

L = 102 inches

Uplift along the long side of the mechanical unit; therefore use load combination ASCE 41-13 Equation 13.7b:

$T = \Omega_0 F_p(H_1)/D - 0.9W_p/(2)$
$= (2.5)(2,800 \text{ lbs})(36 \text{ in})/(54 \text{ in}) - 0.9(2,700 \text{ lbs})/(2)$
$= 3,450$ lbs (net uplift)

$T/3$ bolts = 1,150 lbs per bolt in tension

Uplift along the short side of the mechanical unit:

$T = (2.5)(2,800 \text{ lbs})(36 \text{ in})/(104 \text{ in}) - 0.9(2,700 \text{ lbs})/(2)$
$= -1,208$ lbs

$T/2$ bolts = 604 lbs per bolt in tension

Horizontal shear per bolt:

$\Omega_0 F_p/6$ bolts = (2.5)(2,800) lbs/6 bolts = 1,170 lbs per bolt

Using one of the several commercially available software tools for concrete anchorage, the capacity of the existing 1/2-inch expansion anchors is determined with the following loads and design parameters and with the geometry and eccentricities shown in Figure 4-22. The anchorage is evaluated as force-controlled using strength design, nominal material properties and phi equal to 1.0.

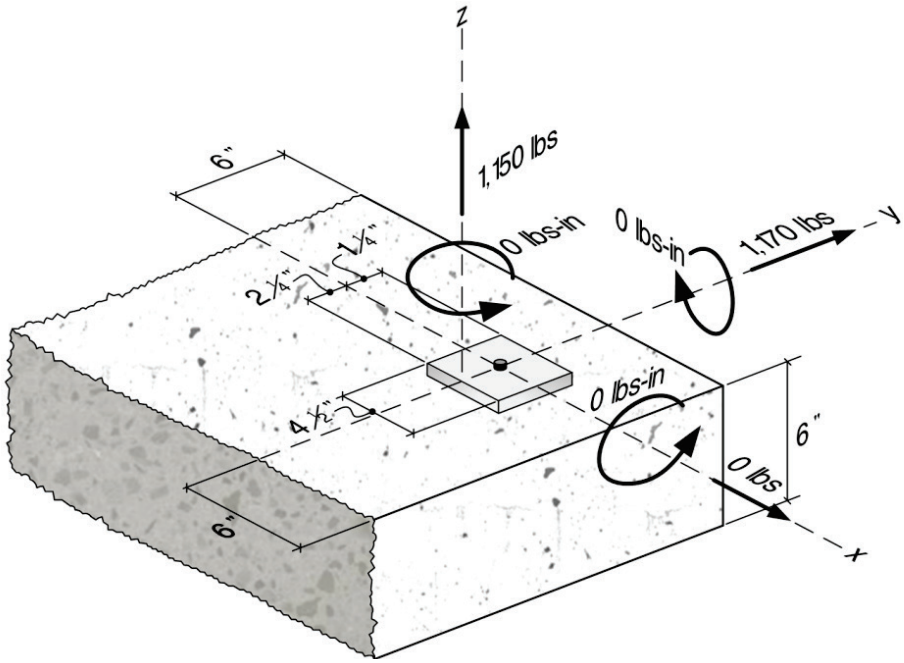


Figure 4-22 Anchor loading diagram. Geometry (in) and loading (lbs).

For the long side of the unit

T = 1.150 lbs per bolt

V = 1.170 lbs per bolt

F_y Anchor = 36 ksi

f_c' = 4 ksi lightweight concrete

Results as follows adjusted for ϕ = 1.0 for the different failure modes:

Effective Tension Load: 3,474 lbs (Tension load from anchor program with eccentrically loaded plate)

Acceptance of Anchors Long Side of Unit

Steel strength: 0.32

Pullout strength: 0.93

Breakout strength 0.92

Acceptance of Anchors in Shear

Steel failure: 0.21

Pryout strength: 0.15

Concrete edge failure in y-direction: 0.48

Combined Shear and Tension Loads

Combined: 1.18

Acceptance of Anchors Short Side of Unit

Steel strength: 0.17

Pullout strength: 0.49

Breakout strength 0.48

Acceptance of Anchors in Shear

Steel failure: 0.21

Pryout strength: 0.15

Concrete edge failure in direction y: 0.48

Combined Shear and Tension Loads

Combined: 0.81

Step 7: Retrofit or Accept

Those components not meeting the acceptance criteria per ASCE 41-13 § 13.7.3 could be retrofitted per ASCE 41-13 § 13.5. For the Acceptance Criteria for this example, ASCE 41-13 § 13.3 refers to § 13.7 and ultimately § 13.7.1.3.2 for the Position Retention Performance Level.

Acceptance of Anchor

For the long side of the mechanical unit, the demand/capacity of the anchors is 0.93 maximum in tension, 0.48 in shear, and 1.17 combined shear and tension. For the short side of the mechanical unit, the demand/capacity of the anchors is 0.49 in tension, 0.48 in shear, and 0.81 combined shear and tension. Therefore, the anchors do not meet the acceptance criteria along the long side of the unit and are acceptable along the short side of the unit. Retrofit of the unit anchorage is required to satisfy the Performance Objective.

Acceptance of Prying Action of Support Angle

From previous calculations:

t_{min} = 0.081 inches < 0.1875 inch thickness of support angle

Therefore, support angle is acceptable.

No retrofit is required to satisfy the Performance Objective.

Retrofit of Anchorage at Long Side of Mechanical Unit

The anchors along the long side of the unit do not meet the acceptance criteria. One option to mitigate this structural concern is to add two 1/2-inch expansion anchors embedded 4 inches along both sides of the unit for a total of five anchors each side of the unit. This would result in an applied anchor load of 690 lbs in tension and 702 lbs in shear. The resulting shear and tension going through the same analysis as above is 0.55 maximum in tension, 0.29 in shear, and 0.71 combined shear and tension. With the additional anchors, the acceptance criteria are met.

4.8.7 Comparison of ASCE 7, ASCE 41-13, and ASCE 41-17 Seismic Design Criteria for Internally Isolated Mechanical Unit Anchorage

Table 4-5 provides a comparative summary of the various design criteria for anchoring an internally isolated mechanical unit, using ASCE 7-10, ASCE 41-13, and ASCE 41-17. Although the criteria are similar, there are subtle differences in some aspects that are important to appreciate in evaluating existing anchors or in designing for new anchors.

Table 4-5 Comparison of Design Criteria ASCE 7-10, ASCE 41-13, and ASCE 41-17 for Anchorage of Internally Isolated Mechanical Unit

Seismic Design Criteria	ASCE 7-10	ASCE 41-13	ASCE 41-17
Seismic Horizontal Force F_p	$F_p = \frac{0.4a_pS_{DS}W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)$ (ASCE 7-10 Eq. 13.3-1) $a_p = 2.5$, $R_p = 2.0$, $I_p = 1.0$ S_{DS} is based on 2/3 MCE_R	$F_p = \frac{0.4a_pS_{XS}W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+\frac{2x}{h}\right)$ (ASCE 41-13 Eq. 13-1) $a_p = 2.5$, $R_p = 2.0$, $I_p = 1.0$ S_{XS} is based on BSE-1E	$F_p = \frac{0.4a_pS_{XS}W_p}{\left(\frac{R_p}{I_p}\right)}\left(1+\frac{2x}{h}\right)$ (ASCE 41-17 Eq. 13-1) $a_p = 2.5$, $R_p = 2.0$, $I_p = 1.0$ S_{XS} is based on BSE-1E
Seismic Vertical Force F_{pv}	$\pm 0.2S_{DS}W_p$ (ASCE 7-10 § 13.3.1)	Not required for this type of nonstructural component	Not required for this type of nonstructural component
Design Load Combinations	$(1.2+02.S_{DS})D + \Omega_0 Q_E$ $(0.9 - 0.2S_{DS})D \pm \Omega_0 Q_E$ (ASCE 7-10 § 12.14.3.2.2) Ω_0 per ASCE 7-10 § Table 13.6-1, which is equal to 2.5 for this example.	$Q_{UF} = 1.2W_P \pm F_P$ (ASCE 41-13 Eq. 13.7a) $Q_{UF} = 0.9W_P \pm F_P$ (ASCE 41-13 Eq. 13.7b) Ω_0 per ACI 318-11 and ASCE 7 § Table 13.6-1, equal to 2.5 for this example.(See discussion in Section 4.8.6 (Anchor Capacity) of this *Example Application Guide* for the applicability of Ω_0.	$Q_{UF} = 1.2W_P + F_{PV} \pm \Omega_0 F_P$ (ASCE 41-17 Eq. 13.6a) $Q_{UF} = (0.9W_P - F_{PV}) \pm \Omega_0 F_P$ (ASCE 41-17 Eq. 13.6b) $F_{PV}= 0$, or not consider as noted above for this type of nonstructural component. Ω_0 from ASCE 7-16 Table 13.6-1, which is equal to 2 for this example.

Chapter 5
Foundations

5.1 Overview

This chapter provides discussion and example application of evaluating and modeling the foundation, soil, and soil-structure interaction (SSI) of buildings as presented in ASCE 41-13 Chapter 8 (ASCE, 2014). Foundations are at the critical interface between the soil and the building structure. They support the structure by distributing gravity and wind loads from the structure to the soil, and also impart seismic ground accelerations that generate the inertial forces in the structure above. The type of foundation, and the performance of the foundation elements and supporting soil, can greatly influence the behavior of the building. The most common usage of the ASCE 41-13 foundation provisions, and therefore the major focus of this chapter, is the modeling and evaluation of shallow foundations; in addition, general guidance is provided for collection of soil and foundation information, shallow foundation lateral loads, and geologic hazards, such as liquefaction, deep foundations, and SSI.

This chapter illustrates the following:

- **Section 5.2:** General design considerations for evaluating existing foundations.
- **Section 5.3:** Review of the various foundation provisions that are located not only in ASCE 41-13 Chapter 8, but also other locations throughout the standard.
- **Section 5.4:** The ASCE 41-13 approach to foundation evaluation and discussion of footing and foundation flexibility.
- **Section 5.5:** Requirements in ASCE 41-13 for information gathering and condition assessment of existing footings and the supporting soil. Some recommendations and items for consideration regarding the scope and approach to the geotechnical investigation are included.
- **Section 5.6:** Determination of capacities and load-deformation characteristics of foundations, including expected capacities, foundation stiffness, bounding requirements, bearing pressure distribution, and force-controlled and deformation-controlled actions.

- **Section 5.7:** An overview of the ASCE 41-13 principles for evaluation and retrofit of shallow foundations, a summary of the three methods of modeling foundations, and an example for each method.
- **Section 5.8:** Shallow foundation lateral load provisions of ASCE 41-13.
- **Section 5.9:** ASCE 41-13 requirements for deep foundation evaluation and retrofit.
- **Section 5.10:** Soil-structure interaction effects, including kinematic interaction and radiation damping, and provides an example of the determination of base slab averaging and embedment effects.
- **Section 5.11:** A general overview of the ASCE 41-13 approach to liquefaction evaluation and mitigation.

It is noted that ASCE 41-13 § 8.6 on seismic earth pressure is not covered.

5.2 Foundation Design Considerations

When evaluating a structure, it is important to consider the impact of the foundation on the superstructure from all seismic-related sources, including forces due to seismic accelerations as well as deformations at the foundation caused by the earthquake. Some key factors to consider when using ASCE 41-13, or any other design standard, include the effects of the following geological and geotechnical phenomena on the building response, which will be discussed in greater detail in the sections ahead:

- Soil response
    - Change in soil characteristics due to shaking, such as liquefaction
    - Transmission and alteration of the earthquake energy to the building's site due to soil site class, geologic, and other considerations
- Soil-structure-interface response
    - Modification of the earthquake input from the subsurface media to the building structure based on the geometry of the building (kinematic interaction)
    - Flexibility and modification of the building's dynamic characteristics, including foundation and radiation damping, at the soil-foundation interface (inertial interaction)

As with design of structures for non-earthquake loads, foundations are critical components to control vertical and lateral deformations imposed on the building. For non-earthquake loads, the tolerable deformations are limited to those that do not impair building function and do not cause a perceptible nuisance. As such, foundation deformation from non-earthquake loads does not typically impose demands back on the structural components they support. For seismic loads, there are transient deformations (during the earthquake) and permanent deformations after the earthquake, both of which impose structurally significant force and deformation demands on the building structure. In some cases, structural members subjected to these demands can lose vertical load-carrying capability. The soil response may also impose deformation demands on the building structure, which can be significant when soil yields, or when liquefaction, seismic-induced settlement, ground rupture, or lateral spreading occurs.

The following potential sources of imposed force and deformation demands on a building as a result of foundation characteristics and geologic hazards are discussed in ASCE 41-13 Chapter 8:

- Foundation sliding: Shallow foundations slide; piles deflect or shear off
- Foundation overturning: Pile and shallow footings rotate and can move up or down
- Settlement from shaking-induced densification
- Settlement and lateral-spreading from liquefaction
- Ground rupture

5.3 ASCE 41-13 Foundation Provisions

Proper application of ASCE 41-13 Chapter 8 depends on the provisions of other chapters of ASCE 41-13. Of course, building foundations can be of many types and materials—steel piles, concrete mats, wood poles, and masonry footings to name a few—and therefore foundation evaluation pursuant to ASCE 41-13 Chapter 8 must be done in conjunction with the applicable chapters for each material. When applying those chapters, keep in mind that each foundation element (or its connections to the superstructure) must be treated as a force-controlled element unless deformation-controlled acceptance criteria are explicitly provided in the relevant material chapters or, for nonlinear procedures, the engineer can show explicitly that it behaves in a deformation-controlled manner as defined by ASCE 41-13 § 7.5.1.2. ASCE 41-13 § 7.6 provides requirements for testing to determine modeling parameters and acceptance criteria.

Foundation provisions are also located in the initial ASCE 41-13 chapters covering general evaluation and retrofit requirements (Chapter 3), Tier 1 screening (Chapter 4), Tier 2 deficiency-based evaluation and retrofits

(Chapter 5), Tier 3 systematic evaluation and retrofit (Chapter 6), and in a number of locations in the Chapter 7 analysis procedures and acceptance criteria. A summary of key foundation-related provisions follows.

Provisions in ASCE 41-13 Chapters 3-6

- ASCE 41-13 § 3.2.4 defines acceptable sources of information for determination of site and foundation conditions, and requires a site-specific subsurface investigation for Enhanced Performance Objectives.
- ASCE 41-13 § C4.2.3 notes the need to look for existing geotechnical reports on site soil conditions and to establish site and soil parameters when conducting Tier 1 screenings.
- ASCE 41-13 § 4.3.4 notes that Benchmark Building provisions cannot be satisfied if there are liquefaction, slope failure, or surface fault rupture hazards at the site, unless they have been mitigated in the lateral force-resisting system and foundation design.
- ASCE 41-13 § 5.4.3 provides foundation check requirements for Tier 2 deficiency-based evaluation and retrofit. It also points out that there are no Tier 2 evaluation procedures for liquefaction, slope failure, or surface fault rupture, and that these geological site hazards need to use a Tier 3 evaluation procedure.
- ASCE 41-13 § 6.3 points to the requirements of ASCE 41-13 Chapter 8 for Tier 3 evaluation and retrofit.

Foundation Requirements in ASCE 41-13 Chapter 7

- ASCE 41-13 Chapter 7 provides overall requirements regarding how the foundation is to be modeled: ASCE 41-13 § 7.2.3.5 requires that the foundation be modeled considering the flexibility at the base of the structure, and refers the user to ASCE 41-13 § 7.2.7 for SSI, § 8.2.2 for geologic hazards, and § 8.4 for acceptance criteria.
- ASCE 41-13 § 7.2.7 recognizes that the presence of a massive, stiff foundation can change the nature of the free-field earthquake ground motions, which are the basis for the design spectra. If such changes are expected to increase the seismic demand (which is uncommon), then ASCE 41-13 § 7.2.7 requires that SSI be considered. Typically, SSI effects will lower the demand, and § 7.2.7 allows the engineer to incorporate this by reducing the design spectral acceleration per ASCE 41-13 § 8.5. Per ASCE 41-13 § 7.2.7.1 and § 7.2.7.2, the engineer has the option of incorporating SSI either through a simplified method (in conjunction with ASCE 7) or by explicitly modelling the elements. Note

that one must use a flexible foundation model to incorporate soil-structure interaction.

- ASCE 41-13 § 7.2.8 requires that the effects of overturning on foundations and geotechnical components be performed per the requirements in ASCE 41-13 Chapter 8. ASCE 41-13 Equations 7-5 and 7-6 shall not be used to evaluate the acceptability of the foundation and soil at the soil-structure interface. Refer to Section 5.7.1 of this *Guide* for further discussion.

Foundation Provisions in Material Chapters 9-12

- Steel: ASCE 41-13 § 9.9.4 requires connections between piles and caps to be considered force-controlled.
- Concrete: ASCE 41-13 § 10.12.3 requires all components of existing foundations and all new retrofit materials and components to be considered force-controlled, but caps the required capacity at 125% of the capacity of the supported vertical component.
- Masonry: ASCE 41-13 § 11.6.2 requires all footings to be considered force-controlled, and to be treated as elastic with no inelastic capacity unless shown otherwise per ASCE 41-13 § 7.6.
- Wood: ASCE 41-13 § 12.6.2 requires wood piles and wood pole structures subject to axial and flexural loads to be considered deformation-controlled using *m*-factors from ASCE 41-13 Table 12-3, and refers to ASCE 41-13 Chapter 8 for acceptability of supporting soils.

5.4 ASCE 41-13 Approach to Foundation Evaluation

The first step in evaluating the expected performance of an existing foundation is characterizing the foundation elements and supporting soils; this is done per ASCE 41-13 § 8.2. Ideally, drawings with foundation design information would be available; otherwise, exploratory investigation, which can include destructive and nondestructive investigation and/or testing will be required. The soil properties required for the structural modeling of the foundation are typically determined in conjunction with a geotechnical consultant, and communication is important to ensure that the specific parameters required for the analyses are requested in advance. Immediate Occupancy or Damage Control Performance Levels require more extensive characterization of engineering soil properties, as described in ASCE 41-13 § 8.2.1.1.2. Some sites will present geologic hazards such as fault rupture, liquefaction, dynamic settlement, landslides and flooding. Such hazards are typically identified by the checklists or by the consulting geotechnical engineer, and ASCE 41-13 § 8.3 provides requirements for mitigating those hazards.

Useful Tip

ASCE 41-13 differentiates between a footing (the structural element in contact with the soil) and the foundation (the soil/structure system). A flexible (or rigid) footing refers to the footing element itself, whereas a flexible (or rigid) foundation refers to the footing/soil system. Therefore, a structure can have a rigid footing but a flexible foundation (due to soil springs).

With the soils and foundations characterized, the engineer must decide how to incorporate foundation behavior into the model, and to choose acceptance criteria consistent with the performance objectives. ASCE 41-13 § 7.2.3.5 requires that foundation flexibility be considered and permits a rigid (fixed) or flexible base (building's boundary condition) assumption subject to the requirements for soil-structure interaction of ASCE 41-13 § 7.2.7 and foundation acceptability of ASCE 41-13 § 8.4. Therefore, per the requirements of ASCE 41-13, foundations may be designed with a rigid (fixed) base assumption regardless of the actual flexibility of the foundation, except for buildings that are sensitive to foundation movement or rotation and are evaluated to the Immediate Occupancy Performance Level. Where foundations are modeled as fixed base, relatively stringent acceptance criteria are provided to limit the deformation imposed on the superstructure.

It is important to distinguish between rigid (fixed) or flexible foundations and rigid or flexible footings, as discussed in ASCE 41-13 § 8.4.2.1. A rigid base (also known as fixed base) or flexible base foundation refers to the flexibility of the structural footing and soil system as a boundary condition of the building, whereas a rigid or flexible footing refers to the structural footing element itself relative to the supporting soil stiffness. Therefore, the foundation may be modeled and evaluated with a combination of foundation and footing flexibility. For instance, a structure can have a rigid footing with a flexible foundation where the flexibility of the soil is modeled with springs. Figure 5-1 illustrates potential combinations for footing and foundation flexibility.

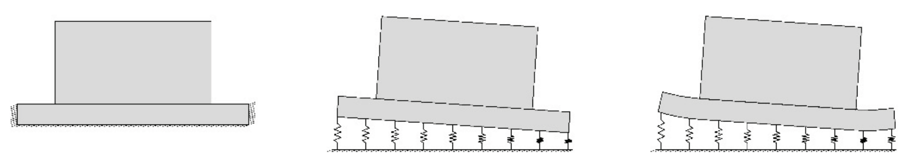


Figure 5-1 Illustration of footing and foundation flexibility.

If the simplifying assumption of a perfectly rigid foundation is determined to be appropriate for the site and structure, then ASCE 41-13 § 8.4.2 through § 8.4.2.3.2.1 provide the model requirements and acceptance criteria. Refer to Section 5.7.2 of this *Guide* for more information on determining footing flexibility. Should it be determined that foundation flexibility is important to the structural response, the designer has the option of using simplified point-wise foundation springs with presumed rigid footings (Method 1 – Figure

5-11); distributed soil springs below a presumed rigid footing (Method 2 – Figure 5-13); or flexible footings supported by soil springs (Method 3 – Figure 5-15). This decision process and sample calculations are provided in the following sections.

5.5 Soil and Foundation Information and Condition Assessment

ASCE 41-13 § 3.2.4 and § 8.2 specify information about the foundation and subsurface soil characteristics that is required for an evaluation or retrofit. Existing drawings or site investigation may be used to determine foundation type, materials, and configuration. Per ASCE 41-13 § 3.2.4, a site reconnaissance must be performed to observe the existing conditions, including evidence of poor foundation performance, as well as discrepancies between the as-built conditions and the construction drawings. Varying levels of detail on the subsurface soil conditions are required depending on the Performance Objective. Site-specific geotechnical information is required for Enhanced Performance Objectives, such as Immediate Occupancy, or where seismic-geologic site hazards are determined to be present. Information on design foundation loads and soil conditions may be gathered from existing documentation, including existing drawings or geotechnical reports. Geotechnical reports from adjacent sites may provide useful information for the subject building, although the potential for variation in subsurface soil should be considered and uncertainty in actual soil behavior incorporated into the foundation evaluation and design.

> **Useful Tip**
>
> An effective data gathering scope is best achieved when the potential benefits of the investigation are understood. Broad, regional maps may not capture unique, site-specific issues.

To minimize the risk of not identifying a potential hazard and to obtain accurate soil data parameters, ideally a geotechnical engineer (and in some cases an engineering geologist) should work directly with the structural engineer and the building owner or the owner's representative to review existing available data to establish the scope of additional investigation, if required, and to provide both geologic and geotechnical recommendations for the evaluation and retrofit of the building. The need for an engineering geologist should be determined in consultation with the geotechnical engineer. Some situations that typically require an engineering geologist include sites with rock subsurface conditions, particularly on sloped sites, sites with a history or evidence of landsliding, sites within an earthquake fault zone or when required by the governing jurisdiction. NIST GCR 12-917-21 report, *Soil-Structure Interaction for Building Structures* (NIST, 2012), Section 6.3.2 and Section 6.3.3 contain sample checklists with recommendations for information to be provided to the geotechnical engineer and material to be included in the geotechnical report. For essential facilities, particularly where there are potential geologic concerns, the engagement of subject matter experts is prudent and may be required by the Authority Having Jurisdiction. In cases where the owner and local jurisdiction deem the use of existing data appropriate as the sole basis for the foundation evaluation and design, the engineer should consider potential geotechnical issues, variables, and unknowns in the structural evaluation and retrofit. The following discussion provides guidance on some of the issues to be considered and sources of information that are available.

For site-specific subsurface characterization, the following approach is suggested:

- **Observation:** Visual observation of the structure, its foundation and surroundings areas should be performed to look for signs of foundation movement (total or differential) in order to identify the historic performance of the building's foundation. Telltale signs of distress due to settlement that are observed at sites with a history of seismic activity may in some circumstances indicate an elevated risk of geologic issues, such as liquefaction. However, the absence of observable settlement does not preclude the possibility of geologic hazards being present on the site.
- **Internet research:** Websites containing regional or local reports related to geologic and seismic hazards and subsurface conditions can provide relevant information on the subject site.
- **Desktop research:** Geotechnical reports (of the site and neighboring sites), construction drawings, test results, and other available documents directly related to the building should be reviewed. These documents provide the most valuable sources of data and extensive efforts to locate and review these documents may be warranted as they can lead to significant savings in in-situ investigation costs. Potential sources for geotechnical reports are the owner, building departments, and on-site archives or mechanical rooms. When original geotechnical reports are available, it is good practice to suggest that the document be reviewed by a geotechnical engineer.
- **In-situ investigation:** A subsurface investigation provides the most comprehensive understanding of geotechnical and geological conditions. The engineer should work closely with the geotechnical engineer to ensure that sufficient exploration and sampling are performed and that the specific geotechnical information required for the seismic evaluation and retrofit is conveyed to the geotechnical engineer. Typically, both field and laboratory testing would be performed as part of this investigation. Although ASCE 41-13 does not provide specific

recommendations for in-situ investigation methods, the following investigative options may be considered, and close collaboration with the geotechnical engineer in determining the appropriate scope is recommended.

- Drill rig to bore, retrieve, and log soil profile. Once drilled, down-hole piezometers and seismic shear wave velocity testing can be performed, which provide insight into groundwater conditions and dynamic soil properties, respectively.
- Cone penetration tests (CPTs) can be performed to supplement and enhance data collected from borings. CPTs provide continuous profiling of subsurface stratigraphy and in-situ measurements of various soil parameters, negating some of the shortcomings associated with borehole sampling, such as discrete sampling intervals and sample disturbance. CPTs are quicker to conduct than borings and enable larger site areas to be investigated at a lower cost.
- Seismic cone penetration tests (SCPTs), which can be performed as part of the CPT, provide estimates of shear wave velocity, a valuable indicator of the dynamic properties of soil and rock. Shear wave velocity is related to the small-strain shear modulus, an integral component of dynamic soil response under seismic loading.
- Standard penetration tests (SPTs) provide the typical N-value representing blow counts, which is correlated to various soil properties. This is the traditional approach for geotechnical investigations and is typically the basis for foundation design values and settlement estimates, particularly in granular soils.
- For shallow foundations up to approximately 15 feet, test pits can be excavated and shored to obtain soil samples, log subsurface conditions, and investigate the depth and configuration of existing foundation elements. Test pits are particularly useful in identifying the transition between native soil and fill.
- Laboratory tests can be used to validate soil classifications and to provide engineering parameters for interpretation and use by engineers. Testing includes, but is not limited to:
    - Soil gradation
    - Plasticity
    - Corrosivity
    - Moisture content and dry density

- Shear strength
- Expansion potential
- Compressibility (consolidation characteristics)

The scope of the data investigation and reporting for a building depends upon specific performance objectives, regulatory requirements, and the extent of available data. In some cases, the cost of extensive investigation and testing can be justified by developing a more efficient retrofit design through the use of more accurately determined soil and foundation properties as opposed to the default values prescribed by the standard.

In addition to the geotechnical investigation, the geologic assessment seeks to identify site-specific hazards such as ground (fault) rupture, lateral spreading due to liquefaction, settlement, landslide, and ground motion (shaking). This requires regional geologic knowledge as well as information on the 30 meters (100 feet) of soil directly below the building. The latter is quantified by the soil's shear wave velocity, $\overline{v}_s$, which is a key parameter in determining the intensity of ground shaking at the surface and is used to determine the soil site class in ASCE 7-10 (ASCE, 2010). This represents the soil amplification factor of the seismic wave as it passes from the epicenter to the building foundation, as shown in Figure 5-2.

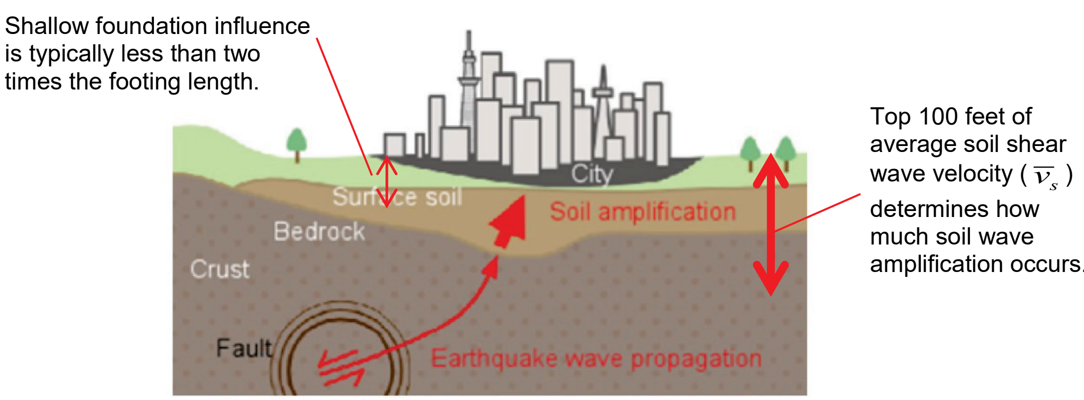


Figure 5-2 Soil depth of interest for geologic and geotechnical conditions. Figure from Ichimura et al. (2015), reprinted with permission from IEEE © 2018.

The minimum required depth and scope of investigation are specified in ASCE 41-13 § 8.2.1.1.2. Typically, the geotechnical effects on a shallow foundation are limited to approximately two times the footing length. For a deep foundation, the required depth of knowledge depends on the underlying soil characteristics and the depth of the deep foundation elements.

Therefore, the use of $\overline{v}_s$ for geotechnical parameters of a shallow foundation, and sometimes a deep foundation depending on the depth, can be misleading, particularly where a layer of weak, soft soils above stiff soil directly supports a shallow footing. In general, if there are significant variations in the soil stratification, further investigation may be warranted.

5.6 Expected Foundation Capacities and Load-Deformation Characteristics (ASCE 41-13 § 8.4)

5.6.1 Geotechnical Information

As previously mentioned, existing geotechnical reports or investigations may be used to assist in determining soil characteristics and strength. However, it is important to note that design strength values (bearing pressure or skin friction) provided in geotechnical reports in the past were traditionally based on lower bound, allowable stress values. Hence, these values must be modified in order to account for the expected strength of the foundation soil. Additionally, load-deformation characteristics have traditionally been based on long-term loading, primarily dead load, which does not reflect the anticipated load-deformation characteristics associated with short-term earthquake loads, and may underestimate the system stiffness. Therefore, it is beneficial where practical, or when required by ASCE 41-13, to develop site-specific expected capacities and short-term load-deformation characteristics. The structural engineer should work closely with the geotechnical engineer to specify the required information and to understand what, if any, safety factors have been applied to the reported values.

5.6.2 Derivation of Strength Capacities

Expected foundation capacities are determined using either a prescriptive or site-specific approach. Prescriptive expected capacities are based on available documentation from the original construction, such as construction documents or geotechnical reports, or from the estimated gravity loading on the foundations. The prescriptive approach accounts for the factor of safety traditionally used in Allowable Stress Design (ASD) of foundations to resist dead and live loads (typically 3). With this approach, the allowable capacity is based on combined dead and live loading and should not be based on dead load only or on short-term wind or seismic loads.

As an alternative, a site-specific geotechnical investigation can be performed to develop expected ultimate foundation capacities. The following examples illustrate the three prescriptive approaches. Note that for linear analysis procedures where the foundation soil is classified as deformation-controlled, these expected bearing capacities are used to develop axial and moment capacities which are multiplied by the appropriate *m*-factors specified in ASCE 41-13 § 8.4 when evaluating the acceptability of the foundation soil.

5.6.2.1 Spread Footing Expected Capacity Example

An existing building is founded on shallow spread footings. The original construction documents contain the design soil pressure of 2,500 psf as shown in Figure 5-3.

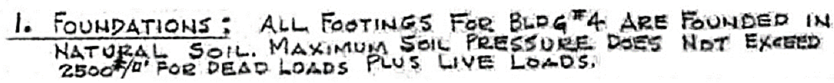

1. FOUNDATIONS: ALL FOOTINGS FOR BLDG #4 ARE FOUNDED IN NATURAL SOIL. MAXIMUM SOIL PRESSURE DOES NOT EXCEED 2500#/□' FOR DEAD LOADS PLUS LIVE LOADS.

Figure 5-3 Design soil bearing pressure from existing drawings.

The prescriptive expected bearing capacity is determined using ASCE 41-13 Equation 8-1.

$$q_c = 3q_{\text{allow}} \qquad \text{(ASCE 41-13 Eq. 8-1)}$$
$$= 3(2{,}500 \text{ psf})$$
$$= 7{,}500 \text{ psf}$$

5.6.2.2 Deep Foundation Expected Capacity Example

An existing building is supported by deep pile foundations. The original construction documents contain information on the design pile capacities as shown in Figure 5-4. The design pile capacities are 20 tons for wood piles, 27.5 tons for composite piles, and 35 tons for cast-in-place concrete piles.

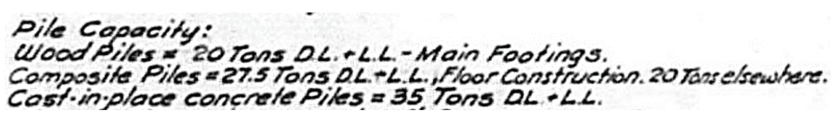

Pile Capacity:
Wood Piles = 20 Tons D.L.+L.L.-Main Footings.
Composite Piles = 27.5 Tons D.L.+L.L., Floor Construction. 20 Tons elsewhere.
Cast-in-place concrete Piles = 35 Tons DL+L.L.

Figure 5-4 Design pile capacities from existing drawings.

The prescriptive expected vertical capacity is determined using ASCE 41-13 Equation 8-2.

$$Q_c = 3Q_{\text{allow}} \qquad \text{(ASCE 41-13 Eq. 8-2)}$$

Therefore, the expected capacities are 60 tons, 82.5 tons, and 105 tons for wood, composite, and cast-in-place concrete piles, respectively.

5.6.2.3 Expected Capacity from Gravity Load Example

An existing one-story building contains interior columns at a 20-foot grid spacing, which support the roof framing. The columns are supported by shallow, normal-weight concrete spread footings which are 2’-6” square by 2’-0” deep. The roof dead load, including roof framing, is determined to be 25 psf. Per ASCE 7-10, the roof live load is 20 psf.

The dead load is calculated to include the roof dead load and the self-weight of the concrete footing. The unit weight of the concrete is 150 pcf. The weight of the concrete footing, W_{ftg}, is:

$$W_{ftg} = (2.5 \text{ ft})(2.5 \text{ ft})(2 \text{ ft})(150 \text{ pcf}) = 1{,}875 \text{ lbs}$$

The dead load from the roof, W_{roof}, is:

$$W_{\text{roof}} = (20 \text{ ft})(20 \text{ ft})(25 \text{ psf}) = 10{,}000 \text{ lbs}$$

The total dead load, Q_D, is:

$$\begin{aligned} Q_D &= W_{ftg} + W_{\text{roof}} \\ &= 1{,}875 \text{ lbs} + 10{,}000 \text{ lbs} = 11{,}875 \text{ lbs} \end{aligned}$$

Per ASCE 41-13 § 7.2.2, the live load, Q_L, is 25% of the unreduced live load of ASCE 7-10:

$$Q_L = 0.25(20 \text{ ft})(20\text{ft})(20 \text{ psf}) = 2{,}000 \text{ lbs}$$

The snow load, Q_S, is zero for this building.

The gravity load supported by the footing is calculated per ASCE 41-13 Equation 7-1. Note that ASCE 41-13 does not have a distinction between roof live load and floor live load, as there is in ASCE 7-10 and its load combinations.

$$\begin{aligned} Q_G &= 1.1(Q_D + Q_L + Q_S) && \text{(ASCE 41-13 Eq. 7-1)} \\ &= 1.1(11{,}875 \text{ lbs} + 2{,}000 \text{ lbs} + 0 \text{ lbs}) = 15{,}263 \text{ lbs} \end{aligned}$$

The footing bearing pressure under gravity load is:

$$(15{,}263 \text{ lbs})/[(2.5 \text{ ft})(2.5 \text{ ft})] = 2{,}442 \text{ psf}$$

The expected bearing capacity of the foundation is determined per ASCE 41-13 Equation 8-3:

$$\begin{aligned} q_c &= 1.5Q_G && \text{(ASCE 41-13 Eq. 8-3)} \\ &= 1.5(2{,}442 \text{ psf}) = 3{,}663 \text{ psf} \end{aligned}$$

Note that determining the expected foundation capacity based on the gravity load will typically provide a lower capacity than that determined from the specified allowable capacities. For example, the allowable bearing pressure for this structure could be 2,500 psf as in the previous example with an expected bearing capacity of 7,500 psf. Based on the calculated gravity load, the expected bearing capacity using this method is only approximately 3,600 psf. ASCE 41-13 § 8.4.1.1 does not explicitly specify whether the calculated gravity load should include the weight of the foundation element. However, since footing weight is part of the dead load measured at the footing interface, it is reasonable to include the footing weight in this calculation.

5.6.3 Bounding of Soil Load-Deformation Characteristics

To analyze and design with any material, a fundamental understanding of the material properties and variability in strength and stiffness is required. As soil has inherently more variation than most engineering materials, ASCE 41-13 explicitly incorporates the potential variations through bounding the load-deformation characteristics used in the analysis. ASCE 41-13 § 8.4.2 requires the application of an upper bound of two times the expected values and a lower bound of one-half of the expected values for any foundation, as shown in Figure 5-5.

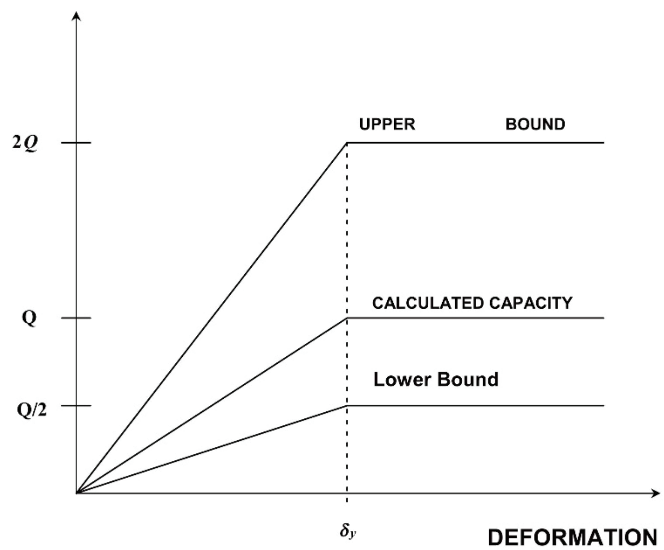


Figure 5-5 Idealized load-deformation behavior (ASCE 41-13 Figure 8-1a). Printed with permission from ASCE.

Alternatively, per ASCE 41-13 § C8.4.2, if specific data are available about the load-deformation characteristics of the soil supporting the existing foundation, the coefficient of variation, C_v, may be determined and the upper and lower bounds calculated by multiplying and dividing by $(1 + C_v)$, where C_v may not be less than 0.5. Further information on bounding is discussed in NIST GCR 12-917-21 Section 6.2.7 and Section 6.3.4.

5.6.4 Derivation of Expected Foundation Stiffness

Expected elastic soil properties are developed based on the expected properties of the soil directly beneath the footing. Specifically, these properties should represent the dynamic stiffness of the foundation under short-term (seismic) loading.

As previously discussed, geotechnical information is typically provided at an ASD level for long-term gravity loading. ASCE 41-13 seeks to utilize an expected soil shear modulus along with the appropriate Poisson's ratio in determining foundation stiffness characteristics. Close coordination with the geotechnical engineer is recommended to ascertain appropriate force-deformation relationships that are representative of dynamic loading and ultimate capacities, as well as a proper understanding and determination of the variables used to calculate prescriptive values of stiffness within ASCE 41-13.

5.6.4.1 Expected Foundation Stiffness Example

The following example illustrates the calculation of the effective shear modulus and Poisson's ratio per ASCE 41-13 § 8.4.2.2. See Figure 5-6 for the foundation dimension definitions, which are taken from ASCE 41-13 Figure 8-2.

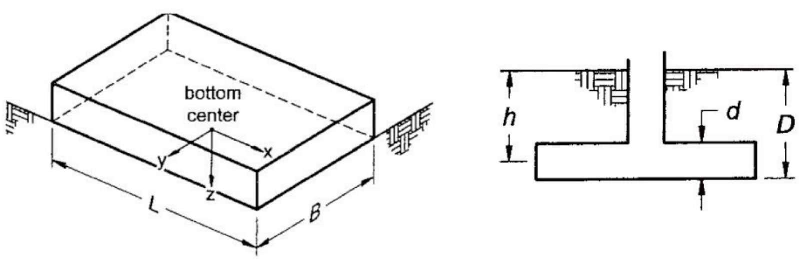


Figure 5-6 Foundation dimensions (ASCE 41-13 Figure 8-2). Printed with permission from ASCE.

Foundation information:

Rectangular footing

$B = B_f = 6$ feet

$L = L_f = 10$ feet

$D = D_f = 2$ feet (depth to foundation soil interface)

$d = 2$ feet (depth of footing)

Design information:

Sandy fill

Site Class D

$S_{XS} = 1.0$ g

$\gamma = 120$ pcf

$v_{s0} = 900$ ft/sec

γ is the in-situ total unit weight of the soil at the base of the footing. v_{s0} is the shear wave velocity at low strains in the region immediately below the soil-footing interface. Where soil properties are relatively uniform within one to two footing widths below this interface, the value at depth $D_f + (\sqrt{B_f L_f})/2$ may be used. As previously discussed, the shear wave velocity directly below the footing may be different than the shear wave velocity within the top 30 meters, $\bar{v}_s$, depending on the soil properties and layers below the structure. For this example, the average shear wave velocity, $\bar{v}_s$, for Site Class D is used to approximate v_{s0}, and the bounding of the stiffness values is intended to capture the uncertainty in the shear wave velocity below the footing. Where variations in soil properties are known to exist on the site or the engineer is not comfortable with these assumptions, a geotechnical engineer should be consulted.

Per ASCE 41-13 § 8.4.2.2, Poisson's ratio may be taken as 0.25 for any soil other than saturated clays for which the value is 0.50. The initial shear modulus, G_0, is calculated per ASCE 41-13 Equation 8-4. This equation may be used for any soil type but may underestimate the shear modulus if the shear wave velocity, v_{s0}, is measured prior to consolidation under expected vertical loads, such as if a shear wave velocity is provided in a report that was developed prior to the construction of the building.

$$G_0 = \frac{\gamma v_{s0}^2}{g} \qquad \text{(ASCE 41-13 Eq. 8-4)}$$

$$= (120\text{ pcf})(900\text{ ft/sec})^2(1\text{ k}/1{,}000\text{ lbs})/(32.2\text{ ft/sec}^2)$$

$$= 3{,}019\text{ ksf}$$

The effective shear modulus is then determined per ASCE 41-13 Table 8-2. Since $S_{XS}/2.5 = 0.4$, it is Site Class D, and $G/G_0 = 0.50$.

$$G/G_0 = 0.50$$

$$G = 0.50G_0$$

$$= 0.50(3{,}019\text{ ksf})$$

$$= 1{,}510\text{ ksf}$$

The elastic stiffness of the foundation can then be calculated for translation, rocking, or torsion using the effective shear modulus, Poisson's ratio and foundation dimensions. As an example, translational stiffness along the x-axis and rocking stiffness about the y-axis are calculated for the footing described above.

Translation along x-axis:

$$K_{x,\mathrm{sur}} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right] \qquad \text{(ASCE 41-13 Fig. 8-2)}$$

$$= \frac{(1{,}510\ \mathrm{ksf})(6)}{2-0.25}\left[3.4\left(\frac{10}{6}\right)^{0.65} + 1.2\right]$$

$$= 30{,}7547\ \mathrm{kip/ft}$$

Rocking about y-axis:

$$K_{yy,\mathrm{sur}} = \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right] \qquad \text{(ASCE 41-13 Fig. 8-2)}$$

$$= \frac{(1{,}510\ \mathrm{ksf})(6)^3}{1-0.25}\left[0.47\left(\frac{10}{6}\right)^{2.4} + 0.034\right]$$

$$= 711{,}259\ \mathrm{kip\text{-}ft/rad}$$

An upper and lower bound of these stiffness values, multiplied by 2 and 0.5 respectively, would be used in the evaluation unless more refined load-deformation information was available, as discussed in Section 5.6.3 of this *Guide*.

5.6.5 Bearing Pressure Distribution

Depending on the flexibility of the footing, as well as loading and dimensions, the distribution of bearing pressures on the soil below will vary. Conventional foundation design has traditionally evaluated the bearing pressure distribution under vertical and overturning forces as triangular or trapezoidal. The expected moment capacity of the footing with respect to the soil in ASCE 41-13 allows for concentrated (rectangular) stresses at the footing edges, particularly as uplift occurs at the opposite edge of the footing, as shown in Figure 5-7, where q is the bearing pressure under vertical load; P is the axial load including gravity and seismic loads; M is the applied moment; q_c is the expected bearing capacity of the soil and M_c is the expected moment strength of the footing as limited by the soil calculated per ASCE 41-13 Equation 8-10. The modeling parameters and acceptance criteria for linear and nonlinear procedures are dependent on the ratio of vertical stress to ultimate soil bearing pressure, which is defined as the critical contact area, A_c ($A_c = P/q$).

5.6.6 Force-Controlled vs. Deformation-Controlled Actions

In conjunction with the soil acceptance criteria, the material-specific requirements for the structural component are determined from the relevant material chapters of ASCE 41-13, as discussed in Section 5.3 of this *Guide*. The requirements for typical foundation materials are provided in Table 5-1, below.

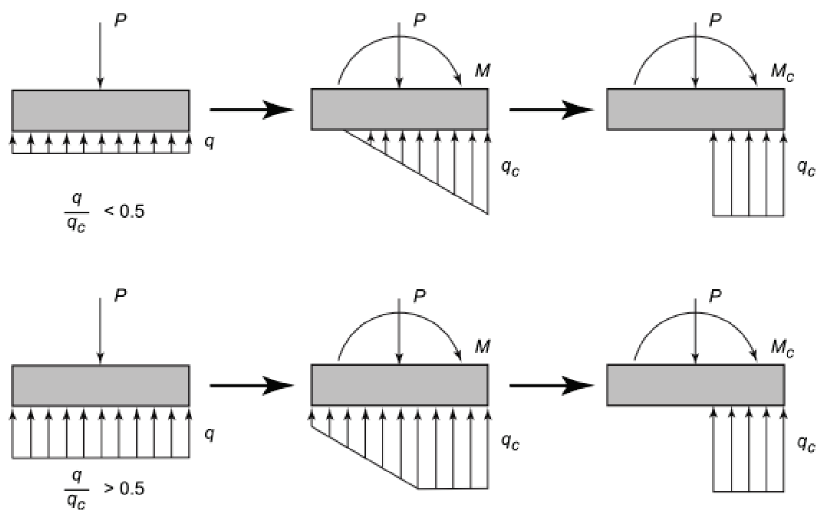


Figure 5-7 Bearing pressure distributions for rectangular and I-shaped rocking footings (FEMA, 1997a).

Table 5-1 Material-Specific Structural Foundation Requirements

Foundation Material	ASCE 41-13 Section	Action Type
Steel	§ 9.9.4	Deformation-controlled for steel pile; Force-controlled for connection from pile to pile cap
Concrete	§ 10.12.3	Force-controlled; the required capacity is limited by 125% of the capacity of the supported vertical component
Masonry	§ 11.6.2	Force-controlled and modeled as elastic with no inelastic deformation capacity unless demonstrated through ASCE 41-13 § 7.6
Wood	§ 12.6.2	Flexure and axial loads are considered deformation-controlled with *m*-factors per ASCE 41-13 Table 12-3. Acceptability of soil below wood footings determined per ASCE 41-13 Chapter 8.

The action classification and acceptance criteria for each material are intended to align with ASCE 41-13 § 7.5.1.2, which defines the load-deformation characteristics of force-controlled and deformation-controlled actions. Different action classifications and acceptance criteria could be developed for nonlinear procedures based on ASCE 41-13 § 7.5.1.2 and § 7.6. For instance, where a force-controlled foundation element is shown to have the ductile behavior of a deformation-controlled action, modeling parameters and acceptance criteria derived from testing per ASCE 41-13 § 7.6 may be utilized in the evaluation and retrofit of the foundation. There is some debate within the profession whether certain items, such as concrete grade beams or flexure within a footing, can be treated as deformation-controlled actions using *m*-factors for nonconforming concrete beams. This is a topic that continues to be debated by the standards committee but is not explicitly permitted for the linear procedures. For nonlinear procedures, the development of acceptance criteria is explicitly permitted by the process described above.

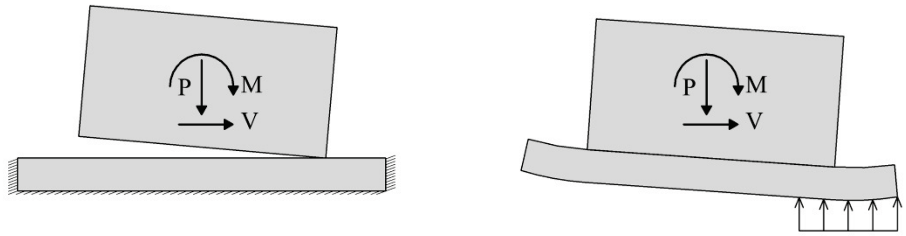
5.7 Shallow Foundation Evaluation and Retrofit

5.7.1 Overview

During an earthquake event, the demands placed on the soil are significantly larger than those associated with non-earthquake loads. These demands are deemed to be acceptable provided the structural and nonstructural components are able to accommodate the deformations and/or to resist the associated forces imposed. These forces and/or deformations may be determined in ASCE 41-13 using fixed-base or flexible-base assumptions, depending on the relative flexibility of the structural footing as compared to the foundation soil. Where the deformations of the foundation and the soil are not explicitly captured with a flexible-base analysis, the acceptance criteria are intended to take into account the actual effect of foundation flexibility on the structure.

Even when meeting the foundation acceptance criteria, significant deformation is likely to occur at the foundation, particularly for performance objectives less stringent than Immediate Occupancy. For shallow foundations, the response is highly dependent on the total axial load, including gravity loads and additional axial load due to seismic forces, during the earthquake. Depending on the magnitude of axial load, foundation uplift, sliding, ratcheting, and/or settlement can occur. Generally, rocking foundations that undergo uplift and do not overturn are stable. However, the occurrence of any of the above mechanisms will dramatically increase force and deformation demands on the superstructure as compared to building response under non-earthquake loads or based on analysis using a fixed-base assumption.

ASCE 41-13 addresses overturning effects on the structure above the footing, the footing itself, and the foundation soil. Overturning of the structure above the footing, as illustrated in Figure 5-8, is evaluated for global stability per the requirements of ASCE 41-13 § 7.2.8. The foundation soil is evaluated based on ASCE 41-13 Chapter 8 requirements. The structural footing is evaluated based on the Chapter 8 requirements and the appropriate material chapter, depending on the type of foundation.



(a)
Overturning of structure above footing
(ASCE 41-13 § 7.2.8)

(b)
Overturning at footing and soil
(ASCE 41-13 Chapter 8 and
material chapters 9-12)

Figure 5-8 Overturning evaluation for structure, footing, and foundation soil.

In addition to the global stability evaluation of the superstructure overturning above the foundation, ASCE 41-13 § 7.2.8 requires overturning at the soil-foundation interface to be evaluated using the provisions of ASCE 41-13 Chapter 8, not ASCE 41-13 Equations 7-5 or 7-6, as was frequently performed with ASCE 41-06. The original intent of the overturning evaluation contained in ASCE 41-13 § 7.2.8, as documented in FEMA 273 (FEMA, 1997a), was to address structures and their individual vertical elements that overturn at their base to confirm global stability. Although global building overturning is very rare, displacements caused by overturning elements, such as walls at their base, can result in significant stress to elements that frame into the wall (or are reliant on the wall to limit deformations). Elements such as wood-sheathed shear walls may not have positive attachments between levels, in which case they have a propensity to overturn at their base (see Figure 5-9).

In keeping with design practice for new buildings using an *R*-factor in accordance with ASCE 7, a μ_{OT} factor is used in ASCE 41-13 § 7.2.8.1 to assess these types of elements and account for the fact that the LSP and LDP seismic forces are not reduced (see Section 2.2 of this *Guide* for a comparison of ASCE 7-10 and ASCE 41-13 design principles). This approach recognizes that the available dead load to restore the element from overturning on a rigid base results in a ductile response and tends to self-right the wall and prevent the element from tipping over.

ASCE 41-13 § 7.2.8 states that overturning effects on the footing and foundation-soil interface should be based on ASCE 41-13 Chapter 8. Therefore, the provisions of ASCE 41-13 § 7.2.8 should not be used to determine the forces imposed by overturning on the footing and foundation-soil interface. The primary reason for this is that the soil or deep foundation does not represent a rigid base; rather, the underlying soil may be subject to elastic and plastic behavior, resulting in significant deformation. To accurately assess and to include this soil flexibility in the structure assessment, ASCE 41-13 Chapter 8 is to be followed. Within ASCE 41-13 Chapter 8, a rigid base may be assumed, and *m*-factors are provided that may be applied to the restoring dead load to resist overturning, as shown in the example in Section 5.7.4.1 of this *Guide*, but the reduction of earthquake forces is much lower to prevent excessive soil yielding.

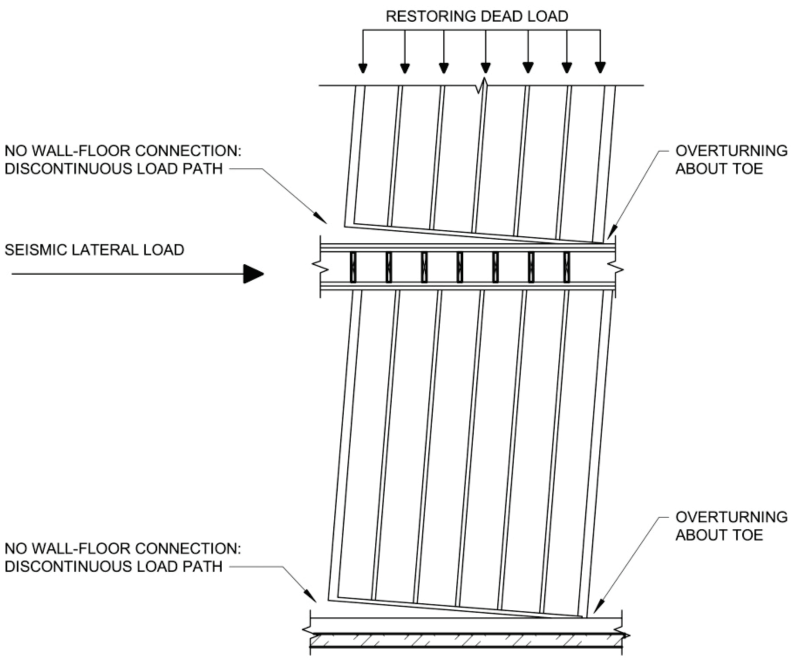


Figure 5-9 Overturning according to ASCE 41-13 § 7.2.8.

The background to the shallow foundation approach within ASCE 41-13, which has changed significantly from its predecessors, is to provide foundation acceptance criteria to include and limit building deformations arising from foundation deformation. The changes to the foundation provisions in ASCE 41-13 are summarized in Kutter et al. (2016). The acceptance criteria are based on the vertical loading, including gravity and seismic forces, on the footing as well as the shape of the footing. It is important to note that the vertical load is based on the expected seismic loading, not the unreduced seismic forces obtained from a linear static procedure (LSP) or linear dynamic procedure (LDP). The expected seismic load may be limited by the capacity of a component in the load path and may be determined from a capacity-based analysis. The elastic force demand determined from a linear analysis will overestimate the actual expected force that can be transmitted to the foundation. In this case, the use of the higher axial seismic force from the linear analysis will yield more stringent

ASCE 41-17

ASCE 41-13 § 8.4.2.4.4 currently states that if a nonlinear dynamic seismic analysis (NDP) "accurately captures characteristics of settling, soil plasticity, and gapping, the acceptability of soil displacements shall be based on the ability of the structure to accommodate the displacements calculated by the NDP within the acceptance criteria for the selected performance objective." Therefore, in this case, the acceptance criteria for the structural components are used in lieu of the acceptance criteria for foundations in Chapter 8. However, the subsequent sentence states: "If these characteristics are adequately captured by the NDP, the acceptability of soil displacements shall be based on the foundation rotation limits in Table 8-4." This statement contradicts the previous sentence and has been corrected in ASCE 41-17 to state: "If these characteristics are ***NOT*** adequately captured by the NDP, the acceptability of soil displacements shall be based on the foundation rotation limits in Table 8-4."

acceptance criteria than should be required when the expected seismic force is used. ASCE 41-17 has been modified to clarify this requirement.

Alternatively, if foundation transient and permanent displacements are explicitly captured in a nonlinear dynamic procedure (NDP), the intent of ASCE 41-13 is to permit the structure to be evaluated with structural component acceptance criteria, rather than foundation soil acceptance criteria, under these displacements. However, ASCE 41-13 § 8.4.2.4.4 is incorrectly worded and has been corrected in ASCE 41-17. Note that the analysis procedures to accurately capture permanent foundation deformations are complicated and are dependent on accurate modeling of soil load-deformation characteristics. There are new methods available in the industry to better determine building performance and residual deformations, such as FEMA P-58 (FEMA, 2012c), but it is important to include foundation flexibility and post-yield behavior in the analysis to capture the contribution of foundation displacements on the overall performance of the structure.

5.7.2 Foundation Modeling Approaches

In general, foundation evaluation and design in ASCE 41-13 allows for separate assessment of deformations due to rocking (overturning), sliding, and settlement. However, the modeling parameters and acceptance criteria in ASCE 41-13 are based on rocking-dominant behavior. The standard provides three methods of modeling and evaluation of shallow foundations. The selection of the appropriate method is dependent on a fixed (rigid) base or flexible base (building's boundary condition) assumption and the flexibility of the footing relative to the soil. The footing flexibility assessment should take the soil bearing pressure distribution, for instance, whether uplift occurs, as well as the strength of the foundation element into consideration as stated in ASCE 41-13 § 8.4.2.1. Where the foundation is flexible relative to the soil or yielding of the structural foundation occurs, the footing is classified as flexible. Methods 1 and 2 are intended for rigid foundations relative to the soil, and Method 3 is used for flexible foundations relative to the soil. ASCE 41-13 § 7.2.3.5 introduces foundation modeling requirements and references ASCE 41-13 Chapter 8 for foundation modeling and acceptability as well as ASCE 41-13 § 7.2.7 for soil-structure interaction requirements. The global stability evaluation of ASCE 41-13 § 7.2.8 should also be performed. Figure 5-10 provides a flowchart to assist with determining which method is appropriate.

The footing dimension, L, used to assist in determining the footing flexibility in ASCE 41-13 Equation C8-1 is straightforward for an isolated rectangular footing. For complex footing geometry, such as L-shaped or basement conditions, ASCE 41-13 provides minimal guidance on the appropriate length. Section 5.6.7.1 of this *Guide* provides some guidance for a mat foundation. For other conditions, engineering judgment is required.

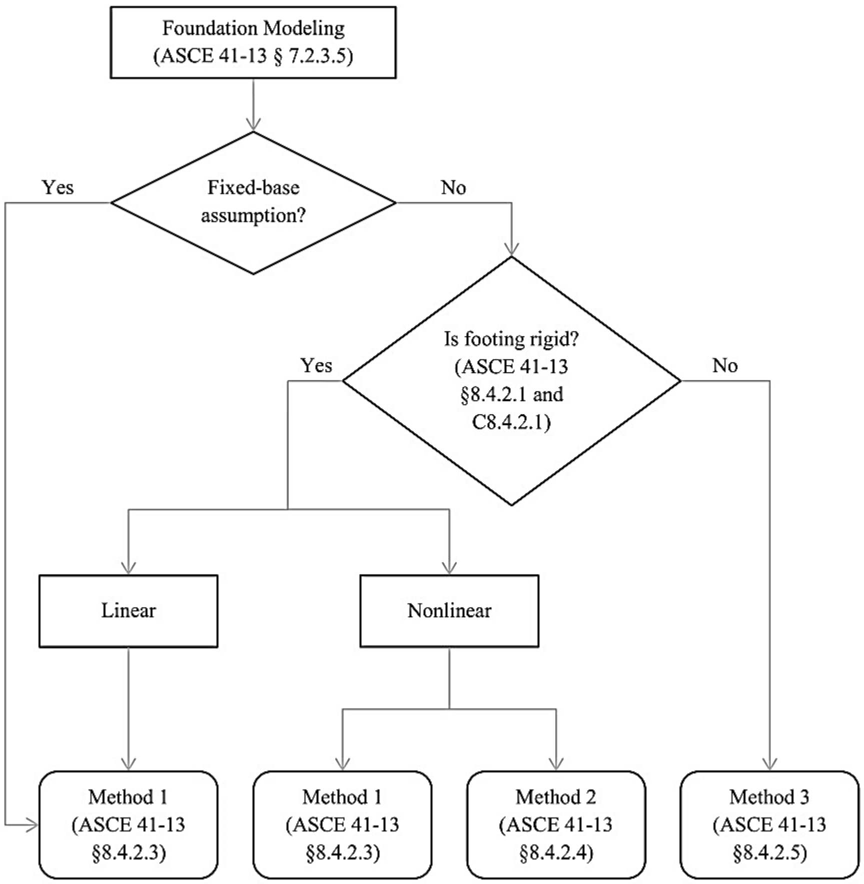


Figure 5-10 Overall flowchart with reference to sections in ASCE 41-13.

5.7.2.1 Method 1

Method 1 uses uncoupled moment, shear, and axial springs to model rigid foundations such that the moment and shear behaviors are independent of the axial load. Method 1 may be used for any linear or nonlinear procedure (LSP, LDP, NSP, or NDP). Analysis procedure selection is discussed in Chapter 4 of this *Guide*. For linear procedures, the foundation soil is classified as deformation-controlled and may be modeled as a fixed or flexible base at the discretion of the engineer, as illustrated in Figure 5-11, where k_{sv}, k_{sh}, and k_{sr} represent vertical, horizontal, and rotational stiffness, respectively.

Where a fixed base is assumed, prescriptive *m*-factors are applied that are typically more conservative than with a flexible base assumption, where *m*-factors are determined from ASCE 41-13 Table 8-3. For nonlinear procedures, the flexibility of the foundation soil is modeled explicitly with modeling parameters and acceptance criteria per ASCE 41-13 Table 8-4. A flowchart for the Method 1 procedure is shown in Figure 5-12.

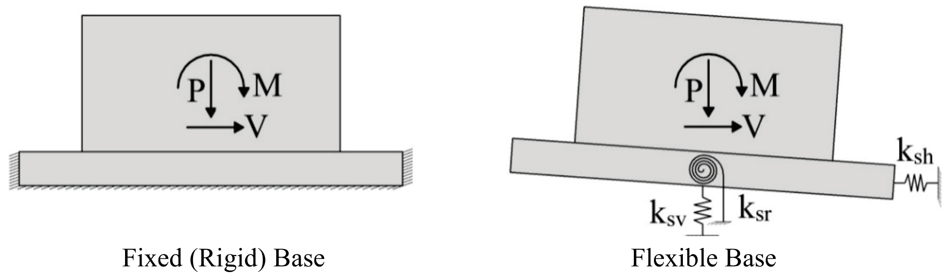


Figure 5-11 Method 1 foundation modeling approaches.

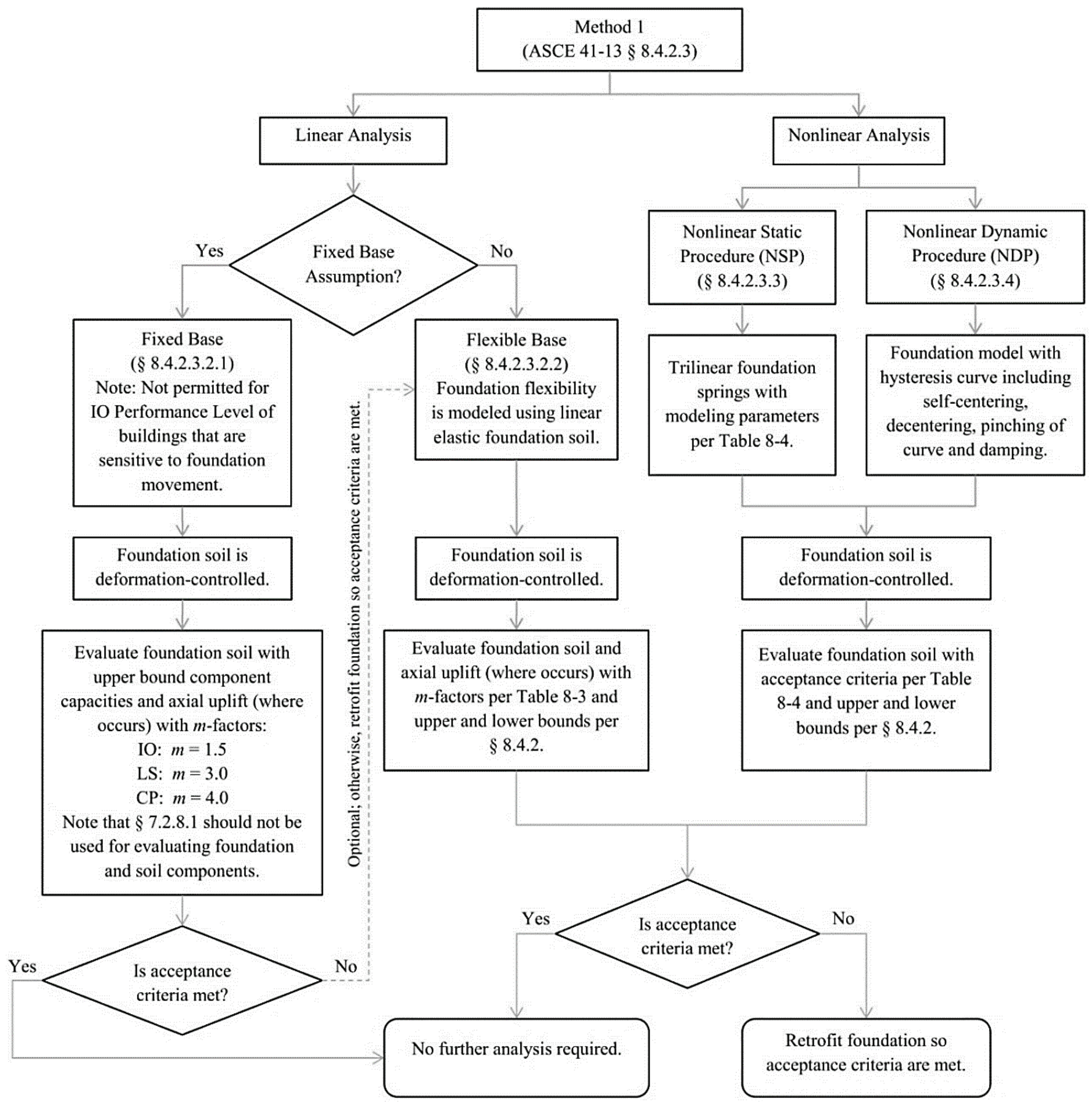


Figure 5-12 Method 1 flowchart with reference to sections in ASCE 41-13.

In ASCE 41-06, the foundation provisions assumed uncapped strength and infinite soil ductility with no consideration of potential consequences of foundation rocking such as permanent settlement. ASCE 41-13 requires foundations to be evaluated in linear procedures with *m*-factors, similar to other deformation-controlled structural components. Where a foundation is evaluated and is shown to be adequate using the *m*-factors provided in ASCE 41-13, the foundation is expected to experience limited inelastic deformations due to bearing capacity failure of the soil.

Note that ASCE 41-17 includes a modification to the Method 1 evaluation for uplift with linear procedures. Method 1 requires the evaluation of two different actions for linear procedures: soil bearing capacity and overturning stability. In ASCE 41-13, the same *m*-factors are applied to both of these actions. However, uplift due to overturning that overcomes the restoring dead load is ductile and self-centering as the footing comes back down. Due to uplift being deemed more ductile and likely self-centering, larger *m*-factors are permitted to be applied to the restoring dead load in ASCE 41-17. These *m*-factors are similar to the μ_{OT} factors provided for rigid body rotation in ASCE 41-13 § 7.2.8.

5.7.2.2 Method 2

As noted in ASCE 41-13 § 7.2.8, Method 2 "is recommended for nonlinear procedures and is anticipated to be too involved for linear procedures." Method 2 provides an alternative approach for rigid foundations that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moment. Method 2 is the preferred approach when there is significant variation in axial load. The moment-rotation and vertical load-deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions of the foundation to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness provided in ASCE 41-13 Figure 8-2 (see Figure 5-13). The same modeling parameters and acceptance criteria used in Method 1 apply to Method 2. A flowchart for the Method 2 procedure is shown in Figure 5-14.

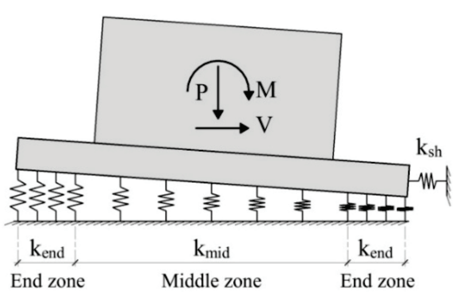


Figure 5-13 Method 2 foundation modeling.

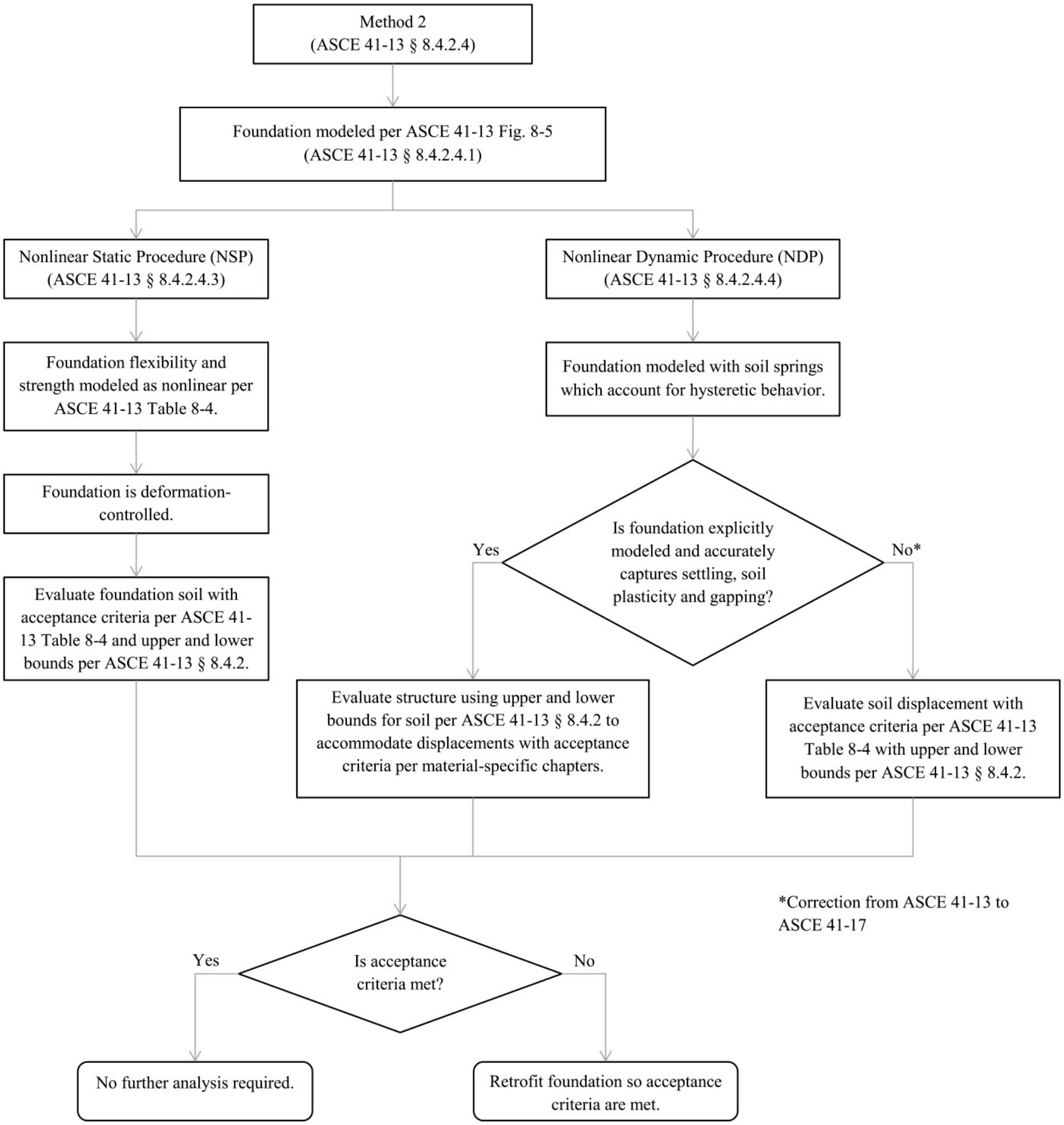


Figure 5-14 Method 2 flowchart with reference to sections in ASCE 41-13.

5.7.2.3 Method 3

For non-rigid foundations where the structural foundation is flexible relative to the soil, Method 3 uses compression-only springs with uniform strength and stiffness beneath the structural footing (see Figure 5-15). Method 2 utilizes springs of different stiffness at the middle and end zones to model a rigid foundation with coupled axial and overturning. Method 3 uses springs with uniform stiffness because the flexible footing is not as sensitive to coupling between axial and overturning and can redistribute forces based on the relative stiffness between the soil and footing. The stiffness used in Method 3 is based on theoretical solutions for beams and plates in contact with elastic supports. Foundation rotation due to deformation of the soil is governed by the same acceptance criteria and the structural foundation element is also evaluated based on acceptance criteria for the component itself. A flowchart for the Method 3 procedure is shown in Figure 5-16.

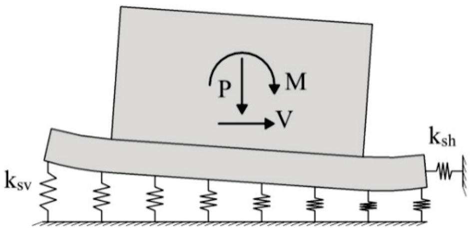


Figure 5-15 Method 3 foundation modeling.

5.7.3 Governing Jurisdiction Discussion

Given the nonlinear complexity of the foundation system and the importance of providing an effective and cost-efficient retrofit scope, dialogue and pre-application meetings with the owner, subject matter experts, the governing jurisdiction, and other stakeholders are highly recommended. This is particularly true where code interpretations or analysis procedures not explicitly addressed, such as LSP or LDP with Method 3, are utilized. Since the findings from the analysis are required to determine which method and process are most relevant and favorable, ongoing dialogue with all stakeholders is recommended. In addition to the broader project issues, the following foundation-specific items are recommended to be considered in developing the basis of design and design criteria for review and approval with the aforementioned stakeholders:

> **Useful Tip**
> Foundation approach, modeling parameters, and acceptance criteria should be developed in collaboration with the Authority Having Jurisdiction prior to developing Construction Documents.

- Entire building analysis procedure (LSP, LDP, NSP, or NDP)
- Procedure for classifying foundation flexibility relative to the soil stiffness
- What foundation flexibility is included and excluded from the building model
- Effective foundation sizes for determining A_c/A
- Derivation of axial and moment actions on foundation
- Use of nonlinear analysis
- Procedure for limit-state analysis to determine the expected axial force including seismic loads by evaluating the maximum force that can be

transmitted to the foundation (note that ASCE 41-17 allows for a capacity-based determination of axial force)

- Effective footing width and length, where not discernable from the foundation configuration
- Foundation structure: force-controlled and essentially elastic per ASCE 41-13 § 10.12.3 or deformation-controlled as demonstrated through ASCE 41-13 § 7.5.1.2

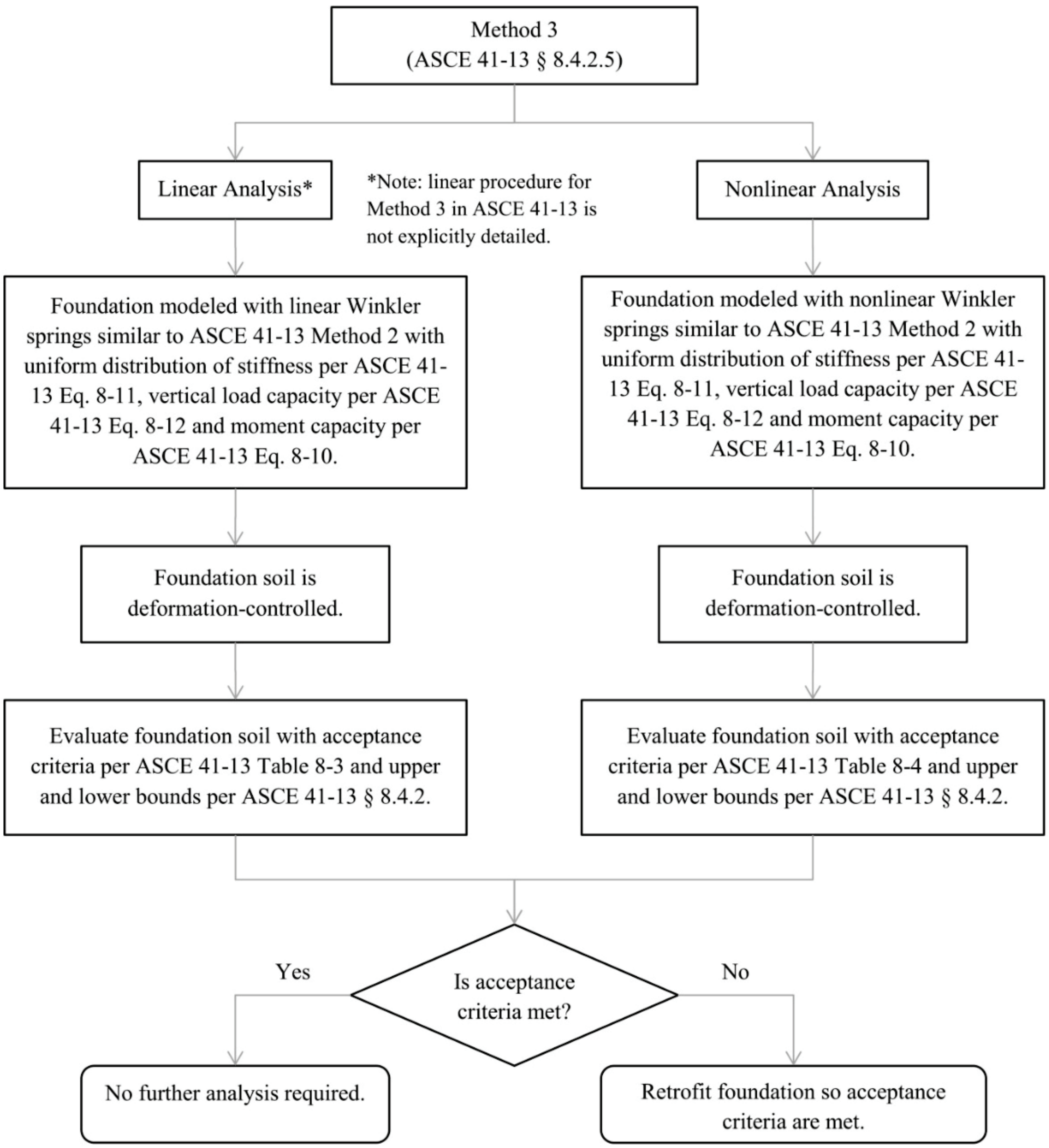


Figure 5-16 Method 3 flowchart with reference to sections in ASCE 41-13.

5.7.4 Method 1 Example

The following example includes an analysis of an existing shallow foundation system when a new reinforced concrete shear wall is added between two existing concrete columns supported on spread footings that are tied together with a grade beam.

Various foundation configurations are explored, including both I-shaped and rectangular footings beneath the new shear wall.

The building is evaluated using LSP with Method 1 provisions. The existing foundation is first assessed with the foundation modeled as a fixed base (ASCE 41-13 § 8.4.2.3.2.1). The foundation is then evaluated and a retrofit is designed using a flexible base assumption (ASCE 41-13 § 8.4.2.3.2.2).

The geometric properties of the footing, as defined in Figure 5-17 and Figure 5-18, are given as:

L = 33 ft

B = 8 ft

t_f = 8 ft

t_w = 2 ft

h = 2 ft

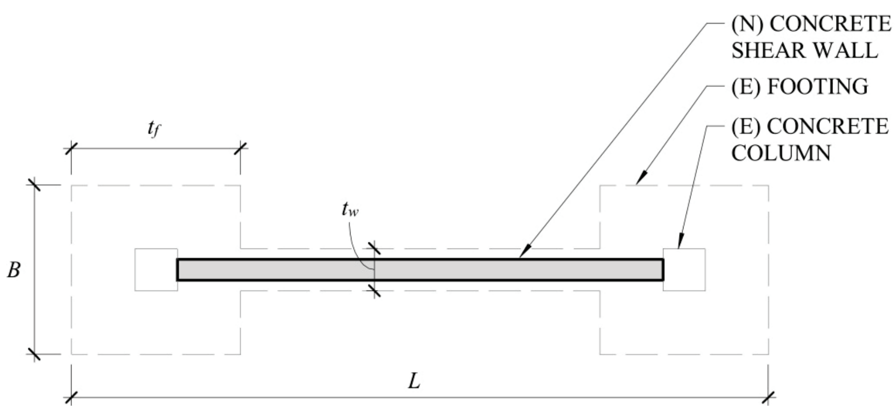


Figure 5-17 Footing and shear wall plan.

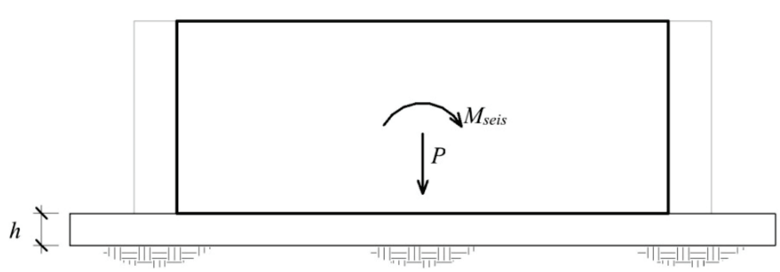


Figure 5-18 Footing and shear wall elevation.

The axial load, P, due to gravity, including the self-weight of the footing, and seismic loads and overturning moment, M_{seis}, on the footing at the base of the shear wall, as defined in Figure 5-18, are given as:

P = 494 k

M_{seis} = 33,000 kip-ft

A geotechnical report is available from the original construction of the building with an allowable bearing pressure for dead (including footing weight) plus live loads of:

q_{allow} = 4 ksf (ASCE 41-13 Eq. 8-1)

Therefore, the expected bearing capacity, q_c, is:

$q_c = 3q_{allow} = 3(4 \text{ ksf}) = 12 \text{ ksf}$ (ASCE 41-13 Eq. 8-1)

The area of the footing, A_f, is:

$$
\begin{aligned}
A_f &= 2Bt_f + (L-2t_f)t_w \\
&= 2(8 \text{ ft})(8 \text{ ft}) + (33 \text{ ft} - 2(8 \text{ ft}))(2 \text{ ft}) \\
&= 162 \text{ ft}^2
\end{aligned}
$$

5.7.4.1 Method 1 Fixed Base Example

A fixed base assumption is used to perform an initial evaluation of the existing footing. ASCE 41-13 § 8.4.2.3.2.1 requires the consideration of both the bearing capacity of the soil and overturning stability, where overturning is resisted by the restoring dead load multiplied by an *m*-factor. The existing foundation condition can be approximated as two isolated footings coupled together by the essentially rigid superstructure above, as shown in Figure 5-19. In this case, the web of the footing under the wall is ignored to simplify the analysis. The loading due to gravity and seismic loads at the footings of each end are calculated. Note that the axial load, P, includes only gravity loads in this case.

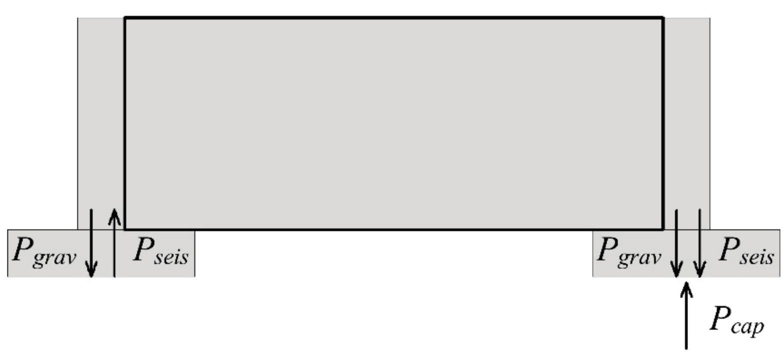


Figure 5-19 Method 1 fixed base example.

$P_{\text{grav}} = P/2 = (494 \text{ k})/2 = 247 \text{ k}$

$Q_E \quad = P_{\text{seis}} = M_{\text{seis}}/(L - t_f) = (33{,}000 \text{ kip-ft})/(33 \text{ ft} - 8 \text{ ft}) = 1{,}320 \text{ k}$

The force demand on the compression footing, Q_{UD}, is the sum of the gravity and seismic forces per ASCE 41-13 Equation 7-34. The gravity load is determined using the load combination of ASCE 41-13 Equation 7-1.

$Q_G \quad = 1.1P_{\text{grav}} = 1.1(247 \text{ k}) = 272 \text{ k}$

$Q_{UD} = Q_G + Q_E = 272 \text{ k} + 1{,}320 \text{ k} = 1{,}592 \text{ k}$

The compression force is resisted by the upper-bound bearing capacity of the soil with an m-factor of 3.0 for the Life Safety Performance Level. Per ASCE 41-13 § 8.4.2.3.2.1, it is permitted to take the upper-bound bearing capacity, which per ASCE 41-13 § 8.4.2 is two times the expected capacity.

$2q_c = 2(12 \text{ ksf}) = 24 \text{ ksf}$

The axial capacity of the 8-foot square footing is:

$Q_{CE} \quad = P_{cap} = (B)(t_f)(2q_c) = (8 \text{ ft})(8 \text{ ft})(24 \text{ ksf}) = 1{,}536 \text{ k}$

$mQ_{CE} = (3.0)(1{,}536 \text{ k}) = 4{,}608 \text{ k}$

$mQ_{CE} > Q_{UD}$ OK

To evaluate the overturning stability, the uplift force on the tension side of the wall is compared to the restoring dead load with an m-factor of 3.0 applied for the Life Safety Performance Level. The tension force is:

$Q_{UD} \quad = Q_E = P_{\text{seis}} = 1{,}320 \text{ k}$

The restoring dead load is based on the factored gravity load calculated with ASCE 41-13 Equation 7-2:

$Q_{CE} \quad = 0.9P_{\text{grav}} = 0.9(247 \text{ k}) = 222 \text{ k}$

$mQ_{CE} \; = (3.0)(222 \text{ k}) = 666 \text{ k}$

$mQ_{CE} < Q_{UD}$ NO GOOD

Therefore, the existing footing is not adequate for overturning stability. As recommended in ASCE 41-13 § C8.4.2.3.2.1, the foundation will be modeled as flexible with m-factors from ASCE 41-13 Table 8-3 to further evaluate and retrofit the existing footing. Note that the structural footing capacity should also be evaluated as part of the foundation analysis as is done in the flexible base example. When evaluating the strength of the footing on the compression side, the expected bearing capacity (not the upper bound capacity) would be used.

5.7.4.2 Method 1 Flexible Base Example

In order to find the rocking moment capacity, M_c, the vertical bearing pressure, q, is calculated:

$$q = \frac{P}{A_f} = \frac{494 \text{ k}}{162 \text{ ft}^2} = 3.05 \text{ ksf}$$

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \qquad \text{(ASCE 41-13 Eq. 8-10)}$$

$$= \frac{(33 \text{ ft})(494 \text{ k})}{2}\left(1 - \frac{3.05 \text{ ksf}}{12 \text{ ksf}}\right)$$

$$= 6{,}079 \text{ kip-ft}$$

Missing area ratio (MAR) for the I-shaped footing is determined per ASCE 41-13 Figure 8-3 and the terminology in ASCE 41-13 Table 8-3:

$$A_{\text{rect}} = BL = (8 \text{ ft})(33 \text{ ft}) = 264 \text{ ft}^2$$

$$\text{MAR} = \frac{A_{\text{rect}} - A_f}{A_{\text{rect}}}$$

$$= \frac{264 \text{ ft}^2 - 162 \text{ ft}^2}{264 \text{ ft}^2}$$

$$= 0.39$$

The critical contact area, A_c, is:

$$A_c = \frac{P}{q_c} = \frac{494 \text{ k}}{12 \text{ ksf}} = 41.2 \text{ ft}^2$$

The critical contact area ratio is:

$$\frac{A_c}{A_f} = \frac{41.2 \text{ ft}^2}{162 \text{ ft}^2} = 0.25$$

The length of the contact area, L_c, is:

$$L_c = \frac{A_c}{B} = \frac{41.2 \text{ ft}^2}{8 \text{ ft}} = 5.15 \text{ ft}$$

$$\frac{B}{L_c} = \frac{8 \text{ ft}}{5.15 \text{ ft}} = 1.55$$

ASCE 41-13 Table 8-3 is used to interpolate between the Life Safety Performance Level m-factors considering the length of the contact area and the critical contact area and missing area ratios shown above. The corresponding m-factor is found to be $m = 2.4$.

$$mM_c = 2.4(6{,}079 \text{ kip-ft}) = 14{,}590 \text{ kip-ft}$$

$$M_{OT} = M_{\text{seis}} = 33{,}000 \text{ kip-ft}$$

$$mM_c < M_{OT} \qquad \text{NO GOOD}$$

If the I-shaped footing is augmented to become a complete rectangle, the additional footing weight, W_{add}, should be considered in the value of P, since the bearing capacity is not based on a net allowable bearing pressure and is defined as the vertical load on the soil at the footing interface:

$$W_{\text{add}} = (L - 2t_f)Bh\gamma$$

$$= (33\text{ ft} - 2(8\text{ ft}))(8\text{ ft})(2\text{ ft})(150\text{ pcf}/1000) = 41\text{ k}$$

$$P = 494\text{ k} + 41\text{ k} = 535\text{ k}$$

The area of the footing, A_f, is:

$$A_f = (8\text{ ft})(33\text{ ft}) = 264\text{ ft}^2$$

The vertical bearing pressure and rocking moment capacity are:

$$q = \frac{535\text{ k}}{264\text{ ft}^2} = 2.03\text{ ksf}$$

$$M_c = \frac{(33\text{ ft})(535\text{ k})}{2}\left(1 - \frac{2.03\text{ ksf}}{12\text{ ksf}}\right)$$

$$= 7{,}334\text{ kip-ft}$$

The factors for determining the appropriate m-factor are:

$$A_c = \frac{535\text{ k}}{12\text{ ksf}} = 44.6\text{ ft}^2$$

$$\frac{A_c}{A_f} = \frac{44.6\text{ ft}^2}{264\text{ ft}^2} = 0.17$$

$$L_c = \frac{44.6\text{ ft}^2}{8\text{ ft}} = 5.58\text{ ft}$$

$$\frac{B}{L_c} = \frac{8\text{ ft}}{5.58\text{ ft}} = 1.43$$

Next, ASCE 41-13 Table 8-3 is used to interpolate between the Life Safety Performance Level m-factors considering the A_c/A_f and B/L_c values calculated above. The corresponding m-factor is found to be $m = 3.17$.

$$mM_R = 3.17(7{,}334\text{ kip-ft}) = 23{,}249\text{ kip-ft}$$

$$M_{OT} = M_{\text{seis}} = 33{,}000\text{ kip-ft}$$

$mM_c < M_{OT}$ NO GOOD

An additional 1’-6” of width is provided on each side of the rectangular footing, resulting in a total width of 11 ft. The total dead load including the additional footing weight can be calculated as follows:

$$P = 535 \text{ k} + (3 \text{ ft})(33 \text{ ft})(2 \text{ ft})(150 \text{ pcf}/1000) = 565 \text{ k}$$

The area of the footing, A_f, is:

$$A_f = (11 \text{ ft})(33 \text{ ft}) = 363 \text{ ft}^2$$

The vertical bearing pressure and rocking moment capacity are:

$$q = \frac{565 \text{ k}}{363 \text{ ft}^2} = 1.56 \text{ ksf}$$

$$M_c = \frac{(33 \text{ k})(565 \text{ ft})}{2}\left(1 - \frac{1.56 \text{ ksf}}{12 \text{ ksf}}\right)$$

$$= 8{,}111 \text{ kip-ft}$$

The factors for determining the appropriate *m*-factor are:

$$A_c = \frac{565 \text{ k}}{12 \text{ ksf}} = 47.1 \text{ ft}^2$$

$$\frac{A_c}{A_f} = \frac{47.1 \text{ ft}^2}{363 \text{ ft}^2} = 0.13$$

$$L_c = \frac{47.1 \text{ ft}^2}{11 \text{ ft}} = 4.28 \text{ ft}$$

$$\frac{B}{L_c} = \frac{11 \text{ ft}}{4.28 \text{ ft}} = 2.57$$

Next, ASCE 41-13 Table 8-3 is used to interpolate between the Life Safety Performance Level *m*-factors considering the A_c/A_f and B/L_c values calculated above. The corresponding *m*-factor is found to be $m = 4.6$.

$$mM_R = 4.6(8{,}111 \text{ kip-ft}) = 37{,}311 \text{ kip-ft}$$

$$M_{OT} = M_{\text{seis}} = 33{,}000 \text{ kip-ft}$$

$$mM_c > M_{OT} \quad \text{OK}$$

Now that the footing size is adequate, the existing footing is evaluated for strength. The footing including retrofit is shown in Figure 5-20 and Figure 5-21. The following strength checks, as shown in Figure 5-22, will be performed on the existing footing:

- Flexure
- One-way shear
- Two-way shear

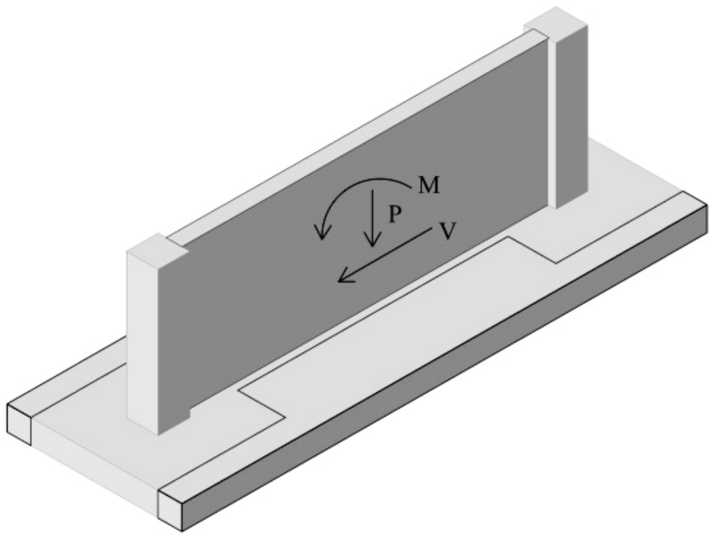


Figure 5-20 Footing with retrofit.

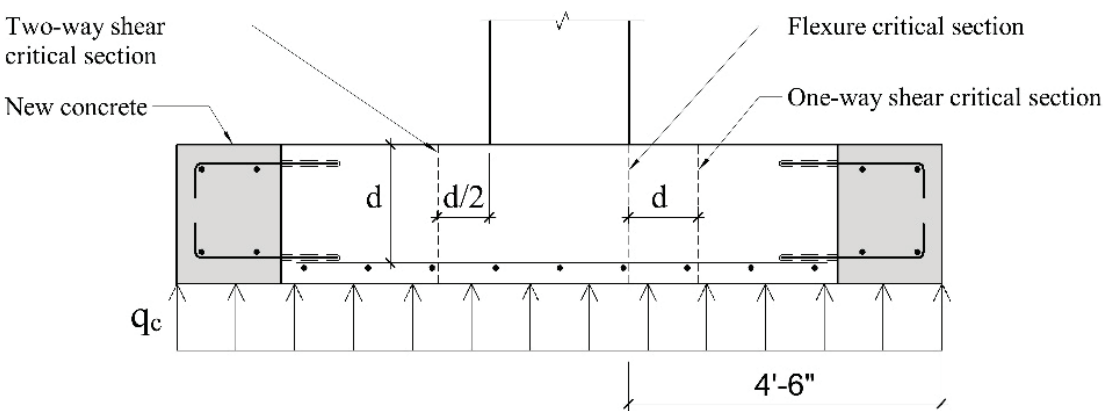


Figure 5-21 Footing retrofit section.

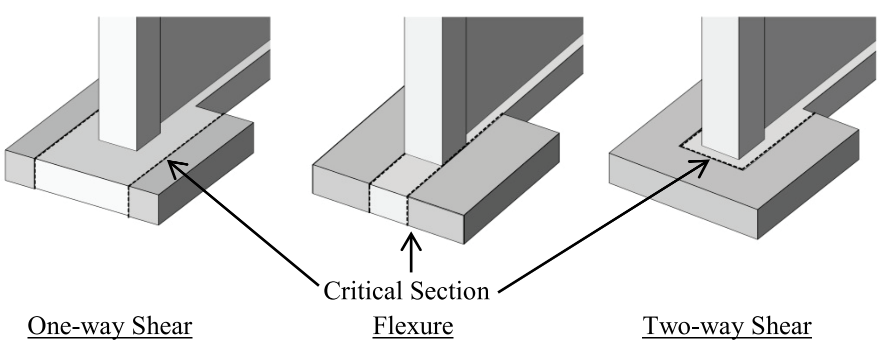


Figure 5-22 Footing strength evaluations.

Note that shear friction at the interface between the new and existing concrete would also need to be evaluated, but is outside of the scope of this example.

The concrete sections are evaluated as force-controlled per ASCE 41-13 § 10.12.3 with strength calculated per ACI 318-14 (ACI, 2014). The following information is provided on the construction drawings:

f'_c = 3,000 psi (specified and lower-bound)

f_y = 60 ksi (specified and lower-bound)

d = 20 inches

#6 @ 12" on center each direction top and bottom

One-way shear is evaluated parallel to the wall with the critical section located a distance, d, from the face of the wall. A one-foot segment of footing is evaluated.

The one-way shear demand in the footing is based on the expected bearing strength, q_c:

$$
\begin{aligned}
V_u &= q_c(4.5\text{ ft} - d)(1\text{ ft}) \\
&= (12\text{ ksf})(4.5\text{ ft} - 1.67\text{ ft})(1\text{ ft}) \\
&= 34\text{ k}
\end{aligned}
$$

The shear capacity of the one-foot segment is:

$$V_n = 2\lambda\sqrt{f'_c}b_w d \qquad \text{(ACI 318-14 22.5.5.1)}$$

$$
\begin{aligned}
&= 2(1.0)\sqrt{3{,}000\text{ psi}}(12\text{ in.})(20\text{ in.})(1\text{ k}/1{,}000\text{ lbs}) \\
&= 26\text{ k}
\end{aligned}
$$

Therefore, the existing portion of the foundation is not adequate for one-way shear and requires retrofit to meet the performance objective.

Flexure is evaluated parallel to the wall with the critical section located at the face of the wall. A one-foot segment of the footing is evaluated.

The flexural demand in the footing is:

$$
\begin{aligned}
M_u &= q_c(4.5\text{ ft})^2(1\text{ ft})/2 \\
&= (12\text{ ksf})(4.5\text{ ft})^2(1\text{ ft})/2 \\
&= 122\text{ kip-ft}
\end{aligned}
$$

The moment capacity of the one-foot segment is calculated using commercial software and is determined to be:

M_n = 47 kip-ft

Therefore, the foundation also requires retrofit for flexural demands.

Two-way shear (or punching shear) is evaluated where the existing 24-inch square column transfers the vertical load, P, from the wall to the foundation due to rocking. P is a force-controlled action determined from ASCE 41-13 Equation 7-35. For this example, C_1, C_2 and J are 1.0. Q_G and Q_E are the values determined in the Method 1 fixed base example.

$$P = Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

$$= 272 + \frac{1320}{(1.0)(1.0)(1.0)} = 1{,}592 \text{ kips}$$

Alternatively, a limit-state analysis could be performed to determine the maximum axial force based on the expected soil bearing capacity.

The critical perimeter occurs a distance of $d/2$ away from the edge of the concrete column. The length of the critical perimeter is:

$$b_o = 4(24 \text{ in} + 20 \text{ in}) = 176 \text{ in}$$

The two-way shear capacity is given by the lesser of the three equations in ACI 318-14 §22.6.5.2. For this condition, the governing two-way shear capacity is:

$$V_c = 4\lambda\sqrt{f_c'}b_o d$$

$$= 4(1.0)\sqrt{3{,}000 \text{ psi}}(176 \text{ in.})(20 \text{ in.})$$

$$= 771 \text{ k}$$

The two-way shear capacity is less than the axial load, P. Therefore, the foundation is also inadequate for punching shear. Based on the strength evaluation, the footing would require additional strengthening beyond that shown in Figure 5-22. This retrofit could include additional concrete placed above the footing with dowels into the existing concrete.

5.7.5 Method 2 Example

In this example, soil spring properties are determined for a footing based on the requirements of the Method 2 procedure. The footing is rigid relative to the foundation soil and is supporting a concrete shear wall. Soil springs are initially calculated based on the specific provisions in ASCE 41-13. Then, soil springs are tuned based on information contained in the source document from which Method 2 was developed to demonstrate how Method 2 springs can be calibrated to approximately match Method 1 stiffness.

Foundation information:

Rectangular footing

Width, $B = 6$ ft

Length, $L = 20$ ft

Depth, $d = 2$ ft

Section modulus, $S = BL^2/6 = (6\text{ ft})(20\text{ ft})^2/6 = 400\text{ ft}^3$

Design information:

Clayey soil

Site Class D

$S_{XS} = 1.0$ g

$N_{60} = 25$ (from geotechnical engineer)

$v = 0.35$ (from geotechnical engineer)

$q_c = 7.5$ ksf (from geotechnical engineer)

$p_a = 2.12$ ksf (atmospheric pressure)

Loading information:

$P = 500$ k

$M = 600$ kip-ft

Per ASCE 41-13 § 8.4.2.4.1, the stiffness of the footing is represented by multiple Winkler springs as shown in Figure 5-23. The springs at the ends of the footing are stiffer than those in the middle of the footing. The springs are intended to be tuned approximately to match the vertical and rotational stiffness calculated in Method 1.

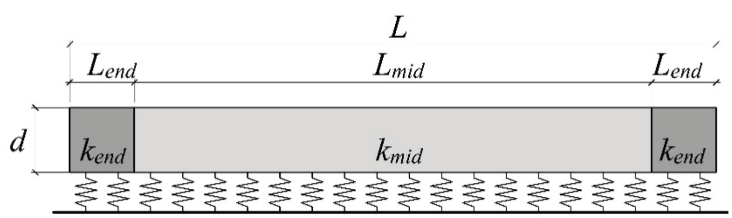


Figure 5-23 Method 2 diagram.

The end length per ASCE 41-13 Figure 8-5 is defined as:

$L_{end} = B/6 = 6\text{ ft}/6 = 1$ foot

The soil shear modulus for clayey soils may be calculated as:

$$G_0 = 20p_a(N_{60})^{0.77} \qquad \text{(ASCE 41-13 Eq. 8-5)}$$
$$= 120(2.12 \text{ ksf})(25)^{0.77} = 3{,}033 \text{ ksf}$$

The effective shear modulus ratio, G/G_0, is taken from ASCE 41-13 Table 8-2 based on site class and $S_{XS}/2.5$.

$$S_{XS}/2.5 = 1.0/2.5 = 0.4 \rightarrow G/G_0 = 0.50$$

$$G = (G/G_0)G_0 = 0.50(3{,}033 \text{ ksf}) = 1{,}517 \text{ ksf}$$

The end and middle stiffness per unit length are given in ASCE 41-13 Figure 8-5.

$$k_{\text{end}} = \frac{6.83G}{1-v} = \frac{6.83(1{,}517 \text{ ksf})}{1-0.35} = 15{,}940 \text{ kip/ft/ft}$$
$$k_{\text{mid}} = \frac{0.73G}{1-v} = \frac{0.73(1{,}517 \text{ ksf})}{1-0.35} = 1{,}704 \text{ kip/ft/ft}$$

To tune these springs, ASCE 41-13 § C8.4.2.4.1 states that Method 2 springs should be compared to the vertical and rotational stiffnesses determined from Method 1. The Method 1 stiffness is calculated per Figure 8-2.

$$K_{z,\text{sur}} = \frac{GB}{1-v}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$$
$$= \frac{(1{,}517 \text{ ksf})(6 \text{ ft})}{1-0.35}\left[1.55\left(\frac{20 \text{ ft}}{6 \text{ ft}}\right)^{0.75} + 0.8\right]$$
$$= 64{,}747 \text{ kip/ft}$$

$$K_{yy,\text{sur}} = \frac{GB^3}{1-v}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$$
$$= \frac{(1{,}517 \text{ ksf})(6 \text{ ft})^3}{1-0.35}\left[0.47\left(\frac{20 \text{ ft}}{6 \text{ ft}}\right)^{2.4} + 0.034\right]$$
$$= 4{,}278{,}350 \text{ kip-ft/radian}$$

The vertical displacement, δ_{z1}, of the footing using Method 1 stiffness and the applied load, as shown in Figure 5-24, is:

$$\delta_{z1} = P/K_{z,\text{sur}} = (500 \text{ k}/64{,}747 \text{ k/ft})(12 \text{ in}/1 \text{ ft}) = 0.093 \text{ in}$$

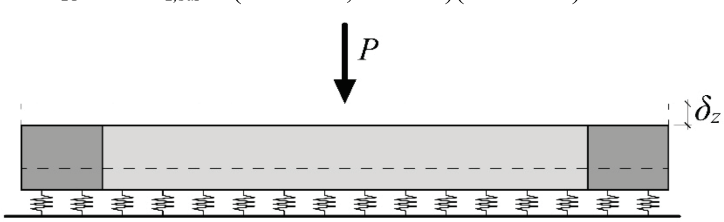


Figure 5-24 Vertical displacement.

The rotational displacement with Method 1 stiffness, θ, is:

$$\theta = M/K_{yy,\text{sur}} = (600 \text{ kip-ft}/4{,}278{,}350 \text{ kip-ft/radian}) = 0.00014 \text{ radians}$$

Assuming a rigid body rotation of the footing, the vertical displacement, δ_{yy1}, at the center of end zone of the footing due to the moment, as shown in Figure 5-25, is:

$$\delta_{yy1} = (L/2 - L_{\text{end}}/2)\sin(\theta)$$
$$= (20 \text{ ft}/2 - 1 \text{ ft}/2)\sin(0.00014)(12 \text{ inches}/1 \text{ foot}) = 0.016 \text{ in}$$

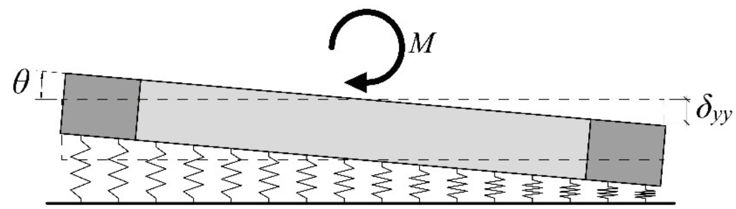


Figure 5-25 Rotational displacement.

To compare the Method 2 stiffness to the Method 1 stiffness, the footing may be modeled with the Method 2 spring stiffness with the vertical and moment loads applied. The displacement and rotation of the footing can be determined and compared to the Method 1 results. Alternatively, the comparison can be performed using hand calculations by comparing displacement at the center of the end zone under vertical load resisted by all Method 2 springs and under a moment resisted by the end zone springs only.

The vertical stiffness of all springs under the footing, K_z, is the summation of the stiffness per unit length multiplied by the length.

$$K_z = k_{\text{end}}(2L_{\text{end}}) + k_{\text{mid}}(L - 2L_{\text{end}})$$
$$= (15{,}940 \text{ ksf})(2)(1 \text{ ft}) + (1{,}704 \text{ ksf})(20 \text{ ft} - (2)(1 \text{ ft})) = 62{,}552 \text{ kip/ft}$$

The vertical displacement, δ_{z2}, of the footing using Method 2 stiffness and the applied load is:

$$\delta_{z2} = P/K_z = (500 \text{ k}/62{,}552 \text{ kip/ft})(12 \text{ in}/1 \text{ ft}) = 0.096 \text{ in}$$

The displacement at the center of the end zone due to the moment is calculated as follows:

$$f = M/S = 600 \text{ kip-ft}/400 \text{ ft}^3 = 1.5 \text{ ksf (at the end of the footing)}$$

The resultant force, F, in the end zone is:

$$F = [f - (f/(L/2))(L_{\text{end}}/2)]BL_{\text{end}}$$
$$= [1.5 \text{ ksf} - (1.5 \text{ ksf}/(20 \text{ ft}/2))(1 \text{ ft}/2)](6 \text{ ft})(1 \text{ ft}) = 8.55 \text{ k}$$

The displacement, δ_{yy2}, at the center of the end zone is:

$$\delta_{yy2} = F/[(k_{\text{end}})(L_{\text{end}})]$$
$$= 8.55\text{ k}/[(15{,}940\text{ kip/ft/ft})(1\text{ ft})](12\text{ in}/1\text{ ft}) = 0.0064\text{ in}$$

In comparing the Method 1 and Method 2 results using the calculated displacements, the vertical stiffness of Method 2 (62,552 kip/ft) is within 4% of the Method 1 value (64,747 kip/ft). However, the rotational stiffness calculated with Method 1 is 2.5 times that calculated with Method 2 (comparing the 0.016 in Method 1 displacement at the center of the end zone with the 0.0064 in Method 2 displacement at the same location). The difference in stiffness calculated with Method 1 and Method 2 can vary significantly depending on the geometry of the foundation.

ASCE 41-13 § 8.4.2.4.1 requires that the Method 2 springs be tuned to approximately match the stiffness from the elastic solutions used in Method 1, but does not provide specific guidance on how to tune the springs beyond the reference to Gajan et al. (2010) in ASCE 41-13 § C8.4.2.4.1. Therefore, it is recommended to utilize information provided in Gajan et al. (2010) to tune the Method 2 springs by varying the factors used to determine k_{end} and k_{mid}. The stiffness intensity ratio, $k_{\text{end}}/k_{\text{mid}}$, is determined from Figure 5-26 based on the footing aspect ratio, B/L.

$B/L = 6/20 = 0.3$

$k_{\text{end}}/k_{\text{mid}} = 2.6$ (Figure 5-26)

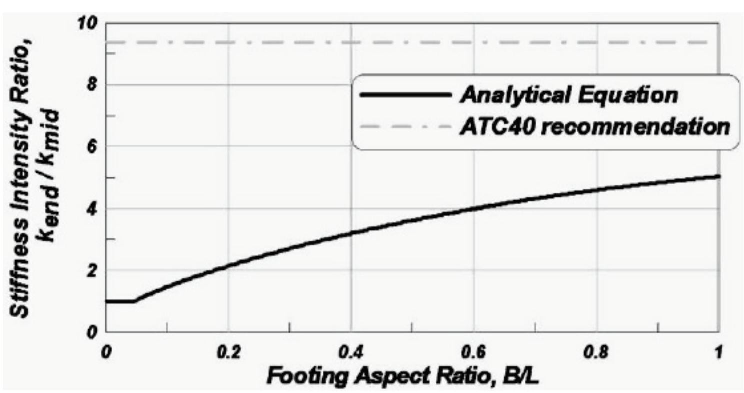


Figure 5-26 Stiffness intensity ratio versus aspect ratio (Harden and Hutchinson, 2009).

The end length is determined from Figure 5-27 based on the footing aspect ratio.

$L_{\text{end}}/B = 0.33$

$L_{\text{end}} = (L_e/B)B = 0.33(6\text{ ft}) = 2\text{ feet}$

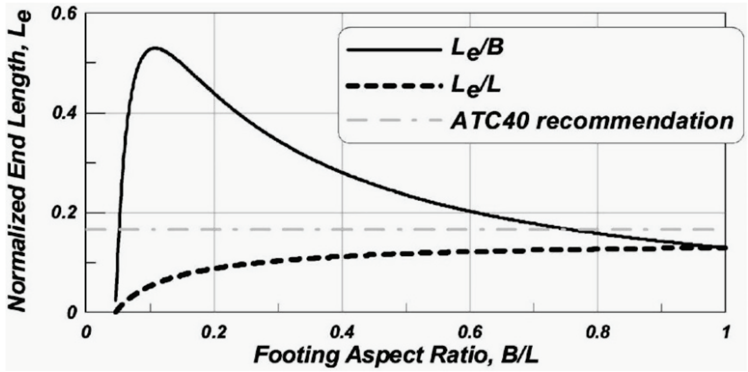


Figure 5-27 End length versus aspect ratio (Harden and Hutchinson, 2009).

To match the rotation calculated in Method 1, F is calculated with the revised end length in order to determine the target k_{end}:

$$F = [f - (f/(L/2))(L_{\text{end}}/2)]BL_{\text{end}}$$
$$= [1.5 \text{ ksf} - (1.5 \text{ ksf}/(20 \text{ ft}/2))(2 \text{ ft}/2)](6 \text{ ft})(2 \text{ ft}) = 16.2 \text{ k}$$

$$k_{\text{end}} = F/[(\delta_{yy1})(L_{\text{end}})]$$
$$= (16.2 \text{ k})/[(0.016)(1 \text{ ft}/12 \text{ in})(2 \text{ ft})] = 6{,}075 \text{ kip/ft/ft}$$

The target k_{mid} is determined from the stiffness intensity ratio:

$$k_{\text{mid}} = k_{\text{end}}/(k_{\text{end}}/k_{\text{mid}}) = (6{,}075 \text{ kip/ft/ft})/2.6 = 2{,}337 \text{ kip/ft/ft}$$

K_z is calculated as above:

$$K_z = (6{,}075 \text{ ksf})(2)(2 \text{ ft}) + (2{,}337 \text{ ksf})(20 \text{ ft} - (2)(2 \text{ ft})) = 61{,}692 \text{ kip/ft}$$

This vertical stiffness of 61,692 kip/ft is within 5% of that calculated with Method 1 (64,747 kip/ft). Therefore, the revised stiffness values are judged to be adequately tuned to the elastic stiffness values of Method 1. Where additional tuning is required, k_{end} and k_{mid} may be varied by trial and error until both vertical and rotational stiffness approximately match Method 1.

To develop the load-deformation relationship for each spring, the spacing of the springs must be determined. ASCE 41-13 § 8.4.2.4.2 specifies that the spacing of the springs be sufficient to capture yielding of the soil at the edge of the footing under bearing loads and ASCE 41-13 § C8.4.2.4.2 recommends that the spacing be less than $L_c/2$. Gajan et al. (2010) suggest a minimum of 25 springs along the length of the footing. For this example, a spring spacing, s_{sp}, of 1 foot is selected. The stiffness of the end and middle springs are:

$$K_{\text{end}} = s_{sp}k_{\text{end}} = (1 \text{ ft})(6{,}075 \text{ kip/ft/ft}) = 6{,}075 \text{ kip/ft}$$

$$K_{\text{mid}} = s_{sp}k_{\text{mid}} = (1 \text{ ft})(2{,}337 \text{ kip/ft/ft}) = 2{,}337 \text{ kip/ft}$$

The expected strength, q_{sp}, of the springs is:

$$q_{sp} = q_c s_{sp} B = (7.5 \text{ ksf})(1 \text{ ft})(6 \text{ ft}) = 45 \text{ k}$$

The end spring yield displacement, $\delta_{y\text{end}}$, is:

$$\delta_{y\text{end}} = q_{sp}/K_{\text{end}} = (45 \text{ k}/6{,}075 \text{ kip/ft})(12 \text{ in}/1 \text{ ft}) = 0.09 \text{ in}$$

The middle spring yield displacement, $\delta_{y\text{mid}}$, is:

$$\delta_{y\text{mid}} = q_{sp}/K_{\text{mid}} = (45 \text{ k}/2{,}337 \text{ kip/ft})(12 \text{ in}/1 \text{ ft}) = 0.23 \text{ in}$$

Per ASCE 41-13 § 8.4.2.4.2, the tension capacity of the spring shall be set at zero. The force-deformation curve for each spring is shown in Figure 5-28 and Figure 5-29.

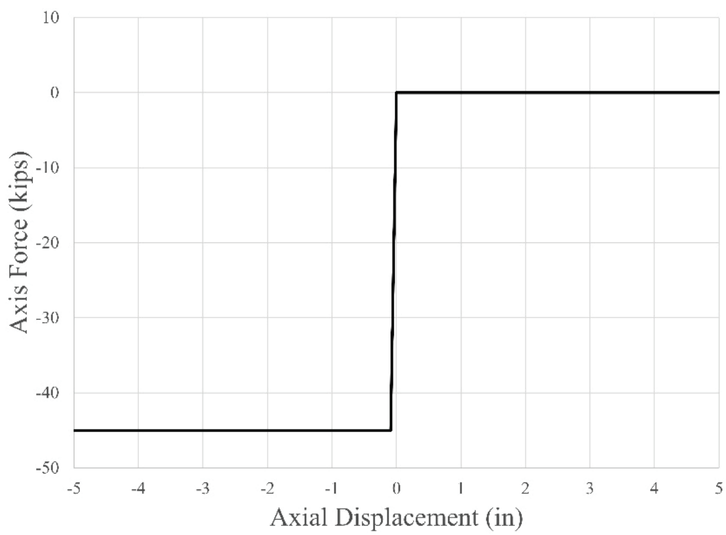


Figure 5-28 End spring force-displacement curve.

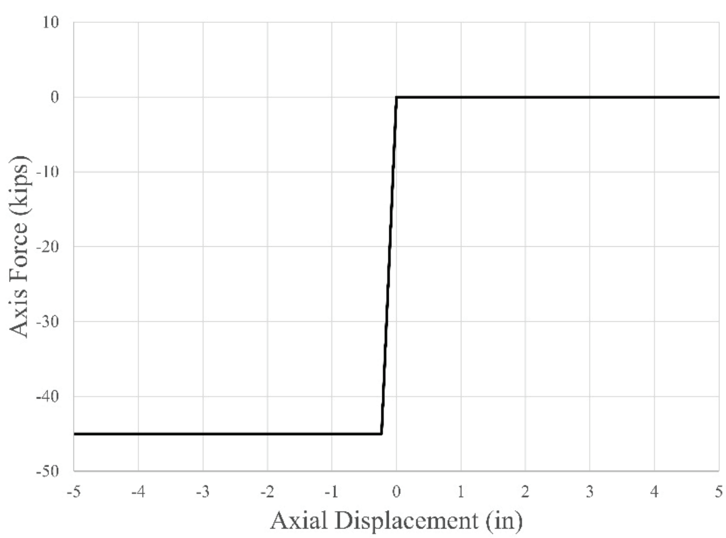


Figure 5-29 Middle spring force-displacement curve.

The acceptance criteria for the footing are determined based on the following parameters:

$A_f \quad = BL = (20\text{ ft})(6\text{ ft}) = 120\text{ ft}^2$

$A_c \quad = P/q_c = (500\text{ k})/(7.5\text{ ksf}) = 67\text{ ft}^2$

$L_c \quad = A_c/B = (67\text{ ft}^2)/(6\text{ ft}) = 11.2\text{ ft}$

$A_c/A_f = (67\text{ ft}^2)/(120\text{ ft}^2) = 0.56$

$B/L_c = (6\text{ ft})/(11.2\text{ ft}) = 0.54$

Using linear interpolation in ASCE 41-13 Table 8-4, the total footing rotation for the Life Safety Performance Level is 0.0026 radians. Springs with these properties would be included in the analytical model and the rotation of the footing would be evaluated against these acceptance criteria to determine its adequacy under the design loads. By implementing the nonlinear methodology, the component actions in the superstructure caused by foundation deformation are more accurately captured. By meeting the acceptance criteria, there is greater confidence that potential foundation settlement issues due to the rocking response under high axial loads have been avoided.

5.7.6 Method 3 Example

Where a structural footing is found to be flexible relative to the soil, ASCE 41-13 § 8.4.2.1 requires that Method 3 be used (see the flowchart in Figure 5-10). The acceptance criteria for Method 3 are defined in ASCE 41-13 § 8.4.2.5.3, which references ASCE 41-13 Table 8-4 for nonlinear modeling parameters and total foundation rotation limits. Therefore, all buildings with "flexible" foundations relative to the soil must be evaluated using either an NSP or NDP analysis, or at least with nonlinear modeling of the foundation system based on a limit-state analysis to determine the maximum force that can be delivered to the foundation. The use of a fixed-base model of the building superstructure in conjunction with a separate foundation model using the Method 3 approach for the foundation, a commonly used two-step analysis, is not permitted. The intent and requirement of Method 3 is to modify the building response with the incorporation of foundation flexibility.

To achieve accurate bearing areas, A_c, and soil pressure distribution, a limit-state analysis is used to derive demands and a nonlinear soil-structure-interaction analysis is required, which in turn provides expected bending and shear demands on the foundation structural components. This is not readily achievable with LSP and LDP procedures; hence, Method 3 adopts nonlinear modeling parameters and acceptance criteria. The following general guidance is provided for nonlinear analysis procedures, followed by an approach for Method 3 linear analysis procedures.

5.7.6.1 Method 3 Nonlinear Analysis

For nonlinear models (NSP or NDP), the applied loads on the foundation are based on the expected strength of the components supported by the foundation in order to provide consistency between the soil bearing pressure distribution and the structural flexibility of the footing in accordance with ASCE 41-13 § 8.4.2.1. The foundation rotation capacity is defined per the foundation rotation acceptance criteria in ASCE 41-13 Table 8-4 and is compared directly to the rotation at the base of the vertical elements supported by the foundation at the target displacement. The resultant demand actions (moment and shear) on the concrete footing are evaluated and designed in accordance with ASCE 41-13 Chapter 10.

Where buildings have interconnected strip footings that support vertical elements of the seismic force-resisting system, the footing may be modeled as a beam element with modeling parameters and acceptance criteria from ASCE 41-13 Chapter 10. The footing should be modeled with the appropriate width and with a sufficient quantity of nodes provided along the its length for the attachment of vertical Winkler springs, which represent the unit subgrade spring coefficient, k_{sv} (ASCE 41-13 Eq. 8-11). Where the footing is modeled as a shell element with Winkler springs connected at the mesh nodes, the tributary area to a node would define the Winkler spring modeling parameters.

A key step in applying the acceptance criteria is to determine the compression index factor, A_c/A_f, which requires judgment in determining the effective, tributary footing size to the vertical element. This is discussed below using an example from FEMA 274 (FEMA, 1997b) (see Figure 5-30), which consists of three concrete shear walls on a flexible grade beam connected (slaved in lateral translation) at the top with a rigid axial link. The example considers different levels of soil stiffness, represented by K_s (which is equivalent to k_{sv} in ASCE 41-13), under the limit-state loading from the concrete shear walls to evaluate the displacement, $D_{\max}$, that will occur due to the foundation flexibility, which would be added to the wall deflection at the target displacement.

For the topmost case with very flexible soil, the contact area is over half the length of the footing and there is a linear soil bearing pressure distribution where in contact with the soil. The three walls are imposing an essentially rigid body rotation onto the soil, as if they are one solid wall element (see Figure 5-31).

In this case, A_c/A_f based on the entire footing length, L, is deemed appropriate. The use of Method 1 or 2 could be considered based on this analysis finding. Continuing with Method 3, the axial load level, P, is the summation of both the gravity and seismic loads. Where there are no elements framing into the three walls, the seismic axial load may approach zero. When there are other elements connected to the walls (either in the same line or orthogonal), there would be either a downward or upward seismic axial load to be included in the total axial load, P, for the determination of A_c/A_f in ASCE 41-13 Table 8-4. For the evaluation of the rotation demand, a line could be drawn between the centers of the walls at their base and the rotation angle would be measured to the horizontal axis.

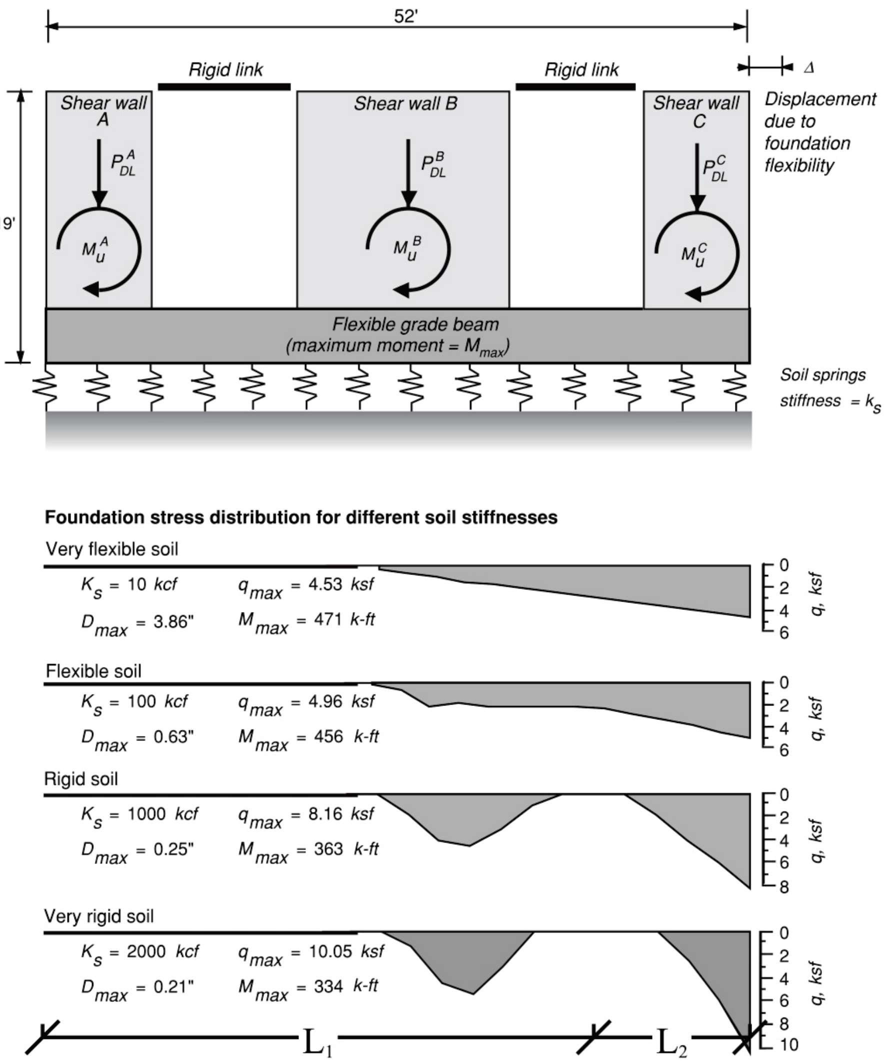


Figure 5-30 Method 3 nonlinear example (from FEMA, 1997b).

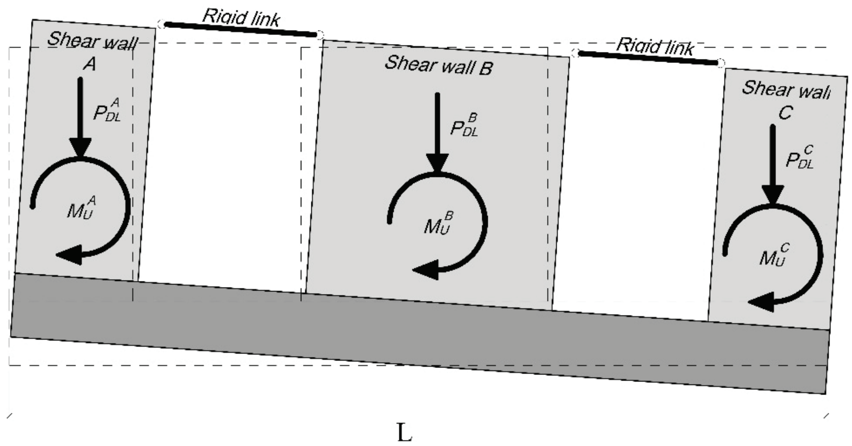


Figure 5-31 Very flexible soil rotation.

For the bottom case with very rigid soil, the left-most wall segment is uplifting and the other two are rotating as if on individual footings (see Figure 5-32). In this case, two actions should be evaluated:

- The left-most and center walls should be evaluated as if they share one footing with the length defined as L_1 and the axial load equal to the summation of the loads on those two walls.
- The right-most wall should be evaluated with an A_c/A_f based on a footing length of L_2 and its associated axial load, P.

The rotation demands are obtained from the effective rotation at the base of the wall(s).

As can be seen for the cases in between, each one would be approached based on their soil distribution, which is a reflection of the strip footing curvature.

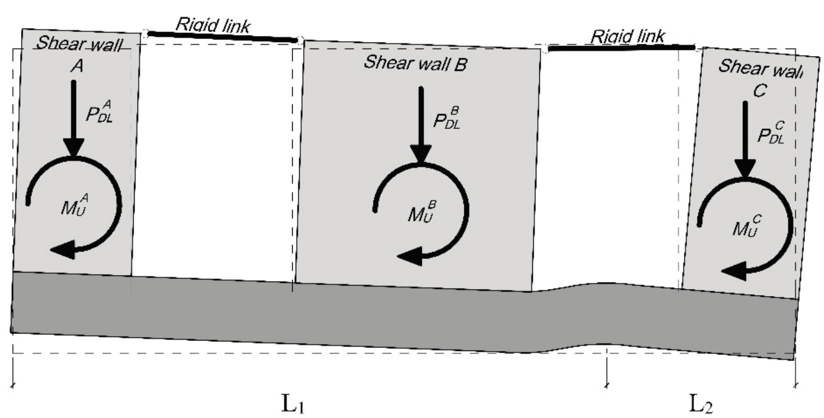


Figure 5-32 Very rigid soil rotation.

For a more complex building, such as a split-level basement with a mat foundation, a mesh with the concrete stiffness modified per ASCE 41-13

Chapter 10 and a similar soil spring approach is recommended. Judgment regarding the appropriate effective foundation width is required to establish A_c/A_f, and a similar review of the analysis findings, in particular the soil bearing pressure distribution, can be the basis for determining the width. In all of these nonlinear cases, the nonlinear Winkler springs should include zero tension capacity for a shallow foundation to represent gapping (soil separation from the structure), but need not capture non-recoverable soil plastic deformation.

If, however, the soil is modeled to explicitly capture settlement and soil plasticity, which requires an NDP analysis, the intent of ASCE 41-13 is to allow for the acceptability of the soil displacements to be determined based on the evaluation of structural components with their acceptance criteria subject to displacements explicitly captured by the NDP analysis similar to the provisions for Method 2 in ASCE 41-13 § 8.4.2.4.4. As discussed in Section 5.7.1 of this *Guide*, ASCE 41-17 has corrected the wording in this section to permit this approach.

5.7.6.2 Method 3 Linear Analysis

ASCE 41-13 does not provide guidance on a linear analysis procedure for Method 3. Approaches have been developed to perform linear analysis using the Method 3 procedure, but there is no consensus on the appropriate application of ASCE 41-13 to this procedure and is therefore outside the scope of this document.

5.8 Shallow Foundation Lateral Load

Shallow foundation lateral load provisions are contained in ASCE 41-13 § 8.4.2.6 and allow for the use of a simplified passive pressure mobilization curve (ASCE 41-13 Figure 8-6) to determine what fraction of the ultimate passive pressure is engaged based on lateral displacement of the footing. Given the highly nonlinear force-displacement relationship of this curve, it is not practical to use this curve when developing a lateral spring for use in an analytical model. Per ASCE 41-13 § C8.4.2.6, the nonlinear force-displacement response of shallow footings may be characterized as elastic-perfectly plastic using the initial, effective stiffness and the total expected capacity in conjunction with the upper and lower bounds previously described. The effective stiffness may be calculated per ASCE 41-13 § 8.4.2 as discussed in Section 5.6.4 of this *Guide* or provided by a geotechnical engineer. The total expected capacity includes contributions from sliding resistance at the bottom of the footing and passive pressure on the face of the footing. This force-deformation relationship is adequate to capture the nonlinear behavior in a practical manner, in lieu of a more complex relationship developed using ASCE 41-13 Figure 8-6. As an alternate, shallow foundation response may be analyzed based on test data.

5.8.1 Shallow Foundation Lateral Load Example

The elastic-perfectly plastic lateral load-deformation relationship (described above) is determined for the footing from the example in Section 5.6.4.1 of this *Guide*, where the translational stiffness, $K_{x,\mathrm{sur}}$, was determined to be 30,750 kip/ft. Additional design information is provided:

N = 100 k (vertical dead load on footing, including footing weight)

μ = 0.35 (coefficient of friction)

p_{ult} = 1,000 pcf (ultimate passive pressure resistance)

The ultimate passive pressure capacity is:

$$P_{\mathrm{ult}} = p_{\mathrm{ult}} d^2 B/2 = (1{,}000)(2^2)(6)/2 = 12 \text{ k}$$

The lateral capacity due to traction at the base of the footing is:

$$T = N\mu = (100 \text{ k})(0.35) = 35 \text{ k}$$

The total lateral load capacity of the footing is:

$$P_{\mathrm{ult}} + T = 12 \text{ k} + 35 \text{ k} = 47 \text{ k}$$

The load-deformation curve for the footing is then developed using the stiffness and total lateral load capacity as shown in Figure 5-33. Note that the upper and lower bounds (calculated in Section 5.6.3) should be evaluated in assessing lateral foundation component actions.

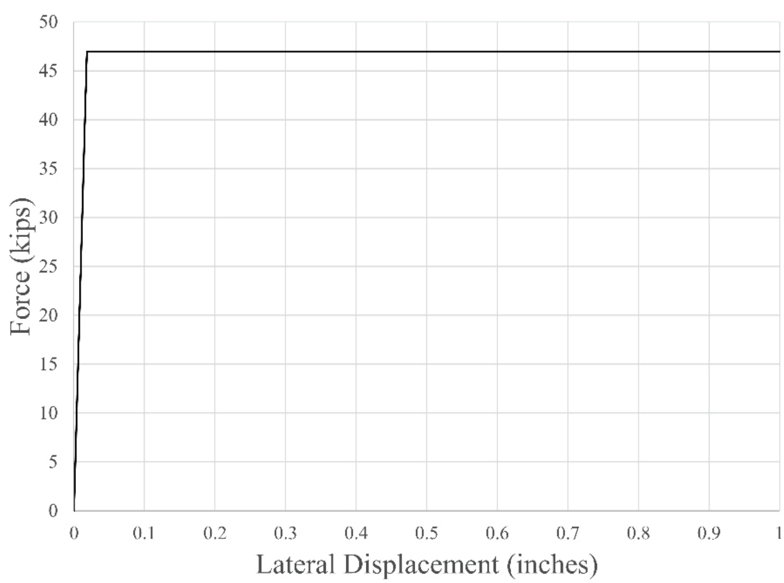


Figure 5-33 Lateral load force-deformation curve.

5.9 Deep Foundation Evaluation and Retrofit

The approach to evaluation and retrofit of deep foundations in ASCE 41-13 is similar to that for shallow foundations. A fixed-base or flexible-base assumption may be made for both linear and nonlinear analysis procedures. For fixed-base linear procedures, the foundation soil is classified as deformation-controlled, and the capacity is evaluated with m-factors and upper-bound component capacities. For fixed-base nonlinear procedures, the foundation soil is classified as force-controlled and upper-bound component capacities may be used. Where a flexible base is assumed for linear or nonlinear procedures, the soil is assumed to have unlimited ductility, and soil strength capacity need not be evaluated. The structural components are evaluated based on the component acceptance criteria at the selected Performance Objective to determine their ability to accommodate the soil displacements. For the nonlinear flexible-base assumption with the Immediate Occupancy Performance Objective, the structure and foundation are evaluated based on the permanent, non-recoverable soil displacement. It is worth noting that ASCE 41-06 had similar infinite ductility assumptions for shallow foundations, which was removed in the ASCE 41-13 version.

The determination of relative stiffness between the soil and structure is directly incorporated into the deep foundation procedures and the load-deformation characteristics are bounded per ASCE 41-13 § 8.4.3. Separate provisions are required for piles 24 inches or less in diameter and piles greater than 24 inches in diameter.

- For piles with diameter less than or equal to 24 inches, a simplified approach assuming flexible piles and rigid soil is used with individual and group pile stiffness determined by the summation of AE/L per ASCE 41-13 Equation 8-13.
- For larger piles, a more in-depth analysis is required. The derivation of axial and overturning force-deformation relationships requires explicit inclusion of the soil and pile properties, such as p-y (horizontal), t-z (vertical skin friction), and q-z (end bearing) curves, as well as the appropriate area and flexural structural properties of the pile, in the numerical model. This approach assumes both pile and soil are flexible, and that their stiffnesses are in series.

General procedures for deep foundation evaluation and retrofit are outlined in the flowchart in Figure 5-34.

Geotechnical and structural analyses are recommended where end bearing piles are subject to uplift or where friction piles are loaded near their upper-

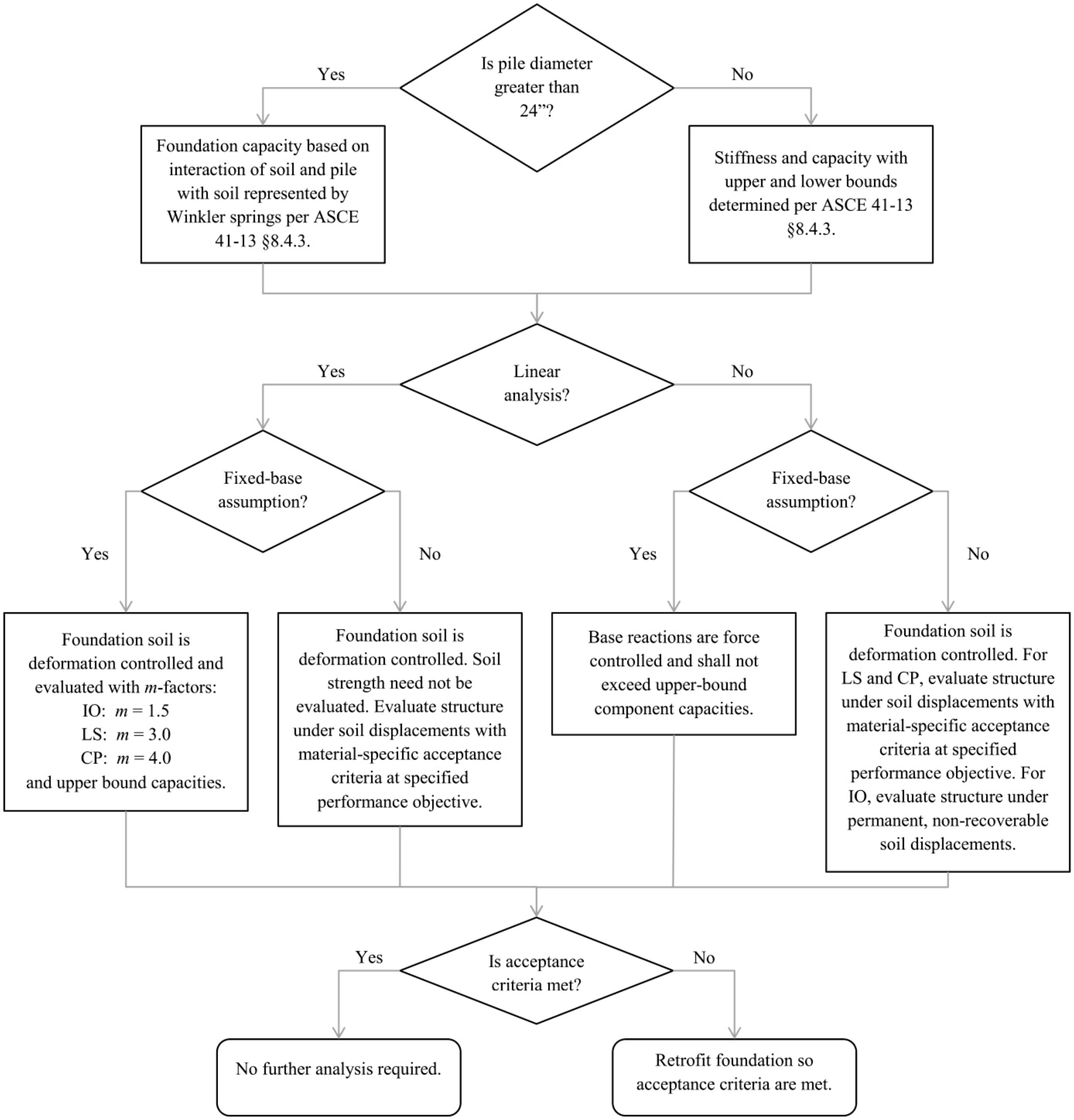


Figure 5-34 Deep foundation evaluation and retrofit flowchart with reference to sections in ASCE 41-13.

bound limit. Pile plunging or large permanent deformations are not considered to have a high probability of occurrence in ASCE 41-13, based on the assumption that there is significant pile axial overstrength (FEMA 274). However, soil yielding may cause accumulation of permanent displacements, similar to shallow foundations. Therefore, cyclic loading and potential impacts due to settlement are recommended to be discussed and evaluated with a geotechnical engineer or subject matter expert with deep foundation analysis experience.

The bounding requirements provided in ASCE 41-13 § 8.4.3 are important to follow, particularly where end bearing piles experience uplift since less tension strength and stiffness are anticipated for bearing piles compared to friction piles, which will in turn affect the overturning (rocking) stiffness of end bearing pile groups. Additionally, the soil may not be rigid as assumed in the procedure for determining axial and overturning stiffnesses for piles less than 24 inches in diameter. It is important to note that the bounding may not sufficiently capture the foundation performance in weaker soils or for highly loaded foundations, where the upper-bound ultimate capacity of the pile is used.

Lateral loading and deformation on the piles is critical to evaluate. Displacement compatibility is required between the lateral displacement of the pile head and the passive pressure on the pile cap. Therefore, the full lateral pile strength capacity may only be achieved after a flexural hinge occurs at the top of the pile or at some depth below grade depending on the soil stiffness relative to the pile stiffness, or yielding of the passive soil pressure. The adequacy of the pile ductility and determination of when the hinge forms will depend on the detailing of the pile. The provisions of the material-specific chapters of ASCE 41-13 along with a soil-structure interaction analysis are required. See "Limit State Design of Piles, Pile Caps, and Grade Beams" in *SEAOC Blue Book: Seismic Design Recommendations* (SEAOC, 2008a) for further information.

Once the pile, pile group, and passive modelling parameters have been developed, the distribution of lateral forces in the building can be determined from the building structural analysis. The distribution of forces may vary significantly based on the configuration of the building. For example, considering a pile-supported building with a basement condition without a robust structural diaphragm interconnecting the foundations, the initial stiffness of the structural system may attract the earthquake loads to the perimeter walls. Based on the concentration of loads at those lines of resistance, lateral yielding may occur at the foundation and cause higher inter-story drift demands on interior columns. This results in higher shear and inelastic demands on the gravity columns at that floor, which leads to a reduced reliability of maintaining vertical load support.

When evaluating deep foundation retrofit options, displacement compatibility and differential foundation stiffness should be considered per ASCE 41-13 § 8.7 and § C8.7. The addition of piles in a retrofit or underpinning of an adjacent existing foundation locally stiffens the foundation and may change the load distribution within the building. During an earthquake, this stiffness incompatibility would likely show more damage where the existing building transitions from existing shallow foundation to the pile retrofit portion of the building. The difference in vertical stiffness will also increase the risk of long-term differential settlement damage, particularly for unreinforced masonry buildings.

5.10 Kinematic Interaction and Radiation Damping Soil-Structure Interaction Effects

Soil-structure interaction (SSI) can have an effect on the seismic demands on a building, which impacts the response of the structure and the foundation and supporting soil. In the context of ASCE 41-13, SSI includes three primary sources:

- Flexibility of the foundation at the foundation-soil interface, which can be incorporated into the analysis through the three methods previously discussed
- Kinematic effects (base slab averaging and embedment effects) which influence the accelerations transmitted to the structure
- Foundation damping which dissipates energy through radiation and hysteretic soil damping

Soil-structure interaction is introduced in ASCE 41-13 § 7.2.7 with specific provisions contained in ASCE 41-13 § 8.5. Typically, SSI effects will reduce seismic forces. In these cases, SSI may be included in the structural analysis at the discretion of the designer, but is not required by ASCE 41-13. In the rare occasions that SSI effects increase spectral accelerations, such as for near-field and soft-soil sites, SSI effects must be included in the evaluation of the structure. NIST GCR 12-917-21 (NIST, 2012) contains significant background and discussion on the application and effects of soil-structure interaction.

ASCE 41-17 Revision

ASCE 41-17 has a number of revisions to SSI provisions including the following:

- The building must be modeled with flexible base conditions if kinematic effects are to be included;
- RRS_e shall not be taken less than the value calculated with a maximum embedment of 20 feet; and
- The maximum cumulative reduction due to SSI shall not be greater than 30%.

5.10.1 Example of Kinematic Interaction Effects for 3-Story Building over Basement

The following example illustrates the calculation of kinematic interaction effects for a large single story building with a basement. The following building and seismic characteristics are provided:

- 1-story concrete shear wall building with rigid diaphragms over basement
- Mat slab foundation

- Embedment depth, e = 20 ft (from ground surface to bottom of foundation)
- Building footprint = 250'-0" by 250'-0"
- Building period, T = 0.15 seconds. Note that when calculating base slab averaging and embedment effects, the building period should be based on a model with flexible base conditions (soil springs) with stiffness determined per the requirements for the selected foundation modeling approach: Method 1 (ASCE 41-13 § 8.4.2.3), Method 2 (ASCE 41-13 § 8.4.2.4.1), or Method 3 (ASCE 41-13 § 8.4.2.5.1). Building period determination based on a fixed-base model will result in an unconservative reduction.
- Soil Site Class D
- S_{XS} = 1.00

Since the building is not located in soil Site Classes A, B, E or F, the diaphragms are rigid, and the foundations are laterally connected by a concrete slab, both base slab averaging and embedment effects are permitted to be incorporated into the analysis. Note that these provisions have been modified in ASCE 41-17.

The effects of base slab averaging are calculated per ASCE 41-13 § 8.5.1.

Area of the foundation footprint:

$$A_{\text{base}} = (250\text{ ft})(250\text{ ft}) = 62{,}500\text{ ft}^2$$

Effective foundation size:

$$b_e = \sqrt{A_{\text{base}}} = \sqrt{62{,}500} = 250\text{ ft} < 260\text{ ft} \qquad \text{(ASCE 41-13 Eq. 8-18)}$$

$$b_0 = 0.0001 \times \left(\frac{2\pi b_e}{T}\right) \qquad \text{(ASCE 41-13 Eq. 8-17)}$$

$$= 0.0001 \times \left(\frac{2\pi(250\text{ ft})}{0.20\text{ s}}\right) = 0.785$$

Note that the period shall not be taken as less than 0.20 seconds for this calculation, per ASCE 41-13 § 8.5.1.1.

For $b_0 \leq 1$,

$$B_{bsa} = 1 + b_0^2 + b_0^4 + \frac{b_0^6}{2} + \frac{b_0^8}{4} + \frac{b_0^{10}}{12} \qquad \text{(ASCE 41-13 Eq. 8-16)}$$

$$= 1 + 0.785^2 + 0.785^4 + \frac{0.785^6}{2} + \frac{0.785^8}{4} + \frac{0.785^{10}}{12}$$

$$= 2.16$$

The ratio of response spectra (*RRS*) for base slab averaging is calculated:

$$RRS_{bsa} = 0.25 + 0.75 \times \left\{ \frac{1}{b_0^2} \left[1 - \exp(-2b_0^2) \times B_{bsa} \right] \right\}^{1/2} \quad \text{(ASCE 41-13 Eq. 8-15)}$$

$$= 0.25 + 0.75 \times \left\{ \frac{1}{0.785^2} \left[1 - \exp(-2(0.785^2)) \times 2.16 \right] \right\}^{1/2}$$

$$= 0.83$$

Embedment effects are calculated per ASCE 41-13 § 8.5.1.2.

The shear wave velocity at low strains is provided based on a site-specific geotechnical investigation.

v_{s0} = 750 ft/s

The effective shear modulus ratio is interpolated from the values in ASCE 41-13 Table 8-2.

$S_{XS}/2.5 = 1.00/2.5 = 0.40$

$G/G_0 = 0.50$

$n = \sqrt{G/G_0} = \sqrt{0.50} = 0.71$

The effective shear wave velocity:

$v_s = nv_{s0} = (0.71)(750 \text{ ft/s}) = 533 \text{ ft/s}$

The *RRS* factor for embedment is calculated:

$$RRS_e = 0.25 + 0.75 \times \cos\left(\frac{2\pi e}{Tv_s} \right) \quad \text{(ASCE 41-13 Eq. 8-16)}$$

$$= 0.25 + 0.75 \times \cos\left(\frac{2\pi(20 \text{ ft})}{(0.20 \text{ s})(533 \text{ ft/s})} \right)$$

$$= 0.53$$

Note that the period shall not be taken as less than 0.20 seconds for this calculation and RRS_e must be greater than or equal to 0.50 per ASCE 41-13 Equation 8-19.

Therefore, the total reduction is the spectral acceleration due to kinematic interaction effects is:

$RRS_{bsa} \times RRS_e = (0.83)(0.53) = 0.44$

The combined effect of base slab averaging and embedment should also not be less than 0.50 per ASCE 41-13 § 8.5.1. The spectral acceleration used for the calculation of the pseudo lateral force is modified to account for kinematic interaction effects:

$S_{XS} = (0.50)(1.00\text{g}) = 0.50\text{g}$

Note that ASCE 41-17 § 7.2.7 has been modified to limit the total reduction of pseudolateral force and target displacement due to SSI to 30% of that calculated without SSI.

5.10.2 Discussion of Foundation Damping

ASCE 41-13 § 8.5.2 addresses foundation damping. Inertial interaction effects, such as foundation damping, typically have a larger impact on stiff structures, such as shear wall or braced frames, which are supported by softer soils. NIST GCR 12-917-21 discusses a variety of parameters that can be used to evaluate the effect of foundation damping on a structure. The most important parameter is the structure-to-soil stiffness ratio, $h/(V_sT)$, where h is the height to the center of mass of the first mode shape, V_s is the shear wave velocity and T is the building's fundamental period. Foundation damping can increase significantly with higher structure-to-soil stiffness ratios; $h/(V_sT)$ values of less than 0.1 (flexible building on stiff soil or rock) typically indicate negligible inertial SSI effects, and larger $h/(V_sT)$ ratios generally relate to more significant effects. Note that the foundation damping equations in the NIST document are different than those presented in ASCE 41-13; the NIST equations are incorporated into the foundation provisions of ASCE 41-17.

5.11 Liquefaction Evaluation and Mitigation

A number of seismic-geologic site hazards are addressed in ASCE 41-13 § 8.2.2, including fault rupture, liquefaction, settlement, landsliding, and flooding, as well as mitigation approaches in ASCE 41-13 § 8.3. This section focuses on the qualitative liquefaction provisions in ASCE 41-13 § 8.2.2.2, which provides a general framework for the designer and the Authority Having Jurisdiction to utilize in determining the scope and extent of the liquefaction analyses and the impact of liquefaction on the structure. Because of the complexity and uncertainties with liquefaction, it is recommended that a geotechnical specialist be consulted as required. Where liquefiable soils are present, the liquefaction analyses should include potential liquefaction-induced effects such as lateral spreading, settlement, slope stability, bearing capacity failure, and flotation of buried structures.

In general, the effects of liquefaction are evaluated with three analyses:

- Upper bound: ASCE 41-13 § 8.2.2.2.1 requires a mathematical model with a flexible foundation condition be analyzed assuming no liquefaction has occurred.

- Lower bound: ASCE 41-13 § 8.2.2.2.1 requires a mathematical model with a flexible foundation condition be analyzed with seismic hazard parameters, response spectrum, or acceleration response histories modified to account for liquefaction. Strength and stiffness of the foundation is also reduced based on the effects of liquefaction.
- Post-liquefaction: ASCE 41-13 § 8.2.2.2.2 requires a nonlinear mathematical model to be analyzed with estimated differential settlement and lateral spreading applied to foundation elements to assess the structure following liquefaction.

Mitigation recommendations for liquefaction, along with other seismic-geologic site hazards, are briefly discussed in ASCE 41-13 § 8.3 and § C8.3. In some cases, it may not be economically practical to retrofit the structure or provide ground improvements to mitigate the hazards.

Chapter 6

Tier 1 Screening and Tier 2 Deficiency-Based Evaluation and Retrofit

6.1 Overview

This chapter provides discussion and example application of the Tier 1 screening and Tier 2 deficiency-based evaluation and retrofit procedures presented in ASCE 41-13 (ASCE, 2014). The Tier 1 and 2 procedures provide design professionals a quick and effective method to screen buildings for potential deficiencies and to focus on evaluating and potentially retrofitting only the deficiencies discovered. The Tier 3 systematic evaluation procedure is much more detailed and thorough and is addressed in the remaining chapters and example problems of this *Example Application Guide*. The Tier 1 and Tier 2 procedures are discussed in detail below followed by an example of a tilt-up concrete building to demonstrate the application of these methods.

6.2 Tier 1 Screening (ASCE 41-13 Chapter 4)

The Tier 1 Screening Procedure is effective at quickly identifying potentially hazardous seismic deficiencies in a building by using checklists and quick check calculations of major building components. This procedure is limited to specific common building types as outlined in ASCE 41-13 § 3.3.1 and Table 3-1 and the number of stories in ASCE 41-13 Table 3-2. The method is structured to assess buildings to the Basic Performance Objective for Existing Buildings (BPOE) at either the Immediate Occupancy (IO) or Life Safety (LS) Performance Levels as outlined in ASCE 41-13 Table 2-1. The Tier 1 procedure is not applicable to Basic Performance Objective Equivalent to New Building Standards (BPON).

The Tier 1 procedure requires only a single level assessment to the BSE-1E Seismic Hazard Level, which is unlike the Tier 3 procedure that requires a two-level assessment when evaluating to the BPOE. The Tier 1 procedure does not require an assessment to the BSE-2E Seismic Hazard Level as it implies the structure is deemed to comply with this seismic hazard.

ASCE 41-17 Revision

The seismic hazard used in the Tier 1 and Tier 2 procedures in ASCE 41-17 changes from BSE-1E to BSE-2E for Risk Category I through III buildings along with corresponding performance level changes to the Quick Check procedures and evaluation statements from Life Safety to Collapse Prevention. Risk Category IV buildings are assessed in ASCE 41-17 for both the BSE-1E and BSE-2E seismic hazards. The increase in seismic hazard is effectively offset by the relaxation in Performance Level and should result in only minimal change between ASCE 41-17 and ASCE 41-13.

The Tier 1 procedure utilizes checklists to quickly screen for deficiencies. The standard provides these checklists for evaluating each common building type, nonstructural components, and foundations for various levels of seismicity and performance levels. ASCE 41-13 Table 4-7 provides a matrix of the required checklists that can be found in Chapter 16. These checklists are available in electronic format for purchase from ASCE to be easily incorporated into a seismic evaluation report.

If the building complies with the benchmark buildings indicated in ASCE 41-13 Table 4-6 and the review and assessment provisions of ASCE 41-13 § 4.3.1 through § 4.3.4, then the building is deemed to comply with BPOE, and no further evaluation or checklists for the building are required. However, the nonstructural checklists are still required. The review and assessment provisions of ASCE 41-13 § 4.3.1 through § 4.3.4 include confirming that the building is in substantial compliance with the building code under which it was designed. This is not an exhaustive check, but the evaluating engineer should verify that the lateral system proportioning and element detailing appear to meet the minimum requirements of that code.

When performing a Tier 1 screening, an on-site investigation and condition assessment of the building are necessary. Material testing is not required, and it is permitted to use default material values indicated in ASCE 41-13 § 4.2.3. There is no need to apply a knowledge factor when using the Tier 1 procedure as it is built into the procedure.

At the conclusion of completing the applicable checklist for the building, all the "noncompliant" and "unknown" items require further investigation to demonstrate compliance with the applicable performance objective. In order to pass the Tier 1 screening, all "unknown" items need to be investigated until the information becomes known and the item can be fully assessed. The "noncompliant" items should be evaluated with the Tier 2 procedure in order to comply with the BPOE performance objective. At the evaluating engineer's discretion, the "unknown" items can be investigated as part of the Tier 2 procedure during a more thorough investigation; however, the building would not be deemed to pass a Tier 1 screening with "unknown" items.

6.3 Tier 2 Deficiency-Based Evaluation and Retrofit (ASCE 41-13 Chapter 4)

The Tier 2 deficiency-based evaluation and retrofit procedure is used after completion of the Tier 1 procedure. The process is to perform a more detailed evaluation of the "noncompliant" and "unknown" items identified in the Tier 1 procedure, and if the item is still determined to be noncompliant, then the building is deemed to not comply with the targeted Performance

Objective. The building or the deficient component may be retrofitted and Tier 1 checklist revised to satisfy the targeted Performance Objective. Each checklist statement in ASCE 41-13 Chapter 16 references an applicable section in the Tier 2 procedure for further evaluation. ASCE 41-13 Figure 5-1 illustrates the Tier 2 evaluation process and is helpful in providing guidance on how to evaluate various components of the building.

Based on the "noncompliant" items identified in the Tier 1 procedure, the Tier 2 procedure may require a full building analysis in order to determine the demands on the potentially deficient items. Per ASCE 41-13 § 5.2.4, only linear analysis methods are permitted for the Tier 2 procedure. If nonlinear analysis methods are to be used, the Tier 3 procedure is required. When determining the capacity of elements in the Tier 2 procedure, ASCE 41-13 § 5.2.6 requires the knowledge factor to equal 0.75 unless data collection and material testing comply with ASCE 41-13 § 6.2.4. See Section 6.6.1 of this *Guide* for further discussion on knowledge factors.

The intent of the Tier 2 deficiency-based procedure is to limit the scope of the evaluation or retrofit to only those "noncompliant" and "unknown" items identified in the Tier 1 procedure. This is a much simpler and focused approach as compared to the Tier 3 procedure where all the components of the building are evaluated. As with the Tier 1 procedure, the Tier 2 procedure uses a single-level assessment to the BSE-1E Seismic Hazard Level, which is unlike the Tier 3 procedure that requires a two-level assessment when evaluating to the BPOE.

The scope of the Tier 2 deficiency-based retrofit need not expand beyond what is necessary to modify the building to comply with the Tier 1 screening or Tier 2 deficiency-based evaluation. The strengthened building should be evaluated to confirm that the strengthened building complies with the intended Tier 1 and Tier 2 performance objective to ensure that the strengthening did not simply shift the deficiency to another critical component. ASCE 41-13 § 2.2.3 addresses similar concerns when addressing Limited Performance Objectives and Partial Retrofit Objectives by requiring that the retrofit design not result in a reduction in the Structural Performance Level, not create a new structural irregularity or make an existing structural irregularity more severe, and not result in an increase in the seismic force to any component that is deficient in capacity to resist such forces.

Commentary

The provisions in ASCE 41-13 § 2.2.3 require the retrofit design for Limited Performance Objectives to essentially not result in a worse condition for the existing components.

Example Summary

Building Type: PC1

Performance Objective: BPOE (Basic Performance Objective for Existing Buildings) – Life Safety Performance at BSE-1E Seismic Hazard Level

Risk Category: II

Location: Anaheim, California

Level of Seismicity: High

Evaluation Procedure: Tier 1 and Tier 2

Analysis Procedure: Linear Static (LSP)

Reference Documents:

NDS-2012

SDPWS-2008

ACI 318-11

AISC 360-10

6.4 Example Building Tilt-up Concrete (PC1)

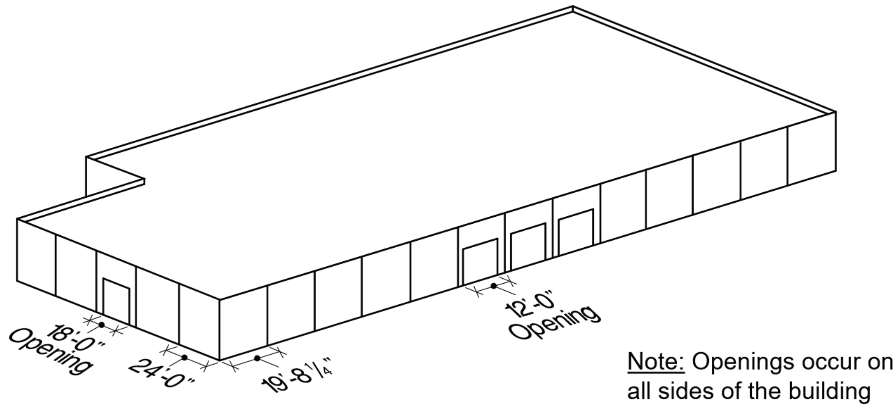
6.4.1 Overview

The example illustrates the seismic evaluation and retrofit of a 1960s tilt-up warehouse building with a panelized wood roof system using the ASCE 41-13 Tier 1 Screening Procedure and the Tier 2 Deficiency-Based Retrofit Procedure. The roof is framed with glued laminated (glulam) beams, purlins, and plywood sheathed diaphragm as illustrated in Figure 6-1. The walls are precast concrete tilt-up panels. The foundation system is continuous spread footings. The example utilizes the same building (shown in Figure 6-1) and Level of Seismicity as the example in Chapter 3 of the *2009 IEBC SEAOC Structural/Seismic Design Manual* (SEAOC, 2012) to provide a direct comparison to the procedure in Chapter A2, Earthquake Hazard Reduction in Existing Reinforced Concrete and Reinforced Masonry Wall Buildings with Flexible Diaphragms, of the *International Existing Building Code* (ICC, 2012b).

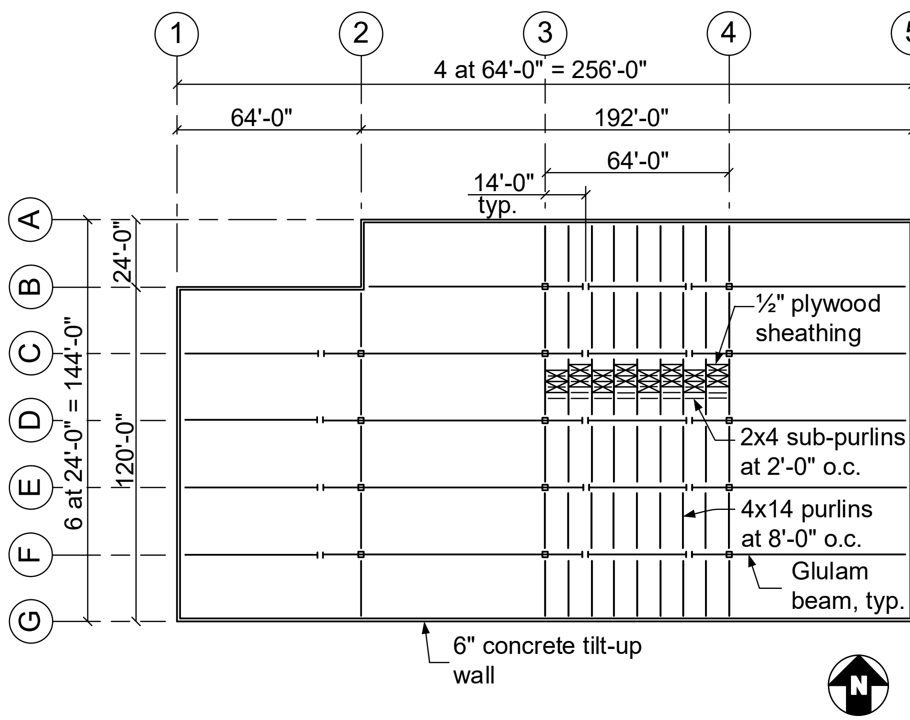
This example illustrates the process of performing a Tier 1 screening of the entire building to identify potential deficiencies and applying the Tier 2 procedure to evaluate the potential deficiencies and to strengthen only those elements the Tier 2 procedure confirmed as deficient with regard to the roof-to-wall anchorage and subdiaphragm analysis for east-west loading and the collector analysis along Gridline B.

This example illustrates the following:

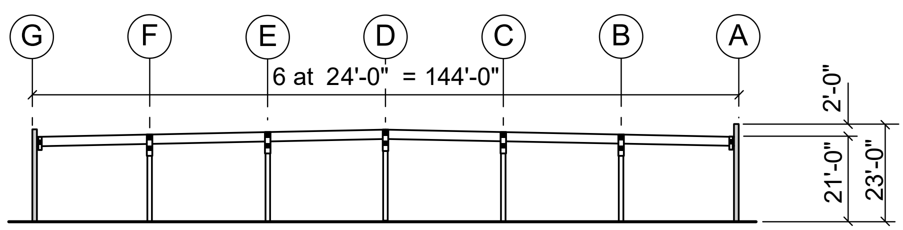
- **Section 6.4.2:** Illustrate building geometry and identify building loads
- **Section 6.4.3:** Identify the selected performance objective for the Tier 1 and 2 procedures (ASCE 41-13 Table 2-1)
- **Section 6.5:** Assess Tier 1 eligibility and identify Tier 1 checklists required (ASCE 41-13 Table 3-2, Table 4-6, Table 4-7)
- **Section 6.5.1:** Calculate pseudo seismic force for use in completing Tier 1 checklist statements (ASCE 41-13 § 4.5.2.1)
- **Section 6.5.2:** Complete Tier 1 checklists (ASCE 41-13 Chapter 16 and Appendix C)
- **Section 6.5.3:** Summary of Tier 1 screening noncompliant items
- **Section 6.6:** Overview of scope of Tier 2 evaluation and retrofit (ASCE 41-13 Chapter 5)
- **Section 6.6.1:** Outline the data collection program and resulting knowledge factors (ASCE 41-13 § 5.2.6)



(a) Tilt-up building



(b) Roof framing plan of tilt-up building



(c) Typical cross-section

Figure 6-1 Tilt-up building geometry.

- **Section 6.6.2:** Determine out-of-plane wall-roof anchorage loads, and evaluate sub-purlin-to-wall anchorage, adhesive anchor, development of load into diaphragm, and purlin for combined bending and axial tension loading for east-west direction loading only (ASCE 41-13 § 5.7.1.1, § 5.2.4, and § 7.2.11)
- **Section 6.6.3:** Evaluate the subdiaphragm, subdiaphragm chord, and diaphragm continuous crosstie for wall anchorage loading in east-west direction loading only (ASCE 41-13 § 5.6.1.2, § 7.2.11)
- **Section 6.6.4:** Determine pseudo seismic forces on roof diaphragm in order to evaluate collector at Gridline B (ASCE 41-13 § 7.4.1.3)
- **Section 6.6.5:** Evaluate the collector and connections at Gridline B (ASCE 41-13 § A5.2.1, § 7.5.2.1.2)
- **Section 6.6.6:** Summary of Tier 2 retrofit

The following items are not addressed in this example:

- Tier 1 screening or Tier 2 evaluation and retrofit of nonstructural components
- Tier 2 evaluation and retrofit of roof-to-wall anchorage for north-south loading direction
- Tier 2 evaluation and retrofit of subdiaphragms for north-south loading direction
- Tier 2 evaluation and retrofit of collectors at re-entrant corner along Gridline 2
- Tier 2 evaluation and retrofit of out-of-plane analysis of panels (see Section 4.7 of this *Guide* for similar example of reinforced masonry wall for out-of-plane loading)

6.4.2 Building Geometry and Loads

6.4.2.1 Building Geometry

The building is a one-story concrete tilt-up warehouse constructed in the 1960s as shown in Figure 6-1. The roof is framed with glued laminated beams, purlins, and plywood roof sheathing using a panelized system. The connection details between roof elements and walls are shown in Figure 6-2 to Figure 6-4. The floor is slab-on-grade, and the walls are supported by continuous footings.

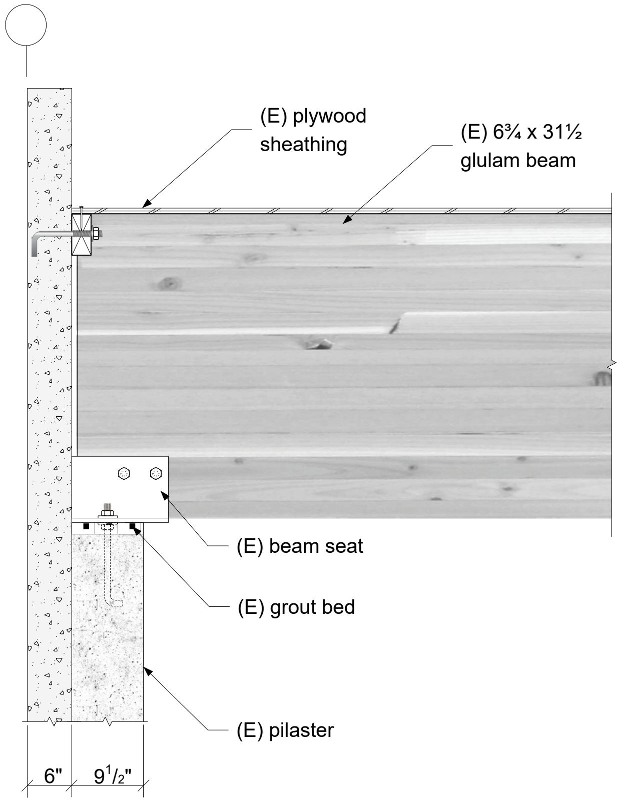


Figure 6-2 Section at existing glulam beam support at pilaster.

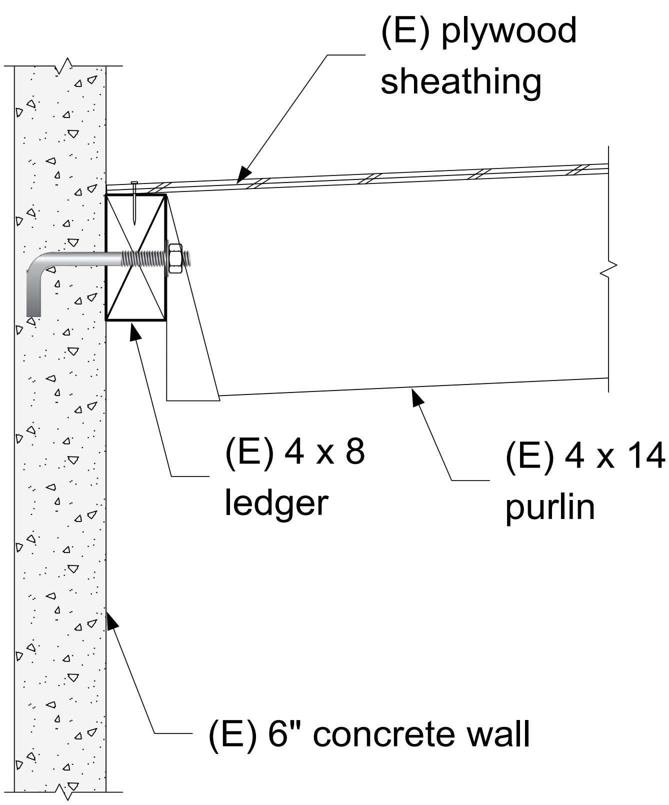


Figure 6-3 Section at purlin support at ledger.

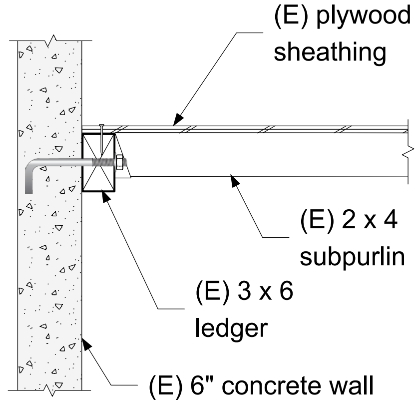


Figure 6-4 Existing sub-purlin support at ledger.

6.4.2.2 Building Information and Loads

The following building information is based on the existing building drawings and on-site investigation:

- Date built: 1967
- Seismic and site data:
  - Location: Anaheim, California
  - Site Class: D
  - Risk Category: II (warehouse)
- Building story height: One-story, 21 feet
- Walls
  - Thickness: 6 inches
  - Height: 23 feet
  - Reinforcing steel: #5 at 16 inches on center vertical and #5 at 18 inches on center horizontal
  - Reinforcing steel strength: 60,000 psi
  - Concrete strength: 3,000 psi
  - Normal weight concrete: 150 pcf
  - Dead load: 75 psf (= W_{psf})
- Roof (material design values of wood components obtained from ANSI/AWC NDS-2012, *National Design Specification for Wood Construction* (AWC, 2012) Supplement)
  - Diaphragm sheathing: 15/32-inch wood structural panel (Structural 1), 10d at 4/6/12 pattern (boundary, edge, field), blocked

- Girders: 6¾×31½ glulam beam girders – 24F-V4 DF/DF for simple span and 24F-V8 DF/DF for spans with cantilevers
- Purlins: 4×14 Douglas Fir-Larch, Select Structural
- Sub-purlins: 2×4 Douglas Fir-Larch, No. 1
- Ledger: 3×6 and 4×8 Douglas Fir-Larch, No. 1
- Dead load: See Table 6-1.
- Live load: 20 psf, reducible

Table 6-1 Roof Dead Loads

Component	Dead Load (psf)
Roofing (3 layers rolled roofing)	3.0
Sheathing (15/32-inch)	1.4
Sub-purlins (2×4 at 24 inches on center)	0.7
Purlins (4×14 at 8 feet on center)	1.4
Girders (6¾×31½ glulam beams)	2.2
Ceiling (5/8-inch gypsum board)	3.1
MEP/miscellaneous components*	2.2
Total	14.0

* No partition allowance included.

- Wall anchorage
  - Girders: Steel seat, see Figure 6-2
  - Purlins: Ledger anchor bolt, see Figure 6-3
  - Sub-purlins: Ledger anchor bolt, see Figure 6-4
- Collector connections at re-entrant corner
  - Line B: Steel bucket, see Figure 6-27
  - Line 2: None
- Material properties of new components
  - Light-gauge metal strap yield and tensile strength, f_y = 50,000 psi and f_u = 65,000 psi (per manufacturer's evaluation report)
  - Anchor rod yield strength, f_y = 36,000 psi

6.4.3 Performance Objective

In accordance with ASCE 41-13 § 4.1.2 and § 5.2.1, the Performance Objective is the Basic Performance Objective for Existing Buildings (BPOE) using the BSE-1E Seismic Hazard Level, and per ASCE 41-13 Table 2-1 for Risk Category II, the Life Safety Performance Level will be evaluated.

6.5 Tier 1 Screening of Example Building

Useful Tip

The Tier 1 checklists contained in ASCE 41-13 Appendix C are available as fillable PDF forms and can be purchased through ASCE Library at the following website: http://ascelibrary.org/doi/book/10.1061/9780784478660.

The format used in this example uses the same format as the examples illustrated in ASCE 31-3 Appendix A (ASCE, 2003). Note that this format is different than the fillable PDF forms available through the ASCE Library above, but the overall approach is the same.

In accordance with ASCE 41-13 § 3.3.1.1, the Tier 1 procedure requires that the building comply with one of the common building types in ASCE 41-13 Table 3-1 and meet the limitations on size (number of stories) in ASCE 41-13 Table 3-2. This building clearly complies with the PC1 definition in ASCE 41-13 Table 3-1. Prior to entering ASCE 41-13 Table 3-2, the Level of Seismicity needs to be determined.

The Level of Seismicity is determined in accordance with ASCE 41-13 Table 2-5 and is computed with the mapped BSE-2N response acceleration parameters to calculate the BSE-1N Seismic Hazard Level. The BSE-2N response acceleration parameters obtained through the online tools as described in Section 3.3 of this *Guide* and are as follows for this site:

$S_{S,BSE\text{-}2N} = 1.500\text{g}$

$S_{1,BSE\text{-}2N} = 0.579\text{g}$

$S_{DS,BSE\text{-}1N} = (2/3)F_a S_{S,BSE\text{-}2N} = (2/3)(1.00)(1.500\text{g}) = 1.000\text{g} > 0.5\text{g}$

$S_{D1,BSE\text{-}1N} = (2/3)F_v S_{1,BSE\text{-}2N} = (2/3)(1.500)(0.579\text{g}) = 0.579\text{g} > 0.2\text{g}$

Per ASCE 41-13 Table 2-5, the Level of Seismicity is High.

The number of stories is not more than the two-story limitation in ASCE 41-13 Table 3-2 for High Level of Seismicity, so the building qualifies for the Tier 1 procedure. However, the building does not qualify as a Benchmark Building indicated in ASCE 41-13 Table 4-6 since this 1960s tilt-up was constructed before the 1997 *Uniform Building Code* (ICC, 1997) and is not deemed to comply, so the structural checklists are required to be completed.

In accordance with ASCE 41-13 Table 4-7 for High Level of Seismicity and Life Safety Performance Level, the following checklists from ASCE 41-13 Chapter 16 are required:

- Life Safety Basic Configuration Checklist (ASCE 41-13 § 16.1.2LS)
- Life Safety Structural Checklist for Building Type PC1 (ASCE 41-13 § 16.12LS)
- Life Safety Nonstructural Checklist (ASCE 41-13 § 16.17): This is not included as part of this example.

In accordance with ASCE 41-13 § 4.2.1, an on-site investigation was conducted to verify general conformance of the existing conditions to provided construction documents. These documents provided the information contained in Section 6.4.2 of this *Guide*. The investigation followed the general recommendations of ASCE 41-13 Table 4-1, and no defects or deterioration were discovered. The investigation included a site assessment of the exterior and interior of the building, and there was no evidence of settlement, cracks, or distress in the concrete wall panels and the wood framing did not have visible evidence of decay, distress, or damage.

6.5.1 Pseudo Seismic Force

The Tier 1 screening checklists require analysis using the Quick Check procedure, which are used to calculate the stiffness and strength of certain building components. This requires the pseudo seismic force to be calculated in accordance with ASCE 41-13 § 4.5.2.1.

As discussed in Section 6.2 of this *Guide*, the Tier 1 screening procedure is based on the BSE-1E Seismic Hazard Level. The spectral response acceleration parameters, S_{XS} and S_{X1}, for the BSE-1E Seismic Hazard Level have been obtained through the online tools as described in Section 3.3 of this *Guide* for this site are:

$S_{XS,BSE\text{-}1E}$ = 0.801g

$S_{X1,BSE\text{-}1E}$ = 0.443g

Pseudo seismic forces are computed per ASCE § 4.5.2.1 as follows:

$$V = CS_aW \qquad \text{(ASCE 41-13 Eq. 4-1)}$$

where:

C = 1.0 per ASCE 41-13 Table 4-8, building type PC1

S_a = response spectral acceleration parameter at the fundamental period of the building

$$S_{a,BSE\text{-}1E} = \frac{S_{X1,BSE-1E}}{T} \leq S_{XS,BSE-1E} \qquad \text{(ASCE 41-13 Eq. 4-4)}$$

The building fundamental period, T, is calculated using the empirical period formulation in accordance with ASCE 41-13 § 4.5.2.4, as follows:

$$T = C_t h_n^{\beta} = (0.020)(21\text{ ft})^{0.75} = 0.20 \text{ seconds}$$

where:

C_t = 0.020 for all other framing systems

h_n = height above the base to the roof level = 21 ft

β = 0.75 for all other framing systems

$S_{a,BSE\text{-}1E} = \dfrac{0.443\text{g}}{0.20\text{ s}} = 2.22\text{g}$, but exceeds $S_{XS,BSE\text{-}1E} = 0.80\text{g}$,

therefore,

$S_a = 0.80\text{g}$

W is the effective seismic weight of the building.

$V = CS_aW = (1.0)(0.80\text{g})W = 0.80W$

6.5.2 Tier 1 Checklists

In this section, the checklist items are indicated in *italics* (printed with permission from ASCE) and its evaluation pertaining to the example building follows each checklist item. Each evaluation statement is marked "compliant" (C), "noncompliant" (NC), "not applicable" (N/A), or "unknown" (U). Each statement refers to ASCE 41-13 Appendix A for additional commentary. Each checklist is segregated into "Low Seismicity," "Moderate Seismicity," and "High Seismicity." For the High Level of Seismicity, the checklist items associated with Low and Moderate Seismicity are also required to be completed.

Life Safety Basic Configuration Checklist (ASCE 41-13 § 16.1.2LS)

Low Seismicity

Building System

General

C NC N/A U *LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)*

The building contains a complete load path. However, the collectors at the re-entrant corner are deficient and are addressed in the checklist item "TRANSFER TO SHEAR WALLS" below.

C NC N/A U *ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent* *building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)*

There are no adjacent buildings; item is not applicable.

C NC N/A U *MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)*

The building contains no mezzanines; item is not applicable.

Building Configuration

C NC N/A U *WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)*

The building is one-story; item is not applicable.

C NC N/A U *SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)*

The building is one story; item is not applicable.

C NC N/A U *VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)*

The building does not contain any discontinuous vertical elements.

C NC N/A U *GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting* *system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)*

The building is one story; item is not applicable.

C NC N/A U *MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)*

The building is one story; item is not applicable.

C NC N/A U *TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)*

The building has well-distributed perimeter walls that are essentially rigid with a flexible diaphragm, therefore it is compliant for torsion.

Useful Tip

The California Geologic Survey under the Department of Conservation maintains regulatory maps of fault, liquefaction, and landslide zones in accordance with the Alquist-Priolo Earthquake Fault Zoning Act (1972) and Seismic Hazards Mapping Act (1990). The maps are available at the following website: http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

Other states in regions of high seismicity have similar mapping projects.

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Geologic Site Hazards

C NC N/A U *LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)*

The site does not contain liquefaction-susceptible soils based on a review of the Anaheim Quadrangle Seismic Hazard Zones map published by the California Department of Conservation, Division of Mines and Geology shown in Figure 6-5.

C NC N/A U *SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted* *movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)*

The site is generally flat and remote from slopes susceptible to earthquake-induced slope failures or rockfalls. A review of the Anaheim Quadrangle Seismic Hazard Zones map, published by the California Department of Conservation, Division of Mines and Geology, shown in Figure 6-5 does not indicate any earthquake-induced landslides in the region.

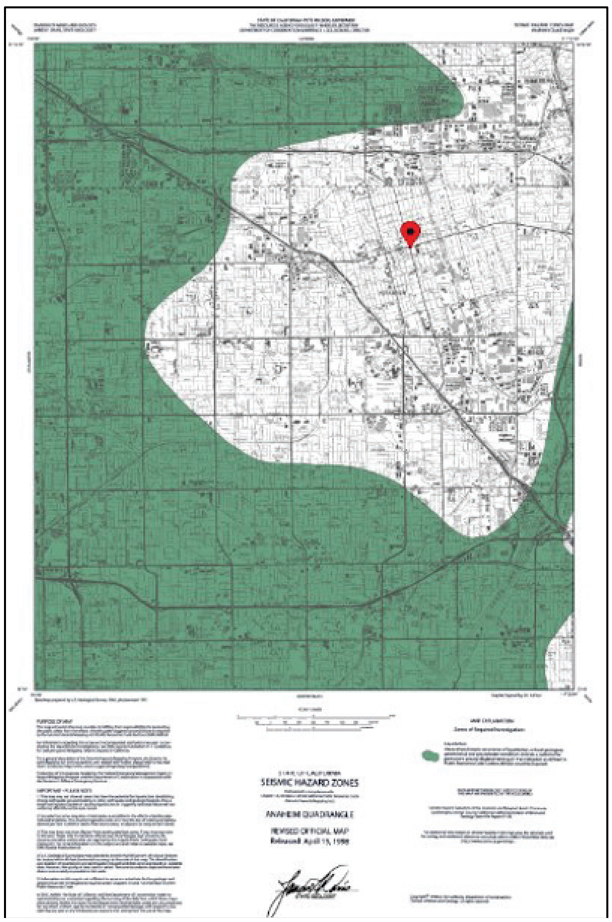


Figure 6-5 Seismic hazard zones map of Anaheim quadrangle with approximate site location indicated.

C NC N/A U *SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)*

The site does not contain active faults in the vicinity based on a review of the California Department of Conservation, Division of Mines and Geology mapping portal website for the Anaheim Quadrangle.

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Foundation Configuration

C NC N/A U *OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/ height) is greater than 0.6S_a. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)*

The tilt-up panels are detailed to each act as an independent wall element and do not have connections at vertical panel joints to transfer overturning forces to adjacent panels (the pilasters at adjoining panel edges are independent from each other and not integrally cast). Therefore, the lowest ratio of the least horizontal dimension of the seismic-force-resisting system to the building height (21 ft) occurs along Gridlines A, B, and G where the panel lengths are 19.7 ft:

l/h = 19.7 ft/21 ft = 0.94

$0.6S_a$ = 0.6(0.80g) = 0.48

0.94 > 0.48; therefore, overturning is compliant.

C NC N/A U *TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)*

The Site Class is D; therefore, this item is required to be evaluated. All wall footings and wall panels are connected together with dowels into a common pour strip and all isolated column footings are interconnected by the slab-on-grade.

Life Safety Structural Checklist for Building Type PC1: Precast or Tilt Up Concrete Shear Walls with Flexible Diaphragms (ASCE 41-13 § 16.12LS)

Low Seismicity

Connections

C NC N/A U *WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)*

The existing purlins and sub-purlins connections to the existing walls result in cross-grain bending in the wood ledger; see Figure 6-3 and Figure 6-4. There are no connections to create subdiaphragms at intermediate purlins and girders. The existing girders are connected with beam seats at the exterior walls and are noncompliant by observation; see Figure 6-2.

Commentary

The quick check calculation for flexible diaphragm connection forces in ASCE 41-13 § 4.5.3.7 (Equation 4-13) does not specify what acceptance criteria to use. It is intended that the connection be evaluated as a force-controlled action using the acceptance criteria in ASCE 41-13 § 7.5.2.2.2 using lower-bound material strengths.

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Seismic Force-Resisting System

C NC N/A U *REDUNDANCY: The number of lines of shear walls in each principle direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)*

The building has three lines of resistance in each principle direction; therefore, this item is compliant.

C NC N/A U *WALL SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.*^{2} *or* $2\sqrt{f'_c}$. *(Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1)*

The wall shear stress check will be evaluated for each line of resistance based on the flexible diaphragm

Commentary

The shear stress check for shear walls in ASCE 41-13 § 4.5.3.3 is a conservative check on the average shear demand of all the shear walls in the direction of load in a story. This approach is generally applicable with rigid diaphragm buildings where the load is shared to all walls in the story based on their rigidity. However, for flexible diaphragm buildings, load to each shear wall line is typically based on the tributary diaphragm width, and the story demands may be concentrated on certain shear wall lines that have large tributary areas and are not related to the wall rigidity. For flexible diaphragm buildings, such as in this example, it is an ASCE 41-17 Revision

The Seismic Hazard Level used in the Tier 1 and Tier 2 procedures in ASCE 41-17 changes from BSE-1E to BSE-2E for Risk Category I through III buildings along with corresponding Performance Level changes to the Quick Check procedures and evaluation statements from Life Safety to Collapse Prevention. Risk Category IV buildings are still assessed for both the BSE-1E and BSE-2E Seismic Hazards Levels in ASCE 41-17. The increase in the Seismic Hazard Level is effectively offset by the relaxation in Performance Level and should result in only minimal change between ASCE 41-17 and ASCE 41-13.

tributary width. See the margin box for further discussion on this assumption.

The shear shall not exceed the greater of 100 lb/in.^{2} or $2\sqrt{3{,}000\text{ lb/in.}^2} = 110\text{ lb/in.}^2$.

Effective seismic weight, W:

Roof dead load = 14 psf (per Table 6-1)

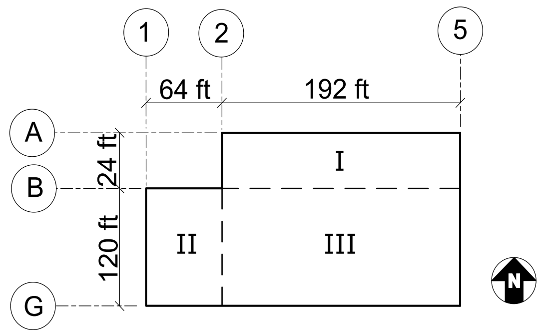


Figure 6-6 Roof plan showing diaphragm segments and dimensions (not to scale).

The effective seismic weight of each diaphragm segment is as follows:

W_{I} = (192 ft)(24 ft)(14 psf/1000 lb/kip) = 65 kips

W_{II} = (120 ft)(64 ft)(14 psf /1000 lb/kip) = 108 kips

W_{III} = (120 ft)(192 ft)(14 psf /1000 lb/kip) = 323 kips

The effective seismic weight of the wall computed as the reaction to the roof assuming pinned base (see Figure 6-1 and Figure 6-8) is as follows:

$$W_{\text{wall}} = \frac{h_p^2}{2h_r} t\gamma_w = \frac{(23\text{ ft})^2}{2(21\text{ ft})}(0.5\text{ ft})\left(\frac{150\text{ lb/ft}^3}{1000\text{ lb/kip}}\right)$$
$$= 0.94\text{k/ft}$$

where:

h_p = height of the parapet from grade = 23 ft

h_r = height of the wall to the roof level = 21 ft

t = thickness of the concrete wall = 0.5 ft

γ_w = density of the concrete wall = 150 lb/ft^3

The pseudo seismic force at each shear wall line including weight of parallel and perpendicular walls and ignoring the reduced weight from wall openings is as follows:

V = 0.80W, per Section 6.5.1

V_1 = 0.80[0.94 kips/ft(120 ft + 2(64 ft/2)) + 0.5(108 kips)]
= 182 kips

V_2 = 0.80[0.94 kips/ft (192 ft + 64 ft + 24 ft) + 0.5(108 kips + 323 kips + 65 kips)]
= 409 kips

V_5 = 0.80[0.94 kips/ft (192 ft + 144 ft) + 0.5(65 kips + 323 kips)]
= 408 kips

V_A = 0.80[0.94 kips/ft (192 ft + 24 ft) + 0.5(65 kips)]
= 188 kips

V_B = 0.80[0.94 kips/ft (64 ft + 144 ft) + 0.5(108 kips + 323 kips + 65 kips)]
= 355 kips

V_G = 0.80[0.94 kips/ft (256 ft + 120 ft) + 0.5(108 kips + 323 kips)]
= 455 kips

The average shear stress is checked in each shear wall line:

$$v_j^{\text{avg}} = \frac{1}{M_S}\left(\frac{V_j}{A_w}\right) < 110 \text{ lb/in.}^2 \quad \text{(ASCE 41-13 Eq.4-9)}$$

where:

M_S = system modification factor per ASCE 41-13 Table 4-9 = 4

V_j = story shear at each level at each line of resistance

A_w = summation of wall area in the direction of loading

$$v_1^{\text{avg}} = \frac{1}{4}\left(\frac{182 \text{ kips}(1000 \text{ lb/kip})}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(120 \text{ ft} - 18 \text{ ft})}\right) = 6 \text{ lb/in.}^2$$

$$v_2^{\text{avg}} = \frac{1}{4}\left(\frac{409 \text{ kips}(1000 \text{ lb/kip})}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(24 \text{ ft})}\right) = 59 \text{ lb/in.}^2$$

$$v_5^{avg} = \frac{1}{4}\left(\frac{408 \text{ kips}\left(1000 \text{ lb/kip}\right)}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(144 \text{ ft} - 18 \text{ ft})}\right) = 11 \text{ lb/in.}^2$$

$$v_A^{avg} = \frac{1}{4}\left(\frac{188 \text{ kips}\left(1000 \text{ lb/kip}\right)}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(192 \text{ ft} - 3(12 \text{ ft}))}\right)$$
$$= 4 \text{ lb/in.}^2$$

$$v_B^{avg} = \frac{1}{4}\left(\frac{355 \text{ kips}\left(1000 \text{ lb/kip}\right)}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(64 \text{ ft})}\right) = 19 \text{ lb/in.}^2$$

$$v_G^{avg} = \frac{1}{4}\left(\frac{455 \text{ kips}\left(1000 \text{ lb/kip}\right)}{(0.5 \text{ ft})(144 \text{ in.}^2/\text{ft}^2)(256 \text{ ft} - 3(12 \text{ ft}))}\right)$$
$$= 7 \text{ lb/in.}^2$$

The average shear stress at each line of resistance is less than 110 lb/in.^{2}.

C NC N/A U *REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3)*

The wall is reinforced with #5 bars at 16 inches on center vertically and #5 bars at 18 inches on center horizontally.

$\rho_v = A_{sv}/ts_v = (0.31 \text{ in.}^2)/(6 \text{ in.})(16 \text{ in.})$

$= 0.0032 > 0.0012$

$\rho_h = A_{sh}/ts_h = (0.31 \text{ in.}^2)/(6 \text{ in.})(18 \text{ in.})$

$= 0.0029 > 0.0020$

The vertical and horizontal wall reinforcing steel ratios exceed both minimum values, respectively.

C NC N/A U *WALL THICKNESS: Thicknesses of bearing walls shall not be less than 1/40 the unsupported height or length, whichever is shorter, nor less than 4* in. *(Commentary: Sec. A.3.2.3.5. Tier 2: Sec. 5.5.3.1.2)*

The tilt-up panels are 6 inches thick with an unsupported height of 21 feet, resulting in a height to thickness ratio of 42, which exceeds 40.

Diaphragms

C NC N/A U *TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2* in. *(Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)*

The building does not contain precast concrete diaphragms or topping slabs; item is not applicable.

Connections

C NC N/A U *WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)*

The existing purlins and sub-purlins are not anchored to the existing walls for out-of-plane loading, which results in cross-grain bending in the wood ledger; see Figure 6-3 and Figure 6-4 of this *Guide*.

C NC N/A U *TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)*

The existing diaphragms are connected to the tilt-up panel shear walls to transfer in-plane shear; however, the collectors at the re-entrant corner at the intersection of Gridlines B and 2 in Figure 6-1 of this *Guide* do not have in-plane collector connections to the walls nor connections at purlins and girder splices that extend the full depth of the diaphragm. (Note: ASCE 41-13 § A.5.2.1 indicates that where walls do not extend the full depth of the diaphragm, the connection in this checklist item also includes the collectors and their connections to deliver concentrated loads.)

C NC N/A U *TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame* *elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)*

The building does not contain precast concrete diaphragms or topping slabs; item is not applicable.

C NC N/A U *GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)*

The girders are connected to the columns with a premanufactured bolted steel bucket assembly. The girders are connected to the concrete pilasters with steel bucket assembly bolted to the girder and embedded anchor bolts to the pilaster; see Figure 6-2 of this *Guide*.

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Seismic Force-Resisting System

C NC N/A U *DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)*

The building does not contain rigid or semi-rigid diaphragms; item is not applicable.

C NC N/A U *WALL OPENINGS: The total width of openings along any perimeter wall line constitutes less than 75% of the length of any perimeter wall when the wall piers have aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1)*

The wall line with the largest percentage of openings is wall line A with (12 ft)(3)/(192 ft) = 19% < 75%. There are localized wall piers with aspect ratios exceeding 2-to-1; however, the stiffness of adjacent solid panels protects these piers from in-plane loads and drift.

Diaphragms

C NC N/A U *CROSSTIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)*

There are no continuous connection splices at purlins and girder lines to form continuous crossties between diaphragm chords.

C NC N/A U *STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)*

The building does not contain straight sheathed diaphragms; item is not applicable.

C NC N/A U *SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)*

The diaphragms are all wood structural panels (plywood).

C NC N/A U *DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec.5.6.2)*

The building does not contain diagonally sheathed or unblocked diaphragms; item is not applicable.

C NC N/A U *OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)*

The diaphragms are all wood structural panels (plywood).

Commentary

The "MINIMUM NUMBER OF WALL ANCHORS PER PANEL" checklist item does not require an evaluation of the strength of the anchors or cross-grain tension as those are evaluated in separate checklist items, "WALL ANCHORAGE" and "WOOD LEDGERS," respectively. This checklist item is simply to check that there are at least two anchors per panel into the diaphragm.

Connections

C NC N/A U *MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors from each precast wall panel into the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4)*

The existing wall panels are connected to the diaphragm with anchor bolts in the ledger and contain at least two per panel.

C NC N/A U *PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6. Tier 2: Sec. 5.7.3.4)*

All wall footings and wall panels are connected together with dowels into a common pour strip.

C NC N/A U *UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)*

The building does not contain pile caps; item is not applicable.

C NC N/A U *GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2)*

The girder seat connection to the pilaster does not contain reinforcing steel ties securing the anchor bolts, and the girders do not contain independent wall anchors to the tilt-up panels, see Figure 6-2 of this *Guide*.

6.5.3 Tier 1 Screening Summary

The Tier 1 Screening identified a number of noncompliant items in the checklist in Section 6.5.2, which are summarized pictorially below in Figure 6-7.

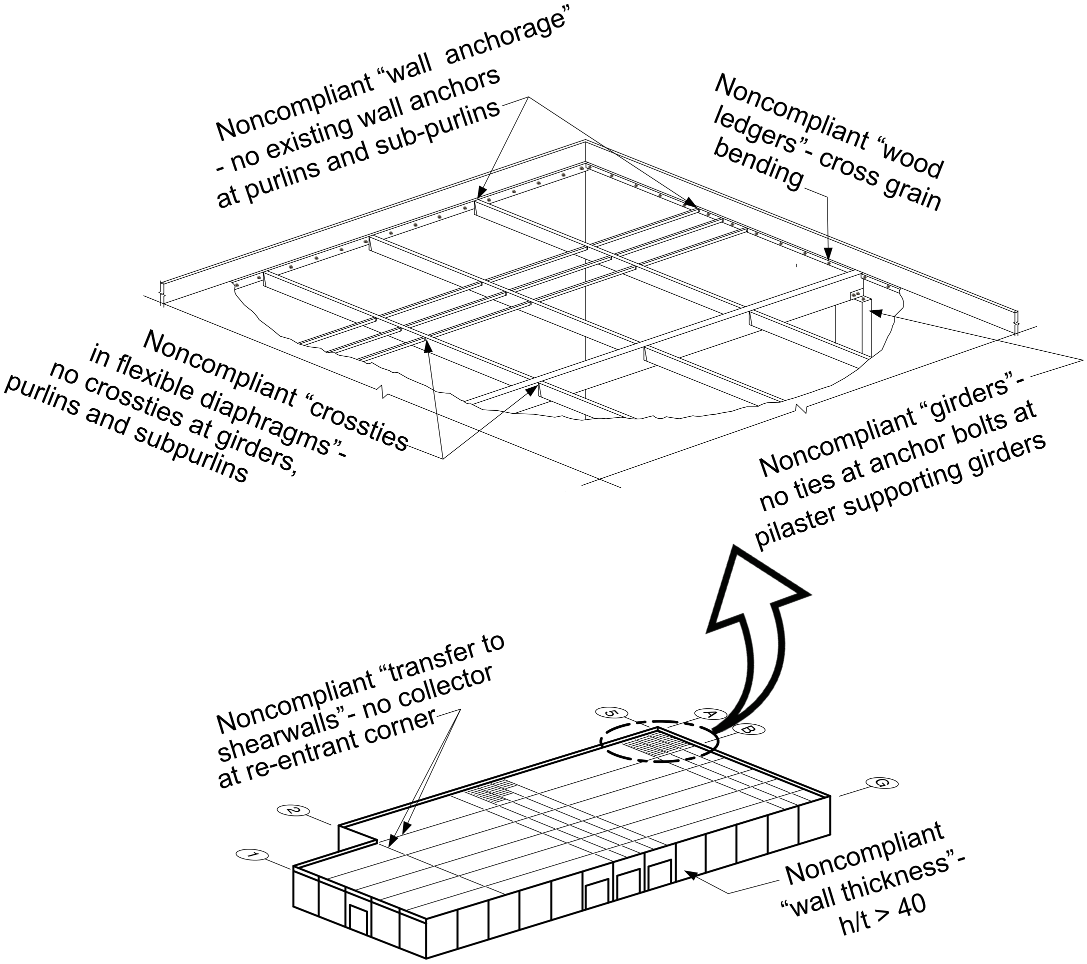


Figure 6-7 Tilt-up building Tier 1 screening noncompliant items.

6.6 Tier 2 Evaluation and Retrofit of Example Building

The Tier 2 evaluation and retrofit will focus on the noncompliant items found in the Tier 1 screening in Section 6.5 of this *Guide*. As indicated in Section 6.4.1 of this *Guide*, only those deficiencies related to the roof-to-wall anchorage and subdiaphragm analysis for east-west loading direction and the collector analysis along Gridline B will be evaluated in this example. The figures used for the roof-to-wall anchorage will illustrate the framing along Gridline 1, which is the same as that on Gridline 5. The roof-to-wall anchorage and subdiaphragm analysis for north-south loading direction and the collector analysis along Gridline 2 are not addressed in the example, however, the evaluation is similar. The noncompliant checklist item regarding the concrete panel thickness are not addressed in the example; however, see Section 4.7 of this *Guide* for similar example of a reinforced masonry wall analysis for out-of-plane loading.

In order to determine the scope of the Tier 2 evaluation and retrofit for a noncompliant item, the checklist statement includes a reference to a section in ASCE 41-13 § 5.4 to § 5.7 that provides specific direction as to scope of the evaluation required. At the beginning of each section below, the applicable ASCE 41-13 sections referenced from the checklists are indicated with an explanation of the scope of the required Tier 2 evaluation.

The Tier 2 evaluation and retrofit uses the same Performance Objective and Level of Seismicity as Tier 1 screening, which is Life Safety Performance Level for the BSE-1E Seismic Hazard Level.

The Tier 2 retrofit strategy for this building used in the example is to install the strengthening from the underside of the structure so reroofing is not required. With this approach, the existing diaphragm nailing patterns will be used and not enhanced.

6.6.1 Data Collection and Material Properties

When using the Tier 2 procedure, ASCE 41-13 § 5.2.6 requires that the knowledge factor, κ, for existing components be taken as 0.75 unless the data collection complies with ASCE 41-13 § 6.2.4.1 and Table 6-1 that outline the level of testing required, as-built information needed, condition assessment required, and material properties determination in accordance with the selected Performance Level. This information is needed to gain knowledge of the existing structure. The following assumptions are made for this example: (1) original design drawings have been obtained that indicate specified material properties; (2) no testing has been performed, but some inspections have been performed as outlined below; and (3) a condition assessment of the building has been performed and found that there is no deterioration or damage. See Section 3.5 of this *Guide* for further discussion on data collection and condition assessments.

κ is determined on an individual component basis as determined by the level of knowledge obtained for that component per ASCE 41-13 § 6.2.4.1. In accordance with ASCE 41-13 Table 6-1 and the stated Performance Level of Life Safety for this example, the usual level of data collection is required to use of a κ of 1.0. It is not permissible to use a κ equal to 0.9 since ASCE 41-13 § 5.2.6 requires the Tier 2 procedure use a κ equal to either 0.75 or 1.0.

The following summarize the κ used for each component and the corresponding data collection performed for this example. The values below are based on the usual level of data collection.

- **Existing Concrete Walls:** Concrete cores samples were not obtained and tested in accordance with ASCE 41-13 § 10.2.2.4.1 Bullet (1); therefore, $\kappa = 0.75$ when evaluating components based on concrete strength. Reinforcing steel testing samples are not required in accordance with ASCE 41-13 § 10.2.2.4.1 Bullet 3, as the existing design strength was specified on the original drawings; therefore, $\kappa = 1.0$ when evaluating components based on reinforcing steel strength only.
- **New Concrete Anchors:** Post-installed anchors in the existing concrete walls use $\kappa = 0.75$ when evaluating components based on existing concrete strength and $\kappa = 1.0$ when evaluating components based on existing reinforcing steel as noted above for existing concrete walls. $\kappa = 1.0$ when evaluating new components not affected by concrete strength (e.g., steel anchor strength). This refinement of multiple κ factors for evaluating concrete anchors is not explicitly stated in ASCE 41-13, but is a reasonable approach. A more common approach would be to apply one κ factor for evaluating all concrete anchor strength calculations.
- **Existing Wood Diaphragms:** The existing roofing was not partially removed to verify the diaphragm fastener spacing in accordance with ASCE 41-13 § 12.2.2.4.1 Bullet 1; therefore, $\kappa = 0.75$ when evaluating component strength based on diaphragm fasteners.
- **Existing Wood Members (Sawn):** As indicated above, the diaphragm fastener spacing was not verified, but this should not cause $\kappa = 0.75$ to be applied to all wood components if reasonable confirmation of the material properties can be made. The grade stamps on the purlins and subpurlins were visible and verified to match that specified on the existing building drawings; therefore, $\kappa = 1.0$ when evaluating component strengths based on the purlins or subpurlins.
- **Existing Wood Members (Glulam):** As with the purlins and subpurlins, an attempt was made to observe the glulam beam grade stamps; however, as is often the case, the grade stamps were on the top of the beams and not visible. For this example, since no visual confirmation could be made on the glulam beam grade stamps, $\kappa = 0.75$ is used when evaluating component strengths based on the glulam beams.
- **New Components:** For new components where the strength does not directly rely on existing materials, κ is 1.0.

6.6.2 Wall-Roof Anchorage for East-West Direction Seismic Loads

This section will evaluate the wall-roof anchorage based on the "noncompliant" Tier 1 checklist items "Wall Anchorage" and "Wood Ledgers." The "Wall Anchorage" checklist item directs the user to ASCE 41-13 § 5.7.1.1 for the Tier 2 procedure, which requires that a more detailed analysis of the wall anchorage system be performed in accordance with ASCE 41-13 § 5.2.4. ASCE 41-13 § 5.2.4 Bullet 8 requires that the wall anchorage be analyzed in accordance with ASCE 41-13 § 7.2.11. The "Wood Ledgers" checklist item directs the user to ASCE 41-13 § 5.7.1.3 for the Tier 2 procedure, which indicates that there is no evaluation available to demonstrate compliance with wood ledgers loaded in cross-grain bending. The strengthening proposed will revise the out-of-plane wall anchorage load path such that it will no longer rely on cross-grain bending.

This section will determine the out-of-plane seismic forces for wall anchorage, evaluate the sub-purlin to concrete wall anchorage device, evaluate the adhesive anchor to concrete panel anchorage, evaluate the development of the sub-purlin load into the diaphragm, and evaluate the combined anchorage tension and gravity bending load on the purlin.

This example will evaluate the wall anchorage along Gridlines 1 and 5 for loading in the east-west direction. A separate analysis is required for the wall anchorage along Gridlines A and G for loading in the north-south direction; however, this example does not repeat the similar effort required to do so.

6.6.2.1 Calculate Out-of-Plane Wall Anchorage Force, F_p

The provisions for the evaluation of out-of-plane seismic loads for wall anchorage to diaphragms are contained in ASCE 41-13 § 7.2.11.1. The out-of-plane seismic loads will be computed for the Life Safety Performance Level for the BSE-1E Seismic Hazard Levels per Section 6.6 of this *Guide*. The out-of-plane force is determined as follows:

$F_{p,BSE\text{-}1E} = 0.4 S_{XS,BSE\text{-}1E}\, k_a\, k_h \chi W_p$ (ASCE 41-13 Eq. 7-9)

where:

$S_{XS,BSE\text{-}1E}$ = 0.801g per Section 6.5.1

k_a = $1.0 + L_f/100$, where L_f = 144 ft (ASCE 41-13 Eq. 7-11)

= 1.0+(144 ft)/(100) = 2.44, but need not exceed 2.0 for flexible diaphragms

= 2.0

k_h = 1.0 for flexible diaphragm (ASCE 41-13 Eq. 7-12)

χ = 1.3 for Life Safety per ASCE 41-13 Table 7-2.

$F_{p,BSE\text{-}1E}$ = 0.4(0.801g)(2.0)(1.0)(1.3)(W_p) = 0.83W_p

Compare to $F_{p,\min}$:

$F_{p,\min}$ = $0.2k_a\chi W_p$ = 0.2(2.0)(1.3)W_p = 0.52W_p(ASCE 41-13 Eq. 7-10)

$F_{p,BSE\text{-}1E}$ = 0.83W_p

Using statics to solve for the reaction at the roof per Figure 6-8, as follows:

$F_{p,BSE\text{-}1E}$ = 0.83W_{psf}(23 ft)(23 ft/2)/(21 ft) = 10.5W_{psf}

W_{psf} = 75 psf per Section 6.4.2.2 of this *Guide*

$F_{p,BSE\text{-}1E}$ = 10.5(75 psf) = 788 plf

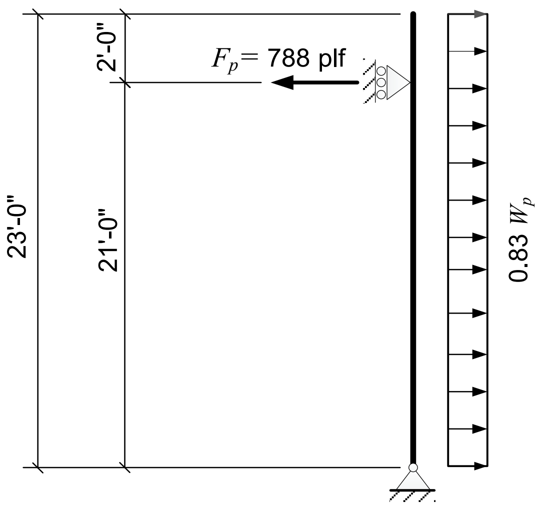


Figure 6-8 Loading diagram for wall anchorage design.

This example conservatively ignores any fixity provided at the base of the wall and the stiffening effects of the pilasters on the two-way bending action of the wall panel and the resulting reduced load to the sub-purlins. As noted earlier, this example does not evaluate the adequacy of the wall anchorage at the girder-to-pilaster support.

The acceptance criteria for the out-of-plane wall anchorage to the diaphragm stipulated in ASCE 41-13 § 7.2.11 require the wall anchorage, sub-diaphragm, and cross ties to be evaluated as force-controlled actions. When evaluating the behavior of force-controlled actions, ASCE 41-13 § 7.5.1.3 requires that the lower-bound component strengths, Q_{CL}, be used.

The acceptance criteria for force-controlled actions using linear analysis procedures are outlined in ASCE 41-13 § 7.5.2.1.2. This section is intended to be applicable to components that resist the pseudo seismic forces calculated in accordance with ASCE 41-13 § 7.4.1.3, and is not applicable to structural integrity, F_p, forces from ASCE 41-13 § 7.2.11, therefore the C_1, C_2, or J factors in ASCE 41-13 § 7.5.2.1.2 cannot be applied to the F_p forces. These factors are applicable to displacement-based design when using pseudo seismic forces. Furthermore, when evaluating connectors (such as nails or bolts) used to link wood components to other wood or metal components, these are required to be evaluated as force-controlled actions with F_p forces and the provisions in ASCE 41-13 § 12.3.3.1 that permit the use of deformation-controlled actions for these connectors are not applicable. The acceptance criteria for force-controlled actions with F_p forces is as follows per ASCE 41-13 § 7.5.2.2.2:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

The knowledge factor, κ, is per Section 6.6.1 of this *Guide*.

6.6.2.2 Sub-Purlin Anchorage to Wall Panel

This section will evaluate the anchorage of the sub-purlin to the wall panel along Gridlines 1 and 5 for loading in the east-west direction. ASCE 41-13 § 7.2.11.1 requires that the maximum spacing of wall anchors to not exceed 8 feet on center unless it can be demonstrated that the wall has adequate capacity to span horizontally a further distance. The sub-purlins are spaced at 2 feet on center, and the bay is 24 feet wide between continuous cross ties at the girder gridlines. This example will evaluate wall anchors spaced at 8 feet on center that will split the bay into thirds. The load to each anchor is as follows:

$$F_{p,BSE\text{-}1E} = (788 \text{ plf})(8 \text{ ft}) = 6{,}300 \text{ lbs per anchor at 8 ft o.c.}$$

The proposed strengthening is to install two wall anchorage hardware devices on sub-purlins at 8 feet on center with a new sistered rafter as illustrated in Figure 6-9. This strategy is necessary so the anchorage devices do not impose an eccentric load and cause weak axis bending on the sub-purlins. Single-sided wall anchorage on small members usually results in inadequate weak axis bending. Furthermore, adding a sistered rafter to the existing 2x sub-purlin is often needed to both resist the large out-of-plane wall anchorage forces for combined tension and bending and provide adequate embedment for the wall anchorage hardware fasteners.

The wall anchorage hardware device and its fasteners will be evaluated as a force-controlled component as noted earlier. See Section 4.6.5.5 of this *Guide* for how to determine the lower-bound strength of manufactured hold-downs and wall anchorage hardware. In this example, the approach outlined in Section 4.6.5.5 of this *Guide* is used, adjusting the "allowable tension loads" value listed in an evaluation report and converts that value to a load and resistance factor design (LRFD) value. For the wall anchorage hardware device selected, the evaluation report lists the following 160% allowable design values for seismic applications, which already includes a load duration factor, C_D, of 1.6.

Wall anchorage hardware "allowable tension load (160%)" = 3,075 lbs

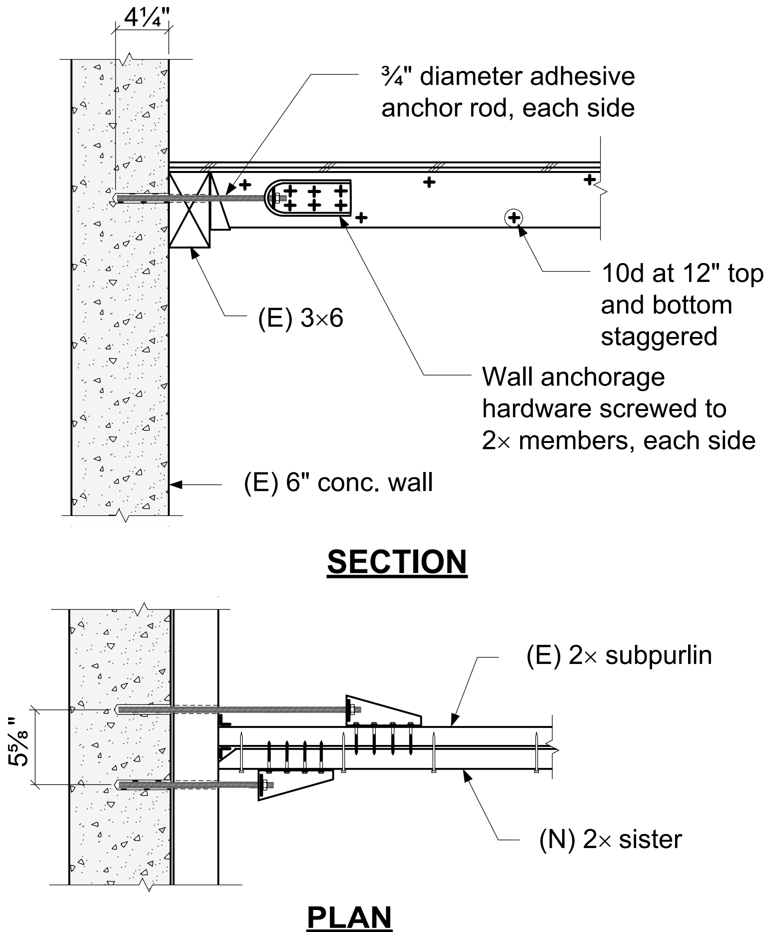


Figure 6-9 Proposed wall anchorage detail.

As outlined in Section 4.6.5.5 of this *Guide*, this published 160% allowable tension value for the wall anchorage hardware is divided by 1.6 (to remove C_D), then multiplied by 3.32 (the format conversion factor, K_F, applicable for connections from NDS-2012) to obtain the LRFD value with a $\phi = 1.0$ which is the expected strength. Per ASCE 41-13 § 12.2.2.5, the lower-bound strength is obtained by multiplying the expected strength by 0.85. The lower-bound strength for two wall anchorage devices is as follows:

Q_{CL} = (3,075 lbs/1.6)(3.32)(0.85)(2) = 10,850 lbs

The knowledge factor is equal to 1.0 per Section 6.6.1 of this *Guide*.

κQ_{CL} = (1.0)(10,850 lbs) = 10,850 lbs

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

10,850 lbs > 6,300 lbs; therefore, wall anchorage hardware is adequate.

The adhesive anchor attaching the wall anchorage hardware to the existing tilt-up panel will be evaluated as a force-controlled components per ASCE 41-13 § 10.3.6.2 where the lower bound strength equals the anchor strength in accordance with ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2011), Appendix D with $\phi = 1.0$. The additional 0.75 seismic reduction factor in ACI 318-11 Section D.3.3.4.4 applied to the concrete failure modes to determine the design tensile strength of concrete anchors is required to be applied as the concrete failure modes have a reduced capacity under cyclic loads.

The evaluation of the adhesive anchor shall satisfy one of the design options in ACI 318-11 Section D.3.3.4.3. The design loads for out-of-plane wall anchorage, F_p, in ASCE 41-13 § 7.2.11 are evaluated as force-controlled actions, and deemed to satisfy the design option in ACI 318-11 Section D.3.3.4.3 Bullet (d), so no further amplification is required. This is similarly stated in 2012 IBC, *International Building Code* (ICC, 2012a), Section 1905.1.9 and 2015 IBC, *International Building Code* (ICC, 2015a), Section 1905.1.8 where out-of-plane wall anchorage forces need not comply with the ductile anchor requirements of ACI 318-11 § D.3.3.

Per the wall anchorage hardware manufacturer's evaluation report, the anchor size for the selected device is 3/4-inch diameter. The anchor will be a ¾-inch diameter adhesive anchor with 4 1/4-inch embedment, ASTM F1554, Grade 36 threaded rod. The embedment of 4 1/4 inches complies with the maximum embedment permitted for a 3/4-inch diameter anchor per the adhesive anchor product evaluation report in a 6-inch thick concrete element. As shown in Figure 6-10, the adhesive anchors are 5.625 inches apart. The adhesive anchor design will assume the concrete wall is cracked as this is the typical assumption for walls subject to seismic loading. Uncracked concrete should not be assumed unless it can be proven otherwise.

Adhesive anchor steel strength is checked in tension per ACI 318-11 Section D.5.1. The anchor bolt properties are taken from the referenced tables in *Notes on ACI 318-11 Building Code Requirements for Structural Concrete with Design Applications* (PCA, 2013):

$N_{sa} = A_{se,N} f_{uta}$ (ACI 318-11 Eq. D-2)

$A_{se,N}$ = effective cross-sectional area

= 0.335 in.^{2} (PCA Notes Table 34-2)

f_{uta} = specified tensile strength

= 58,000 psi (PCA Notes Table 34-1)

N_{sa} = (0.335 in.^{2})(58,000 psi) = 19,430 lbs × 2 anchors = 38,860 lbs

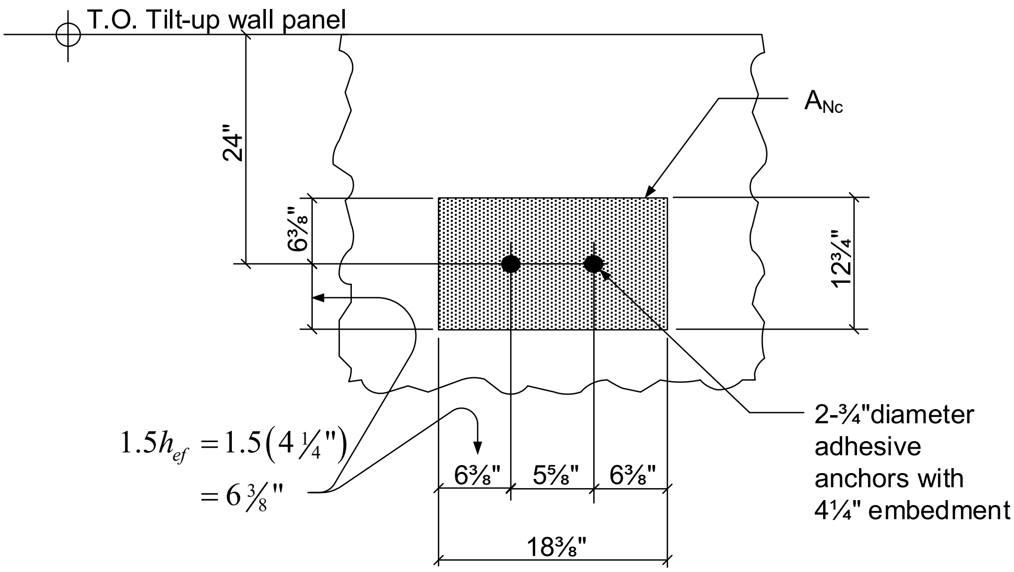


Figure 6-10 Projected concrete failure area of group of adhesive anchors.

Adhesive anchor concrete breakout strength is checked in tension per ACI 318-11 Section D.5.2:

$$N_{cbg} = 0.75\frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \quad \text{(ACI 318-11 Eq. D-4)}$$

A_{Nc} = (12.75 in.)(18.375 in.) = 234 in.^{2}, see Figure 6-10 of this *Guide*

A_{Nco} = $9h_{ef}^2$ = 9(4.25 in.)^2 = 163 in.^{2} (ACI 318-11 Eq. D-5)

$\Psi_{ec,N}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-8)

$\Psi_{ed,N}$ = 1.0, no edge effects since $c_{a,\min} \geq 1.5h_{ef}$ (ACI 318-11 Eq. D-9)

$\Psi_{c,N}$ = 1.0, cracked concrete

$\Psi_{cp,N}$ = 1.0, no splitting effects $c_{a,\min} \geq c_{ac}$ (ACI 318-11 Eq. D-11)

N_b = $N_b = k_c\lambda_a\sqrt{f'_c}h_{ef}^{1.5}$ (ACI 318-11 Eq. D-6)

k_c = 17, post installed anchor in cracked concrete per the Concrete Breakout Design Information table in product evaluation report

λ_a = 1.0, normal weight concrete

f'_c = 3,000 psi

h_{ef} = 4.25 inches embedment

$$N_b = 17(1.0)\sqrt{3000 \text{ psi}}\left(4.25 \text{ in.}\right)^{1.5} = 8{,}158 \text{ lbs}$$

$$N_{cbg} = \frac{\left(234 \text{ in.}^2\right)}{(163 \text{ in.}^2)}(1.0)(1.0)(1.0)(1.0)(8{,}158 \text{ lbs}) = 11{,}711 \text{ lbs}$$

The lower bound strength of the adhesive anchor concrete breakout strength in tension per ACI 318-11 Section D.3.3.4.4 with $\phi = 1.0$:

$$0.75\phi N_{cbg} = 0.75(1.0)(11{,}711 \text{ lbs}) = 8{,}783 \text{ lbs}$$

Adhesive anchor bond strength in tension is checked per ACI 318-11 Section D.5.5:

$$N_{ag} = \alpha_{N,seis}\,\frac{A_{Na}}{A_{Nao}}\psi_{ec,Na}\psi_{ed,Na}\psi_{cp,Na}N_{ba} \qquad \text{(ACI 318-11 Eq. D-19)}$$

$\alpha_{N,seis}$ = 0.75, reduction factor per manufacturer's evaluation report

A_{Na} = (21.4 in.)(27.0 in.) = 578 in.^{2}, see Figure 6-11 of this *Guide*. Note how per ACI 318 procedures that the projected area for bond strength in Figure 6-11 is different that the projected area for breakout in Figure 6-10.

$$c_{Na} = 10d_a\sqrt{\frac{\tau_{uncr}}{1100}} \qquad \text{(ACI 318-11 Eq. D-21)}$$

d_a = 0.75 inches, diameter of anchor

τ_{uncr} = 2,220 psi, uncracked concrete, temperature range A per the Bond Strength Design Information table in product evaluation report (c_{Na} is always calculated based on uncracked concrete per ACI 318-11 § RD.5.5.1)

$$c_{Na} = 10(0.75 \text{ in.})\sqrt{\frac{2{,}220 \text{ psi}}{1100}} = 10.7 \text{ inches}$$

A_{Nao} = $(2c_{Na})^2 = (2(10.7 \text{ in.}))^2 = 458$ in.^{2} (ACI 318-11 Eq. D-20)

$\Psi_{ec,Na}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-23)

$\Psi_{ed,Na}$ = 1.0, no edge effects since $c_{a,\min} \geq c_{Na}$ (ACI 318-11 Eq. D-24)

$\Psi_{cp,Na}$ = 1.0, no splitting effects $c_{a,\min} \geq c_{ac}$ (ACI 318-11 Eq. D-26)

N_{ba} = $\lambda_a \tau_{cr} \pi d_a h_{ef}$ (ACI 318-11 Eq. D-22)

λ_a, d_a, h_{ef} as indicated above.

τ_{cr} = 1,185 psi, cracked concrete, temperature range A per the Bond Strength Design Information table in product evaluation report since wall is assumed cracked as discussed above.

$$N_{ba} = (1.0)(1{,}185\text{ psi})(3.14)(0.75\text{ in.})(4.25\text{ in.}) = 11{,}860\text{ lbs}$$

$$N_{ag} = (0.75)\frac{(578\text{ in.}^2)}{(458\text{ in.}^2)}(1.0)(1.0)(1.0)(11{,}860\text{ lbs}) = 11{,}226\text{ lbs}$$

The lower bound strength of the adhesive anchor bond strength in tension per ACI 318-11 Section D.3.3.4.4 with $\phi = 1.0$:

$$0.75\phi\, N_{ag} = 0.75(1.0)(11{,}226\text{ lbs}) = 8{,}420\text{ lbs}$$

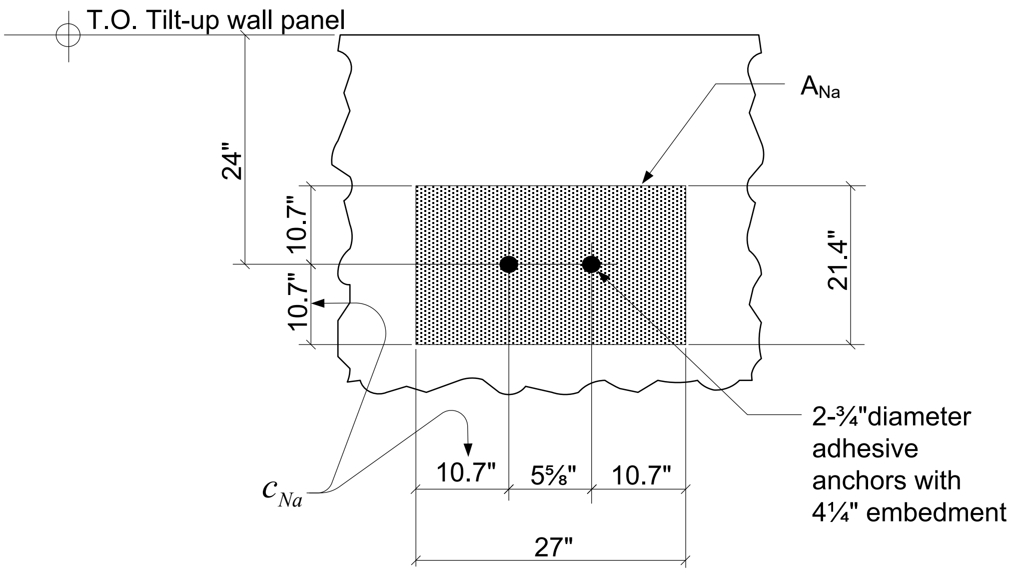


Figure 6-11 Projected influence area of group of adhesive anchors for calculation of bond strength.

The adhesive anchor design results are summarized in Table 6-2 for the acceptance criteria indicated below for force-controlled components with κ per Section 6.6.1 of this *Guide*.

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

The adhesive anchors are adequate for all design checks.

Table 6-2 Adhesive Anchor Design Results

Design Check	Load Q_{UF} (lbs)	Lower Bound Strength Q_{CL} (lbs)	κ	κQ_{CL} (lbs)	Status
Steel Strength	6,300	38,860	1.0	38,860	OK
Concrete Breakout Strength	6,300	8,783	0.75	6,587	OK
Bond Strength	6,300	8,420	0.75	6,315	OK

6.6.2.3 Subpurlin and Subpurlin Fasteners

The development of the out-of-plane wall anchorage forces into the diaphragm will be evaluated as a force-controlled component as noted earlier. The sub-purlins are spaced at 2 feet on center and the 4 feet by 8 feet diaphragm sheathing panels are staggered as shown in Figure 6-1, so every other sub-purlin has diaphragm edge nailing, while the others have field nailing. If the out-of-plane anchorage force is transferred by an additional sub-purlin bay into the diaphragm, then one of the sub-purlins would have field nailing and the other edge nailing on two panel edges due to the diaphragm sheathing stagger. Per Section 6.4.2.2 of this *Guide*, the existing diaphragm nailing is 10d commons with a spacing of 4 inches on center at diaphragm boundaries, 6 inches on center at panel edges, and 12 inches on center field nailing. The configuration of the sub-purlin and the associated nailing is illustrated in Figure 6-12 and Figure 6-15.

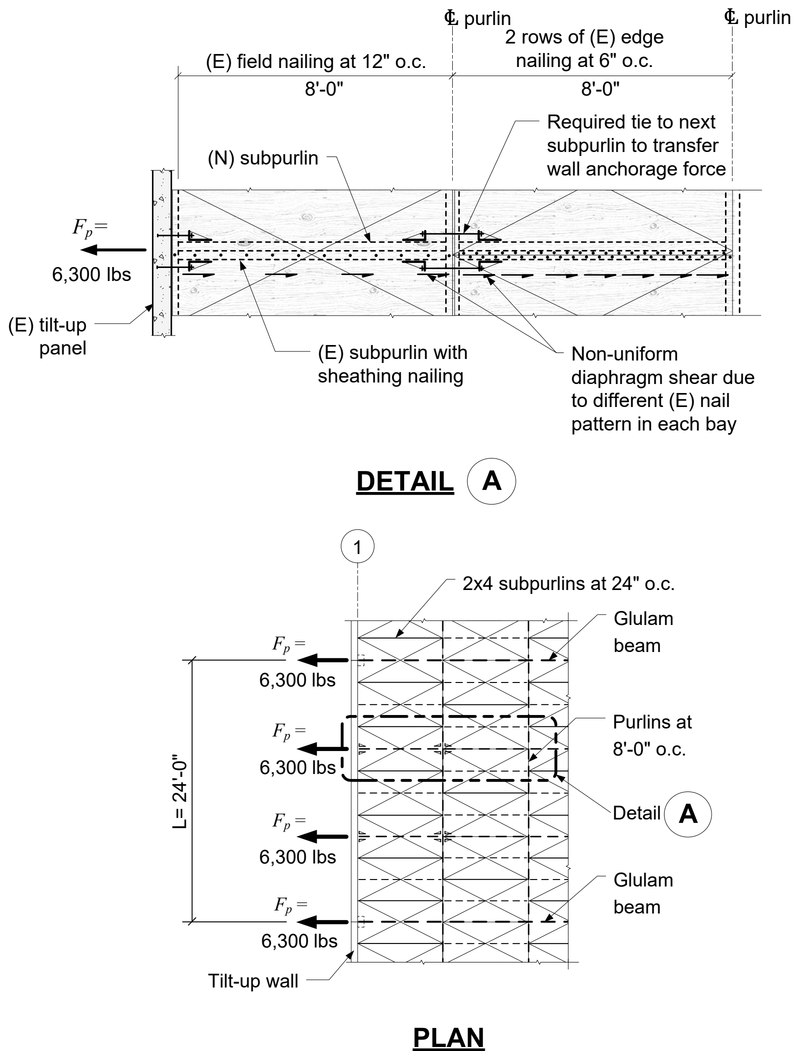


Figure 6-12 Existing diaphragm layout and nailing to existing sub-purlins.

The total number of nails from the diaphragm into the sub-purlins over the two bays is as follows (see Figure 6-15 for additional clarity):

n = (8 ft)(12 in./ft)/(12 in.) + (2)(8 ft)(12 in./ft)/(6in.) = 40 nails

The lower bound component strength of wood connections per ASCE 41-13 § 12.3.2.3.1 is calculated as the load and resistance factor design values in NDS-2012 with a resistance factor, ϕ, taken equal to 1.0.

Z' = per NDS-2012 Table 10.3.1: All adjustment factors = 1.0 except C_{di} = 1.1 and K_F = 3.32. Per NDS-2012 Section N3.3 and Table N3, λ = 1.0 for load combinations with seismic.

Z = 78 lbs per NDS-2012 Table 11R for Douglas Fir-Larch, 15/32" wood structural panel side member, and 10d common nail

$Z' = ZK_F\phi\lambda C_{di}$ = (78 lbs)(3.32)(1.0)(1.0)(1.1) = 285 lbs/nail

Z' = (285 lbs/nail)(40 nails) = 11,400 lbs

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

The knowledge factor is equal to 0.75 per Section 6.6.1.

κQ_{CL} = (0.75)(11,400 lbs) = 8,550 lbs

8,550 lbs > 6,300 lbs, the anchorage loads can be developed 2 bays into the diaphragm.

The connections between the new sistered sub-purlin and existing sub-purlin will be evaluated in Section 6.6.3 of this *Guide*.

The strengthened sub-purlin will be evaluated as a force-controlled component for combined out-of-plane wall anchorage tension loads and gravity loads. The strengthened sub-purlin will not be evaluated for out-of-plane compression loads since when the wall pushes inward on the diaphragm, the load is distributed to all the sub-purlins spaced at 2 feet on-center as opposed to the tension load which is applied through the anchorage to sub-purlins at 8 feet on-center. The loading diagram for sub-purlin is shown in Figure 6-13.

The sub-purlin is evaluated for combined gravity and axial loads as a force-controlled component per ASCE 41-13 § 7.5.2.1.2. The gravity load combination per ASCE 41-13 § 7.2.2 is as follows:

$Q_G = 1.1(Q_D+Q_L+Q_S)$ (ASCE 41-13 Eq. 7-1)

Useful Tip

When evaluating force-controlled actions, the lower bound component strength of wood members and connections is calculated as the load and resistance factor design values in NDS-2012 with a resistance factor, ϕ, taken equal to 1.0 per ASCE 41-13 § 12.3.2.3.1. When evaluating deformation-controlled actions, the expected strength of wood members and connections is 1.5 times this value per ASCE 41-13 § 12.3.2.2.1.

> **Commentary**
>
> The load combinations in ASCE 41-13 § 7.2.2 do not require roof live load to be combined with seismic loads, which is consistent with ASCE 7-10. For other live loads, ASCE 41-13 § 7.2.2 indicates that a portion of the live loads obtained in accordance with ASCE 7-10 be considered in the load combinations included therein.

Q_D = The dead load on the sub-purlin is 9 psf of the 14 psf total roof dead load indicated in Table 6-1 after subtracting the weight of the purlins, glulam beams, and 1.4 psf of the miscellaneous dead loads

Q_L = 0, roof live load need not be applied simultaneously with seismic per ASCE 41-13 § 7.2.2

Q_S = 0, no snow load

Q_G = 1.1(9 psf) = 9.9 psf

w_G = (9.9 psf)(2 ft) = 19.8 lbs/ft

$M_G = wL^2/8$ = (19.8 lbs/ft)(8 ft)^2/8 = 158 ft-lbs

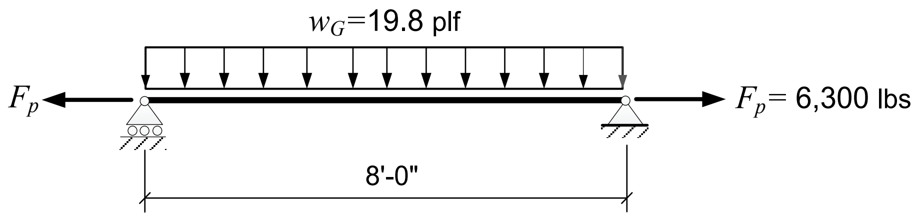


Figure 6-13 Sub-purlin free-body diagram for combined tension and bending loads.

The out-of-plane anchorage load, F_p, is 6,300 lbs per Section 6.6.2 of this *Guide*.

The lower-bound strength of the member is determined per ASCE 41-13 § 12.3.2.3.1 using NDS-2012 with strength reduction factor equal to 1.0. Combined bending and tension is checked per NDS-2012 Section 3.9.1:

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \le 1.0 \text{ , and} \qquad \text{(NDS-2012 Eq. 3.9-1)}$$

$$\frac{f_b - f_t}{F_b^{**}} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-2)}$$

The properties of each 2×4 sub-purlin are as follows:

DF-L No. 1, S_x = 3.06 in.^{3}, A = 5.25 in.^{2}

The moment and axial load will be shared equally between the existing and sistered sub-purlins as illustrated in Figure 6-12. The bending stress in the sub-purlins are as follows:

$f_b = M_G/S_x$ = (158 ft-lbs)(12 in./ft)/((3.06 in.^{3})(2)) = 310 psi

The axial tension stress in the sub-purlins are as follows:

$f_t = F_p/A$ = (6,300 lbs)/((5.25 in.^{2})(2)) = 600 psi

The lower bound reference bending design value, F_b^*, is obtained by multiplying F_b by all applicable adjustment factors, except C_L, per NDS-2012 Section 3.9.1 and Table 4.3.1 as follows:

$F_b^* = F_b C_M C_t C_F C_{fu} C_i C_r K_F \phi \lambda$

F_b = 1,000 psi for DF-L No. 1

C_F = 1.5, C_r = 1.15 per NDS-2012 Supplement Table 4A

C_M, C_t, C_{fu}, C_i = 1.0

K_F = 2.54 per NDS-2012 Table 4.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

λ = 1.0 per NDS-2012 Section N3.3 and Table N3 for load combinations with seismic

F_b^* = (1000 psi)(1.0)(1.0)(1.5)(1.0)(1.0)(1.15)(2.54)(1.0)(1.0)
= 4,382 psi

The lower bound reference bending design value, F_b^{**}, is obtained by multiplying F_b by all applicable adjustment factors, except C_V, per NDS-2012 Section 3.9.1 and Table 4.3.1 as follows:

$F_b^{**} = F_b C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi \lambda$

C_L = 1.0, top of beam is continuously braced per NDS-2012 Section 3.3.3

F_b^{**} = (1000 psi)(1.0)(1.0)(1.0)(1.5)(1.0)(1.0)(1.15)(2.54)(1.0)(1.0)
= 4,382 psi

The lower bound adjusted design strength for tension parallel to grain, per NDS-2012 Table 4.3.1 is as follows:

$F_t' = F_t C_M C_t C_F C_i K_F \phi \lambda$

F_t = 675 psi for DF-L No. 1

C_F = 1.5 per NDS Supplement Table 4A

C_M, C_t, C_i = 1.0

K_F = 2.70 per NDS-2012 Table 4.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

λ = 1.0 per NDS-212 Section N3.3 and Table N3 for load combinations with seismic

F_t' = (675 psi)(1.0)(1.0)(1.5)(1.0)(2.70)(1.0)(1.0) = 2,734 psi

When evaluating the unity check for combined bending and axial tension, the denominator is the lower bound strength, Q_{CL}, and needs to be multiplied by the knowledge factor, κ, to be consistent with the acceptance criteria for force-controlled components as shown below:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = 1.0 per Section 6.6.1 of this *Guide*

Q_{CL} = F_b^*, F_b^{**}, F_t (lower bound strength)

Q_{UF} = f_b, f_t

Combined bending and tension is checked per NDS-2012 Section 3.9.1:

$$\frac{f_t}{\kappa F_t'} + \frac{f_b}{\kappa F_b^*} \leq 1.0 \text{, and} \qquad \text{(NDS-2012 Eq. 3.9-1)}$$

$$\frac{600 \text{ psi}}{1.0(2{,}734 \text{ psi})} + \frac{310 \text{ psi}}{1.0(4{,}382 \text{ psi})} = 0.29 < 1.0 \quad \text{OK}$$

$$\frac{f_b - f_t}{\kappa F_b^{**}} \leq 1.0 \qquad \text{(NDS-2012 Eq. 3.9-2)}$$

$$\frac{310 \text{ psi} - 600 \text{ psi}}{1.0(4{,}382 \text{ psi})} = \text{-}0.07 < 1.0\text{, O.K.}$$

Therefore, the strengthened sub-purlins are adequate.

6.6.3 Subdiaphragm Analysis in East-West Direction Seismic Loads

This section will evaluate the subdiaphragm and subdiaphragm chords based on the “noncompliant” Tier 1 checklist item “Wall Anchorage” discussed in Section 6.5.2. Diaphragm crossties based on the “noncompliant” Tier 1 checklist item “Crossties in Flexible Diaphragms” will also be evaluated. The “Crossties in Flexible Diaphragms” checklist item directs the user to ASCE 41-13 § 5.6.1.2 for the Tier 2 procedure, which requires the crossties to be evaluated for the out-of-plane forces in accordance with ASCE 41-13 § 7.2.11. There is no direct reference to the continuity forces in ASCE 41-13 § 7.2.10 for this noncompliant item when using the Tier 2 procedure, and therefore continuity tie requirements will not be assessed.

As indicated in Section 6.6.2 of this *Guide*, this example will evaluate the wall anchorage in east-west direction only. A separate analysis is required for the north-south direction, however, this example does not repeat the similar effort required to do so.

The out-of-plane wall anchorage forces in ASCE 41-13 § 7.2.11.1 are required to be developed into the diaphragm. For large buildings with multiple bays of girders, purlins, and sub-purlins, the anchorage force is developed into subdiaphragms and transferred into continuous diaphragm ties that are continuous the entire depth of the diaphragm. For out-of-plane loading in the east-west direction for this example, the continuous diaphragm ties are the glulam beams. Each subdiaphagm is analyzed to resist the wall anchorage forces and requires the subdiaphragm shear, subdiaphragm connection to the glulam beam continuous tie, and the chords to be evaluated.

ASCE 41-13 § 7.2.11.1 requires that subdiaphragms have length-to-depth ratio not exceeding 3 to 1. The length of the subdiaphragm is the distance between glulam beam continuous ties. The depth of the subdiaphragm will be evaluated initially as two purlin bays as that was determined previously and depth required to develop the anchorage forces based on the existing nails fastened to the sub-purlins. Figure 6-14 illustrates the configuration and loading on the subdiaphragm for east-west out-of-plane wall anchorage loads.

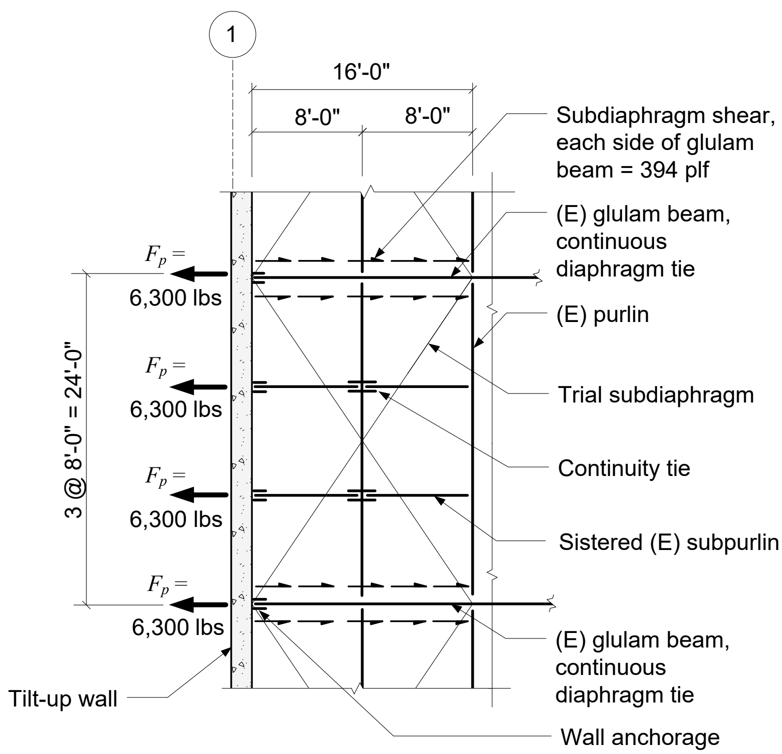


Figure 6-14 Wall anchorage loads on the subdiaphagm for east-west direction.

The wall anchors provided between the glulam beam girders and the pilasters will deliver the wall anchorage forces directly into the glulam beams and this load need not be included in the subdiaphagm analysis. The sub-purlin wall anchor force calculated previously is as follows:

$F_{p,BSE\text{-}1E}$ = 6,300 lbs per anchor at 8 ft o.c.

The length-to-depth ratio of the subdiaphragm is as follows:

24 ft/16 ft = 1.5 < 3.0

The subdiaphragm is evaluated considering only the wall anchorage forces and does not include the seismic inertial loads from the mass of the diaphragm and framing. The subdiaphagm shear on each side of the glulam beam cross tie is the wall anchorage forces in that subdiaphragm bay divided by the depth of each side of the subdiaphragm as follows:

v_{sub} = 2(6,300 lbs)/(16 ft + 16 ft) = 394 plf

Commentary

The subdiaphragms are not evaluated in the north-south direction in the example; however, it should be noted that the nominal unit shear capacity of the diaphragm would be evaluated based on loading Case 4 in SDPWS-2008 Table 4.2A and would result in a reduced diaphragm capacity of v_n = 640 plf unless boundary nailing is applied to all continuous panel joints.

As noted previously, the development of the out-of-plane wall anchorage forces into the diaphragm are evaluated as a force-controlled component. Per Section 6.4.2.2 of this *Guide*, the existing diaphragm is 15/32" wood structural panel sheathing, Structural I, blocked, with 10d commons at a spacing of 4 inches on center at diaphragm boundaries, 6 inches on center at panel edges, and 12 inches on center field nailing. The lower bound component strength of wood diaphragms per ASCE 41-13 § 12.5.3.6.2 and § 12.3.2.3.1 is the load and resistance factor design values in SDPWS-2008, *Special Design Provisions for Wind and Seismic Standard with Commentary* (AWC, 2008), Table 4.2A with a resistance factor, ϕ, taken equal to 1.0. The nominal unit shear capacity of the wood diaphragm per SDPWS-2008 Table 4.2A with a minimum nominal framing width of nailed faces as 2 inches and loading case 2 is as follows:

Q_{CL} = 850 plf

The diaphragm is evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

The knowledge factor is equal to 0.75 per Section 6.6.1 of this *Guide.*

κQ_{CL} = (0.75)(850 plf) = 638 plf

638 plf > 394 plf; therefore, the subdiaphagm has adequate shear capacity.

The subdiaphragm shear calculation above evaluates the subdiaphragm shear adjacent to each side of the glulam beam continuous tie. However, it does not evaluate the adequacy of the subdiaphragm force transfer through the diaphragm nailing into the top of glulam beam considering loading from two adjacent subdiaphragms. As noted above, the anchorage forces into the glulam beam girders at the pilasters are delivered directly into the glulam beams and this load need not be included in the subdiaphagm analysis. The total subdiaphragm force delivered to the glulam beam from adjacent subdiaphragms is as follows:

Q_{UF} = 2(6,300 lbs) = 12,600 lbs

Due to the staggered diaphragm sheathing layout as shown in Figure 6-1, the existing nailing into the top of the glulam beam will alternate such that every other sheathing panel will have diaphragm edge nailing, followed by field nailing. The fastener spacing will be similar to that illustrated in Figure 6-12.

The total number of nails from the diaphragm into the glulam beam over the two bays is as follows:

n = (8 ft)(12 in./ft)/(12 in.) + (2)(8 ft)(12 in./ft)/(6 in.) = 40 nails

The lower bound component strength of the nails as determined in Section 6.6.2 of this *Guide* is as follows:

Z' = 285 lbs/nail

= (285 lbs/nail)(40 nails) = 11,400 lbs

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

The knowledge factor is equal to 0.75 per Section 6.6.1 of this *Guide*.

κQ_{CL} = (0.75)(11,400 lbs) = 8,550 lbs

8,550 lbs < 12,600 lbs; therefore, the nailing into the glulam beam over the subdiaphragm depth of two bays is inadequate.

The subdiaphragm is evaluated by increasing the depth to three bays. The worst case sheathing layout on the glulam beam would be if the first and third bay had field nailing and the second bay had edge nailing. The total number of nails from the diaphragm into the glulam beam over three bays is as follows:

n = (8 ft)(12 in./ft)/(12 in.) + (2)(8 ft)(12 in./ft)/(6 in.) + (8 ft)(12 in./ft)/(12 in.) = 48 nails

Z' = (285 lbs/nail)(48 nails) = 13,680 lbs

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

κQ_{CL} = (0.75)(13,680 lbs) = 10,260 lbs

10,260 lbs < 12,600 lbs; therefore, the nailing into the glulam beam over the subdiaphragm depth of three bays is inadequate.

The subdiaphragm is evaluated by increasing the depth to four bays. The sheathing layout on the glulam beam alternates field and edge nailing in each bay. The total number of nails from the diaphragm into the glulam beam over four bays is as follows:

n = 2[(8 ft)(12 in./ft)/(12 in.)] + 2[(2)(8 ft)(12 in./ft)/(6 in.)] = 80 nails

Z' = (285lbs/nail)(80 nails) = 22,800 lbs

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

κQ_{CL} = (0.75)(22,800 lbs) = 17,100 lbs

17,800 lbs > 12,600 lbs; therefore, the nailing into the glulam beam over the subdiaphragm depth of four bays in adequate.

Next, the connection device at each subdiaphragm continuity tie is evaluated. The load distribution along each continuity ties is assumed to linearly reduce from a maximum at the tilt-up wall panel to zero at the end of the subdiaphragm at a rate corresponding to the diaphragm nail spacing to the sub-purlin in each bay. There are two possible diaphragm nail spacing patterns over the four-bay subdiaphragm depending on which sub-purlin has edge nailing or field nailing applied. Nail Pattern I is illustrated in Figure 6-15 and represents the condition where the first and third bays have field nailing and the second and fourth bays have edge nailing for a total of 80 nails. Nail Pattern II is illustrated in Figure 6-16 and represents the opposite condition where the first and third bays have edge nailing and the second and fourth bays have field nailing for a total of 80 nails. The load from the continuity tie is dissipated more quickly into the subdiaphragm in the bays with edge nailing.

Nail Pattern I:

F_p @ Point A = 6,300 lbs per Section 6.6.2 of this *Guide*

F_p @ Point B = (6,300 lbs)(72 nails/80 nails) = 5,670 lbs

F_p @ Point C = (6,300 lbs)(40 nails/80 nails) = 3,150 lbs

F_p @ Point D = (6,300 lbs)(32 nails/80 nails) = 2,520 lbs

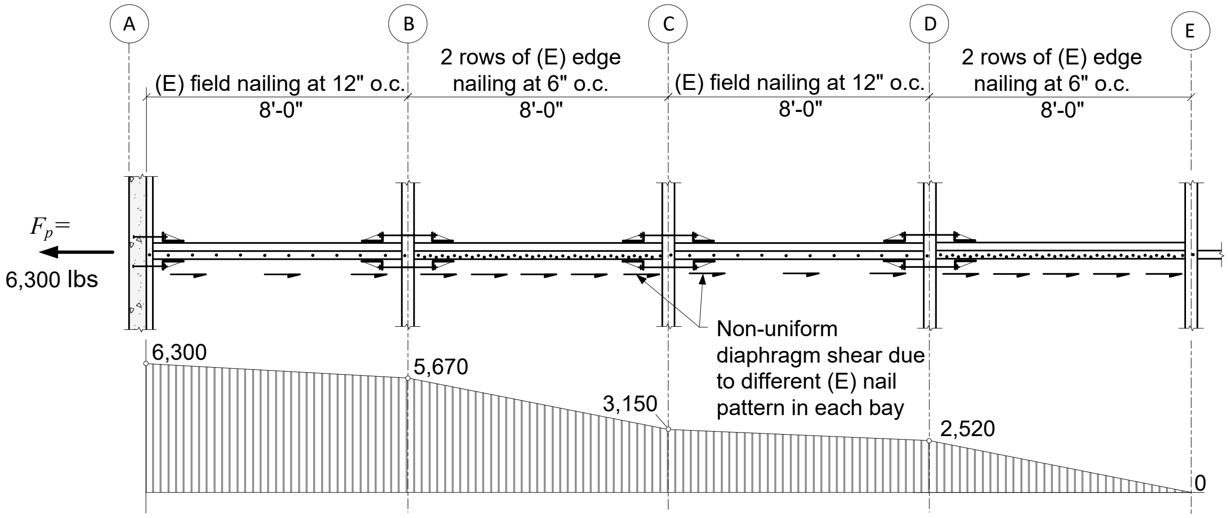


Figure 6-15 Sub-purlin tie load (lbs) for existing Nail Pattern I.

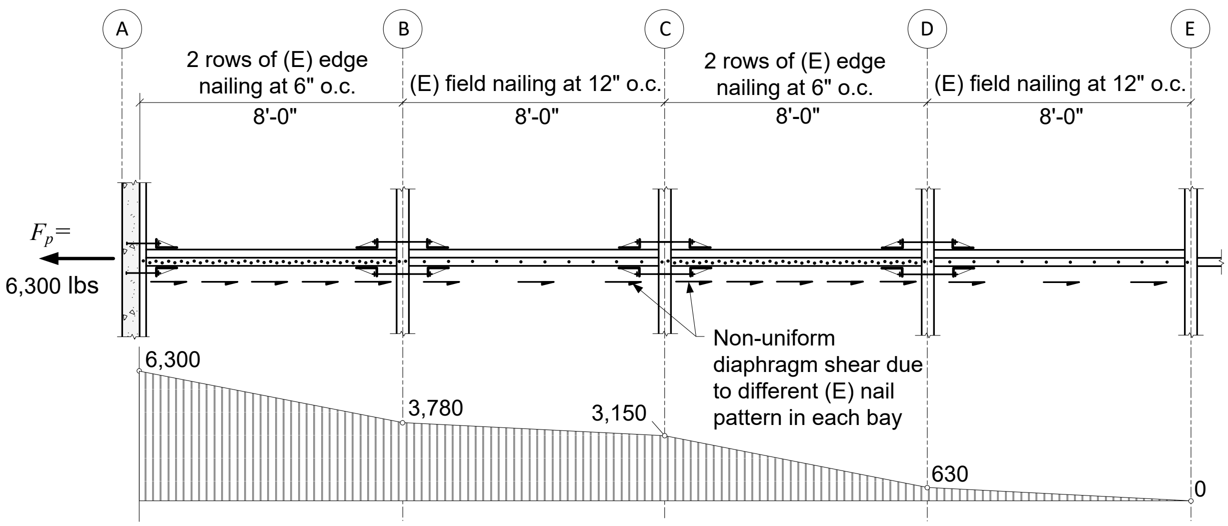


Figure 6-16 Sub-purlin tie load (lbs) for existing Nail Pattern II.

Nail Pattern II:

F_p @ Point A = 6,300 lbs per Section 6.6.2

F_p @ Point B = (6,300 lbs)(48 nails/80 nails) = 3,780 lbs

F_p @ Point C = (6,300 lbs)(40 nails/80 nails) = 3,150 lbs

F_p @ Point D = (6,300 lbs)(8 nails/80 nails) = 630 lbs

As indicated in Section 6.6.2 of this *Guide*, the continuity tie anchorage devices are placed in symmetric pairs on each sub-purlin to prevent an eccentric weak axis moment and the sistered rafter were added to provide adequate embedment for the wall anchorage hardware fasteners. For this retrofit, the same continuity tie anchorage devices will be specified for each bay at Points B, C and D as those for the wall anchorage devices evaluated in Section 6.6.2 at Point A shown in Figure 6-15 and Figure 6-16.

The fasteners that connect the new sistered sub-purlin to the existing sub-purlin are required to transfer a portion of the out-of-plane wall anchorage loads from the wall anchorage devices into the existing sub-purlin since only the existing sub-purlin is connected to the subdiaphragm. The worst condition occurs for Nail Pattern I between Points B and C and between Points D and E and Nail Pattern II between Points A and B and between C and D as shown in Figure 6-15 and Figure 6-16 since these bays have the largest drop in continuity tie forces that are unloaded into the subdiaphragm. Figure 6-17 illustrates a free-body diagram of this load transfer. The net load to be transferred through the fasteners connecting the new and existing sub-purlin is the difference between the tie forces on each end of the new subpurlin.

Q_{UF} = 2,835 lbs – 1,575 lbs = 1,260 lbs

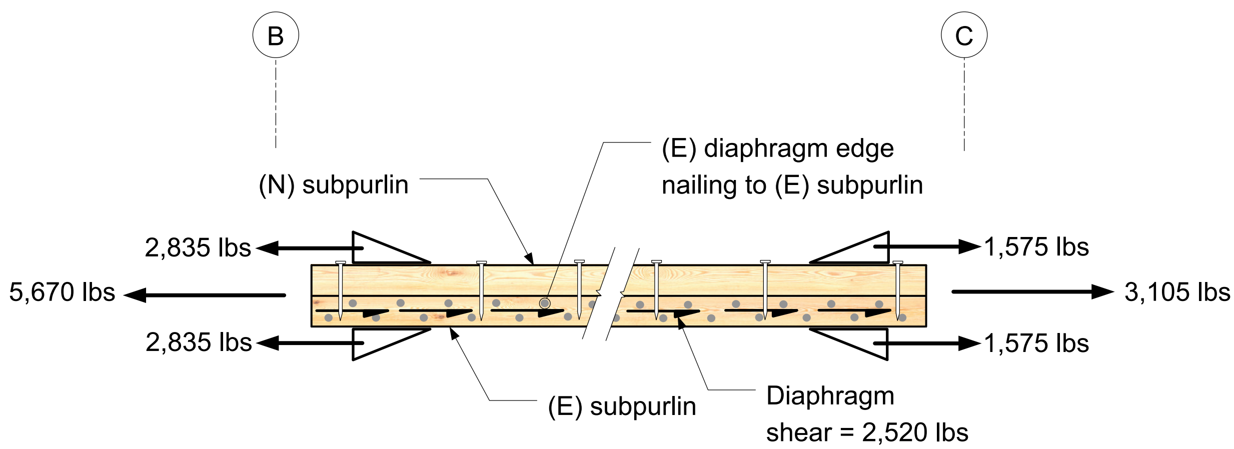


Figure 6-17 Load transfer free-body diagram between new and existing sub-purlin.

The fasteners will be 10d common nails since their length of 3 inches matches the total thickness of the two 2× subpurlins and they will comply with the minimum 10D (1.5 in./0.148 in. = 10.1) penetration into the main member with no reduction factor. The lower bound component strength of wood connections per ASCE 41-13 § 12.3.2.3.1 is calculated as the load and resistance factor design values in NDS-2012 with a resistance factor, ϕ, taken equal to 1.0.

Z' = per NDS-2012 Table 10.3.1: All adjustment factors = 1.0 except K_F = 3.32. Per NDS-2012 Section N3.3 and Table N3, λ = 1.0 for load combinations with seismic.

where:

Z = 118 lbs per NDS-2012 Table 11N for Douglas Fir-Larch, 1-$^{1}/_{2}$" side member, and 10d common nail (0.148")

$Z' = ZC_MC_tC_gC_\Delta C_{eg}C_{di}C_{tn}K_F\phi\lambda C_{di}$
$= (118 \text{ lbs})(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(3.32)(1.0)(1.0)$
$= 392$ lbs/nail

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

The knowledge factor is equal to 1.0 per Section 6.6.1:

$\kappa Q_{CL} = (1.0)(392 \text{ lbs}) = 392$ lbs

The 10d nail spacing, s, over 8 feet long bay (minus half the purlin and ledger thickness) is as follows:

$s = L_{\text{bay}}\kappa Q_{CL}/Q_{UF} = [(8 \text{ ft})(12 \text{ in./ft}) - (3.5 \text{ in.})](392 \text{ lbs})/(1{,}260 \text{ lbs})$
$= 29$ inches

Therefore, provide 10d at 12 inches o.c. staggered.

6.6.3.1 Evaluate Subdiaphragm Chord

The subdiaphragm is required to be evaluated as an independent diaphragm with its own chord. In this case, the chords are the tilt-up panel and the fourth purlin at the interior boundary of the subdiaphragm, as illustrated in Figure 6-18. The tilt-up panel is continuous across the full width of the subdiaphragm and there is no need to evaluate it as the strength of the concrete and horizontal reinforcing bars of the panel would easily comply with the chord demands. The purlin chord will be evaluated for the condition when the wall anchorage forces are pulling away from the building as this is typically the primary concern. When the wall pushes into the building, the load is transferred in bearing and may engage more of the diaphragm. In any case, the purlin acting as a chord in compression as a result of the forces pulling away from the building causes the more severe loading on the purlin. More discussion on this topic is provided in *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up Buildings and Other Rigid Wall/Flexible Diaphragm Structures* (SEAONC, 2001), Section 6.3.9.

The purlin is evaluated for combined gravity and axial as a force-controlled component per ASCE 41-13 § 7.5.2.1.2. The axial force in the purlin is determined based on the subdiaphragm being idealized as a simply supported diaphragm that spans from each continuous cross-tie. In this example, the axial load is determined based on uniform out-of-plane wall anchorage loads, but it could be refined into a series of point loads at each wall anchor.

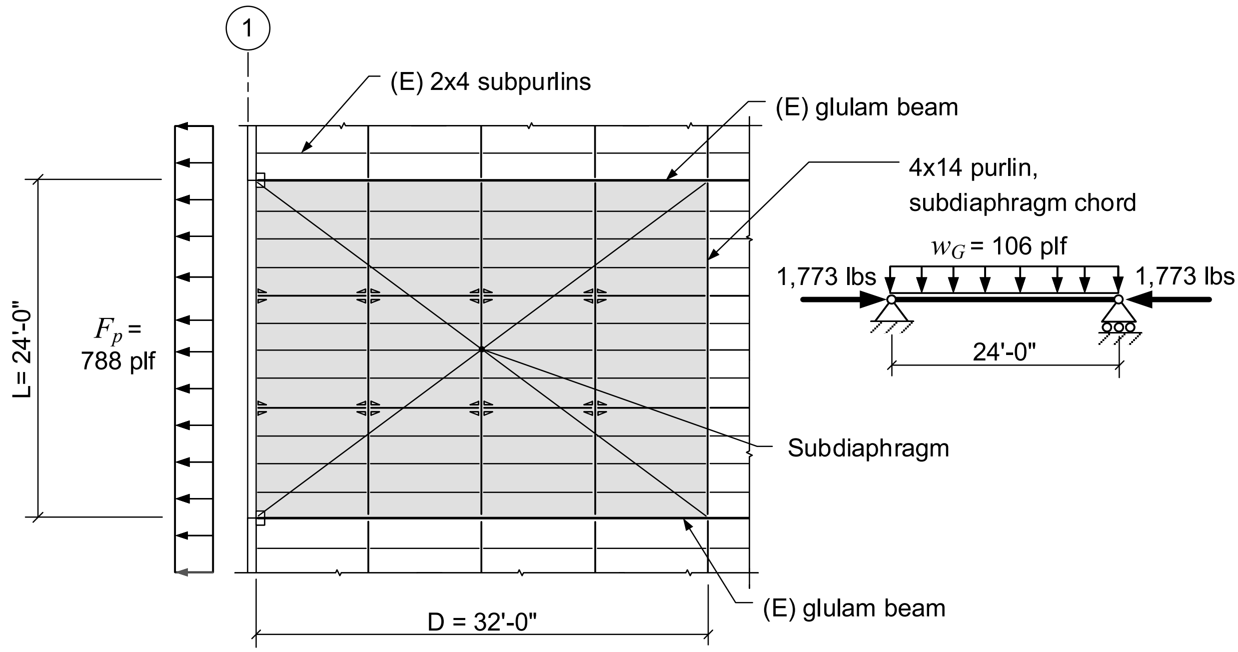


Figure 6-18 Subdiaphragm chord evaluation.

Axial chord force = $Q_{UD} = F_pL^2/8D$ = (788 lbs/ft)(24 ft)^2/[8(32 ft)] = 1,773 lbs

where:

F_p = 788 lbs/ft from Section 6.6.2 of this *Guide*

L = 24 ft, span

D = 32 ft, depth

The gravity load combination per ASCE 41-13 § 7.2.2 is as follows:

$Q_G = 1.1(Q_D+Q_L+Q_S)$ (ASCE 41-13 Eq. 7-1)

Q_D = The dead load on the purlin is 12 psf of the 14 psf total roof dead load indicated in Table 6-1 after subtracting the weight of the glulam beam dead load.

Q_L = 0, roof live load need not be applied simultaneously with seismic per ASCE 41-13 § 7.2.2.

Q_G = 1.1(12 psf) = 13.2 psf

w_G = (13.2 psf)(8 ft) = 106 lbs/ft

$M_G = wL^2/8$ = (106 lbs/ft)(24 ft)^2/8 = 7,632 ft-lbs

The lower-bound strength of the member is determined per ASCE 41-13 § 12.3.2.3.1 using NDS-2012 with strength reduction factor equal to 1.0. Combined bending and compression is checked per NDS-2012 Section 3.9.2.

$$\left[\frac{f_c}{F_c'}\right]^2 + \frac{f_b}{F_b'\left[1-\left(\frac{f_c}{F_{cE}}\right)\right]} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

The properties of the 4×14 sub-purlin are as follows:

DF-L Select Structural, S_x = 102.41 in.^{3}, A = 46.38 in.^{2}

The bending stress in the purlin is as follows:

f_b = M_G/S_x = (7,632 ft-lbs)(12 in./ft)/(102.41 in.^{3}) = 894 psi

The axial compressive stress in the purlin is as follows:

f_c = F_p/A = (1,773 lbs)/(46.38 in.^{2}) = 38 psi

The lower bound adjusted bending design strength per NDS-2012 Table 4.3.1 is as follows:

$F_b' = F_b C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi \lambda$

where:

F_b = 1,500 psi for DF-L Select Structural

C_F = 1.0, per NDS-2012 Supplement Table 4A

C_L = 1.0, top of beam is continuously braced

C_M, C_t, C_{fu}, C_i, C_r = 1.0

K_F = 2.54 per NDS-2012 Table 4.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

λ = 1.0 per NDS-2012 Section N3.3 and Table N3 for load combinations with seismic

F_b' = (1500 psi)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(2.54)(1.0)(1.0)
= 3,810 psi

The lower bound adjusted design strength for compression parallel to grain, per NDS-2012 Table 4.3.1 is as follows:

$F_c' = F_c C_M C_t C_F C_i C_P K_F \phi \lambda$

where:

F_c = 1,700 psi for DF-L Select Structural

C_F = 0.9 per NDS-2012 Supplement Table 4A

C_M, C_t, C_i = 1.0

$$C_P = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad \text{(NDS-12 Eq. 3.7-1)}$$

where:

c = 0.8, sawn lumber

$$F_{cE} = \frac{0.822E'_{\min}}{\left(l_e/d\right)^2}$$

where:

$(l_e/d)_x$ = (24 ft)(12 in./ft)/(13.25 in.) = 21.7, most critical is x-axis

$(l_e/d)_y$ = (0 ft)(12 in./ft)/(3.5 in.) = 0 (braced by diaphragm)

$E'_{\min} = E_{\min}C_MC_tC_iC_TK_F\phi$

where:

$E_{\min}$ = 690,000 psi for DF-L Select Structural

C_M, C_t, C_i, C_T = 1.0

K_F = 1.76 per NDS-2012 Table 4.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

$E'_{\min}$ = (690,000 psi)(1.0)(1.0)(1.0)(1.0)(1.76)(1.0)
= 1,214,400 psi

$$F_{cE} = \frac{0.822(1,214,400\text{ psi})}{\left(21.7\right)^2} = 2,120\text{ psi}$$

$F_c^* = F_cC_MC_tC_FC_iK_F\phi\lambda$ (all adjustment factors except C_P)

F_c^* = (1,700 psi)(1.0)(1.0)(0.9)(1.0)(2.40)(1.0)(1.0) = 3,672 psi

$$C_P = \frac{1+\left(2,120\text{ psi}/3,672\text{ psi}\right)}{2\left(0.8\right)} - \sqrt{\left[\frac{1+\left(2,120\text{ psi}/3,672\text{ psi}\right)}{2\left(0.8\right)}\right]^2 - \frac{\left(2,120\text{ psi}/3,672\text{ psi}\right)}{\left(0.8\right)}}$$

= 0.49

F'_c = (1,700 psi)(1.0)(1.0)(0.9)(1.0)(0.49)(2.40)(1.0)(1.0) = 1,799 psi

When evaluating the unity check for combined bending and axial compression, the denominator is the lower bound strength, Q_{CL}, and needs to be multiplied by the knowledge factor, κ, to be consistent with the acceptance criteria for force-controlled components as shown below:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = 1.0 per Section 6.6.1

$Q_{CL} = F_b'$, F_c' (lower bound strength)

$Q_{UF} = f_b, f_c$

Combined bending and compression is checked per NDS-2012 Section 3.9.2.

$$\left[\frac{f_c}{\kappa F_c'}\right]^2 + \frac{f_b}{\kappa F_b'\left[1-\left(\dfrac{f_c}{F_{cE}}\right)\right]} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

$$\left[\frac{38\text{ psi}}{1.0(1{,}799\text{ psi})}\right]^2 + \frac{894\text{ psi}}{1.0(3{,}810\text{ psi})\left[1-\left(\dfrac{38\text{ psi}}{2{,}120\text{ psi}}\right)\right]} = 0.24 < 1.0\text{, O.K.}$$

Therefore, the existing purlins are adequate to resist the subdiaphragm chord forces.

6.6.3.2 Evaluate Glulam Beam Continuous Cross-Tie

The glulam beam is a continuous cross-tie for the full depth of the building diaphragm. The purpose of this tie is to collect the subdiaphragm wall anchorage loads and dissipate them throughout the entire depth of the diaphragm using the load from ASCE 41-13 § 7.2.11.1. The continuous cross-tie is also used to provide continuity and tie the entire building together; however, as discussed at the beginning of this section, there is no requirement to evaluate the continuity forces in ASCE 41-13 § 7.2.10 for this noncompliant item when using the Tier 2 procedure.

The axial load in the glulam beam cross-tie varies linearly from a value equal to the pilaster wall anchorage force, $F_{p,GLB}$, which is directly connected to the glulam beam, and increases to a maximum at the end of the subdiaphragm. The load in the cross-tie then dissipates into the body of the diaphragm from a maximum at the end of the subdiaphragm to a value of zero at the opposite end of the full diaphragm depth. This load path is illustrated in Figure 6-19. More discussion on this topic is provided in *Guidelines for Seismic*

Evaluation and Rehabilitation of Tilt-up Buildings and Other Rigid Wall/Flexible Diaphragm Structures (SEAONC, 2001), Section 6.3.3.

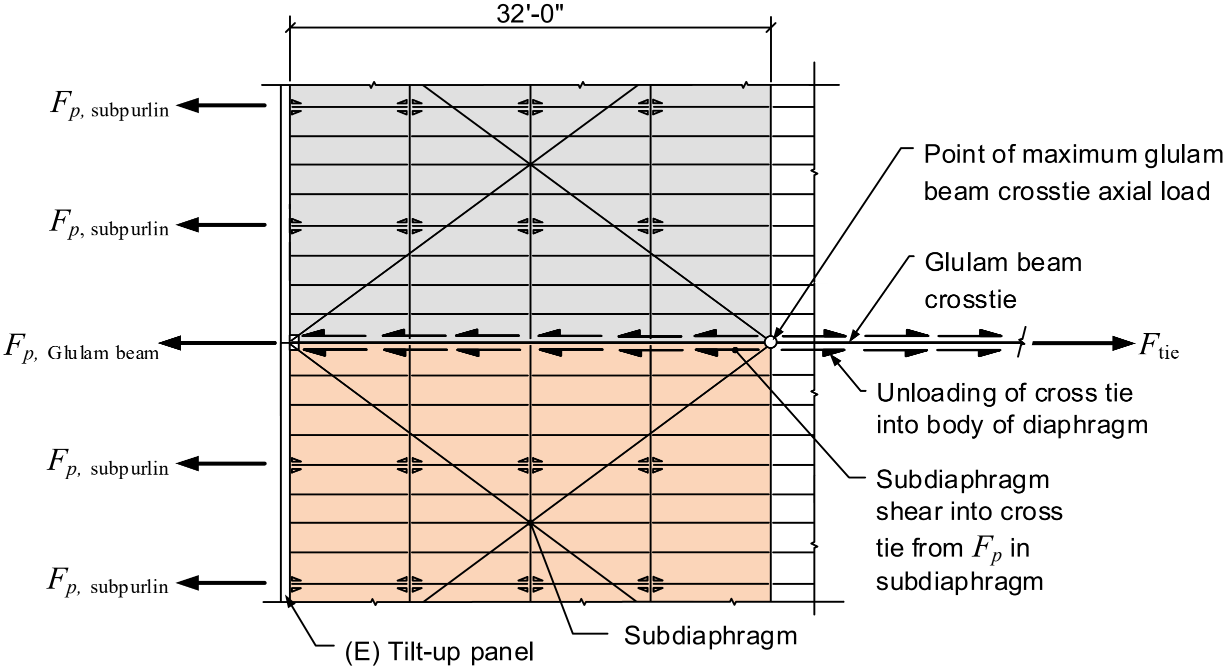


Figure 6-19 Cross-tie loading diaphragm.

The wall anchorage force at the pilaster, $F_{p,GLB}$, is based on the direct connection of the glulam beam to the pilaster. As noted previously, this example ignores pilaster effects from two-way tilt-up panel flexural action, which should be considered as it causes an increase in the glulam beam to tilt-up panel out-of-plane anchorage loads. This load is not needed for the cross-tie analysis since the maximum cross-tie load will occur at the end of the subdiaphragm. The axial load in the cross-tie at the end of the subdiaphragm depth, $F_{\mathrm{tie,sub}}$, will be the full tributary out-of-plane anchorage forces, F_p, to the cross-tie line, which is equal to 788 lbs/ft per Figure 6-18.

$$F_{\mathrm{tie,sub}} = (24\text{ ft})(788\text{ lbs/ft}) = 18{,}900\text{ lbs}$$

The axial load in the cross-tie at the midpoint of the diaphragm at Gridline 3, $F_{\mathrm{tie,mid}}$, is calculated assuming the maximum cross-tie load, $F_{\mathrm{tie,mid}}$, will dissipate linearly along the remainder of the diaphragm length to a value of zero as illustrated in Figure 6-20.

$$F_{\mathrm{tie,mid}} = (18{,}900\text{ lbs})(128\text{ ft})/(256\text{ ft} - 32\text{ ft}) = 10{,}800\text{ lbs}$$

The axial load in the cross-tie at the most heavily loaded splice ($F_{\mathrm{tie,splice}}$) is located 14 feet west of Gridline 2. It is calculated using similar triangles as illustrated in Figure 6-21. The cross-tie forces at various points of the diaphragm are shown in Figure 6-22.

$F_{\text{tie,splice}}$ = (18,900 lbs)(256 ft – 64 ft + 14 ft)/(256 ft – 32 ft)
= 17,400 lbs

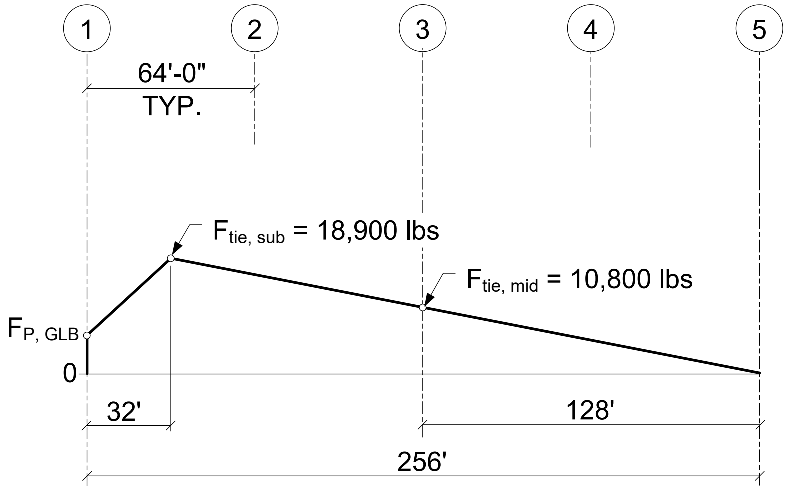


Figure 6-20 Cross-tie loading at midpoint of diaphragm.

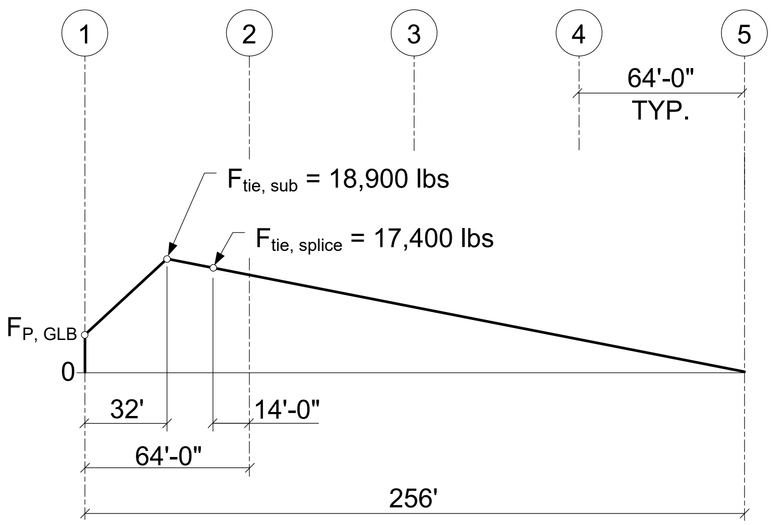


Figure 6-21 Cross-tie loading at splice.

This example will evaluate the glulam beam with a simple span between Gridlines 1 and 2. The continuous glulam beam between Gridlines 4 and 5 and the corresponding cantilever should also be evaluated, but is not part of this example. The collector evaluation in Section 6.6.5 of this *Guide* will evaluate the continuous glulam beam for combined flexure and collector loads. The glulam beam will be evaluated for combined gravity and axial as force controlled component per ASCE 41-13 § 7.5.2.1.2. The highest axial load per Figure 6-22 is 18,900 lbs.

Q_{UF} = 18,900 lbs

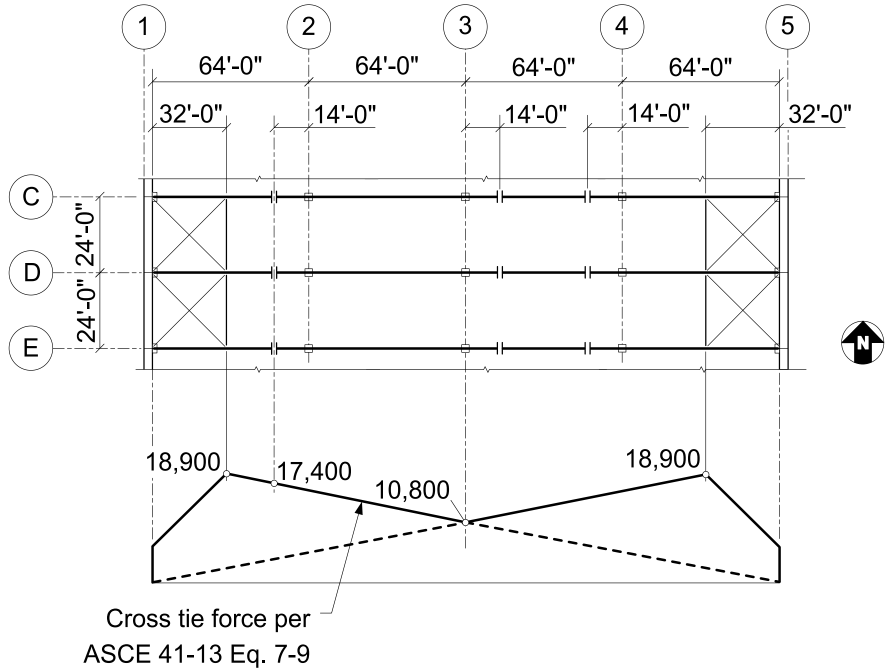


Figure 6-22 Cross-tie layout and loading diagram (lbs).

The gravity load combination per ASCE 41-13 § 7.2.2 is as follows:

$$Q_G = 1.1(Q_D+Q_L+Q_S) \qquad \text{(ASCE 41-13 Eq. 7-1)}$$

where:

Q_D = The dead load on the girder is 14 psf per Table 6-1

Q_L = 0, roof live load need not be applied simultaneously with seismic per ASCE 41-13 § 7.2.2

Q_S = 0, no snow load

Q_G = 1.1(14 psf) = 15.4 psf

w_G = (15.4 psf)(24 ft) = 370 lbs/ft

$M_G = wL^2/8$ = (370 lbs/ft)(50 ft)^2/8 = 115,600 ft-lbs

The lower-bound strength of the member is determined per ASCE 41-13 § 12.3.2.3.1 using NDS-2012 with strength reduction factor equal to 1.0. Combined bending and compression is checked per NDS-2012 Section 3.9.2.

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_b}{F'_b\left[1-\left(\frac{f_c}{F_{cE}}\right)\right]} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

The properties of the 6-3/4 × 31-1/2 glulam beam are as follows:

24F-V4 DF/DF, S_x = 1116 in.^{3}, A = 212.6 in.^{2}

Glued laminated beams manufactured prior to 1970 were not required to have the more stringent grading requirements for the tension laminations that became effective January 1, 1970, and as a result, AITC Technical Note 26, *Design Values of Structural Glued Laminated Timber in Existing Structures* (AITC, 2007), recommends that the bending design values for these beams be reduced 25% for beams deeper than 15 inches, unless the tension laminations are field graded to the newer requirements. For deeper beams, this inspection can be difficult as there is more than one tension lamination laid-up and the second tension lamination cannot be visually graded as only two sides of the lamination are visible. Similarly, this inspection is difficult for cantilever beams where sheathing covers up the top tension laminations. For this example, the bending design value is reduced 25% in accordance with these recommendations by applying the additional 0.75 factor to the bending design values.

The bending stress in the glulam beam is as follows:

$$f_b = M_G/S_x = (115{,}600 \text{ ft-lbs})(12 \text{ in./ft})/(1{,}116 \text{ in.}^3) = 1{,}243 \text{ psi}$$

The axial compressive stress in the glulam beam is as follows:

$$f_c = F_p/A = (18{,}900 \text{ lbs})/(212.6 \text{ in.}^2) = 89 \text{ psi}$$

The lower bound adjusted bending design strength per NDS-2012 Table 5.3.1 is as follows:

$$F_b' = 0.75 F_b C_M C_t C_L C_V C_{fu} C_c C_I K_F \phi \lambda$$

where:

F_b = 2,400 psi for 24F-V4 DF/DF

C_M, C_t, C_{fu}, C_C, C_I = 1.0

$$C_L = \frac{1+\left(F_{bE}/F_b^*\right)}{1.9} - \sqrt{\left[\frac{1+\left(F_{bE}/F_b^*\right)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

(NDS-2012 Eq. 3.3-6)

where:

$$F_{bE} = \frac{1.20E'_{\min}}{R_B^2}$$

where:

$$R_B = \sqrt{\frac{l_e d}{b^2}} = \sqrt{\frac{(166 \text{ in.})(31.5 \text{ in.})}{(6.75 \text{ in.})^2}} = 10.7 \quad \text{(NDS-2012 Eq. 3.3-5)}$$

where:

l_e = 1.73(8 ft)(12 in./ft) = 166 in, per NDS-2012 Table 3.3.3

$E'_{min} = E_{min} C_M C_t K_F \phi$

E_{min} = 950,000 psi for 24F-V4 DF/DF

C_M, C_t = 1.0

K_F = 1.76 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

E'_{min} = (950,000 psi)(1.0)(1.0)(1.76)(1.0)

= 1,672,000 psi

$$F_{bE} = \frac{1.20(1,672,000 \text{ psi})}{(10.7)^2} = 17,525 \text{ psi}$$

$F_b^* = 0.75 F_b C_M C_t C_c C_I K_F \phi \lambda$ (all adjustment factors except C_L, C_V, C_{fu} per NDS-2012 § 3.3.3.8)

F_b^* = 0.75(2,400 psi)(1.0)(1.0)(1.0)(1.0)(2.54)(1.0)(1.0)

= 4,572 psi

$$C_L = \frac{1+\left(17,525 \text{ psi}/4,572 \text{ psi}\right)}{1.9} - \sqrt{\left[\frac{1+\left(17,525 \text{ psi}/4,572 \text{ psi}\right)}{1.9}\right]^2 - \frac{\left(17,525 \text{ psi}/4,572 \text{ psi}\right)}{0.95}}$$

= 0.98

$C_V = (21/L)^{1/x}(12/d)^{1/x}(5.125/b)^{1/x} \leq 1.0$ (NDS-2012 Eq. 5.3-1)

$= (21/50 \text{ ft})^{1/10}(12/31.5 \text{ in.})^{1/10}(5.125/6.75 \text{ in.})^{1/10} = 0.81$

$C_V < C_L$, C_V controls, only apply C_V per NDS-2012 Section 5.3.6 and Table 5.3.1, Footnote 1.

K_F = 2.54 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

λ = 1.0 per NDS-2012 Section N3.3 and Table N3 for load combinations with seismic

F'_b = 0.75(2,400 psi)(1.0)(1.0)(n/a)(0.81)(1.0)(1.0)(1.0)(2.54)(1.0)(1.0)

= 3,704 psi

The lower bound adjusted design strength for compression parallel to grain, per NDS-2012 Table 5.3.1 is as follows:

$F'_c = F_c C_M C_t C_P K_F \phi \lambda$

where:

F_c = 1,650 psi for 24F-V4 DF/DF

C_M, C_t = 1.0

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

(NDS-2012 Eq. 3.7-1)

c = 0.9, glulam

$$F_{cE} = \frac{0.822E'_{\min}}{\left(l_e/d\right)^2}$$

where:

$(l_e/d)_x$ = (50 ft)(12 in./ft)/(31.5 in.) = 19.0

$(l_e/d)_y$ = (24 ft)(12 in./ft)/(6.75 in.) = 42.7, most critical is y-axis

The glulam beam is evaluated for compression buckling assuming there are kicker braces at every third 4×14 purlin at 8 feet on center to brace the bottom flange at 24 feet on center. If they are not present, they will need to be installed as part of the retrofit to the cross-tie gridlines.

$E'_{\min}$ = $E_{\min}C_MC_tK_F\phi$

$E_{\min}$ = 950,000 psi for 24F-V4 DF/DF

C_M, C_t = 1.0

K_F = 1.76 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

$E'_{\min}$ = (950,000 psi)(1.0)(1.0)(1.76)(1.0) = 1,672,000 psi

$$F_{cE} = \frac{0.822(1{,}672{,}000 \text{ psi})}{\left(42.7\right)^2} = 754 \text{ psi}$$

F_c^* = $F_cC_MC_tK_F\phi\lambda$ (all adjustment factors except C_P)

F_c^* = (1,650 psi)(1.0)(1.0)(2.40)(1.0)(1.0) = 3,960 psi

$$C_P = \frac{1+\left(754 \text{ psi}/3{,}960 \text{ psi}\right)}{2\left(0.9\right)} - \sqrt{\left[\frac{1+\left(754 \text{ psi}/3{,}960 \text{ psi}\right)}{2\left(0.9\right)}\right]^2 - \frac{\left(754 \text{ psi}/3{,}960 \text{ psi}\right)}{\left(0.9\right)}}$$

$C_P \quad = 0.19$

$F_c' \; = (1{,}650 \text{ psi})(1.0)(1.0)(0.19)(2.40)(1.0)(1.0) = 752 \text{ psi}$

When evaluating the unity check for combined bending and axial compression, the denominator is the lower bound strength, Q_{CL}, and needs to be multiplied by the knowledge factor, κ, to be consistent with the acceptance criteria for force-controlled components as shown below:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

$\kappa \quad = 0.75$ per Section 6.6.1 of this *Guide*

$Q_{CL} \; = F_b'$, F_c' (lower bound strength)

$Q_{UF} \; = f_b, f_c$

Combined bending and compression is checked per NDS-2012 Section 3.9.2:

$$\left[\frac{f_c}{\kappa F_c'}\right]^2 + \frac{f_b}{\kappa F_b'\left[1-\left(\frac{f_c}{F_{cE}}\right)\right]} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

$$\left[\frac{89 \text{ psi}}{0.75(752 \text{ psi})}\right]^2 + \frac{1{,}243 \text{ psi}}{0.75(3{,}704 \text{ psi})\left[1-\left(\frac{89 \text{ psi}}{754 \text{ psi}}\right)\right]} = 0.53 < 1.0, \text{ O.K.}$$

The existing glulam beams are adequate to resist the continuous cross-tie out-of-plane wall anchorage forces. Kicker braces at each 4×14 purlin at 24 feet on center are required to brace the bottom flange of the glulam beam. If they are not present, they will need to be included as part of the retrofit along Gridlines C through F.

The continuity tie connection 14 feet west of Gridline 2 is evaluated as shown in Figure 6-23. The existing connection is a gravity-only saddle connection. The proposed strengthening is to add a bolted splice plate on each side of the glulam beam. The connection will be evaluated as a force controlled component per ASCE 41-13 § 7.5.2.1.2. The axial load per Figure 6-22 is 17,400 lbs.

$Q_{UF} = 17{,}400 \text{ lbs}$

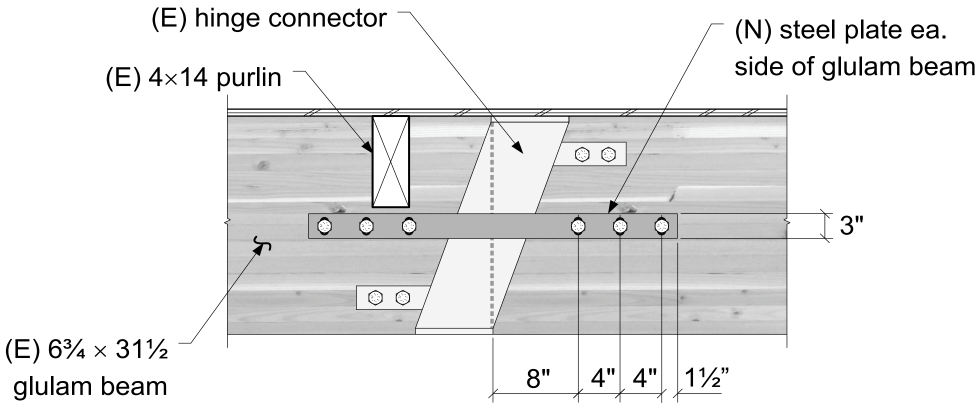


Figure 6-23 Continuity tie connection.

The connection will be three ¾-inch bolts with ¼-inch plates each side. The end distance and spacing provided exceed the minimum required, so no reduction in strength is required with a geometry adjustment factor. The lower bound component strength of wood connections per ASCE 41-13 § 12.3.2.3.1 is calculated as the load and resistance factor design values in NDS-2012 with a resistance factor, ϕ, taken equal to 1.0.

Z' = per NDS-2012 Table 10.3.1: All adjustment factors = 1.0 except K_F = 3.32. Per NDS-2012 Section N3.3 and Table N3, λ = 1.0 for load combinations with seismic.

where:

Z = 3,340 lbs per NDS-2012 Table 11I for Douglas Fir-Larch, 6-3/4" main member, 1/4" plate side member, and ¾" diameter bolt

$$Z' = ZC_M C_t C_g C_\Delta C_{eg} C_{di} C_{tn} K_F \phi \lambda$$

$$= (3\text{ bolts})(3{,}340\text{ lbs})(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(3.32)(1.0)(1.0)$$

$$= 33{,}266\text{ lbs}$$

The fasteners are evaluated with the acceptance criteria for force-controlled components as follows:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = 0.75 per Section 6.6.1 of this *Guide*

κQ_{CL} = (0.75)(33,266 lbs) = 24,950 lbs

24,950 lbs > 17,400 lbs; therefore, three 3/4" diameter bolts are adequate.

The bolted splice plate on each side of the glulam beam is evaluated assuming the plate is ASTM A36 steel. The lower bound component strength of steel connections per ASCE 41-13 § 9.3.2.3 is calculated as the load and resistance factor design values in AISC 360-10, *Specification for Structural Steel Buildings* (AISC, 2010b), with a resistance factor, ϕ, taken equal to 1.0.

Tension yielding, R_{NY}, and tension rupture, R_{NR}, is checked per AISC 360-10 Section J4.1:

$$R_{NY} = F_y A_g = (36{,}000 \text{ psi})(3 \text{ in.})(1/4 \text{ in.}) \quad \text{(AISC 360-10 Eq. J4-1)}$$
$$= 27{,}000 \text{ lbs}$$

$$R_{NR} = F_u A_e \quad \text{(AISC 360-10 Eq. J4-1)}$$

A_e = effective net area using the effective diameter of the hole, d_{eff}, equal to 1/16-inch larger than the nominal hole diameter per AISC 360-10 Section B4.3b, so d_{eff} = 3/4 inch diameter bolt + 1/16 inch for standard hole + 1/16 inch per AISC 360-10 Section B4.3b = 7/8 inch.

$$= (58{,}000 \text{ psi})(3 \text{ in.} - 7/8 \text{ in.})(1/4 \text{ in.}) = 30{,}813 \text{ lbs}$$

Tension yielding governs. The plate is evaluated with the acceptance criteria for force-controlled components as follows:

$$\kappa Q_{CL} > Q_{UF} \quad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = 1.0 per Section 6.6.1 of this *Guide*

κQ_{CL} = (1.0)(27,000 lbs) = 27,000 lbs

27,000 lbs > 17,400 lbs; therefore, 1/4 inch × 3 inch plate is adequate.

6.6.4 Pseudo Seismic Force on Roof Diaphragm

The diaphragm forces will be calculated in this section in order to determine the forces on the collectors at the re-entrant corner in Section 6.6.5 of this *Guide*. There were no noncompliant checklist items in the Tier 2 evaluation that require the global diaphragm forces to be assessed.

The diaphragm forces will be determined using the linear static procedure as permitted in ASCE 41-13 § 7.3.1. The building does not contain any of the characteristics that would prohibit the linear static procedure outlined in ASCE 41-13 § 7.3.1.2. The linear static procedure utilizes the pseudo seismic force, V, in ASCE 41-13 § 7.4.1.3 to calculate the seismic demands on elements. See Section 4.3.2 of this *Guide* for a more detailed example of determining the pseudo seismic force.

Pseudo seismic forces will be computed for only the Life Safety Performance Levels for the BSE-1E Seismic Hazard Levels in accordance with ASCE 41-13 § 5.2.1.

$V = C_1C_2C_mS_aW$ (ASCE 41-13 Eq. 7-21)

where:

C_1 and C_2:

In this example, the simplified alternate method will be used to determine the combined factors C_1C_2 per ASCE 41-13 Table 7-3. See Section 4.3.2 of this *Guide* for further discussion on the C_1 and C_2 factors. In order to determine the combined C_1C_2 factors using ASCE 41-13 Table 7-3, the fundamental period, T, of the building is required, and it was determined as 0.20 seconds in Section 6.5.1 using the same empirical period formulation as Method 2 in ASCE 41-13 § 7.4.1.2.2. For one-story buildings with flexible diaphragms, there is the option of using Method 3 of ASCE 41-13 § 7.4.1.2.3, which contains a more accurate period formulation in Equation 7-19. However, this formula requires an analysis to determine the in-plane wall and diaphragm displacements which is beyond the scope of this example. FEMA P-1026, *Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedures* (FEMA, 2015b), contains a simplified period formulation for rigid wall-flexible diaphragm buildings and is discussed in the adjacent "Useful Tip" box.

Useful Tip

FEMA P-1026 indicates that the dynamic behavior of tilt-up buildings is dominated by the diaphragm's response instead of the walls' response, and the base shear related to the diaphragm response may be determined using the period of the diaphragm, T_{DIAPH}, while the base shear related to the in-plane wall mass may be determined using the period of the walls. This procedure is not easily adaptable to the methodology contained in ASCE 41-13, nor has this method been adopted by ASCE 7 or ASCE 41. The procedure would be an alternate approach and may prove useful when evaluating tilt-up buildings with long span diaphragms.

The selection of the combined factors C_1C_2 per ASCE 41-13 Table 7-3 also requires the determination of m_{max}, which is the largest m-factor for all primary elements of the building in the direction under consideration. The primary mechanisms in the building are yielding of the plywood diaphragm and concrete walls. The m-factor for wood structural panel diaphragms per ASCE 41-13 Table 12-3 is 3.0 for Life Safety. The m-factors for concrete shear walls with low axial and shear loads and no boundary elements per ASCE 41-13 Table 10-21 and Table 10-22 range from 2.5 to 3 for Life Safety. m_{max} is approximately 3 for this structure.

The value for combined factors C_1C_2 per ASCE 41-13 Table 7-3 for a fundamental period of 0.20 seconds and $2 \leq m_{max} \leq 6$ is as follows:

$C_1C_2 = 1.4$

This example only focuses on the evaluation of noncompliant items found in Tier 1. If a full building assessment were performed, validation of the C_1 and C_2 factors should be done in accordance with ASCE 41-13 § 7.4.1.3.1 considering the actual demand-capacity ratios of the controlling components through an iterative process.

C_m:

C_m is obtained per Table 7-4 of ASCE 41-13.

$C_m = 1.0$ for concrete shear wall systems and one-story in height

$S_{a,BSE\text{-}1E} = 0.80g$ per Section 6.5.1 of this *Guide*

W:

W is the effective seismic weight of the building

Therefore,

$V_{BSE\text{-}1E} = C_1C_2C_mS_{a,BSE\text{-}1E}W = 1.4(1.0)(0.80g)W = 1.12W$

Diaphragm forces will be computed per ASCE 41-13 § 7.4.1.3.4. For one-story buildings, the diaphragm forces will be the same as the pseudo base shear forces calculated above since there is no vertical distribution of story forces.

$F_{px} = 1.12W$

Next, diaphragm load in east-west direction is determined. The mass of the diaphragm and wall were derived in Section 6.5.2 of this *Guide* and copied below. See Figure 6-24 for building layout and diaphragm labels.

$W_{\mathrm{I}} = 65$ kips

$W_{\mathrm{II}} = 108$ kips + 323 kips = 431 kips

$W_{wall} = 0.94$ k/ft

The weights per foot of diaphragm for loading in the east-west direction are as follows:

$w_{px\mathrm{I}} = 65$ kips/24 ft + 2(0.94 k/ft) = 4.59 k/ft

$w_{px\mathrm{II}} = 431$ kips/120 ft + 2(0.94 k/ft) = 5.47 k/ft

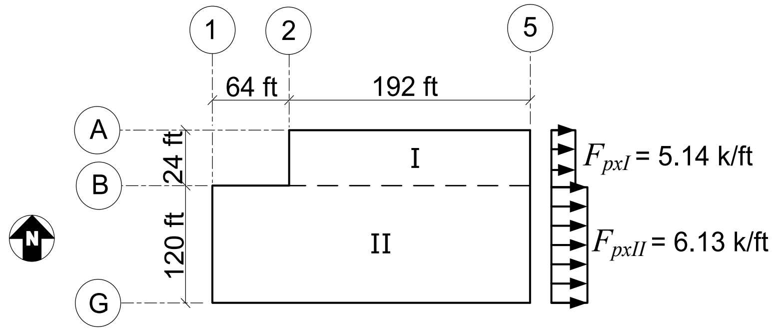


Figure 6-24 Diagraphm forces for east-west loading.

The diaphragm forces per foot of diaphragm for loading in the east-west direction are as follows:

$F_{px\mathrm{I}}$ = 1.12(4.59 k/ft) = 5.14 k/ft

$F_{px\mathrm{II}}$ = 1.12(5.47 k/ft) = 6.13 k/ft

6.6.5 Collector Analysis at Gridline B

The collector along Gridline B will be evaluated assuming an idealized flexible diaphragm with simple spans from Gridlines A to B and B to G. Collectors are evaluated as force-controlled actions, while the collector connections are evaluated as a combination of force-controlled actions and deformation-controlled actions. In accordance with ASCE 41-13 § 12.3.3.1, connectors that link wood-to-wood or wood-to-metal (i.e., nails, bolts) are evaluated as deformation-controlled actions, and the body of connection hardware (i.e., strap) is evaluated as a force-controlled action.

The collector load along Gridline B at Gridline 2 is determined as follows as illustrated in Figure 6-25:

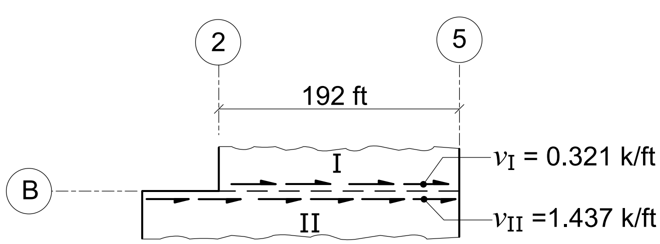


Figure 6-25 Collector loads at Gridline B.

As noted above, the collectors are evaluated as force-controlled actions. The acceptance criteria for force-controlled actions using linear analysis procedures is outlined in ASCE 41-13 § 7.5.2.2.2, as follows:

$\kappa Q_{CL} > Q_{UF}$

The force-controlled action, Q_{UF}, is permitted to be calculated using one of two different methods specified in ASCE 41-13 § 7.5.2.1.2. The first method is to perform a limit-state analysis to determine the maximum force that can be delivered to the component being evaluated based on the expected strength of the component delivering the load. The second method will determine the collector loads based on the pseudo diaphragm forces then reduce those loads by dividing by C_1C_2J. The loads on the collector will be determined with both methods and will be the lesser of the following.

1. The demand using a limit-state analysis considering the expected strength of the diaphragm on each side of the collector per ASCE 41-13 § 7.5.2.1.2 Bullet (1).

   The expected strength of the diaphragm is determined in accordance with ASCE 41-13 § 12.5.3.6.2 based on 1.5 times the load and resistance factor design diaphragm capacities in accordance with SDPWS-2008 Table 4.2A with a resistance factor, ϕ, equal to 1.0. Per Section 6.6.3 of this *Guide*, the nominal shear strength of the diaphragm is 850 plf.

   v_{CE} = 1.5(850 plf) = 1,275 plf

   Q_{UF} = Q_{CE} = 2(1,275 plf)(192 ft) = 490 kips

2. The demand using a limit-state analysis considering the expected strength of the diaphragm nails into the glulam beam collector per ASCE 41-13 § 7.5.2.1.2 Bullet (1).

   The expected strength of the diaphragm nails is determined in accordance with ASCE 41-13 § 12.3.2.2.1 based on 1.5 times the load and resistance factor design capacities in accordance with NDS-2012 with a resistance factor, ϕ, equal to 1.0. Per Section 6.6.2 of this *Guide*, the nominal design capacity of the 10d nail through 15/32 sheathing is 285 lbs.

   Q_{CE} = 1.5(285 lbs) = 428 lbs/nail

   Per Section 6.6.2 of this *Guide*, there are 40 nails per 16 feet attaching the diaphragm to the top of the glulam beam. The expected strength of the diaphragm nails for the entire length of the collector is as follows:

   Q_{UF} = Q_{CE} = (428 lbs/nail)(192 ft)((40 nails)/(16 ft))/(1000 lbs/kip)
   = 205 kips

3. The demand based on the pseudo diaphragm forces divided by C_1C_2J per ASCE 41-13 § 7.5.2.1.2 Bullet (2) in accordance with ASCE 41-13 Equation 7-35.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

where:

Q_G = 0 kips, no gravity axial load in collector

C_1C_2 = 1.4, per Section 6.6.4 of this *Guide*

J = 2.0, for High Level of Seismicity, forces are being delivered by a yielding diaphragm element. With more rigorous calculations, J can be determined as the smallest demand capacity ratio of the components in the load path delivering force to this component.

Q_E = Collector load per pseudo diaphragm forces. The diaphragm shear, v, along each side of the collector.

v_{I} = (5.14 k/ft)(24 ft/2)/(192 ft) = 0.321 k/ft

v_{II} = (6.13 k/ft)(120 ft/2)/(192 ft + 64 ft) = 1.437 k/ft

The collector load is as follows:

Q_E = (0.321 k/ft + 1.437 k/ft)(192 ft) = 338 kips

$$Q_{UF} = \frac{(338 \text{ kips})}{(1.4)(2)} = 121 \text{ kips}$$

Summary:

Q_{UF} = 490 kips, based on expected diaphragm strength

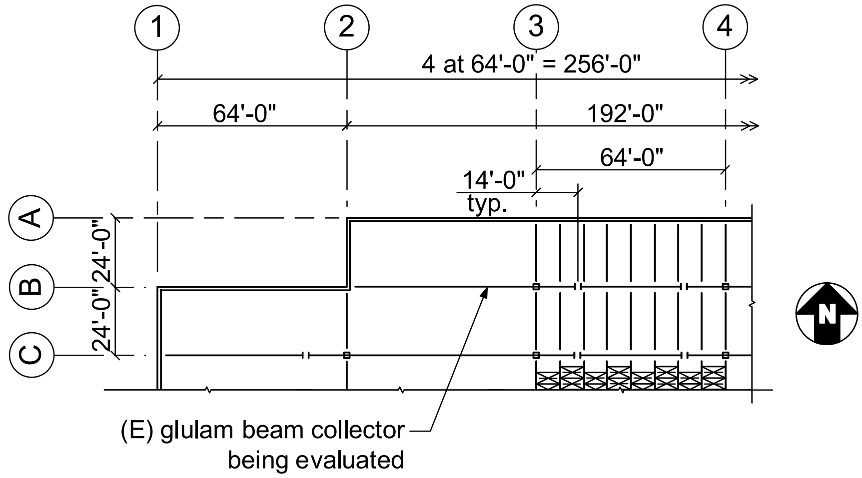
Q_{UF} = 205 kips, based on expected diaphragm to glulam nailing strength

Q_{UF} = 121 kips, based on pseudo diaphragm forces divided by C_1C_2J

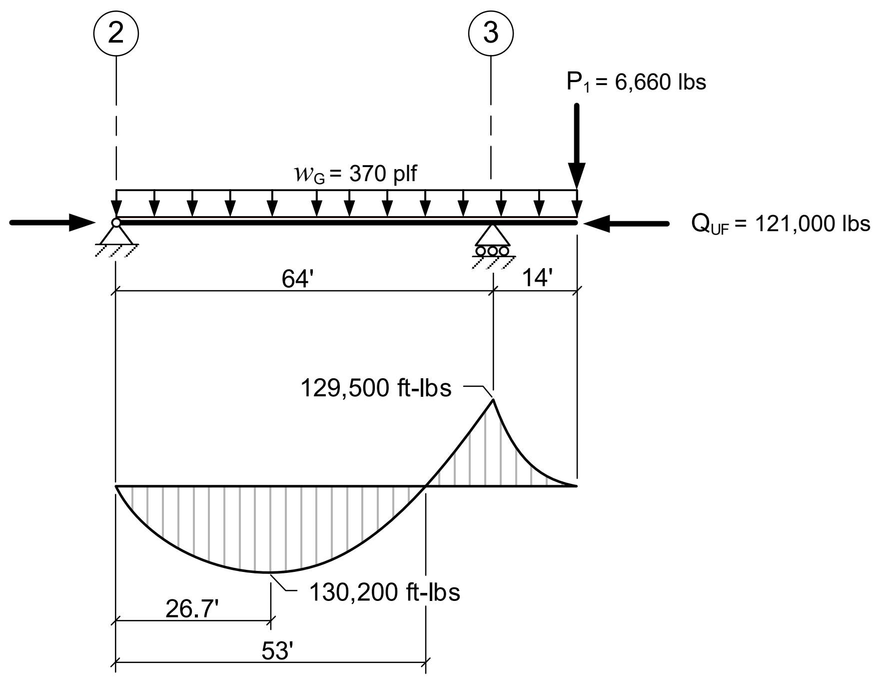
Therefore, Q_{UF} = 121 kips controls.

6.6.5.1 Evaluate Glulam Beam for Collector Loads

The glulam beam at Gridline B between Gridlines 2 and 3 shown in Figure 6-26 will be evaluated for the collector loads at the re-entrant corner. The “Transfer to Shear Walls” Tier 1 checklist item was “noncompliant” and ASCE 41-13 § A.5.2.1 indicates that where walls do not extend the full depth of the diaphragm, that this item includes both the connection and the collector. As discussed at the beginning of this section, the diaphragm shear need not be evaluated in the Tier 2 procedure for this checklist item.



PLAN



BEAM LOADING DIAGRAM

Figure 6-26 Glulam beam loads and moment diagram at Gridline B.

The glulam beam will be evaluated for combined gravity and axial as force controlled component per ASCE 41-13 § 7.5.2.1.2. The collector load is 121,000 lbs derived in the previous section.

Q_{UF} = 121,000 lbs

The gravity load combination per ASCE 41-13 § 7.2.2 is as follows:

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad \text{(ASCE 41-13 Eq. 7-1)}$$

Q_D = The dead load on the girder is 14 psf per Table 6-1

Q_L = 0, roof live load need not be applied simultaneously with seismic per ASCE 41-13 § 7.2.2

Q_S = 0, no snow load

Q_G = 1.1(14 psf) = 15.4 psf

w_G = (15.4 psf)(24 ft) = 370 lbs/ft

P_1 = (370 lbs/ft)((64 ft – 14 ft – 14 ft)/2) = 6,660 lbs (reaction from adjacent beam)

M_G = 130,200 ft-lbs at main span, 129,500 ft-lbs at support

The loading on the glulam beam and associated moment diagram are shown in Figure 6-26.

The lower-bound strength of the member is determined per ASCE 41-13 § 12.3.2.3.1 using NDS-2012 with strength reduction factor equal to 1.0. Combined bending and compression is checked per NDS-2012 Section 3.9.2:

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_b}{F'_b\left[1-\left(\frac{f_c}{F_{cE}}\right)\right]} \leq 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

The properties of the 6-3/4 × 31-1/2 glulam beam are as follows:

24F-V8 DF/DF, S_x = 1116 in.^{3}, A = 212.6 in.^{2}

As discussed in Section 6.6.3 of this *Guide*, the bending design value is reduced 25% in accordance with the AITC Technical Note 26 (AITC, 2007) recommendations by applying an additional 0.75 factor to the bending design values.

The main span of the glulam beam will be evaluated in this example as that span has the largest unbraced compression length of 64 feet in the strong axis and the accumulation of collector forces is maximum in that span. The glulam beam will be evaluated assuming there are kicker braces at every other 4×14 purlin at 8 feet on center to brace the bottom flange at 16 feet on center. If they are not present, they will need to be installed as part of the retrofit to the collector line.

The bending stress in the glulam beam is as follows:

$f_b = M_G/S_x$ = (130,200 ft-lbs)(12 in./ft)/(1116 in.^{3}) = 1,400 psi

The axial compressive stress in the glulam beam is as follows:

$f_c = F_p/A$ = (121,000 lbs)/(212.6 in.^{2}) = 569 psi

The lower bound adjusted bending design strength per NDS-2012 Table 5.3.1 is as follows:

$$F_b' = 0.75 F_b C_M C_t C_L C_V C_{fu} C_c C_I K_F \phi \lambda$$

where:

F_b = 2,400 psi for 24F-V8 DF/DF

C_M, C_t, C_{fu}, C_C, C_I = 1.0

$$C_L = \frac{1+\left(F_{bE}/F_b^*\right)}{1.9} - \sqrt{\left[\frac{1+\left(F_{bE}/F_b^*\right)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

(NDS-2012 Eq. 3.3-6)

where:

$$F_{bE} = \frac{1.20 E'_{\min}}{R_B^2}$$

where:

$$R_B = R_B = \sqrt{\frac{l_e d}{b^2}} = \sqrt{\frac{(396 \text{ in.})(31.5 \text{ in.})}{(6.75 \text{ in.})^2}} = 16.5 \text{(NDS-12 Eq. 3.3-5)}$$

where:

l_e = 2.06(16 ft)(12 in./ft) = 396 in., per NDS-2012 Table 3.3.3, Footnote 1 where l_u=16 ft and l_u/d=6

$E'_{\min} = E_{\min} C_M C_t K_F \phi$

where:

$E_{\min}$ = 950,000 psi for 24F-V8 DF/DF

C_M, C_t = 1.0

K_F = 1.76 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

$E'_{\min}$ = (950,000 psi)(1.0)(1.0)(1.76)(1.0)
= 1,672,000 psi

$$F_{bE} = \frac{1.20(1,672,000 \text{ psi})}{(16.5)^2} = 7,370 \text{ psi}$$

$F_b^* = 0.75 F_b C_M C_t C_c C_I K_F \phi \lambda$ (all adjustment factors except C_L,C_V,C_{fu} per NDS-2012 Section 3.3.3.8)

F_b^* = 0.75(2,400 psi)(1.0)(1.0)(1.0)(1.0)(2.54)(1.0)(1.0)
= 4,572 psi

$$C_L = \frac{1+(7{,}370\text{ psi}/4{,}572\text{ psi})}{1.9} - \sqrt{\left[\frac{1+(7{,}370\text{ psi}/4{,}572\text{ psi})}{1.9}\right]^2 - \frac{(7{,}370\text{ psi}/4{,}572\text{ psi})}{0.95}}$$

$= 0.94$

$C_V = (21/L)^{1/x}(12/d)^{1/x}(5.125/b)^{1/x} \le 1.0$ (NDS-2012 Eq. 5.3-1)

$= (21/53\text{ ft})^{1/10}(12/31.5\text{ in.})^{1/10}(5.125/6.75\text{ in.})^{1/10} = 0.81$

$C_V < C_L$, C_V controls, only apply C_V per NDS-2012 § 5.3.6 and Table 5.3.1, Footnote 1.

K_F = 2.54 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

λ = 1.0 per NDS-2012 Section N3.3 and Table N3 for load combinations with seismic

$F_b' = 0.75(2{,}400\text{ psi})(1.0)(1.0)(\text{n/a})(0.81)(1.0)(1.0)(1.0)(2.54)(1.0)(1.0)$

$= 3{,}704$ psi

The lower bound adjusted design strength for compression parallel to grain, per NDS-2012 Table 5.3.1 is as follows:

$F_c' = F_c C_M C_t C_P K_F \phi \lambda$

where:

F_c = 1,650 psi for 24F-V8 DF/DF

C_M, C_t = 1.0

$$C_p = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

(NDS-2012 Eq. 3.7-1)

c = 0.9, glulam

$$F_{cE} = \frac{0.822E'_{\min}}{(l_e/d)^2}$$

where:

$(l_e/d)_x$ = (64 ft)(12 in./ft)/(31.5 in.) = 24.4

$(l_e/d)_y$ = (16 ft)(12 in./ft)/(6.75 in.) = 28.4, most critical is y-axis

$E'_{\min} = E_{\min} C_M C_t K_F \phi$

where:

E_{min} = 950,000 psi for 24F-V8 DF/DF

C_M, C_t = 1.0

K_F = 1.76 per NDS-2012 Table 5.3.1

ϕ = 1.0, per ASCE 41-13 § 12.3.2.3.1

$$E'_{min} = (950,000 \text{ psi})(1.0)(1.0)(1.76)(1.0) = 1,672,000 \text{ psi}$$

$$F_{cE} = \frac{0.822(1,672,000 \text{ psi})}{(28.4)^2} = 1,704 \text{ psi}$$

$$F_c^* = F_c C_M C_t K_F \phi \lambda \text{ (all adjustment factors except } C_P\text{)}$$

$$= (1,650 \text{ psi})(1.0)(1.0)(2.40)(1.0)(1.0) = 3,960 \text{ psi}$$

$$C_P = \frac{1+(1,704 \text{ psi}/3,960 \text{ psi})}{2(0.9)} - \sqrt{\left[\frac{1+(1,704 \text{ psi}/3,960 \text{ psi})}{2(0.9)}\right]^2 - \frac{(1,704 \text{ psi}/3,960 \text{ psi})}{(0.9)}}$$

$$C_P = 0.40$$

$$F'_c = (1,650 \text{ psi})(1.0)(1.0)(0.40)(2.40)(1.0)(1.0) = 1,584 \text{ psi}$$

When evaluating the unity check for combined bending and axial compression, the denominator is the lower bound strength, Q_{CL}, and needs to be multiplied by the knowledge factor, κ, to be consistent with the acceptance criteria for force-controlled components as shown below:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = 0.75 per Section 6.6.1 of this *Guide*

Q_{CL} = F'_b, F'_c (lower bound strength)

Q_{UF} = f_b, f_c

Combined bending and compression is checked per NDS-2012 Section 3.9.2.

$$\left[\frac{f_c}{\kappa F'_c}\right]^2 + \frac{f_b}{\kappa F'_b\left[1-\left(\dfrac{f_c}{F_{cE}}\right)\right]} \le 1.0 \qquad \text{(NDS-2012 Eq. 3.9-3)}$$

$$\left[\frac{569 \text{ psi}}{0.75(1,584 \text{ psi})}\right]^2 + \frac{1,400 \text{ psi}}{0.75(3,704 \text{ psi})\left[1-\left(\dfrac{569 \text{ psi}}{1,704 \text{ psi}}\right)\right]} = 0.99 < 1.0 \quad \text{OK}$$

The existing glulam beams are adequate to resist the continuous cross-tie out-of-plane wall anchorage forces. Kicker braces at each 4×14 purlin at 16 feet on center are required to brace the bottom flange of the glulam beam. If they are not present, they will need to be included as part of the retrofit along Gridline B.

6.6.5.2 Evaluate Collector Connection at Concrete Wall at Gridlines B/2

The connection of the glulam beam at Gridline B to the concrete wall intersection at Gridline 2 will be evaluated for the collector loads derived in the previous section. This example will not evaluate the collector connection along Gridline 2 for loads in the north-south direction. As indicated above, collector connections are evaluated as a combination of force-controlled actions and deformation-controlled actions. In accordance with ASCE 41-13 § 12.3.3.1, connectors that link wood-to-wood or wood-to-metal (i.e., nails, bolts) are evaluated as deformation-controlled actions, and the body of connection hardware (i.e., rods, plates) is evaluated as a force-controlled action. The proposed collector connection is shown in Figure 6-27.

The following components of the collector connection will be evaluated:

- Bolts connecting HSS to glulam beam
- HSS member
- Rods connecting HSS members
- HSS end plates
- Adhesive anchors connecting HSS to concrete wall

The glulam beam will not be evaluated in this example for the moment about the y-axis as a result of the eccentric loading between the HSS and glulam beam, but should be for a complete evaluation.

6.6.5.3 Evaluate Bolts Connecting HSS to Glulam Beam

The bolted connection between the HSS8×6 and glulam beam is evaluated as shown in Figure 6-27. The connection will be evaluated as a deformation-controlled action per ASCE 41-13 § 12.3.3.1 and ASCE 41-13 § 7.5.2.1.1. The axial collector load per the pseudo diaphragm forces shown in Figure 6-25 (determined earlier in this section noted as Q_E is 338 kips).

$$Q_{UD} = (0.321 \text{ kips/ft} + 1.437 \text{ kips/ft})(192 \text{ ft}) = 338 \text{ kips}$$

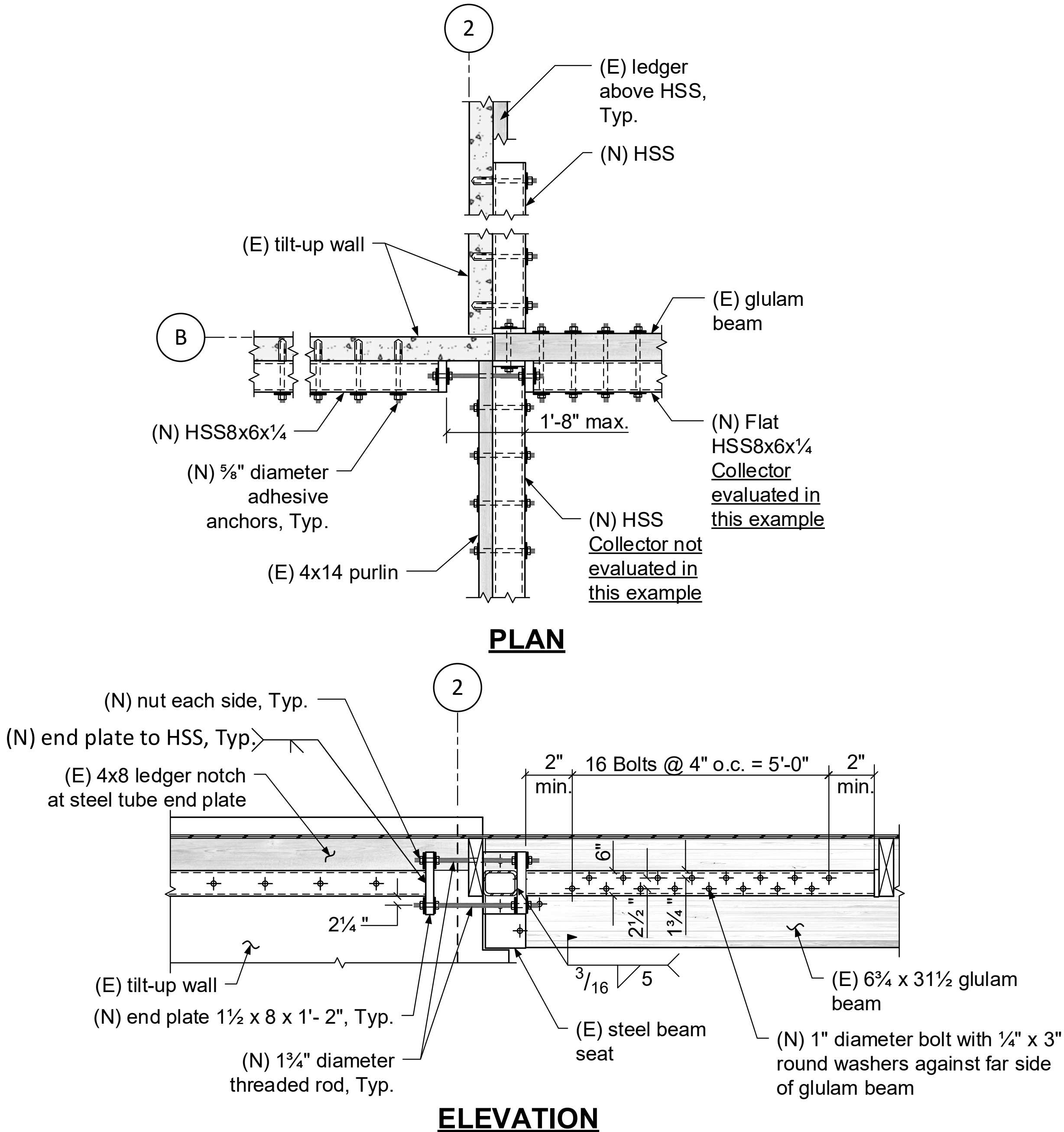


Figure 6-27 Proposed collector connection at Gridlines B/2.

The connection will be sixteen 1-inch diameter bolts at 4 inches on-center staggered with 1/4-inch thick HSS wall on one side. The end distance and spacing provided exceed the minimum required in NDS-2012 Section 11.5.1, so no reduction in strength is required with a geometry adjustment factor, C_Δ. For deformation-controlled actions, the expected strength of the component is used. The expected strength of wood connections per ASCE 41-13 § 12.3.2.2.1 is calculated based on 1.5 times the load and resistance factor design procedures in ANSI/AWC NDS-2012 with a resistance factor, ϕ, taken equal to 1.0.

Z' = per NDS-2012 Table 10.3.1: All adjustment factors = 1.0, except C_g (see below) and K_F = 3.32. Per NDS-2012 Section N3.3 and Table N3, λ = 1.0 for load combinations with seismic.

Z_{ll} = 2,860 lbs per NDS-2012 Table 11D for Douglas Fir-Larch, 6-3/4" main member, 1/4" plate side member, and 1" diameter bolt

C_g = 0.94 per NDS-2012 Table 10.3.6C for A_m/A_s = 34, A_m = 213 in.^{2}, number of fasteners in a row = 8

$Z' = Z_{ll}C_MC_tC_gC_\Delta C_{eg}C_{di}C_{tn}K_F\phi\lambda$

= (2,860 lbs)(1.0)(1.0)(0.94)(1.0)(1.0)(1.0)(1.0)(3.32)(1.0)(1.0)

= 8,925 lbs

Q_{CE}= 1.5Z' (# of bolts) = 1.5(8,925 lbs)(16 bolts)/1000 lbs/kip = 214 kips

The fasteners are evaluated with the acceptance criteria for deformation-controlled actions per ASCE 41-13 § 7.5.2.2.1 as follows:

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

where:

m = component capacity modification factor from Table 12-3 for the entry "Machine Bolts—metal to wood" for Life Safety Performance Level = 2.8

κ = knowledge factor per Section 6.6.1 of this *Guide* is equal to 0.75.

$m\kappa Q_{CE}$ = 2.8(0.75)(214 kips) = 449 kips

449 kips > 338 kips; therefore sixteen 1 inch diameter bolts are adequate.

6.6.5.4 Evaluate HSS Member

The HSS8×6×1/4 member is evaluated for collector tension loads as shown in Figure 6-27. The member will be evaluated as a force-controlled action per ASCE 41-13 § 7.5.2.1.2. The force-controlled axial collector load derived in the previous section.

Q_{UF} = 121 kips

For force-controlled actions, the lower-bound strength of the component is used. The lower-bound strength of steel components per ASCE 41-13 § 9.3.2.3 is calculated based the load and resistance factor design procedures in AISC 360-10 with a resistance factor, ϕ, taken equal to 1.0. The HSS will be evaluated for tensile yielding in the gross section and tensile rupture in the net section in accordance with AISC 360-10 § D2.

The HSS8×6×1/4 has the following properties for ASTM A500, Grade B specification:

A_g = 6.17 in.^{2}, t = 0.233 inches, F_y = 46 ksi, F_u = 58 ksi

Tensile yielding is checked per AISC 360-10 Section D2(a):

$$P_n = F_y A_g = (46 \text{ ksi})(6.17 \text{ in.}^2) = 284 \text{ kips} \qquad \text{(AISC 360-10 Eq. D2-1)}$$

Tensile rupture is checked per AISC 360-10 Section D2(b):

$$P_n = F_u A_e \qquad \text{(AISC 360-10 Eq. D2-2)}$$

where:

$A_e = A_n U$

where:

A_n = net area per AISC 360-10 Section B4.3b. This requires computing the net area across the tube in an orthogonal section cut (*abc*) and a zigzag section cut (*abde*) to determine the most critical section per Figure 6-28. The net area is computed using the effective diameter of the hole, d_{eff}, equal to 1/16-inch larger than the nominal hole diameter per AISC 360-10 Section B4.3b, so d_{eff} = (1 inch diameter bolt + 1/16 inch for standard hole + 1/16 inch per AISC 360-10 Section B4.3b) = 1-1/8 inch. For each gage space that the section cut is staggered through (in this case cut *abde*) the term $s^2/4g$ is added as described in AISC 360-10 Section B4.3b. Since the HSS section has 2 holes at each bolt, the area values for each bolt will be doubled.

$A_n = A_g - A_{\text{holes}} + s^2/4g$

$A_{n(abc)}$ = 6.17 in.^{2} – (2)(1.125 in.)(0.233 in.) = 5.65 in.^{2}, governs

$A_{n(abde)}$ = 6.17 in.^{2} – (4)(1.125 in.)(0.233 in.) + (2)[(4 in.)^2/4(2.5 in.)](0.233 in.)
= 5.87 in.^{2}

U = shear lag factor = $(1 - \bar{x}/l)$, where $\bar{x}$ and l are illustrated in Figure 6-29. AISC 360-10 Table D3.1 does not contain the specific case for this connection.

$\bar{x}$ is defined as the distance from the connection interface to the centroid of the member and l is the length of the connection.

$= 1 - (4 \text{ in.})/(60 \text{ in.}) = 0.93$

$A_e = A_nU = (5.65 \text{ in.}^2)(0.93) = 5.25 \text{ in.}^2$ (AISC 360-10 Eq. D3-1)

$P_n = F_uA_e = (58 \text{ ksi})(5.25 \text{ in.}^2) = 305 \text{ kips} > 284 \text{ kips}$; therefore, tensile yielding governs.

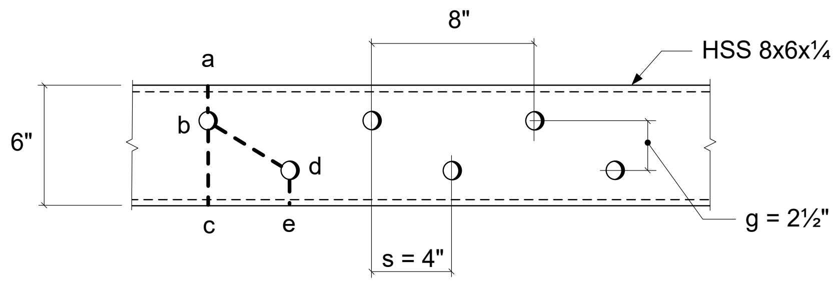


Figure 6-28 Net area sections in HSS.

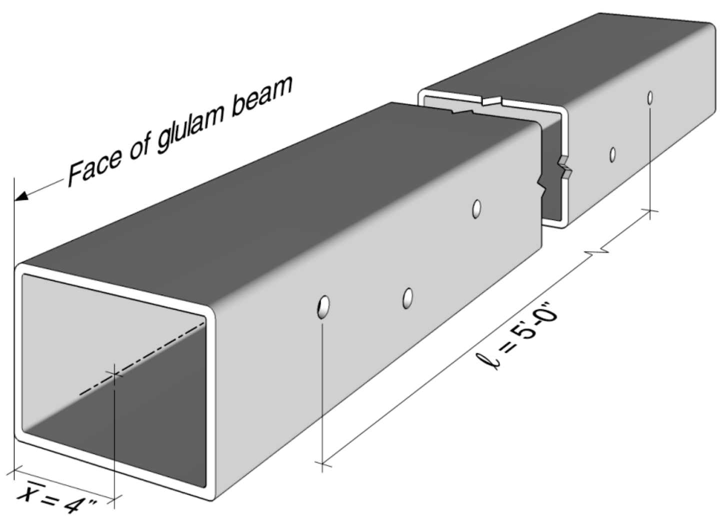


Figure 6-29 Shear lag factor for HSS connection.

The member is evaluated with the acceptance criteria for force-controlled actions per ASCE 41-13 § 7.5.2.2.2 as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

where:

κ = knowledge factor per Section 6.6.1 of this *Guide* is equal to 1.0.

$\kappa Q_{CL} = (1.0)(284 \text{ kips}) = 284 \text{ kips}$

284 kips > 121 kips; therefore, HSS8×6×1/4 with 1-1/16 inch diameter holes is adequate.

6.6.5.5 Evaluate Rods Connecting HSS Members

The rods connecting HSS end plates are evaluated for collector tension loads as shown in Figure 6-27. The rods will be evaluated as a force-controlled action per ASCE 41-13 § 7.5.2.1.2. The force-controlled axial collector load is 121 kips derived in the previous section and resisted by two rods.

Q_{UF} = 121 kips

For force-controlled actions, the lower-bound strength of the component is used. The lower-bound strength of steel components per ASCE 41-13 § 9.3.2.3 is calculated based on the load and resistance factor design procedures in AISC 360-10 with a resistance factor, ϕ, taken equal to 1.0. The rods will be evaluated for compression flexural buckling in accordance with AISC 360-10 Section E3.

The 1-3/4 inch diameter threaded rods have the following properties for ASTM F1554, Grade 36 specification:

A_g = 2.41 in.^{2}, K = 1.53 inches (root diameter)

F_y = 36 ksi, F_u = 58 ksi, E = 29,000 ksi

Flexural buckling is checked per AISC 360-10 § E3:

$$P_n = F_{cr}A_g \qquad \text{(AISC 360-10 Eq. E3-1)}$$

where:

F_{cr} = Determine the critical stress. First determine KL/r for the threaded rod. The effective length factor, K, is 1.0 since both ends of the rods are laterally braced. The maximum unbraced length is 1'-8" per Figure 6-27. Since the rod is threaded, the radius of gyration, r, will be determined based on the root diameter, K, at the threads.

r = d/4 = (1.53 in.)/4 = 0.38 inch

KL/r = (1.0)(20 in.)/(0.38 in.) = 53

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{(29{,}000\text{ ksi})}{(36\text{ ksi})}} = 134$$

53 < 134, F_{cr} is determined with AISC 360-10 Equation E3-2

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y \qquad \text{(AISC 360-10 Eq. E3-2)}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29{,}000 \text{ ksi})}{(53)^2} \qquad \text{(AISC 360-10 Eq. E3-4)}$$

$$= 102 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\frac{(36 \text{ ksi})}{(102 \text{ ksi})}}\right](36 \text{ ksi}) = 31 \text{ ksi}$$

$$P_n = F_{cr}A_g = (2 \text{ rods})(31 \text{ ksi})(2.41 \text{ in.}^2) = 149 \text{ kips}$$

The member is evaluated with the acceptance criteria for force-controlled actions per ASCE 41-13 § 7.5.2.2.2 as follows:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

κ = knowledge factor per Section 6.6.1 of this *Guide* is equal to 1.0.

κQ_{CL} = (1.0)(149 kips) = 149 kips

149 kips > 121 kips; therefore, two 1-3/4 inch diameter ASTM F1554 Grade 36 threaded rods are adequate.

6.6.5.6 Evaluate HSS End Plates

The end plates connecting the HSS to the threaded rods are evaluated for collector tension loads as shown in Figure 6-27 and 6-30. The plates will be evaluated as a force-controlled action per ASCE 41-13 § 7.5.2.1.2. The force-controlled axial collector load, Q_{UF}, is 121 kips, as derived earlier.

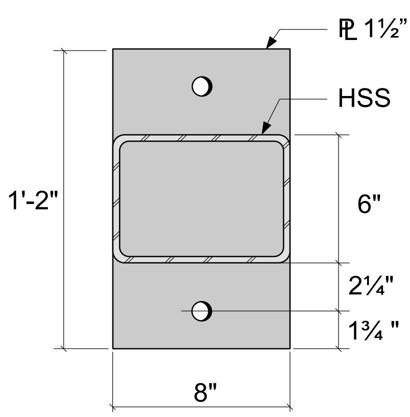


Figure 6-30 HSS end plate.

For force-controlled actions, the lower-bound strength of the component is used. The lower-bound strength of steel components per ASCE 41-13 § 9.3.2.3 is calculated based on the load and resistance factor design procedures in AISC 360-10 with a resistance factor, ϕ, taken equal to 1.0. The plates will be evaluated for flexure yielding in accordance with AISC 360-10 Section F11.

The 1-1/2 inch thick end plate will be ASTM A36 specification with F_y = 36 ksi.

Flexural yielding is checked per AISC 360-10 Section F11:

$M_n = M_p = F_y Z \leq 1.6 M_y$ (AISC 360-10 Eq. F11-1)

where:

$1.6M_y = 1.6SF_y = 1.6(3.0 \text{ in.}^3)(36 \text{ ksi}) = 173$ k-in

where:

$S = bd^2/6 = (8 \text{ in.})(1.5 \text{ in.})^2/6 = 3.0 \text{ in.}^3$

$Z = bd^2/4 = (8 \text{ in.})(1.5 \text{ in.})^2/4 = 4.5 \text{ in.}^3$

$M_p = (36 \text{ ksi})(4.5 \text{ in.}^3) = 162$ k-in. < 173 k-in, OK

$M_n = 162$ k-in.

Calculate the bending moment, M_{UF}, in the plate as a cantilever from the edge of the HSS to the centerline of the threaded rod. Half of the collector load is applied to each end of the plate.

$M_{UF} = Q_{UF}L/2 = (121 \text{ kips})(2.25 \text{ in.})/2 = 136$ k-in.

The component is evaluated with the acceptance criteria for force-controlled actions per ASCE 41-13 § 7.5.2.2.2 as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

where:

κ = knowledge factor per Section 6.6.1 of this *Guide* is equal to 1.0.

$\kappa Q_{CL} = (1.0)(162 \text{ k-in.}) = 162$ k-in

162 k-in > 136 k-in, 1-1/2 thick ASTM A36 plate is adequate.

6.6.5.7 Evaluate Adhesive Anchors Connecting HSS to Concrete Wall

The adhesive anchors in shear connecting the HSS8×6 to the concrete wall are evaluated for collector loads as shown in Figure 6-27. The adhesive anchors will be evaluated as a force-controlled action per ASCE 41-13 § 7.5.2.1.2. The force-controlled axial collector, Q_{UF}, load is 121 kips, as derived in the previous section.

This example employs the model for anchors in concrete provided in ACI 318-11 Appendix D to assess the bolt shear, edge breakout, and pryout failure modes for the collector anchorage to the existing concrete wall. Collectors with large numbers of anchors pose some design challenges that are beyond the model assumptions for anchors in ACI 318-11 Appendix D. The format for this portion of the example will first evaluate the collector connection in accordance with ACI 318-11 Appendix D and then, in a separate subsection, address those other design challenges.

In accordance with ASCE 41-13 10.3.6.2 the lower bound strength equals the anchor strength in accordance with ACI 318-11 Appendix D with $\phi = 1.0$. As discussed in Section 6.4.5.2 of this *Guide*, the additional 0.75 seismic reduction factor in ACI 318-11 Section D.3.3.4.4 applied to the concrete failure modes to determine the design tensile strength of concrete anchors is required to be applied when applying ASCE 41-13 loads; however, this 0.75 factor is not applicable to the design shear strength of anchors and will not be applied in this calculation since the anchors are loaded in shear. It should be noted that the nominal pryout strength for an anchor in shear is calculated in accordance with ACI 318-11 Section D.6.3 and is based on the tensile breakout and bond strength, but there is no requirement in ACI 318-11 Appendix D to apply the 0.75 factor in ACI 318-11 Section D.3.3.4.4 to the shear design of anchors, so the tensile breakout and bond strength calculation for the purpose of computing the pryout strength in shear need not be reduced by the 0.75 seismic reduction factor.

The evaluation of the adhesive anchors shall satisfy one of the design options in ACI 318-11 Section D.3.3.5.3. The design loads for collectors are evaluated as force-controlled actions, and therefore deemed to satisfy the design option in ACI 318-11 Section D.3.3.5.3(c), so no further amplification is required.

The geometry and configuration of the adhesive anchors connecting the HSS8x6 to the concrete wall are shown in Figure 6-31. The proposed connection is thirty 5/8-inch diameter adhesive anchors with 3-1/2 inch embedment, ASTM F1554, Grade 36 threaded rod. The adhesive anchors are 12 inches on-center with the first anchor located 9 inches from the end of the wall. The adhesive anchor design will assume the concrete wall is cracked as this is the typical assumption for walls subject to seismic loading. Uncracked concrete should not be assumed unless it can be proven otherwise.

The acceptance criteria for all modes of failure of the adhesive anchors will be summarized at the end of this section in Table 6-3.

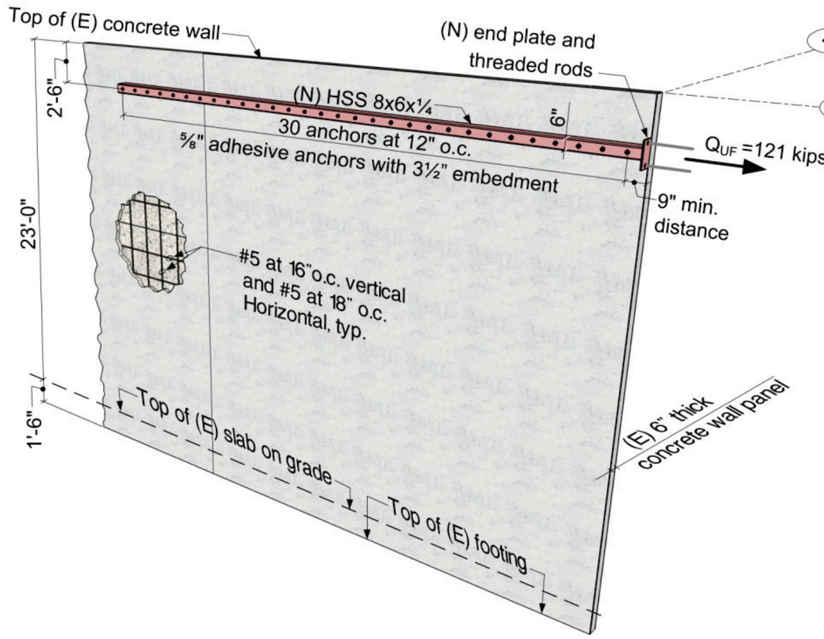


Figure 6-31 Wall elevation - adhesive anchor layout for collector connection.

Table 6-3 Adhesive Anchor Design Results

Design Check	Load QUF (kips)	Lower Bound Strength QCL (kips)	κ	κQ_{CL} (kips)	Status
Steel Strength	121	165	1.0	165	OK
Concrete Breakout Strength, Case 1 (Figure 6-32)	4.03	9.27	0.75	6.95	OK
Concrete Breakout Strength, Case 2a (Figure 6-33)	121	73	0.75	55	NGa
Concrete Breakout Strength, Anchor Reinforcement, Case 2b (Figure 6-34)	121	149	1.0	149	OKa
Concrete Breakout Strength, Perpendicular (Figure 6-35)	121	276	0.75	207	OK
Concrete Pryout Strength (Figure 6-36,6-37)	121	250	0.75	188	OK

* Case 2 is OK provided either Case 2a or 2b meets the acceptance criteria.

Next, adhesive anchor steel strength is checked in shear per ACI 318-11 Section D.6.1. The anchor bolt properties are taken from the referenced tables in *Notes on ACI 318-11 Building Code Requirements for Structural Concrete with Design Applications* (PCA, 2013):

$$V_{sa} = 0.6\alpha_{V,\mathrm{seis}} A_{se,V} f_{uta} \qquad \text{(ACI 318-11 Eq. D-29)}$$

where:

$\alpha_{V,\text{seis}}$ = 0.70, reduction factor per manufacturer's evaluation report

$A_{se,V}$ = 0.226 in.^{2} (PCA Notes Table 34-2)

f_{uta} = 58 ksi (PCA Notes Table 34-1)

V_{sa} = 0.6(0.7)(0.226 in.^{2})(58 ksi) = 5.51 kips × 30 anchors = 165 kips

Adhesive anchor concrete breakout strength is checked in shear per ACI 318-11 Section D.6.2. Since the spacing (12 inches) of the adhesive anchors is greater than the edge distance (9 inches), the evaluation will require the evaluation of two cases as illustrated as Case 1 and Case 2 in ACI 318-11 Figure RD.6.2.1(b). Case 1 is shown in Figure 6-32. Case 2 is evaluated for both concrete breakout without steel reinforcement and with steel reinforcement as shown in Figure 6-33 (Case 2a) and Figure 6-34 (Case 2b), respectively. Since the collector load is primarily parallel to the top edge of the wall and it is loaded near a corner, the concrete breakout is required to be evaluated perpendicular to the top edge of the wall per ACI 318-11 Section D.6.2.1(c) and (d) as shown in Figure 6-35.

Concrete breakout strength is evaluated in shear per ACI 318-11 Section D.6.2 for Case 1 in Figure 6-32. For this case, the collector force is equally distributed to all the anchors and the first anchor closest to the edge is evaluated.

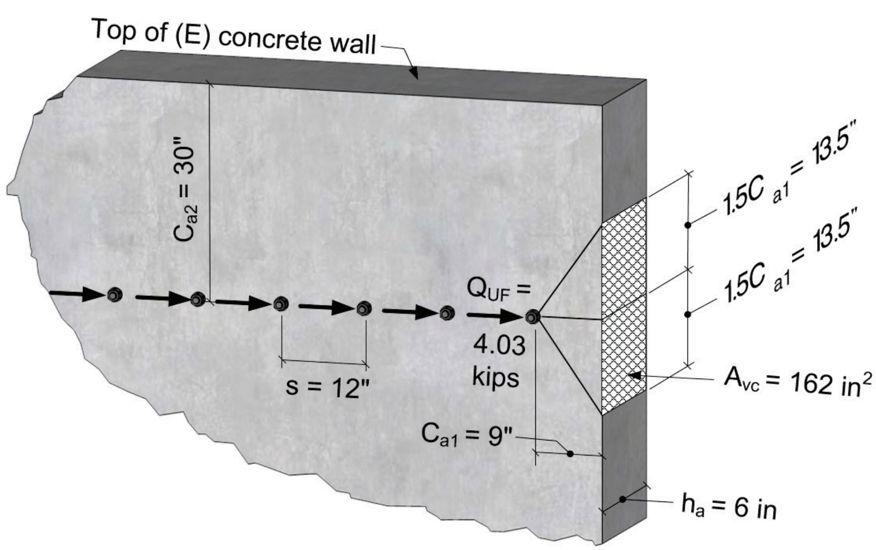


Figure 6-32 Concrete breakout strength for Case 1: Fraction of load distributed to first anchor.

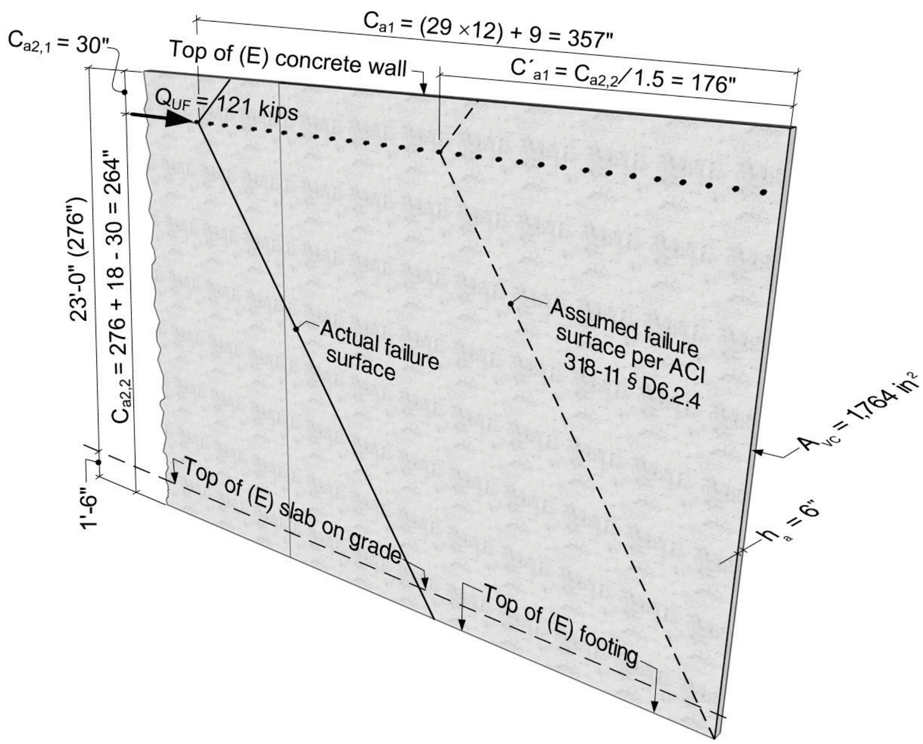


Figure 6-33 Concrete breakout strength for Case 2a: All load resisted by furthest anchor – parallel.

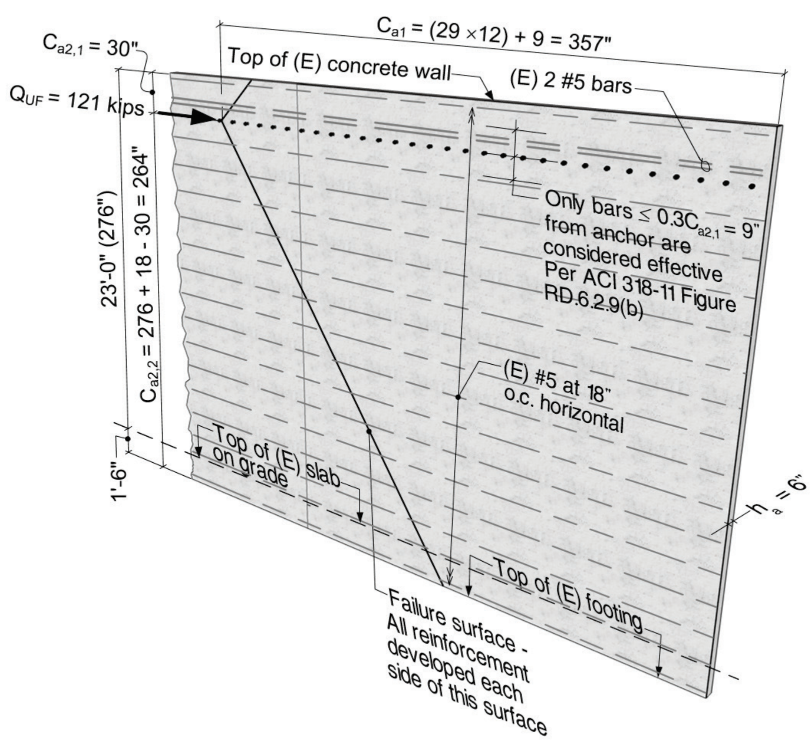


Figure 6-34 Concrete breakout strength for Case 2b: Concrete breakout resisted by horizontal wall reinforcing steel.

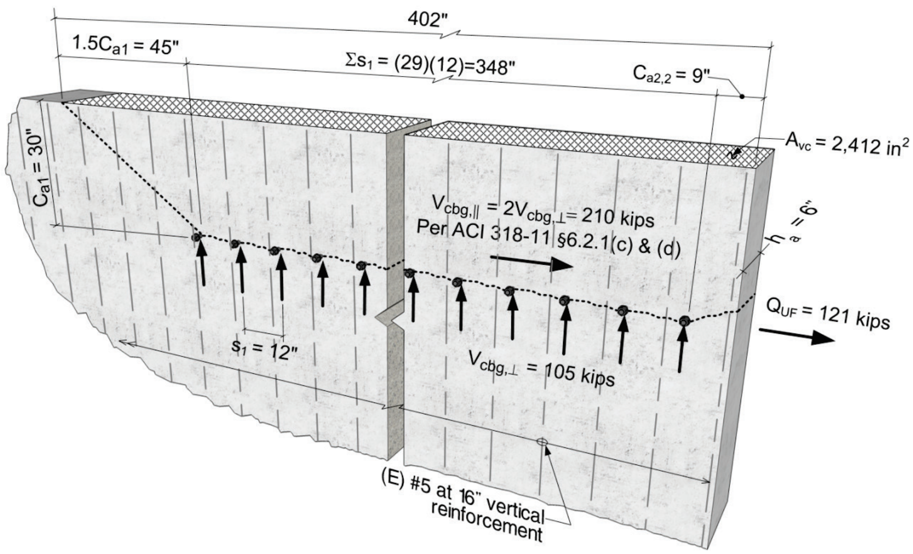


Figure 6-35 Concrete breakout strength perpendicular to load direction.

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad \text{(ACI 318-11 Eq. D-31)}$$

A_{Vc} = (13.5 in. + 13.5 in.)(6) = 162 in.^{2}, See Figure 6-32.

A_{Vco} = $4.5c_{a1}^2$ = 4.5(9 in.)^2 = 365 in.^{2} (ACI 318-11 Eq. D-32)

$\Psi_{ec,V}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-36)

$\Psi_{ed,V}$ = 1.0, no edge effects since $c_{a,2} \geq 1.5c_{a1}$

(ACI 318-11 Eq. D-37)

$\Psi_{c,V}$ = 1.2, cracked concrete and edge reinforcement

$$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} = \sqrt{\frac{1.5(9 \text{ in.})}{(6 \text{ in.})}} = 1.5 \qquad \text{(ACI 318-11 Eq. D-39)}$$

V_b = per ACI 318-11 Section D.6.2.2, smaller of ACI 318-11 Equation D-33 and D-34:

$$= \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} \left(c_{a1}\right)^{1.5} \qquad \text{(ACI 318-11 Eq. D-33)}$$

where:

l_e = h_{ef} = 3.5 inches, anchor length, embedment

d_a = 0.625 inches, anchor diameter

λ_a =1.0, normal weight concrete

f'_c = 3,000 psi

c_{a1} = 9 inches, edge distance

$$= \left(7\left(\frac{3.5\text{ in.}}{0.625\text{ in.}}\right)^{0.2}\sqrt{0.625\text{ in.}}\right)(1.0)$$

$$\times\sqrt{3{,}000\text{ psi}}\,(9\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1000\text{ lbs}}\right)$$

$= 11.6$ kips

and:

$$= 9\lambda_a\sqrt{f_c'}\,(c_{a1})^{1.5} \qquad \text{(ACI 318-11 Eq. D-34)}$$

$$= 9(1.0)\sqrt{3{,}000\text{ psi}}\,(9\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1000\text{ lbs}}\right)$$

$= 13.3$ kips

V_b = min (11.6 kips, 13.3 kips) = 11.6 kips

$$V_{cbg} = \frac{(162\text{ in.}^2)}{(365\text{ in.}^2)}(1.0)(1.0)(1.2)(1.5)(11.6\text{ kips}) = 9.27\text{ kips}$$

Concrete breakout strength in shear is evaluated per ACI 318-11 Section D.6.2 for Case 2a in Figure 6-33. For this case, the entire collector force is resisted by the furthest anchor from the edge to check global group anchor failure in breakout.

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b \qquad \text{(ACI 318-11 Eq. D-31)}$$

A_{Vc} = (30 in. + 264 in.)(6 in.) = 1,764 in.^{2}, See Figure 6-33.

A_{Vco} = $4.5c_{a1}^2$ = 4.5(176 in.)^2 = 139,392 in.^{2}(ACI 318-11 Eq. D-32)

$\Psi_{ec,V}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-36)

$\Psi_{ed,V}$ = $0.7 + 0.3\dfrac{c_{a2}}{1.5c'_{a1}}$, since $c_{a2,1} < 1.5c_{a1}^2$ (ACI 318-11 Eq. D-38)

c_{a2} = 30 inches, smaller of $c_{a2,1}$ and $c_{a2,2}$, see Figure 6-33

c'_{a1} = $c_{a2,2}/1.5$ = 264 in./1.5 = 176 inches, see Figure 6-33

$$\Psi_{ed,V} = 0.7 + 0.3\frac{(30\text{ in.})}{1.5(176\text{ in.})} = 0.73$$

$\Psi_{c,V}$ = 1.2, cracked concrete and edge reinforcement

$$\Psi_{h,V} = \sqrt{\frac{1.5c'_{a1}}{h_a}} = \sqrt{\frac{1.5(176\text{ in.})}{(6\text{ in.})}} = 6.63 \quad \text{(ACI 318-11 Eq. D-39)}$$

V_b = per ACI 318-11 Section D.6.2.2, smaller of ACI 318-11 Equation D-33 and D-34:

$$= \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\left(c'_{a1}\right)^{1.5} \qquad \text{(ACI 318-11 Eq. D-33)}$$

l_e = h_{ef} = 3.5 inches, anchor length, embedment

d_a = 0.625 inches, anchor diameter

λ_a =1.0, normal weight concrete

f'_c = 3,000 psi

c'_{a1} = 176 inches, edge distance, see Figure 6-33

$$= \left(7\left(\frac{3.5\text{ in.}}{0.625\text{ in.}}\right)^{0.2}\sqrt{0.625\text{ in.}}\right)(1.0)\sqrt{3{,}000\text{ psi}} \times (176\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbs}}\right)$$

= 999 kips

$$= 9\lambda_a\sqrt{f'_c}\left(c'_{a1}\right)^{1.5} \qquad \text{(ACI 318-11 Eq. D-34)}$$

$$= 9(1.0)\sqrt{3{,}000\text{ psi}}\,(176\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbs}}\right)$$

= 1,151 kips

V_b = min (999 kips, 1,151 kips) = 999 kips

$$V_{cbg} = \frac{\left(1{,}764\text{ in.}^2\right)}{(139{,}392\text{ in.}^2)}(1.0)(0.73)(1.2)(6.63)(999\text{ kips}) = 73\text{ kips}$$

The concrete breakout strength for Case 2a for global breakout of the anchor group is significantly less than the collector load (Q_{UF} = 121 kips). This evaluation method in ACI 318-11 Section D.6.2.1 does not account for horizontal reinforcing steel in the wall to prevent the concrete breakout along the failure breakout surface shown in Figure 6-33. Furthermore, extending the HSS8×6 further along the length of the wall and adding additional anchors will not result in an increase in concrete breakout strength for Case 2a.

ACI 318-11 Section D.6.2.9 permits a method to evaluate the reinforcing steel to be used instead of the concrete breakout strength provided the reinforcement is developed on each side of the breakout failure surface. The typical #5 reinforcing bars in the wall are easily developed on each side of this breakout failure surface. This method limits the effective width of the reinforcing steel measured from the axis of the bolt group to each side equal to the lesser of $0.5c_{a1}$ and $0.3c_{a2}$. For this example, this results in an effective width of 9 inches each side of this axis as shown in Figure 6-34 and amounts to a pair of #5 bars. This effective width limitation significantly reduces the ability of the designer to engage additional horizontal wall reinforcing steel. As discussed earlier, this limitation does not seem practical for anchor configurations such as that in this example that extends approximately 30 feet along the upper edge of a wall. The calculation below assumes that eight of the eighteen #5 horizontal bars in the wall can be engaged and be effective in preventing failure along the breakout surface. This assumes that the horizontal reinforcing steel in the upper third of the wall is effective, which is clearly a reasonable assumption.

The horizontal wall reinforcing to act as anchor reinforcement is evaluated in accordance with ACI 318-11 Section D.6.2.9 assuming eight #5 bars are effective.

$V_n = A_s F_y$

where:

A_s = 0.31 in.^{2} for #5 bar

F_y = 60 ksi

V_n = (0.31 in.^{2})(60 ksi)(8 bars) = 149 kips

Concrete breakout strength is evaluated in shear perpendicular to the load direction per ACI 318-11 Section D.6.2 as shown in Figure 6-35. For this evaluation, the capacity of concrete breakout in shear perpendicular to the top edge of the concrete panel is computed and the capacity of concrete breakout parallel to this edge is equal to twice the perpendicular breakout capacity per ACI 318-11 Section D.6.2.1(c) and (d).

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad \text{(ACI 318-11 Eq. D-31)}$$

where:

A_{Vc} = (402 in.)(6 in.) = 2,412 in.^{2}, See Figure 6-35

$A_{Vco} = 4.5c_{a1}^2 = 4.5(30 \text{ in.})^2 = 4{,}050 \text{ in.}^2$ (ACI 318-11 Eq. D-32)

$\Psi_{ec,V}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-36)

$\Psi_{ed,V}$ = 1.0 per ACI 318-11 § D6.2.1(c) (ACI 318-11 Eq. D-38)

$\Psi_{c,V}$ = 1.2, cracked concrete and edge reinforcement

$$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} = \sqrt{\frac{1.5(30 \text{ in.})}{(6 \text{ in.})}} = 2.74 \qquad \text{(ACI 318-11 Eq. D-39)}$$

V_b = per ACI 318-11 § D.6.2.2, smaller of ACI 318-11 Equation D-33 and D-34:

$$= \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\left(c_{a1}\right)^{1.5} \qquad \text{(ACI 318-11 Eq. D-33)}$$

where:

l_e = h_{ef} = 3.5 inches, anchor length, embedment

d_a = 0.625 inches, anchor diameter

λ_a =1.0, normal weight concrete

f'_c = 3,000 psi

c_{a1} = 30 inches, edge distance, see Figure 6-35

$$= \left(7\left(\frac{3.5\text{ in.}}{0.625\text{ in.}}\right)^{0.2}\sqrt{0.625\text{ in.}}\right)(1.0)\sqrt{3{,}000\text{ psi}} \times (30\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbs}}\right)$$

= 70.3 kips

$$= 9\lambda_a\sqrt{f'_c}\left(c_{a1}\right)^{1.5} \qquad \text{(ACI 318-11 Eq. D-34)}$$

$$= 9(1.0)\sqrt{3{,}000\text{ psi}}\,(30\text{ in})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbs}}\right)$$

= 81.0 kips

V_b = min (70.3 kips, 81.0 kips) = 70.3 kips

$$V_{cbg} = \frac{\left(2{,}412\text{ in.}^2\right)}{(4{,}050\text{ in.}^2)}(1.0)(1.0)(1.2)(2.74)(70.3\text{ kips}) = 138\text{ kips}$$

Per ACI 318-11 Section D.6.2.1 Bullets (c) and (d), the concrete breakout strength parallel to the top edge of the concrete panel is equal to twice the perpendicular breakout capacity.

$$V_{cbg,||} = 2V_{cbg,\perp} = 2(138\text{ kips}) = 276\text{ kips}$$

In lieu of calculating the concrete breakout strength, ACI 318-11 Section D.6.2.9 permits calculating the vertical wall reinforcment strength passing through and developed on each side of the breakout surface shown in Figure 6-35. As shown in Table 6-3, the concrete breakout strength perpendicular to the load direction is adequate for the collector loads, so there was no need to evaluate the vertical wall reinforcing steel at the breakout surface.

The adhesive anchor is evaulated for concrete pryout strength in shear per ACI 318-11 Section D.6.3. The evaluation of concrete pryout strength in shear requires the calculation of both the concrete breakout stength of the anchors in tension per ACI 318-11 Section D.5.2 and the bond strength of adhesive anchors in tension per ACI 318-11 Section D.5.5 and the lesser governs. The concrete pryout strength in shear for a group of anchors is computed as follows per ACI 318-11 Section D.6.3:

$V_{cpg} = k_{cp}N_{cpg}$ (ACI 318-11 Eq. D-41)

where:

k_{cp} = 2.0 for h_{ef} > 2.5 inches

N_{cpg} = the lesser of concrete breakout and bond strength:

$= \dfrac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (ACI 318-11 Eq. D-4)

$= \dfrac{A_{Na}}{A_{Nao}}\psi_{ec,Na}\psi_{ed,Na}\psi_{cp,Na}N_{ba}$ (ACI 318-11 Eq. D-19)

Calculate the adhesive anchor concrete breakout strength in tension per ACI 318-11 Section D.5.2 per Figure 6-36:

$N_{cbg} = \dfrac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$ (ACI 318-11 Eq. D-4)

where:

A_{Nco} = $9h_{ef}^2$ = 9(3.5 in.)^2 = 110 in.^{2} (ACI 318-11 Eq. D-5)

A_{Nc} = nA_{Nco} = 30(110 in.^{2}) = 3,300 in.^{2}, see Figure 6-36, since the projected areas do not overlap with adjacent anchors or edges, A_{Nc} is equal to the projected area of one anchor times the number of anchors, n

$\Psi_{ec,N}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-8)

$\Psi_{ed,N}$ = 1.0, no edge effects since $c_{a,\min} \geq 1.5h_{ef}$ (ACI 318-11 Eq. D-9)

$\Psi_{c,N}$ = 1.0, cracked concrete

$\Psi_{cp,N}$ = 1.0, no splitting effects $c_{a,\min} \geq c_{ac}$ (ACI 318-11 Eq. D-11)

N_b = $k_c\lambda_a\sqrt{f'_c}h_{ef}^{1.5}$ (ACI 318-11 Eq. D-6)

where:

k_c = 17, post installed anchor in cracked concrete per "Concrete Breakout Design Information" Table in product evaluation report

λ_a = 1.0, normal weight concrete

f'_c = 3,000 psi

h_{ef} = 3.5 inches embedment

$$N_b = 17(1.0)\sqrt{3{,}000\text{ psi}}\,(3.5\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbs}}\right) = 6.10\text{ kips}$$

$$N_{cbg} = \frac{(3{,}300\text{ in.}^2)}{(110\text{ in.}^2)}(1.0)(1.0)(1.0)(1.0)(6.10\text{ kips}) = 183\text{ kips}$$

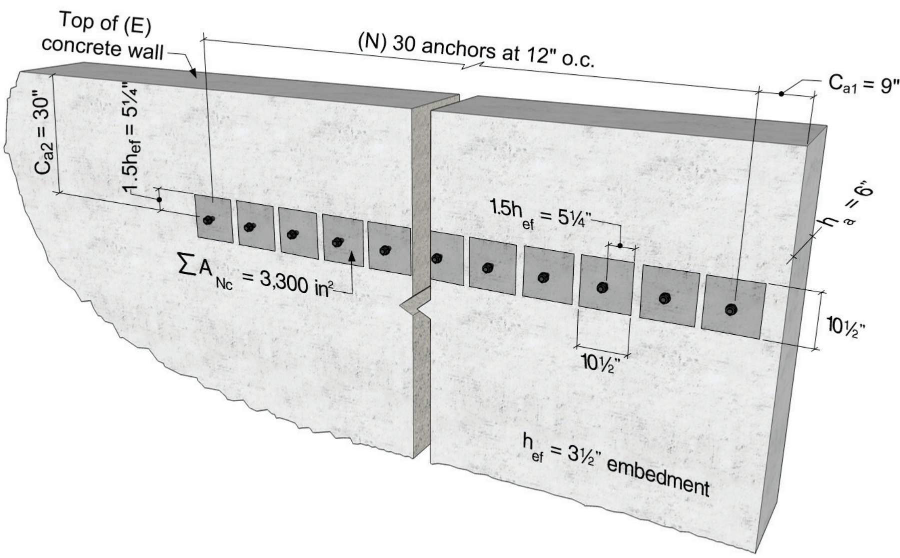


Figure 6-36 Concrete breakout strength in tension – projected area.

Calculate the adhesive anchor bond strength in tension per ACI 318-11 Section D.5.5 per Figure 6-37:

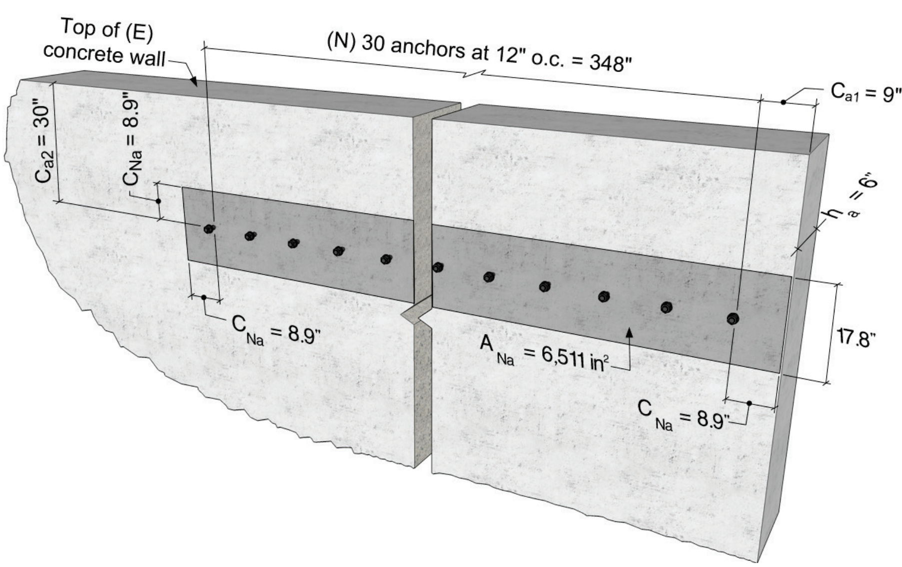


Figure 6-37 Adhesive anchor bond strength in tension – projected area.

$$N_{ag} = \alpha_{N,seis}\frac{A_{Na}}{A_{Nao}}\psi_{ec,Na}\psi_{ed,Na}\psi_{cp,Na}N_{ba} \qquad \text{(ACI 318-11 Eq. D-19)}$$

where:

$\alpha_{N,seis}$ = 0.75, reduction factor per manufacturer's evaluation report

c_{Na} = $10d_a\sqrt{\frac{\tau_{uncr}}{1,100}}$ (ACI 318-11 Eq. D-21)

where:

d_a = 0.625 inches, diameter of anchor

τ_{uncr} = 2,220 psi, uncracked concrete, temperature range A per the Bond Strength Design Information table in product evaluation report (c_{Na} is always calculated based on uncracked concrete per ACI 318-11 Section RD.5.5.1)

c_{Na} = $10(0.625\text{ in.})\sqrt{\frac{2,220\text{ psi}}{1,100}}$ = 8.9 inches

A_{Nao} = $(2c_{Na})^2 = (2(8.9\text{ in.}))^2 = 317\text{ in.}^2$ (ACI 318-11 Eq. D-20)

A_{Na} = (17.8 in.)(8.9 in. + 348 in. + 8.9 in.) = 6,511 in.^{2}, see Figure 6-37

$\Psi_{ec,Na}$ = 1.0, no eccentricity on bolt group (ACI 318-11 Eq. D-23)

$\Psi_{ed,Na}$ = 1.0, no edge effects since $c_{a,\min} \geq c_{Na}$ (ACI 318-11 Eq. D-24)

$\Psi_{cp,Na}$ = 1.0, no splitting effects $c_{a,\min} \geq c_{ac}$ (ACI 318-11 Eq. D-26)

N_{ba} = $\lambda_a\tau_{cr}\pi d_a h_{ef}$ (ACI 318-11 Eq. D-22)

where:

λ_a, d_a, h_{ef} as indicated above.

τ_{cr} = 1,185 psi, cracked concrete, temperature range A per the Bond Strength Design Information table in product evaluation report since wall is assumed cracked as discussed above.

N_{ba} = (1.0)(1,185 psi)(3.14)(0.625 in.)(3.5 in.)(1 kip/1,000 lbs) = 8.14 kips

$$N_{ag} = (0.75)\frac{\left(6,511\text{ in.}^2\right)}{(317\text{ in.}^2)}(1.0)(1.0)(1.0)(8.14\text{ kips}) = 125\text{ kips}$$

Since the concrete bond strength, N_{ag}, is less than the concrete breakout strength, N_{cbg}, bond strength governs. Calculate concrete pryout strength in shear as follows:

$$V_{cpg} = k_{cp}N_{cpg} \qquad \text{(ACI 318-11 Eq. D-41)}$$

where:

k_{cp} = 2.0 for h_{ef} > 2.5 inches

N_{cpg} = the lesser of:

where:

N_{cbg} = 183 kips

N_{ag} = 125 kips

$$V_{cpg} = (2.0)(125 \text{ kips}) = 250 \text{ kips}$$

The adhesive anchor design results are summarized in Table 6-3 for the acceptance criteria indicated below for force-controlled components with κ determined in Section 6.6.1 of this *Guide*.

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

The adhesive anchors are adequate for all design checks in accordance with ACI 318-11 Appendix D.

6.6.5.8 Additional Considerations for Evaluating Adhesive Anchors Connecting HSS to Concrete Wall

Collectors with large numbers of anchors pose some design challenges that are beyond the model assumptions for anchors in ACI 318-11 Appendix D, which were evaluated in the previous section. A proper assessment to address those additional design challenges of the collector anchorage capacity includes: (1) the potential for breakout of the anchors closest to the leading edge; (2) the potential for pryout of the anchors; (3) a reasonable estimate of the degree of shear lag at ultimate; and (4) the interaction of the wall reinforcement with the collector to preclude concrete breakout. A useful analogy for collector response is the behavior of a tension-loaded deformed reinforcing bar, whereby concrete fracture and crushing at each lug closest to the applied load enables transfer of load to propagate over the bar development length. For a collector connected with anchors to a concrete wall, this analogy translates to the formation of diagonal cracks at each anchor location and crushing and spalling of the concrete that leads to softening of the anchor response and distributing the applied collector load with a non-linear distribution over the collector length. For the collector in this example, Figure 6-38 illustrates this behavior where anchors nearest the applied load would have local concrete cracking and spalling and be loaded at or near the anchor steel capacity and the anchors furthest would be less engaged and have less load applied.

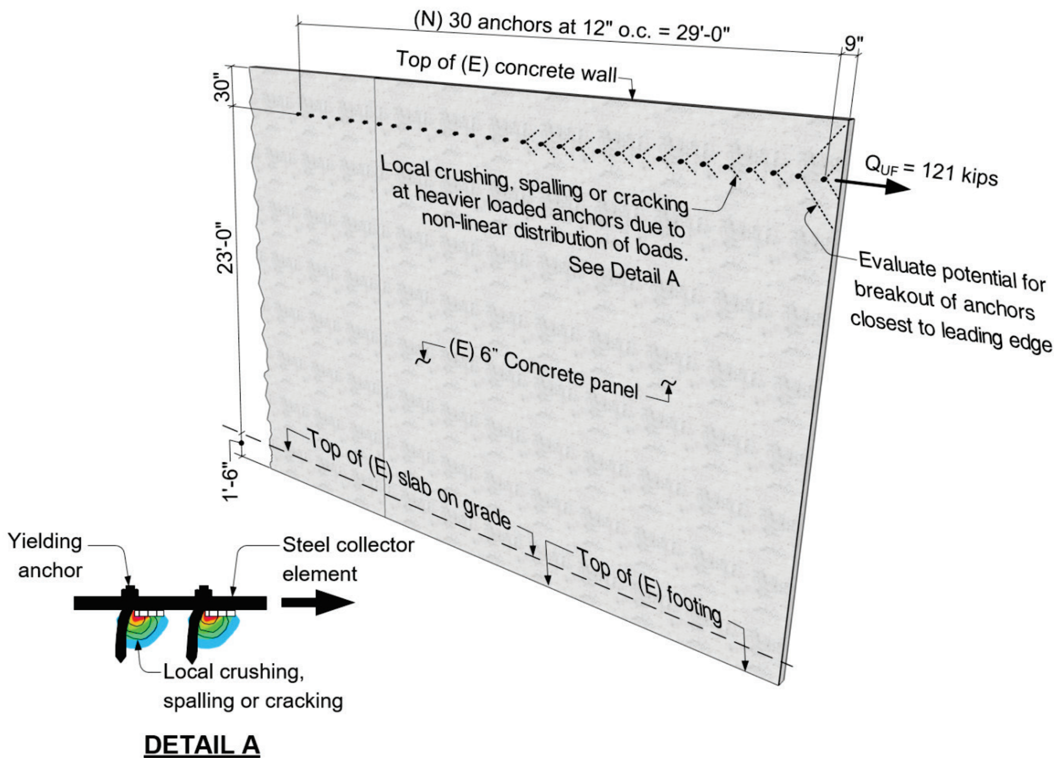


Figure 6-38 Additional considerations for evaluating collector connection.

Following the order indicated above, the first step is to evaluate the concrete breakout strength of the anchors near the leading edge to determine if these anchors need to be excluded from the analysis. This is determined by comparing the concrete breakout strength of the anchor to its steel strength. If the steel strength is greater than the concrete breakout strength, then the anchor may fail in concrete breakout as the collector elongates and overloads the leading anchors. This evaluation is continued for each subsequent anchor from the leading edge until the steel strength is less than the concrete breakout strength, which would indicate the point at which the anchors will deform in a ductile manner and permit the collector force to be distributed to subsequent anchors further from the leading edge. For this example, the first anchor is located 9 inches from the leading edge of the wall and the steel strength and concrete breakout strength (Figure 6-32) for this anchor was determined in the previous subsection as follows:

V_{sa} = 5.51 kips (anchor steel strength)

V_{cbg} = 9.27 kips (concrete breakout strength at 9 inches edge distance)

Since $V_{sa} < V_{cbg}$, steel yielding mode controls and the first anchor can be included to resist the collector force.

The second step is to evaluate the concrete pryout strength of the anchors to ensure that it exceeds the steel strength of the anchor so the anchors will deform in a ductile manner and distribute the collector force to other anchors. The concrete pryout strength (Figure 6-36 and Figure 6-37) for the anchors was determined in the previous section as follows:

V_{sa} = 5.51 kips (anchor steel strength)

V_{cpg} = 250 kips / 30 anchors = 8.33 kips (concrete pryout strength)

Since $V_{sa} < V_{cpg}$, steel yielding mode controls and collector force can be distributed to other anchors.

The third step is to determine a reasonable estimate of the degree of shear lag that will occur as the collector elongates and engages the concrete anchors in a nonlinear manner. This analysis can be performed in a variety of ways from a complex nonlinear finite element analysis to a simple deformation or strength compatibility analysis where a reasonable amount of engineering judgement is needed. This example evaluates the load distribution to the anchors with a capacity-based approach where the leading anchors are fully yielded and anchors further from the applied load are loaded with a linear load distribution. The purpose of determining this distribution is to further evaluate the load transfer into the concrete wall for concrete breakout.

The distribution of load along the length of the collector is split into two segments, one segment near the leading edge of the wall where it is assumed that all the anchors are fully yielded and a second segment near the end of the collector where the load in each anchor is linearly decreasing to zero. This is illustrated in Figure 6-39.

The following equation is the summation of the anchor strengths in each segment equated to the collector load.

$$Q_{UF} = \frac{V_{\text{avg}} L_2}{s} + \frac{V_{sa} L_1}{s}$$

where:

V_{sa} = anchor steel strength = 5.51 kips

V_{avg} = average anchor force in Segment 2 = $V_{sa}/2$

L = length of collector = 357 inches

L_1 = length of collector in Segment 1

L_2 = length of collector in Segment 2 = L - L_1

s = spacing of anchors = 12 inches

Q_{UF} = applied collector load = 121 kips

Substituting V_{avg} and L_2 into the equation for P and solving for L_1 yields:

$$L_1 = \frac{2sQ_{UF}}{V_{sa}} - L = \frac{2(12\text{ in.})(121\text{ kips})}{5.51\text{ kips}} - 357\text{ in.}$$

L_1 = 170 inches = 14.2 ft

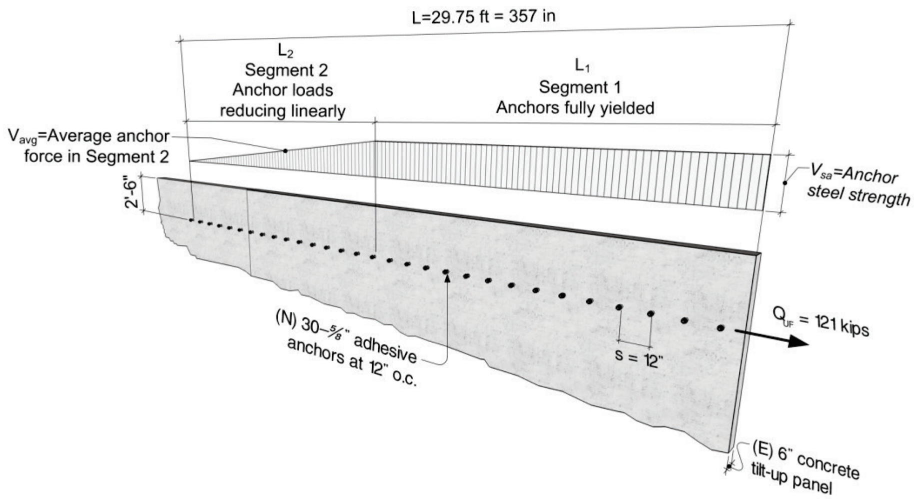


Figure 6-39 Evaluating load distribution in collector.

As a result, at approximately the halfway point along the collector length is the transition between the two segments illustrated in Figure 6-39, which results in approximately two-thirds of the collector force dissipating into the wall within the first segment.

In the previous subsection where the anchors were evaluated with the ACI 318-11 Appendix D, the distribution of the load into the wall was assumed uniform over the entire collector length for evaluating concrete breakout both parallel and perpendicular to the wall. In this subsection, the concrete breakout of the wall is evaluated for a nonlinear distribution of load from the collector into the wall, which is the fourth step outlined earlier.

The concrete breakout for loading perpendicular to leading edge (end of the wall) was evaluated in the previous subsection as illustrated in Figure 6-34 where eight #5 horizontal bars were needed to resist the collector load. This calculation is not repeated here, as the collector will easily engage eight #5 bars over half the collector length. For the configuration in this example, concrete breakout at the top edge of the wall may be critical due to the proximity of the collector to the top of the wall. So concrete breakout parallel to the load direction is evaluated at the top edge of the wall in Segment 1 as illustrated in Figure 6-40.

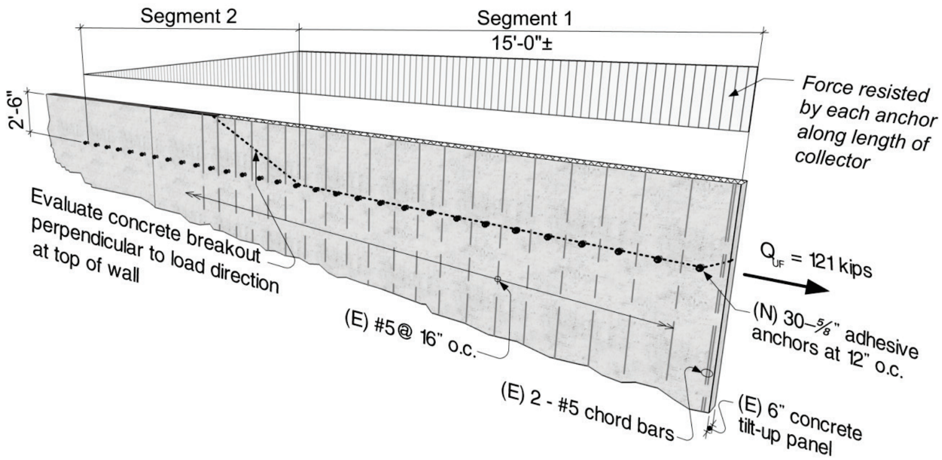


Figure 6-40 Evaluating concrete breakout at the top of the concrete wall.

The demand from the collector in Segment 1 is calculated as follows:

$$Q_{UF1} = \frac{V_{sa}L_1}{s}$$

where:

V_{sa} = anchor steel strength = 5.51 kips

L_1 = length of collector in Segment 1 = 15 feet

s = spacing of anchors = 12 inches

Q_{UF1} = applied collector load in Segment 1

$$Q_{UF1} = \frac{(5.51 \text{ kips})(15 \text{ ft})(12 \text{ in./ft})}{12 \text{ in.}} = 83 \text{ kips}$$

The vertical wall reinforcing is evaluated as anchor reinforcement in accordance with ACI 318-11 Section D.6.2.9.

$$V_n = nA_sF_y$$

where:

n = number of bars developed on each side of breakout surface = (15 ft)(12 in./ft)/16 in. = 11 bars

A_s = 0.31 in.^{2} for #5 bar

F_y = 60 ksi

$$V_n = (11)(0.31 \text{ in.}^2)(60 \text{ ksi}) = 205 \text{ kips}$$

As permitted in ACI 318-11 Section D.6.2.1 Item c, the breakout capacity is permitted to be twice that computed at the top of the wall since it recognizes that the transverse splitting strain is approximately 50% of that in the direction of the applied collector load. For this calculation, rather than doubling the breakout capacity provided by the reinforcing steel, the collector demand in Segment 1 is reduced in half to represent the effective load applied towards the top edge of the wall.

$Q_{UF1}/2 \leq V_n$

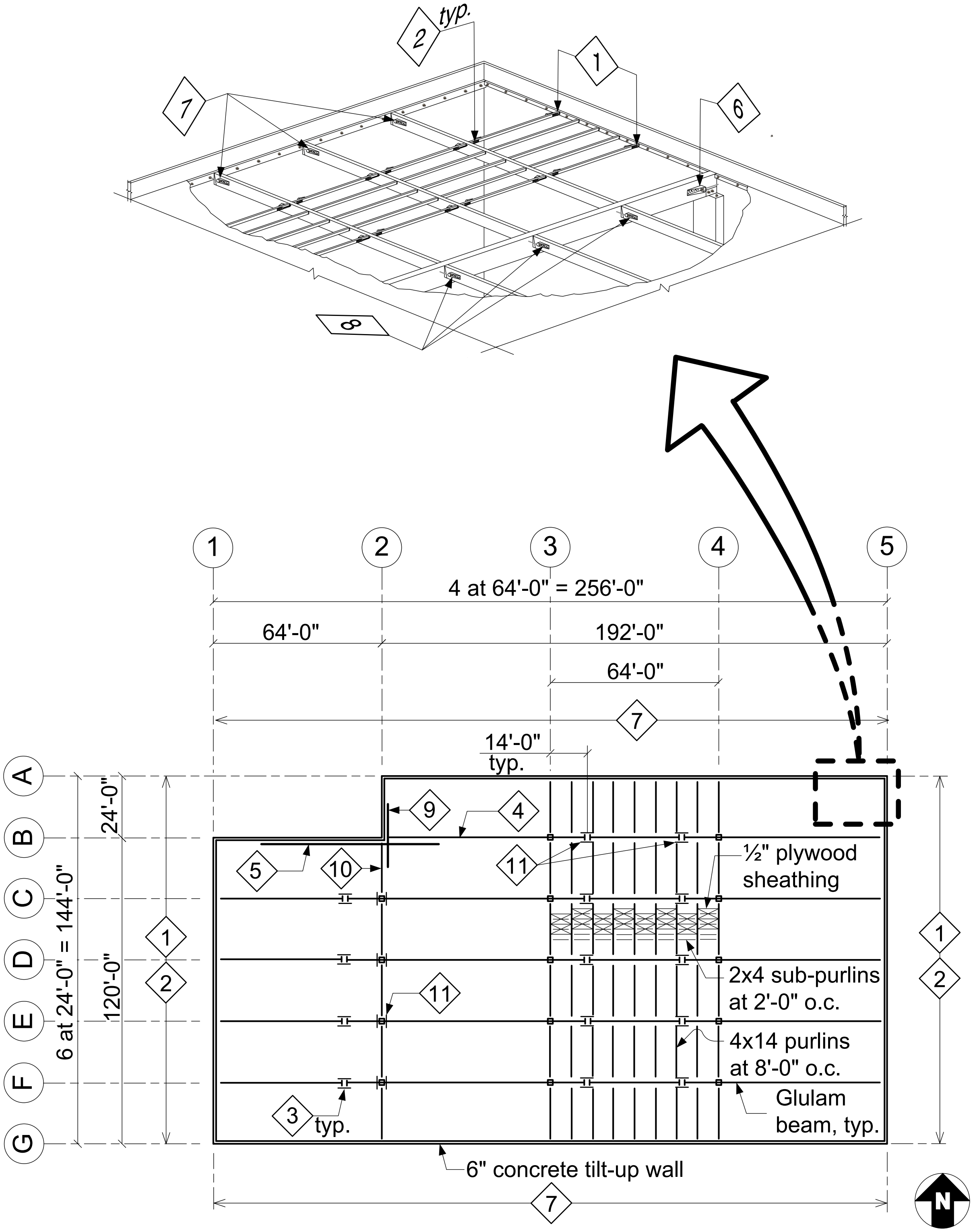
83 kips/2 = 42 kips < 205 kips; therefore, concrete breakout at the top of the wall is adequate.

This subsection presented some additional consideration for the evaluation of collector connections with a large number of anchors to concrete walls. The failure modes and procedures presented are intended to help provide a better understanding of the mechanics involved in these type of connections. There may need to be additional considerations on other anchor configurations that were not presented, such as wall splitting.

6.6.6 Summary of Tier 2 Retrofit

The following summarizes the Tier 2 retrofit measures evaluated in this example followed by additional items that were not evaluated. Each item is keyed into Figure 6-41 in a diamond symbol:

1. Install two adhesive anchors, connection hardware, and sistered sub-purlins at 8 ft o.c. on east and west exterior tilt-up walls per Figure 6-9.
2. Install continuity connection hardware and sistered sub-purlins at 8 ft o.c. for four bays from the east and west exterior tilt-up walls per Figure 6-15.
3. Install continuity strap plates on each side of glulam beam splices at Gridlines C, D, E, and F per Figure 6-23.
4. Install bottom flange bracing at 16 ft o.c. to glulam collector beam on Gridline B and at 24 ft o.c. to glulam cross-tie beam on Gridlines C through F.
5. Install collector-to-wall connection at Gridline B where it intersects Gridline 2 per Figure 6-27 and Figure 6-31.



Roof framing plan of tilt-up building

Figure 6-41 Tier 2 retrofit summary (see text for keynote descriptions).

The following items were not evaluated or addressed in this example but should be included in a complete retrofit. These items are also keyed into Figure 6-41 in a diamond symbol.

6. Installation of adhesive anchors and connection hardware at all glulam beam to wall and pilasters on east and west exterior tilt-up walls
7. Installation of adhesive anchors and connection hardware at purlins at 8 ft o.c. on north and south exterior tilt-up walls
8. Installation of continuity cross-tie connection hardware at purlin splices at glulam beams
9. Installation of collector-to-wall connection at Gridline 2 where it intersects Gridline B as illustrated in Figure 6-27
10. Strengthening of purlin at Gridline 2 for collector loading
11. Installation of collector strap plates on each side of glulam beam splices at Gridline B and at purlin splices at Gridline 2

As discussed in Section 6.3 of this *Guide*, at the conclusion of the Tier 2 deficiency-based retrofit, the strengthened building should be evaluated to confirm that the strengthened building complies with the intended Tier 1 and Tier 2 Performance Objectives to ensure that the strengthening did not simply shift any deficiencies to another critical component. For the example tilt-up building presented, the strengthening to the out-of-plane wall anchorage systems, sub-diaphragms, and collectors did not alter the overall seismic force resisting system's load path in such a way that would cause components previously screened in the Tier 1 evaluation, that were found compliant, to now become non-compliant. As a result, there is no need to reassess the structure with the Tier 1 and Tier 2 procedures.

Chapter 7

Wood Tuck-Under (W1a)

7.1 Overview

This chapter provides discussion and example application of the Tier 3 Systematic Evaluation and Retrofit procedures of ASCE 41-13 (ASCE, 2014) on an apartment building located in Northern California. The building is of a very common type of multi-family, residential construction built in the western U.S. during the 1960s known as the *tuck-under*. Tuck-under buildings are multi-story, wood frame construction with an open area for parking in the lowest story. This feature, coupled with less stringent design requirements at the time, creates a serious seismic deficiency, namely a weak story condition. In addition, shear walls on the front face (over the parking entrance) are typically discontinuous. Tuck-under buildings with these deficiencies have performed poorly in past earthquakes, and are well-known in the structural engineering community to be a major seismic vulnerability of the existing residential building stock. Standards other than ASCE 41-13 are available to the designer, and the selection of which standard is most appropriate is beyond the scope of this chapter. For instance, the Appendix Chapter A4 of the *International Existing Building Code* (IEBC) (ICC, 2015) has been developed to provide semi-prescriptive retrofit guidelines for this type of building, which is referred to as a Soft Weak Open Front (SWOF) building. Also, FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories* (FEMA, 2012b), presents a probability-based evaluation and retrofit methodology for this type of building. This chapter addresses only ASCE 41-13 provisions, but shear wall strength and stiffness properties from FEMA P-807 are incorporated. In addition, the article on "Anchor Bolts in Light-Frame Construction at Small Edge Distances" in SEAOC Blue Book (SEAOC, 2009) presents valuable information that should be reviewed.

> **Example Summary**
>
> **Building Type:** W1a
>
> **Performance Objective:** BPOE and Partial Retrofit
>
> **Risk Category:** II
>
> **Location:** San Jose, California
>
> **Level of Seismicity:** High
>
> **Analysis Procedure:** Linear Static (LSP)
>
> **Retrofit Procedure:** Tier 3
>
> **Reference Documents:**
>
> SDPWS-2008
>
> 2011 AISC Steel Construction Manual
>
> 2013 AISC Seismic Design Manual
>
> FEMA P-807

The example structure is based on an archetype of a typical 1960s vintage apartment building in Northern California presented in Rutherford + Chekene (2000) and corresponds to the Type 1 Model in the referenced report. This example will demonstrate how the weak story condition would be identified in a Tier 1 screening (using the checklists and Quick Checks), and how a subsequent Tier 3 comprehensive evaluation and retrofit of the entire building to Basic Performance Objective for Existing Buildings (BPOE) could be done.

The following evaluation and retrofit aspects are included in this example:

- Because the building was designed and constructed in the 1960s and record drawings are not available, it cannot be considered a Benchmark Building per ASCE 41-13 § 4.3 and therefore not exempt from these requirements for evaluation and retrofit.
- Identification of appropriate Performance Objectives and Target Building Performance Levels (ASCE 41-13 § 2.2, § 2.3).
- Calculation of the seismic BSE-1E and BSE 2E uniform hazard spectra using the General Procedure and determination of the Level of Seismicity (ASCE 41-13 § 2.4 and § 2.5).
- Tier 1 screening including identification of the weak story deficiency through the Tier 1 checklist (ASCE 41-13 § 4.4) and Quick Checks of shear wall loads and building drifts using default material properties and Tier 1 static loading (ASCE 41-13 § 4.5).
- Two evaluations are included in this example: (1) the existing structure in order to estimate the required strength to be added at each story; and (2) the retrofit structure to demonstrate that the acceptance criteria are satisfied.
- Identification of the appropriate Tier 3 analysis method(s) as specified in ASCE 41-13 Chapter 6 and Chapter 7.
- Selection of appropriate strengths, stiffnesses, and acceptance criteria for the existing shear walls (stucco, drywall, and plywood) and retrofit wood structural panels and hold-downs. The requisite strength and stiffness of the retrofit moment frames are determined, but the requirements of ASCE 41-13 Chapter 12 are not included.
- Sizing of retrofit shear walls, hold-downs, and moment frames to satisfy the Tier 3 Acceptance Criteria.

Commentary

Although a Limited Performance Objective for mitigating only the weak story vulnerability might be appropriate for this type of building, a Tier 2 deficiency-based retrofit (ASCE 41-13 Chapter 5) of the torsional irregularity and discontinuous shear walls is not addressed in this example. See Section 3.2 of this *Guide* for an expanded discussion on Performance Objectives.

The following aspects of a full Tier 3 evaluation and retrofit are not included:

- The Tier 3 evaluation and retrofit is informed by a Tier 1 checklist and quick check evaluation, which identifies the torsional irregularity and discontinuous shear walls, but a full Tier 1 checklist evaluation is not presented (ASCE 41-13 § 4.2).
- Nonstructural Life Safety Performance Objective of ASCE 41-13 Table 2-1.

- Full design and detailing of the retrofit steel moment frames.
- Check of the soil bearing capacities under the shallow foundations and potential soil improvement options (ASCE 41-13 Chapter 8).

7.1.1 Performance Objective

The building is classified as Risk Category II and is to be evaluated/retrofit to the Basic Performance Objective for Existing Buildings, BPOE, as defined in ASCE 41-13 § 2.2.1. Per ASCE 41-13 Table 2-1, the Performance Objectives for a Tier 3 evaluation of a Risk Category II structure are twofold: First, for BSE-1E loading (20% in 50 years), the Performance Level must be at least 3-C, that is, Life Safety Structural Performance and Life Safety Nonstructural Performance. Second, for BSE-2E (5% in 50 years), the Structural Performance Level must be at least 5-D, that is, Collapse Prevention Structural Performance. Nonstructural performance is not evaluated at the 5D Performance Level. In addition, the Tier 1 screening checklists and Quick Checks of ASCE 41-13 § 4.5 are to be based on the Life Safety Performance Level. The design spectra associated with the BSE-1E and BSE-2E hazards for this building are discussed below.

7.2 General Building Description

The subject building is modeled after a typical three-story, multi-family, residential structure with two occupied stories over an at-grade ground story that provides both parking area and additional living space. The building is 100 feet long (east-west) by 36 feet wide (north-south). Access to the upstairs units is provided by a light-framed exterior access walkway. (Although this walkway is not included in the model or structural checks, its construction details and attachment to the structure would need to be checked as part of a Tier 3 evaluation.) Figure 7-1 shows a rendering of the building and the open front wall; Figure 7-2 shows the plan of the bottom level (L1); Figure 7-3 shows the plan for the second and third levels (L2, L3); and Figure 7-4 shows a cross section. (The figures show only the walls considered in the evaluation of the existing building. As described in Section 7.5.3 of this *Guide*, walls less than 4 feet long do not contribute significant strength or stiffness and are not shown.) Figure 7-5 shows the deficiencies identified in Sections 7.1 and 7.4 of this *Guide*.

7.2.1 Destructive Evaluation

The original construction drawings were not available for this building, and therefore the construction materials and details described below are based on site inspections. Absent construction documents, comprehensive condition assessment requirements are described in ASCE 41-13 § 12.2.3.2.2.2. Finish

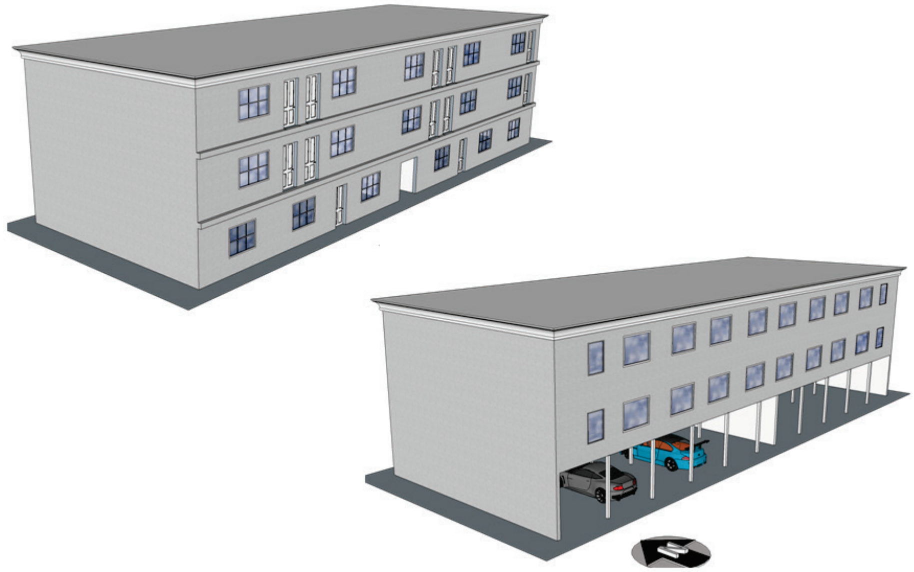


Figure 7-1 Isometric views of structure. Balconies, walkways, stairs and other architectural building elements are not shown in order for the reader to have a clear view of the wall openings and discontinuities in the exterior building elements.

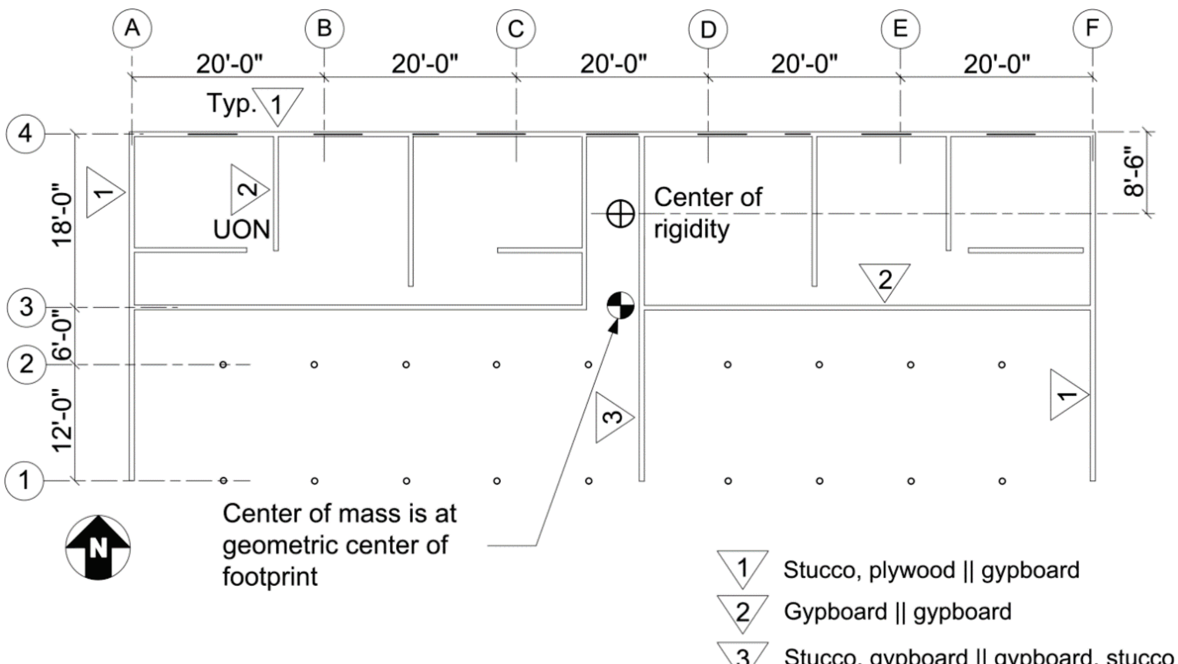


Figure 7-2 Plan of parking level (L1). (See Table 7-4 for wall types) The wall description in the legend lists the sheathing materials from outside to inside face; sheathing on the inner and outer faces are separated by "||".

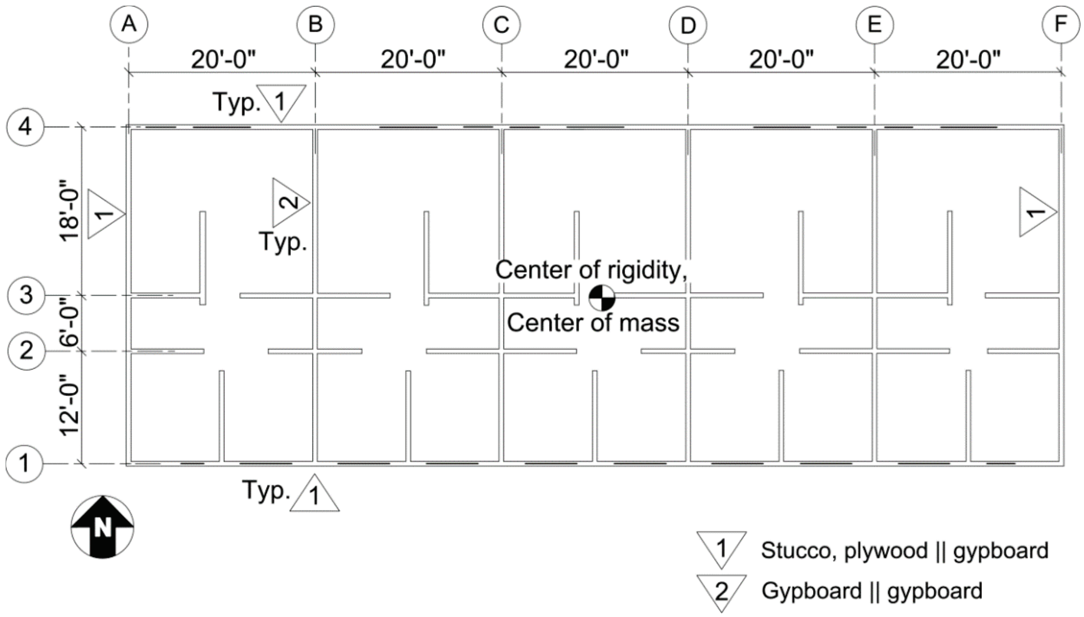


Figure 7-3 Plan of second and third stories (L2 and L3). (See Table 7-4 for wall types.) The wall description in the legend lists the sheathing materials from outside to inside face; sheathing on the inner and outer faces are separated by "||"

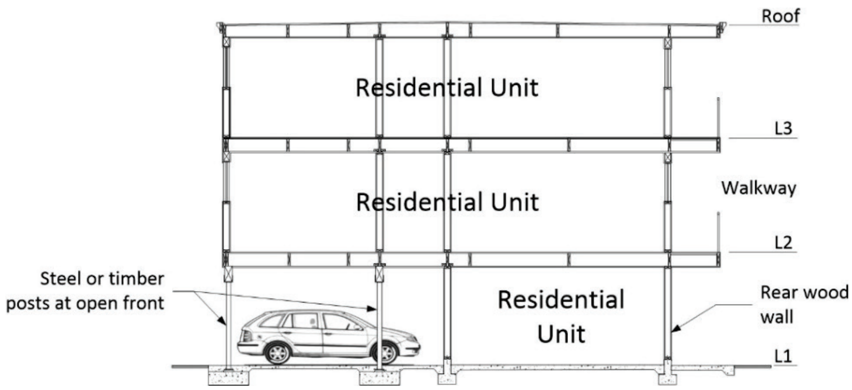


Figure 7-4 Transverse cross section.

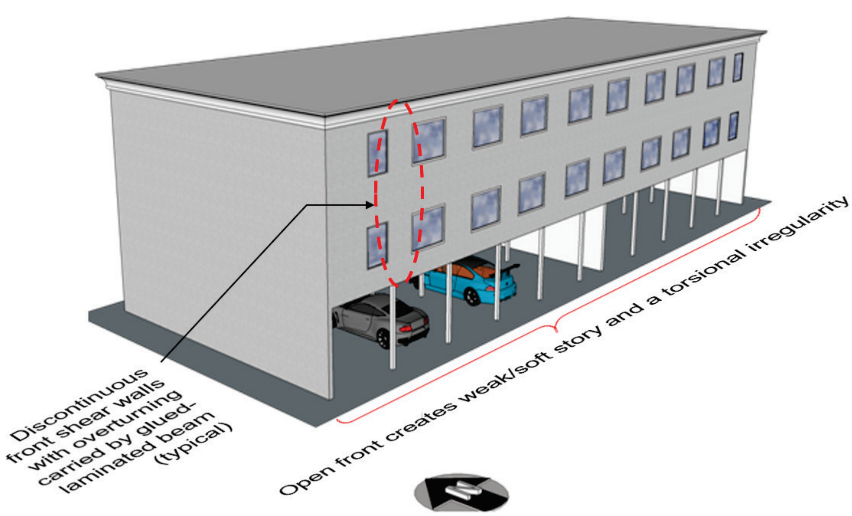


Figure 7-5 Isometric showing major deficiencies.

removal is required to expose 50% of each type of vertical element to expose connections to diaphragms as well as tie-downs and collectors (none found in this case). The materials and construction details were found to be consistent throughout the building; otherwise, additional openings would have been required to better understand the variations. Per ASCE 41-13 § 12.2.4.1, given this level of inspection, a knowledge factor, κ, of 1.0 is permitted for wood construction without requiring the material testing of ASCE 41-13 § 12.2.2.5.

Destructive evaluation of 50% of each existing primary load-resisting element would be very intrusive, requiring substantial effort. In this example, the 50% requirement only applies to the existing primary elements that will be utilized in the ultimate retrofit; one does not need to open 50% of all existing elements. Looking ahead, three fundamental categories of existing plywood walls will be utilized as primary elements in the retrofit: exterior north-south plywood shear walls on the second and third stories and all of the existing plywood shear walls on the front and back walls.

The destructive openings include (note that these categories are not related to the shear wall schedule provided later):

- **Category 1.** Exterior north-south plywood shear walls. There are four of these walls (east and west elevations at second and third stories) because the bottom story walls will be fully exposed during retrofit and need not be opened for inspection; two openings are therefore required. Openings will include top and bottom of wall to document connections to plates and diaphragm, check for presence of hold-downs, and remove sufficient floor finish to verify the diaphragm sheathing and nailing.
- **Category 2.** Exterior east-west shear walls at the back. Three shear walls that extend full height of the second and third stories for a total of six walls, so three openings are required. At the first story, the wall lengths change and door openings perforate two of the walls; one of those will be opened to verify the detailing.
- **Category 3.** Exterior plywood shear walls on the front wall above the garage opening. There are eleven solid sections of wall that extend the entire height of the second and third stories, for a total of 22 walls of this category (as modeled). However, making eleven openings to verify this wall construction would be extreme. A reasonable interpretation of ASCE 41-13 requirements would be to treat these walls as single, perforated shear walls on each story. If so, only one opening on each floor would satisfy the minimum requirements of ASCE 41-13 § 12.2.3.2.2.2, but it may be prudent to open two on each floor for a total of four destructive openings.

Thus, a total of 10 destructive openings of shear walls would satisfy the requirements of ASCE 41-13 § 12.2.3.2.2.2 as shown in Figure 7-6. (The opening on the back wall of the northeast corner is not visible in the figure.) In addition, nailing and sheathing would need to be verified with at least one opening at each level. Conditions exposed in destructive openings often present surprises, and engineering judgment is often required to determine whether additional openings and evaluation are required.

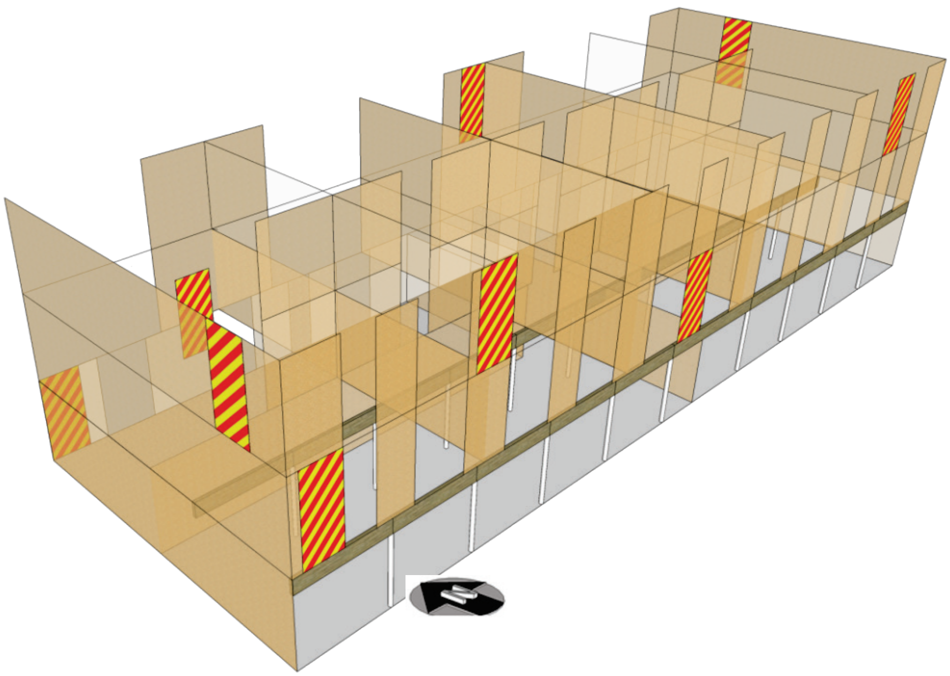


Figure 7-6 Locations of destructive openings per ASCE 41-13 § 12.2.3.2.2.2. (Walls shown filled are those used as primary elements of the retrofit design as described later.)

For the purpose of this example, it is presumed that the condition assessment determined the following general conditions, which are believed to be common in tuck-under buildings of this vintage in western United States:

- Walls are sheathed with a mixture of plywood, stucco, and gypsum wallboard. Exterior walls are stucco over plywood on the outside face and gypsum wallboard on the inside; interior walls are all gypsum wallboard on both sides. The materials/nailing of shear walls and diaphragms are consistent throughout the structure. Plywood shear walls are composed of blocked 3/8-inch plywood on 2× studs with 8d nails spaced at 6 inches on center along the edges and 12 inches on center in the field. Stucco is conventional three-part cement plaster with woven wire mesh reinforcement. Gypsum wallboard is 1/2-inch thick with 5d coolers nails spaced at 7 inches on the edges.

- The materials and nailing of diaphragms were found to be consistent throughout the structure. Floor and roof diaphragms are 1/2-inch plywood with 8d nails spaced at 6 inches on boundaries and 12 inches in the field. All diaphragms are blocked.
- All exposed framing is in good condition (no significant decay or termite damage observed).
- Sill plates of all walls are anchored to concrete foundations with 1/2-inch diameter bolts with 1.5-inch diameter washers, generally spaced at 48 inches.
- Exposed portions of concrete footings are in good condition and appear to be of sound concrete; the slab floor of the garage displays normal and expected shrinkage cracking but is otherwise in good condition.
- No hold-down devices or detailed collector elements were found at any of the shear walls.

Floor framing is 2×12 joists spaced at 24 inches with 1/2-inch plywood floor sheathing, topped with 1-1/2 inches of lightweight concrete (or gypcrete) for improved vibration, sound, and fire performance. Roof framing is 2×12 joists spaced at 24 inches on center with built-up roofing with gravel. Floors and roof bear on wood frame walls throughout except in the parking area where pipe columns, wood posts, and glued-laminated beams provide vertical support.

7.2.2 Dead Loads and Seismic Weight

Building component weights are tabulated in Table 7-1. The values are generally taken from ASCE 7-10 Table C3-1 (ASCE, 2010). The roof and floor dead loads are found to be 19 and 24 lb/ft^2, respectively, including a 2 lb/ft^2 miscellaneous allowance for unknown and unaccounted elements. The seismic weight of each diaphragm is its dead weight plus the tributary weight of all partitions. For this example, the total weights of the exterior walls and interior partitions are determined and assumed to be spread uniformly over the diaphragms. The seismic weight of the roof diaphragm is based on a 4.5-foot tributary height, while the second and third floor diaphragms are based on tributary heights of 9 feet (except along the open front on the first story). As seen in Table 7-2, the weights at the roof, third (L3) and second floor (L2) diaphragms are 106.9, 164.9 and 150.6 kips, respectively, for a total seismic weight of 422.4 kips.

Table 7-1 Flat Loads

Roof Construction	Dead Load (lb/ft^2)
Four-ply felt and gravel	5.5
3/8 plywood	1.2
Insulation	1.5
5/8 gypboard	2.75
2x12 at 24 inches on center, typical	6
Miscellaneous	2
Total	19
Floor Construction	**(lb/ft^2)**
1-1/2 inch lightweight concrete	12
1/2 inch plywood	1.6
Insulation	0
5/8 gypboard	2.75
2x12 at 24 inches on center, typical	6
Miscellaneous	2
Total	24
Partition	**(lb/ft^2)**
1/2 inch gypboard both sides, 2x4 studs	8
Exterior Wall	**(lb/ft^2)**
7/8 inch stucco	10.2
3/8 inch plywood	1.2
Insulation	1.5
1/2 inch gypboard	2.2
2x6 at 16 inches on center, typical	5
Miscellaneous	1
Total*	21
Equivalent Uniform Floor Mass for Exterior Walls	**(lb/ft^2)**
Roof: 272 lineal feet, 4.5 ft high	7
3rd floor: 272 lineal feet, 9 ft high	14
2nd floor: 172 lineal feet, 9 ft high + 100 lineal feet at 4.5 feet high	12
Equivalent Uniform Floor Mass for Interior Partitions	**(lb/ft^2)**
Roof: 400 lineal feet, 4 ft high	4
3rd floor: 400 lineal feet, 8 ft high	7
2nd floor: 650 lineal feet, 4 ft high	6

Note: Conservatively, the exterior wall weights have not been lowered to account for window openings.

Table 7-2 Seismic Weights

Level	Dead Load (psf)	Seismic Weight* (psf)	Area (ft^2)	Weight (kip)
Roof	19	30	3,600	106.9
Third floor (L3)	24	46	3,600	164.9
Second floor (L2)	24	42	3,600	150.6
Total:				422.4

* The seismic weight in psf includes the roof and floor dead loads, plus the tributary exterior wall and interior partition weights applied to the full diaphragm area.

Since most of the building weights are distributed symmetrically, it is assumed here that the mass centroid is at the geometric centroid of the building plan at all floors and roof.

7.3 Site Seismicity

As discussed above, the Tier 3 evaluation and retrofit for this structure requires performance evaluation at both the BSE-1E and BSE-2E Seismic Hazard Levels (ASCE 41-13 § 2.4.1 and Table 2-1). The BSE-1E spectrum is also used for the Tier 1 Quick Checks (ASCE 41-13 § 4.1.2 and § 4.5.2.3). In addition, the Tier 1 screening requires the BSE-1N response spectrum to determine the Level of Seismicity to select the appropriate checklists (ASCE 41-13 § 4.4 and Table 4-7).

7.3.1 Design Spectra

Useful Tip

Per ASCE 41-13 § 7.2.3.6, for wood frame structures with sufficient cross walls, a spectrum with 10% of critical damping is acceptable. This example does not utilize this advantage.

The building site is in San Jose, California in an area of stiff soils (Site Class D). BPOE requires evaluation of the Life Safety(LS) Performance Level for BSE-1E, which is the 20% in 50-year uniform hazard spectrum with 5% of critical damping, but per ASCE 41-13 § 2.4.1.4, need not exceed BSE-1N. In addition, the Collapse Prevention (CP) Performance Level must be evaluated for BSE-2E, which is the 5% in 50-year spectra, but per ASCE 41-13 § 2.4.1.3 need not exceed BSE-2N.

The seismic hazard and design spectra for this site were illustrated in Chapter 3 of this *Guide* using available online tools and those results are repeated here as Figure 7-7 and Table 7-3. For the short period plateau acceleration, the BSE-1E and BSE-2E values are found to be capped by the associated BSE-1N and BSE-2N values of 1.0g and 1.5g, respectively. The 1-second periods are not capped and are defined by the 20% in 50-year and 5% in 50-year values of 0.57g and 0.83g, respectively.

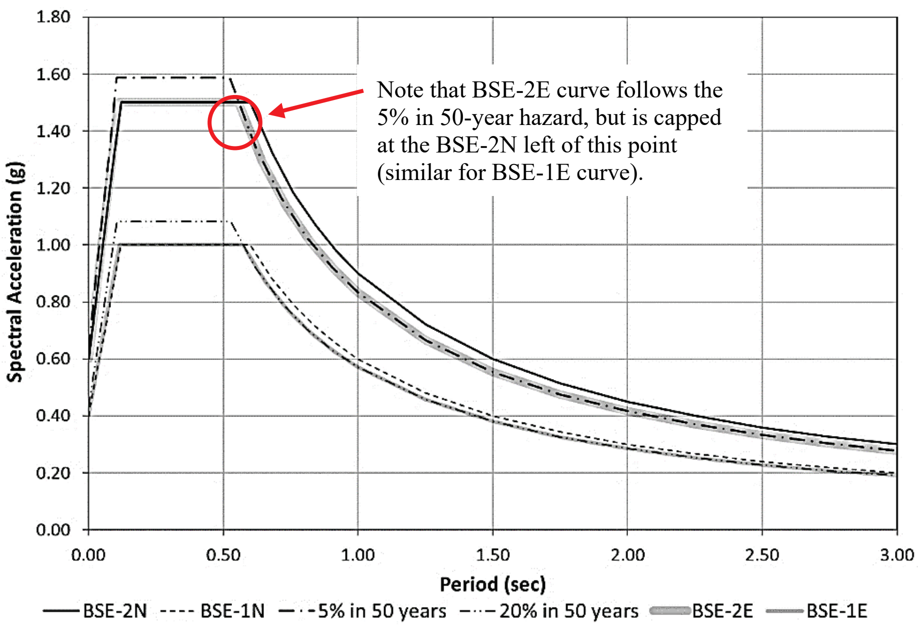


Figure 7-7 Input horizontal spectra for San Jose, California site.

Table 7-3 Spectral Accelerations for Site in San Jose, CA, Site Class D

Value	ASCE 41-13 Section 2.4.1 Spectral Ordinates				Uncapped BSE-2E and BSE-1E	
	BSE-2N	BSE-1N	BSE-2E	BSE-1E	5% in 50yr	20% in 50yr
S_{XS} (g)	1.50	1.00	1.50	1.00	1.59	1.08
S_{X1} (g)	0.90	0.60	0.83	0.57	0.83	0.57
T_o (sec)	0.12	0.12	0.11	0.11	0.10	0.11
T_s (sec)	0.60	0.60	0.56	0.57	0.52	0.53

7.3.2 Level of Seismicity

Tier 1 screening includes completion of a checklist and some simple Quick Checks of shear demand in the shear walls. Selection of the appropriate checklist from ASCE 41-13 Appendix C is based on building type, building Performance Level (LS or IO) and Level of Seismicity. Based on the above seismic hazard values, ASCE 41-13 Table 2-5 assigns this site a High Level of Seismicity because $S_{DS} = 1.50 > 0.50$ and $S_{D1} = 0.90 > 0.20$. As such, the correct checklist for this building is 16.2LS, in addition to the basic configuration checklists 16.1 and 16.1.2LS.

7.4 Tier 1 Analysis (ASCE 41-13 § 4.5)

The deficiencies of tuck-under construction are clearly identified by Tier 1 screening. The Tier 1 screening includes the completion of a checklist and a Quick Check of the demand on the shear walls from a simplified base shear and the total wall length in each direction. For this building, the total base shear of ASCE 41-13 § 4.5 is calculated as follows:

$V \quad = CS_aW$ (ASCE 41-13 Eq. 4-1)

where:

C = Modification factor
= 1.0 for three-story buildings (ASCE 41-13 Table 4-8)

S_a = 1.0g for BSE-1E for a short period structure, Table 7-3

W = Effective seismic weight of building
= 422.4 kips from Table 7-2

$V \quad = 1.0 \times 1.0 \times 422.4 = 422.4$ kips

In the transverse direction, there is 72 feet of plywood shear wall (the central wall is not sheathed with wood structural panel), and thus the average unit demand on the plywood walls is 5.9 klf. When this load is reduced by a system modification factor, M_s, of 4.0 (from ASCE 41-13 Table 4-9 for the Life Safety Performance Level or LS), the unit shear demand is 1,467 plf as shown below, per ASCE 41-13 Equation 4-9.

v_j^{avg} = Average shear stress in shear walls

$$= \frac{1}{M_s} \times \frac{V_j}{A_w}$$ (ASCE 41-13Eq. 4-9)

where:

V = Tier 1 base shear from above
= 422.4 kips

M_s = 4 for LS assessment of wood walls (ASCE 41-13 Table 4-9)

A_w = total length of end plywood walls
= 72 ft from Figure 7-3

v_j^{avg} = (1/4) × (422.4/72) = 1,467 plf

This exceeds the checklist cap of 1,000 plf as prescribed in the Shear Stress Checkbox of the High Seismicity checklist ASCE 41-13 16.2LS, and therefore the building is noncompliant in this regard.

In addition to the noncompliant shear wall demand determined by the Quick Check, this building also exhibits a number of noncompliant building configuration conditions identified in the Tier 1 checklists (ASCE 41-13 Checklists 16.1 and 16.2 LS). These include:

- **Weak Story:** The sum of the shear strengths of the shear walls in the bottom story is less than 80% of the strength in the story above, and the building is thus noncompliant. This was initially determined by engineering judgment based on wall lengths shown in Figure 7-2, Figure 7-3, and Figure 7-5, and confirmed by calculations summarized in Table 7-5 through Table 7-8.
- **Vertical Irregularities:** The upper story front shear walls are not continuous to the foundation, and are thus noncompliant (see Figure 7-5).
- **Torsion:** The estimated distance between the story center of mass (plan centroid in this case) and the story center of rigidity (near the back wall on the bottom story) is greater than 20% of the building width (20% of 36 feet is 7.2 feet), and is thus noncompliant. If only the plywood shear walls are considered, the center of east-west rigidity would be at the back wall, and noncompliance would be obvious by inspection. (Note that if the stiffness of the interior gypsum board shear walls is also considered, later calculations show the eccentricity to be 8.5 feet (Table 7-16), and thus the building would also be noncompliant by calculation.)
- **Redundancy:** The checklist requires two or more lines of shear walls in each principal direction. If one only considers plywood shear walls, there is only one (back wall) on the bottom story in the east-west direction, and this could be interpreted as noncompliant. If the corridor walls along Line 3, which are sheathed with gypsum board without plywood, are considered to be primary elements, then the structure would be compliant in this regard.

Based on the noncompliance indicated both by the presence of irregularities and failure of the Quick Check, a Tier 3 evaluation is warranted.

7.5 Tier 3 Evaluation of the Existing Structure

Based on the checklist results, a decision regarding the evaluation and retrofit of the entire structure based on Tier 3 procedures will be made. Alternatively, depending on owner preference and jurisdiction requirements, one might choose either a Partial Retrofit Objective (ASCE 41-13 § 2.2.3.2) or a Tier 2 Deficiency-Based evaluation and retrofit (ASCE 41-13 § 5.1), with a focus on only retrofitting the weak ground story.

> **Useful Tip**
>
> ASCE 41-13 § 7.3 describes many circumstances for which the LSP cannot be used. This is important to consider up front: if the LSP is selected, then the retrofit will have to preclude all conditions for which the LSP is not allowed. If a flexible diaphragm is assumed and verified, many of the impediments to the LSP are eliminated.

7.5.1 Analysis Procedure

Given the level of effort associated with retrofit of light-frame buildings, and because of the intention to eliminate the weak story and irregularities, a simple linear static procedure (LSP) is considered most appropriate. This

Useful Tip

Note that there is an additional requirement for the use of the LSP that appears in ASCE 41-13 § 7.5.2.2.3. If plastic hinges are expected to form in horizontal members away from the member ends, then linear procedures are not allowed. It is unlikely that this would be the case for a tuck-under building except if steel headers over the open front are used to support narrow shear walls above.

type of analysis can be done with a computer model of the building using commercial software, but is simple enough to do with hand/spreadsheet calculations. This example presents the use of hand/spreadsheet calculations because they are common for this building type and because it will be more informative for the reader.

Based on Tier 1 screening, the option of retrofitting was selected; a full analysis of the existing structure is therefore not required. However, with little additional effort, the same spreadsheet model can be used for both the existing and retrofit buildings. This preliminary analysis of the existing structure will allow estimation of how much additional strength and stiffness are required at each story to meet the acceptance criteria, and will help identify existing shear walls that may need to be enhanced with either additional nailing or hold-downs. (Such a preliminary retrofit scheme would satisfy the requirement in ASCE 41-13 § 1.5.)

For soft-story retrofits, determination of the fundamental building period by the empirical equation of ASCE 41-13 Equation 7-18 is recommended. The open front and sparsity of shear walls on the first story interior can result in a very flexible structure compared to conventional light-frame buildings of three stories. An eigenvalue or Rayleigh (ASCE 41-13 Eq. C7-2) analysis of the existing building model might indicate periods longer than the plateau region of the spectra, and thus lower design base shear. However, because the existing building is being analyzed to identify elements that do not meet the acceptance criteria and to determine the required retrofit strength, the higher base shear associated with a stiffer, retrofitted structure is more appropriate, and the fundamental period of ASCE 41-13 Equation 7-18 is used.

Useful Tip

The Tier 3 BPOE requires that both LS at BSE-1E and CP at BSE-2E be checked. However, both checks are based on ASCE 41-13 Equation 7-36. Since linear procedures are being used, the controlling check can be quickly determined by comparing the CP and LS *m*-factors to the spectral acceleration of the associated hazard curve. In this way, DCRs that satisfy CP performance, in this case, must also satisfy LS.

Note that ASCE 41-13 § 7.2 through § 7.5 have many criteria that must be met to use the LSP for a Tier 3 evaluation, but the analysis model in this section for the existing structure does not need to meet these requirements because it is not being used to demonstrate that acceptance criteria are met, but only to develop a retrofit scheme. Once the retrofit design is complete, conformance to the acceptance criteria will be verified, and the fact that the LSP model satisfies the requirements of ASCE 41-13 will be demonstrated.

7.5.2 Acceptance Criteria

Although BPOE requires two sets of acceptance criteria be checked (LS at BSE-1E and CP at BSE-2E), only the CP criteria need be checked for this example; if the CP criteria are satisfied, the LS criteria will also be satisfied for Deformation-Controlled actions. This is because for linear elastic structural response, the wall demands will be proportional to the base shear. Thus, the wall demands for BSE-2E are 1.5 times the BSE-1E loads, whereas the *m*-factors in ASCE 41-13 Table 12-3 for CP are all less than 1.5 times the associated LS values for the shear wall types in the existing structure. (Looking forward, a retrofit that incorporates moment frames with LS and CP *m*-Factors of 6 and 8, respectively, will also be controlled by CP criteria at this site.) Therefore, the CP acceptance criteria will control the evaluation and retrofit of the tuck-under building, and only CP evaluation and retrofit are included in this example.

7.5.3 Building Model

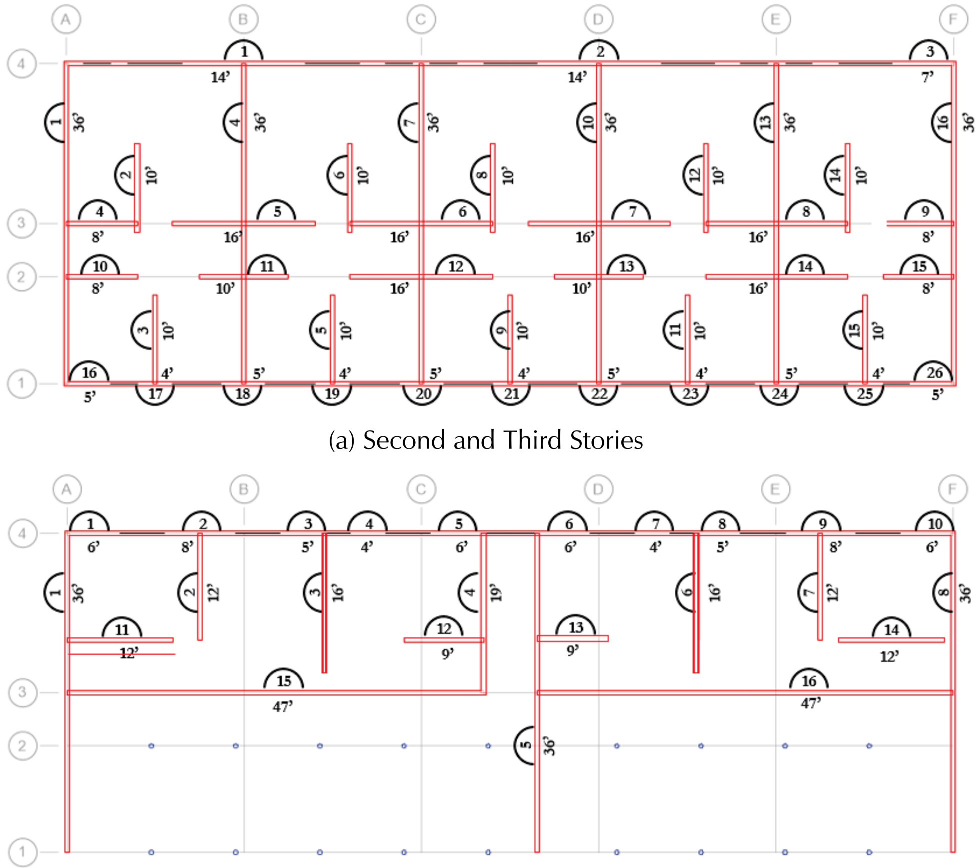
Walls with large height-to-length aspect ratios have diminished strength and stiffness and therefore do not contribute significantly to the overall lateral force-resisting system. It is normal practice for engineers to ignore short walls to reduce the number of calculations; for this example, only the contribution of walls longer than 4 feet (aspect ratio of about 2) will be included. Choosing which walls to omit from the primary elements of the lateral load-resisting system in order to simplify calculations will involve engineering judgment, but omitting walls should never be done with the intent of mitigating the effects of a potential irregularity (ASCE 41-13 § 7.2.3.3). In addition, the ASCE 41-13 Table 12-3 *m*-factors for gypsum wallboard and stucco are capped at aspect ratios of 2.0; stucco or gypsum wallboard shear walls with aspect ratios greater than 2.0 cannot be used for primary load-resisting elements. The upper limit for structural panel shear walls is 3.5. Figure 7-8 presents the wall numbering, locations, orientations, and lengths used in the model of the existing structure, and Figure 7-9 presents the *m*-factors for gypsum wallboard, stucco, and structural panel wall sheathings based on aspect ratio.

Based on the spacing of shear walls and the use of lightweight concrete topping on the second and third floors, engineering judgment might indicate that a rigid diaphragm assumption is most appropriate. In general, a rigid diaphragm evaluation requires significantly more effort than one in which the walls demands are determined by tributary area. The designer should be aware of the different levels of effort required and limitations of each. For this example, the rigid diaphragm assumption is selected, and thus stiffness of the shear walls is required to determine centers of rigidity and rotational stiffness for each floor. This assumption will also require a better analysis of the existing structure, which has an open front and thus relies on diaphragm rotational restraint. Using the rigid diaphragm assumption, all drifts at each floor can be described by only three diaphragm displacements—two

Commentary

ASCE/SEI 7-10 Section 12.3.1.1 allows diaphragms in light-frame buildings constructed of wood structural panels with nonstructural topping (up to 1.5 inches thick) to be modeled as flexible. ASCE 41-13 contains no such explicit exemption.

translations and one rotation. Shear wall loads are then determined by the displacement at each wall times the wall stiffness.



(a) Second and Third Stories

(b) First Story

Figure 7-8 Schematic of shear walls included in the model of existing building. The top plan is for the second and third stories; the bottom plan is for the first story. The wall identification number and length are shown.

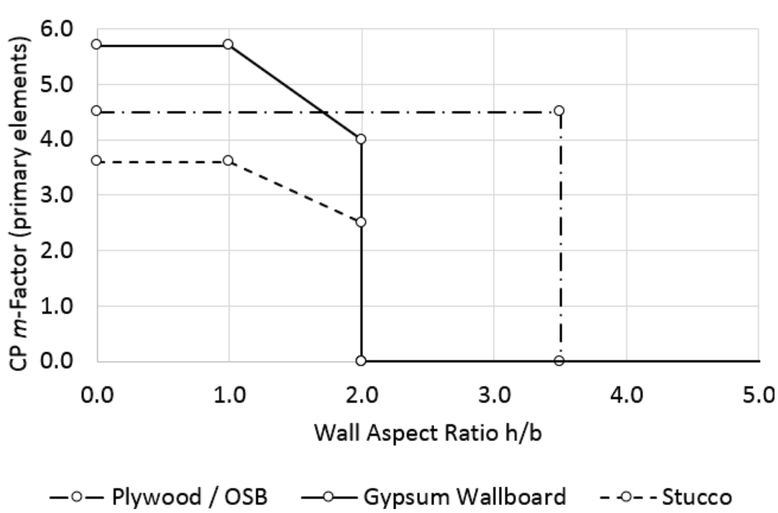


Figure 7-9 Collapse Prevention *m*-factors for existing shear walls.

7.5.4 Shear Walls

Shear wall construction is 2×4 stud framing sheathed on both faces, often with different types of sheathing. The 1994 Northridge Earthquake motivated much testing of these wall types, and the engineer has several sources of engineering properties from which to choose. However, if using anything other than the default properties provided in ASCE 41-13, the qualification of strength and stiffness must be justified per ASCE 41-13 § 12.2.2.3 through testing. Two sources for alternate wall properties include: (1) FEMA P-807 (FEMA, 2012b), which assembles and summarizes many tests of sheathed walls (Appendix D of that document); and (2) lateral testing of perforated stucco walls by the Consortium of Universities for Research in Earthquake Engineering (CUREE) by Arnold et al. (2003), which demonstrates significantly different strengths and stiffness from the tests assembled in FEMA P-807. In general, the engineering properties of sheathed light-framed walls can vary significantly from source to source, and it is important for the engineer to understand the nature of the testing on which the properties are based.

For this example, strength and stiffness properties for wood structural panel shear walls as specified in ASCE 41-13 and for gypsum wallboard and stucco walls as specified in FEMA P-807 (for evaluation of the existing structure) are used. There are three types of existing wall construction:

1. **Exterior Stucco Walls:** These walls are constructed with conventional stucco over 3/8-inch thick, 3-ply plywood on the exterior face, and gypsum wallboard on the interior. For combinations of different sheathing, the default combination rule of ASCE 41-13 § 12.4.1 is to simply include the strongest of the materials, which in this case is the plywood. For plywood shear walls, ASCE 41-13 refers the user to the American Wood Council's *Special Design Provisions for Wind and Seismic* (SDPWS) standard (AWC, 2008). Table 4.3A of that reference provides a unit *yield* strength of 440 pounds per foot and an *apparent* sheathing stiffness, G_a, of 12,000 pounds per inch (8d nails spaced at 6 inches on center along the edges and 12 inches on center in the field). However, ASCE 41-13 Equation 12-2 for deflection of shear walls does not utilize the apparent stiffness (which includes nail deformation), but requires that the nail deformation be treated explicitly. The modulus of rigidity (which SDPWS denotes as $G_v t_v$) can be found in commentary Table C4.2.2A of SDPWS; this is the value to be used in ASCE 41-13 Equation 12-2 for the product Gt (i.e., $G \times t$).

   Per ASCE 41-13 § 12.4.4.6.2, the SDPWS strength values are considered yield strengths, and can be increased by 50% (with $\phi = 1$) such that the

Definitions

General definitions of Q_{CE} and Q_Y are given in ASCE 41-13 § 7.5.1.3. ASCE 41-13 § 12.4.4.6.2 provides specific information for plywood and indicates that SDPWS capacities can be increased by 50% to convert yield to expected strength.

expected strength of these walls Q_{CE} is 660 plf. Note that this is a default, conservative value based on plywood alone, consistent with ASCE 41-13 § 12.4.1. The actual strength and stiffness of these walls, considering both stucco and plywood, could be significantly higher. In situations where the resistance provided by stucco plays a significant role in the building response and the required retrofit effort, one should consider justifying and incorporating more realistic properties for sheathing combinations.

2. **Interior Gypsum Wallboard:** These walls are constructed with conventional gypsum wallboard on both faces. When both faces utilize the same sheathing type, the strength and stiffness for one layer of sheathing can be doubled. Gypsum wallboard is covered in ASCE 41-13 § 12.4.4.10, and ASCE 41-13 Table 12-1 provides a default unit strength of 100 plf (for each face) and a sheathing shear stiffness, G_d, of 8,000 lb/in (each face). ASCE 41-13 § 12.2.2.1.3 permits approved values of material properties other than these defaults if they are based on available historical information or prevailing codes. For this example, as described later, values adopted from FEMA P-807 are used in lieu of the default properties.

3. **Central Garage Wall:** The central north-south wall in the garage is composed of conventional stucco over gypsum sheathing applied to both faces. ASCE 41-13 Table 12-1 provides a unit strength for stucco of 350 plf (for each face) and a sheathing stiffness, G_d, of 14,000 lb/in (each face).

Commentary

Ignoring the strength and stiffness contribution of the weakest sheathing material might be very conservative, and differs substantially from FEMA P-807.

Table 7-4 summarizes the key properties for each of the shear wall types in the existing structure. The wall description in the table lists the sheathing materials from outside to inside face; sheathing on the inner and outer faces are separated by "||". Per ASCE 41-13 § 12.4.1, walls sheathed with a combination of dissimilar materials are by default assigned only the strength and stiffness of the strongest sheathing material of either face; those materials are shown highlighted gray in Table 7-4. For similar sheathing materials, for instance Wall Type 4 which has wood structural panel on both sides, the strength and stiffness is additive. (Q_y is 860 plf for the retrofit OSB plus 440 plf for the original plywood, which equals 1,300 plf in Table 7-4.)

The table provides the yield strength and expected capacity (Q_Y and Q_{CE}) and the full wall density (for calculation of overturning resistance). For wood structural panels, the stiffness terms are as defined in ASCE 41-13 § 12.4.4.6.

$\Delta_y = 8v_y h^3/(EAb) + v_y h/(Gt) + 0.75he_n + (h/b)d_a$ (ASCE 41-13 Eq. 12-2)

where:

Gt = Modulus of rigidity times the panel thickness

EA = Axial stiffness of the end stud

e_n = Expected nail slip

d_a = Deflection at yield of tie-down anchorage, taken as 0.0

Table 7-4 Shear Wall Types and Properties

Wall Type	Description	Q_Y (plf)	Q_{CE} (plf)	Density (psf)	Stiffness per ASCE 41-13 Eq.12-1 and 12-2			
					G_a or G_t (kip/in)	EA (kip)	e_n (in)	d_a (in)
1	Stucco, **Plywood** || Gypboard	440	660	21	25.0	8,400	0.08	0.00
2	Gypboard || Gypboard	284	426	8	3.2	0	0.00	0.00
3	**Stucco**,Gypboard || Gypboard, **Stucco**	444	666	29	7.1	0	0.00	0.00
4	Retrofit OSB (15/32 w/ 8d@4, original plywood opp)	1,300	1,950	22	108.5	16,800	0.08	0.10
5	Retrofit OSB (15/32 w/ 8d@4, original gypboard opp)	860	1,290	10	83.5	16,800	0.08	0.10
7	Retrofit OSB (15/32 w/ 8d@4, both sides)	1,720	2,580	22	167	16,800	0.08	0.10

Note: The strength and stiffness of stucco and gypboard are taken from FEMA P-807 Appendix D (FEMA 2012b). Plywood and OSB values are taken from AWC SDPWS (AWC 2008)

Setting d_a to zero will overestimate the stiffness of existing walls with uplift, but the error introduced by that here is considered acceptable because the as-built condition is being evaluated in order to develop a preliminary retrofit design, not to check satisfaction of acceptance criteria. Walls used as primary elements of the lateral load system in the retrofit structure will either be modeled with an appropriate value of d_a, or be shown to have no uplift (d_a = 0). In general, the d_a term is determined by testing of an assembly, and tabulated values are found in ICC Evaluation Services reports for the hold-down devices selected. For shear walls other than wood structural panel, the sheathing shear deformation and effect of nail slip are combined to an apparent shear modulus, G_d, and per ASCE 41-13 Equation 12-1, the nail slip and bending deflection terms are not used.

For wood structural panels, the key deformation parameters are provided in ASCE 41-13 § 12.2.2.5, § 12.4.4.6.1, and SDPWS Table 4.3. Nail deformation, e_n, is taken as 0.08 inches for 8-penny nails with Structural I panels per ASCE 41-13 § 12.4.4.6.1; the stiffness of the boundary element

EA is conservatively determined by considering one 2×4 stud (1.5 in. × 3.5 in.) with a modulus of elasticity of 1600 ksi (assuming Douglas Fir-Larch, Commercial Grade No. 2).

ASCE 41-13 Table 12-1 provides the apparent shear stiffness, G_d, and expected strength, Q_{CE}, for various wall types, including gypsum wall board and conventional stucco. However, in this example, values from testing provided in FEMA P-807 Appendix D are used. For instance, the values for strength and stiffness provided in ASCE 41-13 Table 12-1 for gypsum wallboard are 8,000 lb/in and 100 plf, respectively. If the wall is sheathed on both sides, the expected strength would be 200 plf, and the yield strength, which is presumed to be two-thirds of the ultimate strength for light frame shear walls, would be (2/3) × 200 = 133 plf and the stiffness would be 16,000 lb/in. Per ASCE 41-13 Equation 12-1, the yield deflection (assuming no uplift) would be only 133plf × 8ft/16,000 lb/in = 0.067 inches, which is not realistic and leads to wall stiffnesses that are not consistent with those associated with the plywood shear walls. (The values for stucco pose a similar problem.) FEMA P-807 Appendix D provides a compilation of shear wall test summaries that include gypsum wall board and stucco shear walls. From FEMA P-807 Table D-14, yield deflection for gypsum wallboard and stucco walls are taken as 0.7 and 0.5, respectively, and expected strengths of 213 plf and 333 plf, respectively. (The strength values Q_{CE} in Table 7-4 are double these to account for sheathing on both sides.) From these, and assuming no uplift, apparent shear stiffness, G_d, is calculated to be 3,200 kip/in and 7,100 kip/in for double-sided gypsum wallboard and stucco, respectively. Table 7-4 summarizes shear wall properties from the various sources described above. For the retrofit walls, d_a in Table 7-4 has been estimated from the ICC reports of the selected hardware supplier. The Collapse Prevention acceptance criteria m-factors for lightweight wood frame shear walls are a function of the wall height-to-length ratio. Figure 7-9 plots the m-factor versus aspect ratio (h/b) of structural wall panel, gypsum wallboard, and stucco walls from ASCE 41-13 Table 12-3.

7.5.5 Model Properties

Useful Tip

Per ASCE 41-13 § 7.2.6, P-Δ effects must be considered. For a light-frame structure being evaluated by the LSP, this would make a negligible difference.

Based on the wall locations and lengths in Figure 7-8 and the wall properties summarized in Table 7-4, the fundamental distribution of wall strengths and the dead load contribution to overturning are summarized in Table 7-5 through Table 7-8. To illustrate, the first row in Table 7-5 that summarizes the north-south oriented shear walls in the upper stories is discussed below:

- Column 1 (Wall) is the wall number, which is unique for each story and each direction. Wall numbers are shown in Figure 7-8.
- Column 2 (Type) is the wall type as summarized the wall construction schedule in Figure 7-3 and listed in Table 7-4.
- Column 3 (Length) is the wall length (ft) as shown Figure 7-8.
- Column 4 (X Location) is the wall location in terms of feet from the west exterior wall as shown Figure 7-8.
- Column 5 (Trib L) is the tributary length of the joist framing supported by the wall, assuming 24 psf for floor dead load and 19 psf for roof dead load. Since these walls are parallel to the framing, they are assigned 1.5 ft.
- Column 6 (v_{CE}) is the strength of the wall for the given type as given in Table 7-4. In this case v_{CE} = 660 plf is the expected strength per foot of wall for Wall Type 1 (existing exterior plywood).
- Column 7 (v_y) is the expected yield strength per foot of wall, taken as two thirds of v_{CE}:

  v_y = 660 × 2/3 = 440 plf for the Wall Type 1

- Column 8 (Q_{CE}) is the wall strength calculated as the length times v_{CE}, converted to kips:

  Q_{CE} = 660 plf × 36 ft = 23.8 kips

- Column 9 (Self W) is the self weight per square foot as shown in Table 7-4; for Type 1 = 21 psf.
- Column 10 (DL) is the supported weight per unit length calculated as the roof (floor) dead load times the tributary length (Trib L). In this case, for roof seismic weight of = 19 psf:

  DL = 19 psf × 1.5 ft = 28.5 plf (accounting for round-off)

- Column 11 (M_{ST}) is the dead load stabilizing moment (ASCE 41-13 Eq. 7-6) provided by weights of the wall itself (Column 9) and the tributary roof area (Column 10).

  M_{ST} = wall density × the height × (wall length)^2/2 + DL × (wall length)^2/2
  = $(21 \times 8 \times 36^2/2) + (28.4 \times 36^2/2)$ = 127.8 kip-ft (accounting for round-off)

Table 7-5 Second and Third Story North-South Shear Walls – Locations and Strengths

Wall (1)	Type and Dimensions: Type (2)	Length (ft) (3)	X Location (ft to west) (4)	Trib *L* (ft) (5)	Expected Strength: v_{CE} (plf) (6)	v_Y (plf) (7)	Q_{CE} (kip) (8)	Overturning Resistance: Self *W* (psf) (9)	DL (plf) (10)	M_{ST} (kip-ft) (11)
1	1	36	0.0	1.5	660	440	23.8	21	28.4	127.8
2	2	10	8.0	1.5	426	284	4.3	8	28.4	4.6
3	2	10	10.0	1.5	426	284	4.3	8	28.4	4.6
4	2	36	20.0	1.5	426	284	15.3	8	28.4	59.9
5	2	10	30.0	1.5	426	284	4.3	8	28.4	4.6
6	2	10	32.0	1.5	426	284	4.3	8	28.4	4.6
7	2	36	40.0	1.5	426	284	15.3	8	28.4	59.9
8	2	10	48.0	1.5	426	284	4.3	8	28.4	4.6
9	2	10	50.0	1.5	426	284	4.3	8	28.4	4.6
10	2	36	60.0	1.5	426	284	15.3	8	28.4	59.9
11	2	10	70.0	1.5	426	284	4.3	8	28.4	4.6
12	2	10	72.0	1.5	426	284	4.3	8	28.4	4.6
13	2	36	80.0	1.5	426	284	15.3	8	28.4	59.9
14	2	10	88.0	1.5	426	284	4.3	8	28.4	4.6
15	2	10	90.0	1.5	426	284	4.3	8	28.4	4.6
16	1	36	100.0	1.5	660	440	23.8	21	28.4	127.8
Σ		**316**					**151**			

Table 7-6 First Story North-South Shear Walls – Locations and Strengths

Wall (1)	Type and Dimensions: Type (2)	Length (ft) (3)	X Location (ft to west) (4)	Trib *L* (ft) (5)	Expected Strength: v_{CE} (plf) (6)	v_Y (plf) (7)	Q_{CE} (kip) (8)	Overturning Resistance: Self *W* (psf) (9)	DL (plf) (10)	M_{ST} (kip-ft) (11)
1	1	36	0	1.5	660	440	23.8	21	36.5	133.1
2	2	12	16	1.5	426	284	5.1	8	36.5	7.2
3	2	16	28	1.5	426	284	6.8	8	36.5	12.9
4	2	19	47	1.5	426	284	8.1	8	36.5	18.1
5	3	36	53	1.5	666	444	24.0	29	36.5	174.6
6	2	16	72	1.5	426	284	6.8	8	36.5	12.9
7	2	12	84	1.5	426	284	5.1	8	36.5	7.2
8	1	36	100	1.5	660	440	23.8	21	36.5	133.1
Σ		**183**					**103**			

Table 7-7 Second and Third Story East-West Shear Walls – Locations and Strengths

Wall (1)	Type and Dimensions				Expected Strength			Overturning Resistance		
	Type (2)	Length (ft) (3)	Y Location (ft to north) (4)	Trib *L* (ft) (5)	v_{CE} (plf) (6)	v_y (plf) (7)	Q_{CE} (kip) (8)	Self *W* (psf) (9)	DL (plf) (10)	M_{ST} (kip-ft) (11)
1	1	14	36	9	660	440	9.2	21	171	33.3
2	1	14	36	9	660	440	9.2	21	171	33.3
3	1	7	36	9	660	440	4.6	21	171	8.3
4	2	8	18	9	426	284	3.4	8	171	7.5
5	2	16	18	18	426	284	6.8	8	341	51.9
6	2	16	18	18	426	284	6.8	8	341	51.9
7	2	16	18	18	426	284	6.8	8	341	51.9
8	2	16	18	18	426	284	6.8	8	341	51.9
9	2	8	18	18	426	284	3.4	8	341	13.0
10	2	8	12	3	426	284	3.4	8	57	3.9
11	2	10	12	3	426	284	4.3	8	57	6.0
12	2	16	12	3	426	284	6.8	8	57	15.5
13	2	10	12	3	426	284	4.3	8	57	6.0
14	2	16	12	3	426	284	6.8	8	57	15.5
15	2	8	12	3	426	284	3.4	8	57	3.9
16	1	5	0	9	660	440	3.3	21	171	4.2
17	1	4	0	9	660	440	2.6	21	171	2.7
18	1	5	0	9	660	440	3.3	21	171	4.2
19	1	4	0	9	660	440	2.6	21	171	2.7
20	1	5	0	9	660	440	3.3	21	171	4.2
21	1	4	0	9	660	440	2.6	21	171	2.7
22	1	5	0	9	660	440	3.3	21	171	4.2
23	1	4	0	9	660	440	2.6	21	171	2.7
24	1	5	0	9	660	440	3.3	21	171	4.2
25	1	4	0	9	660	440	2.6	21	171	2.7
26	1	5	0	9	660	440	3.3	21	171	4.2
Σ							**119**			

Table 7-8 First Story East-West Shear Walls – Locations and Strengths

Wall (1)	Type and Dimensions: Type (2)	Length (ft) (3)	Y Location (ft to north) (4)	Trib *L* (ft) (5)	Expected Strength: v_{CE} (plf) (6)	v_Y (plf) (7)	Q_{CE} (kip) (8)	Overturning Resistance: Self *W* (psf) (9)	DL (plf) (10)	M_{ST} (kip-ft) (11)
1	1	6	36	9	660	440	4.0	21	219	7.0
2	1	8	36	9	660	440	5.3	21	219	12.4
3	1	5	36	9	660	440	3.3	21	219	4.9
4	1	4	36	9	660	440	2.6	21	219	3.1
5	1	6	36	9	660	440	4.0	21	219	7.0
6	1	6	36	9	660	440	4.0	21	219	7.0
7	1	4	36	9	660	440	2.6	21	219	3.1
8	1	5	36	9	660	440	3.3	21	219	4.9
9	1	8	36	9	660	440	5.3	21	219	12.4
10	1	6	36	9	660	440	4.0	21	219	7.0
11	2	12	24	0	426	284	5.1	8	0	4.6
12	2	9	24	0	426	284	3.8	8	0	2.6
13	2	9	24	0	426	284	3.8	8	0	2.6
14	2	12	24	0	426	284	5.1	8	0	4.6
15	2	47	18	18	426	284	20.0	8	438	554.8
16	2	47	18	18	426	284	20.0	8	438	554.8
Σ							**96**			

Table 7-9 through Table 7-12 summarize the wall yield deflections and stiffnesses, composite lateral and torsional story stiffness, and center of rigidity for each story in each direction. For this building, in general, the walls are distributed somewhat symmetrically, and the center of rigidity roughly corresponds with the center of mass. An exception, as expected, is the bottom story where the center of rigidity for the east-west walls is closer to the back stucco wall (see Figure 7-2). This results in a mass-stiffness eccentricity in the first story of 26.5 – 18 = 8.5 feet in the north-south direction. This eccentricity is more than the accidental eccentricity (5% of transverse plan dimension = 1.8 feet) required by ASCE 41-13 § 7.2.3.2.1. The stiffness centroid of second and third stories in the east-west direction are eccentric to the south by 18 – 15.2 = 2.8 feet.

Columns 3 through 6 (δ_{bend}, δ_{shear}, δ_{nail}, δ_{HD}) of Table 7-9 through Table 7-12 are the yield displacements due to bending (elongation of the chords), shear deformation, nail deformation, and uplift of the hold-downs, respectively. These values are determined by either ASCE 41-13 Equation 12-1 or Equation 12-2 for stucco/gypboard or plywood sheathing, respectively. (For wood structural panel shear walls, the shear deformation and nail deformation is treated separately, whereas they are combined into an apparent shear modulus for stucco and gypsum wallboard sheathing).

To illustrate, consider the first row in Table 7-9, which summarizes the north-south oriented walls in the upper stories:

- Column 1 (Wall) is the wall number shown in Figure 7-8
- Column 2 (X Location) is the wall location in terms of distance (feet) from the west exterior wall as shown Figure 7-8
- Column 3 (δ_{bend}) is the component of the deflection of the wall at yield due to bending:

  $\delta_{bend} = 8 \times v_Y \times h^3/(EA \times b)$

  where:

  v_Y = Yield load = 440 plf from Table 7-5

  h = Wall height, taken as 8 feet throughout (9-ft floor-to-floor minus a nominal floor sandwich)

  EA = Product of the boundary member elastic modulus and its area
  = 8400 kips from Table 7-4

  b = Wall length 36 feet from Table 7-5

  $\delta_{bend} = 8 \times 440 \times 8^3/(8400 \times 36) = 0.006$ in

- Column 4 (δ_{shear}) is the component of the deflection of the wall at yield due to shear deformation:

  $\delta_{shear} = v_Y \times h/(Gt)$

  where:

  v_Y = Yield load = 440 plf from Table 7-5

  h = 8 feet is the wall height

  Gt = Product of the modulus of rigidity G and the effective wall thickness t = 25 kip/in as shown in Table 7-4 for Wall Type 1

  $\delta_{shear} = 440 \times 8/25 = 0.141$ in

- Column 5 (δ_{nail}) is the component of the deflection of the wall at yield due to bending, shear, nail deformation and hold-down device extension:

  $\delta_{nail} = 0.75 \times h \times e_n$

where:

h = 8 feet is the wall height

e_n = 0.08 is the nail deformation at yield as shown in Table 7-4 for Wall Type 1, as given in ASCE 41-13 Equation 12-2

δ_{nail} = 0.75 × 8× 0.08 = 0.48 in

- Column 6 (δ_{HD}) is the component of the deflection of the wall at yield due to the hold-down device extension:

δ_{HD} = Taken as zero for the evaluation of the existing structure, as discussed above.

- Column 7 (Δ_y) is the wall yield deflection calculated as the sum of the component deformations calculated above in Columns 3 to 6:

$\Delta_Y = \delta_{\text{bend}} + \delta_{\text{shear}} + \delta_{\text{nail}} + \delta_{\text{HD}}$
= 0.006+0.141+0.48+0.0 = 0.63 in

- Column 8 (Q_y) is the yield strength of the wall:

$Q_Y = (2/3)Q_{CE}$

where:

Q_{CE} = 23.8 kip is the expected strength from Table 7-5

Q_Y = (2/3) × 23.8 = 15.8 kip (accounting for round-off)

- Column 9 (K) is the wall stiffness K calculated as the ratio of Q_y to Δ_y:

$K = Q_Y/\Delta_Y$

where:

Q_Y = (2/3) × 23.8 = 15.8 kip (accounting for round-off)

Δ_Y = 0.63 in is the yield deflection in Column 7

K = 15.8/0.63 = 25.1 kip/in (accounting for round-off)

- Column 10 ($K \times X$) is the wall stiffness times its location, used to calculate the centroid of the story stiffness:

K = 25.27 kip/in from Column 9
where:

X = 0.0 ft from Table 7-5

$K \times X$= 25.27 × 0.0 = 0 kip

- Column 11 (K_θ) is the contribution of each wall to the story torsional stiffness, calculated as the wall stiffness times the square of the distance to the center of stiffness (times 12 to convert units). The location of the center of stiffness X' is calculated in the bottom-right corner of the Table 7-9 as the ratio of the sum of all $K \times X$ entries (Column 10) to the sum of all K entries (Column 9): X' = 89614/149.5 = 599 inches/(12 in/ft) = 49.9 feet.

$$K_\theta = K \times (X' - X \text{ Location})^2$$

where:

K = 25.27 kip/in from Column 9

X' = 49 feet is the *distance* from the west wall to the center of stiffness as calculated above

X Location = 0 feet is the *distance* to this wall from Table 7-5

$$K_\theta = 25.27 \times (49.9 - 0.0)^2 \times (12 \text{ in /ft}) = 755{,}000 \text{ kip-ft/radian}$$
(accounting for round-off)

The total rotational stiffness of the floor is the sum of the contributions from the north-south and east-west walls (Table 7-9 and Table 7-11). Total K_θ = 2,257,199 + 222,850 = 2,480,049 kip-ft/radian.

Table 7-9 Second and Third Story North-South Shear Walls – Yield Deflections and Stiffness

Wall (1)	X Location (ft to west) (2)	Yield Deflection and Stiffness per ASCE 41-13 Eq. 12-1 and 12-2								
		δ_{bend} (in) (3)	δ_{shear} (in) (4)	δ_{nail} (in) 5	δ_{HD} (in) (6)	Δ_Y (in) (7)	Q_Y (kip) (8)	K (kip/in) (9)	$K \times X$ (kip) (10)	K_θ (kip-ft/rad) (11)
1	0.0	0.006	0.141	0.480	0.000	0.63	15.84	25.27	0	756,541
2	8.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	389	85,641
3	10.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	487	77,669
4	20.0	0.000	0.700	0.000	0.000	0.70	10.22	14.60	3,505	157,137
5	30.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	1,460	19,365
6	32.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	1,558	15,676
7	40.0	0.000	0.700	0.000	0.000	0.70	10.22	14.60	7,009	17,333
8	48.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	2,336	184
9	50.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	2,434	0
10	60.0	0.000	0.700	0.000	0.000	0.70	10.22	14.60	10,514	17,714
11	70.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	3,407	19,576
12	72.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	3,505	23,675
13	80.0	0.000	0.700	0.000	0.000	0.70	10.22	14.60	14,018	158,278
14	88.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	4,283	70,488

Table 7-9 Second and Third Story North-South Shear Walls – Yield Deflections and Stiffness (continued)

Wall (1)	X Location (ft to west) (2)	Yield Deflection and Stiffness per ASCE 41-13 Eq. 12-1 and 12-2								
		δ_{bend} (in) (3)	δ_{shear} (in) (4)	δ_{nail} (in) 5	δ_{HD} (in) (6)	Δ_y (in) (7)	Q_Y (kip) (8)	K (kip/in) (9)	$K \times X$ (kip) (10)	K_θ (kip-ft/rad) (11)
15	90.0	0.000	0.700	0.000	0.000	0.70	2.84	4.06	4,381	78,091
16	100.0	0.006	0.141	0.480	0.000	0.63	15.84	25.27	30,327	759,832
Σ								**149.5**	**89,614**	**2,257,199**
Total K_θ										**2,480,049**
X location of center of rigidity:										**49.95 ft**

Table 7-10 First Story North-South Shear Walls – Yield Deflections and Stiffness

Wall (1)	X Location (ft to west) (2)	Yield Deflection and stiffness (Eq. 12-1 and 12-2)								
		δ_{bend} (in) (3)	δ_{shear} (in) (4)	δ_{nail} (in) (5)	δ_{HD} (in) (6)	Δ_y (in) (7)	Q_Y (kip) (8)	K (kip/in) (9)	$K \times X$ (kip) (10)	K_θ (kip-ft/rad) (11)
1	0	0.006	0.141	0.480	0.000	0.63	15.84	25.27	0	777,845
2	16	0.000	0.700	0.000	0.000	0.70	3.41	4.87	935	70,104
3	28	0.000	0.700	0.000	0.000	0.70	4.54	6.49	2,181	39,933
4	47	0.000	0.700	0.000	0.000	0.70	5.40	7.71	4,347	1,228
5	53	0.000	0.500	0.000	0.000	0.50	15.98	31.95	20,320	2,128
6	72	0.000	0.700	0.000	0.000	0.70	4.54	6.49	5,607	35,519
7	84	0.000	0.700	0.000	0.000	0.70	3.41	4.87	4,906	64,988
8	100	0.006	0.141	0.480	0.000	0.63	15.84	25.27	30,327	738,777
Σ								**112.9**	**68,623**	**1,730,523**
Total K_θ										**2,523,628**
X location of center of rigidity:										**50.64 ft**

Table 7-11 Second and Third Story East-West Shear Walls – Yield Deflections and Stiffness

Wall (1)	Y Location (ft to north) (2)	Yield Deflection and Stiffness (Eq. 12-1 and 12-2)								
		δ_{bend} (in) (3)	δ_{shear} (in) (4)	δ_{nail} (in) (5)	δ_{HD} (in) (6)	Δ_Y (in) (7)	Q_Y (kip) (8)	K (kip/in) (9)	$K \times X$ (kip) (10)	K_θ (kip-ft/rad) (11)
1	36	0.015	0.141	0.480	0.000	0.64	6.16	9.68	4,183	50,144
2	36	0.015	0.141	0.480	0.000	0.64	6.16	9.68	4,183	50,144
3	36	0.031	0.141	0.480	0.000	0.65	3.08	4.73	2,042	24,482
4	18	0.000	0.700	0.000	0.000	0.70	2.27	3.25	701	299
5	18	0.000	0.700	0.000	0.000	0.70	4.54	6.49	1,402	599
6	18	0.000	0.700	0.000	0.000	0.70	4.54	6.49	1,402	599
7	18	0.000	0.700	0.000	0.000	0.70	4.54	6.49	1,402	599
8	18	0.000	0.700	0.000	0.000	0.70	4.54	6.49	1,402	599
9	18	0.000	0.700	0.000	0.000	0.70	2.27	3.25	701	299
10	12	0.000	0.700	0.000	0.000	0.70	2.27	3.25	467	405
11	12	0.000	0.700	0.000	0.000	0.70	2.84	4.06	584	507
12	12	0.000	0.700	0.000	0.000	0.70	4.54	6.49	935	811
13	12	0.000	0.700	0.000	0.000	0.70	2.84	4.06	584	507
14	12	0.000	0.700	0.000	0.000	0.70	4.54	6.49	935	811
15	12	0.000	0.700	0.000	0.000	0.70	2.27	3.25	467	405
16	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
17	0	0.054	0.141	0.480	0.000	0.67	1.76	2.61	0	7,261
18	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
19	0	0.054	0.141	0.480	0.000	0.67	1.76	2.61	0	7,261
20	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
21	0	0.054	0.141	0.480	0.000	0.67	1.76	2.61	0	7,261
22	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
23	0	0.054	0.141	0.480	0.000	0.67	1.76	2.61	0	7,261
24	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
25	0	0.054	0.141	0.480	0.000	0.67	1.76	2.61	0	7,261
26	0	0.043	0.141	0.480	0.000	0.66	2.20	3.31	0	9,222
Σ								**117.1**	**21,390**	**222,850**
Total K_θ										**2,480,049**
X location of center of rigidity:										**15.23 ft**

Table 7-12 First Story East-West Shear Walls – Yield Deflections and Stiffness

Wall (1)	Y Location (ft to north) (2)	Yield Deflection and Stiffness (Eq. 12-1 and 12-2)								
		δ_{bend} (in) (3)	δ_{shear} (in) (4)	δ_{nail} (in) (5)	δ_{HD} (in) (6)	Δ_y (in) (7)	Q_Y (kip) (8)	K (kip/in) (9)	$K \times X$ (kip) (10)	K_θ (kip*ft/rad) (11)
1	36	0.036	0.141	0.480	0.000	0.66	2.64	4.02	1,737	4,336
2	36	0.027	0.141	0.480	0.000	0.65	3.52	5.44	2,348	5,861
3	36	0.043	0.141	0.480	0.000	0.66	2.20	3.31	1,432	3,574
4	36	0.054	0.141	0.480	0.000	0.67	1.76	2.61	1,127	2,814
5	36	0.036	0.141	0.480	0.000	0.66	2.64	4.02	1,737	4,336
6	36	0.036	0.141	0.480	0.000	0.66	2.64	4.02	1,737	4,336
7	36	0.054	0.141	0.480	0.000	0.67	1.76	2.61	1,127	2,814
8	36	0.043	0.141	0.480	0.000	0.66	2.20	3.31	1,432	3,574
9	36	0.027	0.141	0.480	0.000	0.65	3.52	5.44	2,348	5,861
10	36	0.036	0.141	0.480	0.000	0.66	2.64	4.02	1,737	4,336
11	24	0.000	0.700	0.000	0.000	0.70	3.41	4.87	1,402	371
12	24	0.000	0.700	0.000	0.000	0.70	2.56	3.65	1,051	278
13	24	0.000	0.700	0.000	0.000	0.70	2.56	3.65	1,051	278
14	24	0.000	0.700	0.000	0.000	0.70	3.41	4.87	1,402	371
15	18	0.000	0.700	0.000	0.000	0.70	13.35	19.06	4,118	16,609
16	18	0.000	0.700	0.000	0.000	0.70	13.35	19.06	4,118	16,609
Σ								**94.0**	**29,905**	**793,105**
Total K_θ										**2,523,628**
X location of center of rigidity:										**26.52 ft**

7.5.6 Loading

Given the story heights (9 feet for all) and the seismic mass distribution in Table 7-3, the base shear, equivalent floor loads, and design story shears for BSE-1E and BSE-2E hazards can be calculated. Parameters used for this calculation are summarized in Table 7-13. The fundamental periods of the structure, T, can be calculated in two ways: the empirical equation method (ASCE 41-13 Eq. 7-18) or by analysis. As discussed above, the soft bottom story will cause the analytical period to be longer than that of the empirical equation. Because this model is being used to estimate the required strength and stiffness of a retrofitted building in which the soft story has been mitigated, the empirical equation is most appropriate and will be used for this evaluation. The empirical equation estimates the period to be 0.24 seconds, which is in the short period (plateau) region of the design spectrum.

$T \quad = C_t\, h_n^{\beta}$ (ASCE 41-13 Eq. 7-18)

where:

C_t = 0.02 for all other buildings (ASCE 41-13 § 7.4.1.2.2)

h_n = 27 feet (building height for three 9-ft floors)

β = 0.75 for all other buildings (ASCE 41-13 § 7.4.1.2.2)

$T \quad = 0.02 \times 27^{0.75} = 0.24$ seconds

The design base shear for Collapse Prevention and Life Safety can be determined from ASCE 41-13 Equation 2-5, but in this case it is simpler to refer back to Figure 7-7. A period of 0.24 seconds falls, by inspection, in the plateau region of the design response spectra, and therefore the design spectral acceleration S_a is simply S_{XS} = 1.5g for Collapse Prevention (BSE-2E) and 1.0g for Life Safety (BSE-1E).

The design base shear V for Collapse Prevention and Life Safety can now be determine from ASCE 41-13 Equation 7-21:

$V \quad = C_1 C_2 C_m S_a W$ (ASCE 41-13 Eq. 7-21)

where:

C_1C_2 = 1.4 (alternate values from ASCE 41-13 Table 7-3)

C_m = 1.0 (ASCE 41-13 Table 7-4)

S_a = 1.5g for CP, 1.0g for LS from Table 7-3

W = building weight = 422.4 kips from Table 7-2

$V \quad = 1.4 \times 1.0 \times 1.5 \times 422.4 = 887$ kips for CP, 591 kips for LS

Factors C_1 and C_2 are adjustments to account for nonlinearity and pinched hysteresis, which can be calculated based on the analysis results, or alternatively ASCE 41-13 provides an approximate combination factor given in ASCE 41-13 Table 7-3. This combined factor C_1C_2 is a function of the maximum m-factor, $m_{\max}$, used for primary elements in the evaluation or retrofit. The intent is to use primarily wood structural panels for the retrofit, and moment frames along the open front. However, the moment frames will be sized for stiffness and likely have low strength demands, and so although the maximum allowed m-factor could be 8, they could be shown to work with a much smaller value. Therefore, by engineering judgment $m_{\max}$ is based on the highest allowable value for plywood/OSB shear walls, which per ASCE 41-13 Table 12-3 is 4.5, and C_1C_2 is selected from the middle column ($m_{\max}$ less than 6) of ASCE 41-13 Table 7-3 as 1.4. C_m is an adjustment factor to account for higher modes, and for wood structures is 1.0

per ASCE 41-13 Table 7-4. The building weight is W, and S_a and V are the spectral acceleration (at $T = 0.24$) and base shear at both Life Safety (BSE-1E) and Collapse Prevention (BSE-2E) demands.

Table 7-13 Lateral Loads for BSE-1E and BSE-2E Seismic Hazard Levels

Design Parameter		
T	0.24 sec	ASCE 41-13 Eq. 7-18
W	422.4 kip	Tabulated weight (Table 7-2)
S_a (BSE-1E)	1 g	ASCE 41-13 Eq. 2-5
S_a (BSE-2E)	1.5 g	ASCE 41-13 Eq. 2-5
C_1C_2	1.4	ASCE 41-13 Table 7-3
C_m	1	ASCE 41-13 Table 7-4
V (BSE-1E)	591 kip	ASCE 41-13 Eq. 7-21
V (BSE-2E)	887 kip	ASCE 41-13 Eq. 7-21
k	1	ASCE 41-13 Eq. 7-25

The vertical distribution of loads to each diaphragm level is calculated according to ASCE 41-13 Equation 7-24 and Equation 7-25, as summarized in Table 7-14 and illustrated in Figure 7-10.

The exponent k in ASCE 41-13 Equation 7-25 is found in ASCE 41-13 § 7.4.1.3.2 to be 1.0 because the period of the structure is less than 0.5 seconds.

To illustrate, consider the first row in Table 7-14:

- Column 1 is the diaphragm level.
- Column 2 (h_i) = 27 feet is the height of level i (in this case the roof)
- Column 3 (w_i) = 106.9 is the seismic weight at level I (roof) from Table 3.
- Column 4 ($w_i \times h_i^k$) = 27×106.9^1 = 2,885 kip-ft is the product of Columns 2 and 3. Note the sum of these entries over all rows is 7,210 kip-ft.
- Column 5 (C_{vx}) = $(w \times h^k)/\sum(w \times h^k)$ = 2,885/7,210 = 0.400 is a vertical distribution factor calculated as Column 3 divided by the sum of the entries in Column 4 (ASCE 41-13 Eq. 7-25).
- Columns 6 and 8 (F_x) are the seismic load at each diaphragm level.

$$F_x = C_{vx} \times V \quad \text{(ASCE 41-13 Eq. 7-24)}$$

where:

C_{vx} = 0.4

V = base shear, = 591.4 kips for BSE-1E (LS) or 887.1 kips for BSE-2E (CP) as shown in Table 7-13

F_x = 0.4 × 591.4 = 237 kips for LS, and 0.4× 887.1 = 355 kips for CP

- Columns 7 and 9 (V_{story}) are the story shear for the story immediately below level i, calculated as the sum of the loads F_x above.
- The resulting equivalent static loads and associated story shears are shown schematically in Figure 7-10.

Table 7-14 Equivalent Static Load per ASCE 41-13 Equation 7-25 (Applicable to Both Horizontal Directions)

Diaphragm Level (1)	h_i (ft) (2)	w_i (kip) (3)	$w_i \times h_i^k$ (kip*ft) (4)	C_{vx} (5)	BSE-1E F_X (kip) (6)	BSE-1E V_{story} (kip) (7)	BSE-2E F_X (kip) (8)	BSE-2E V_{story} (kip) (9)
Roof	27	106.9	2,885	0.400	237	237	355	355
Third	18	164.9	2,969	0.412	244	481	365	720
Second	9	150.6	1,356	0.188	111	592	167	887
Σ		422.4	7,210	1	592		887	

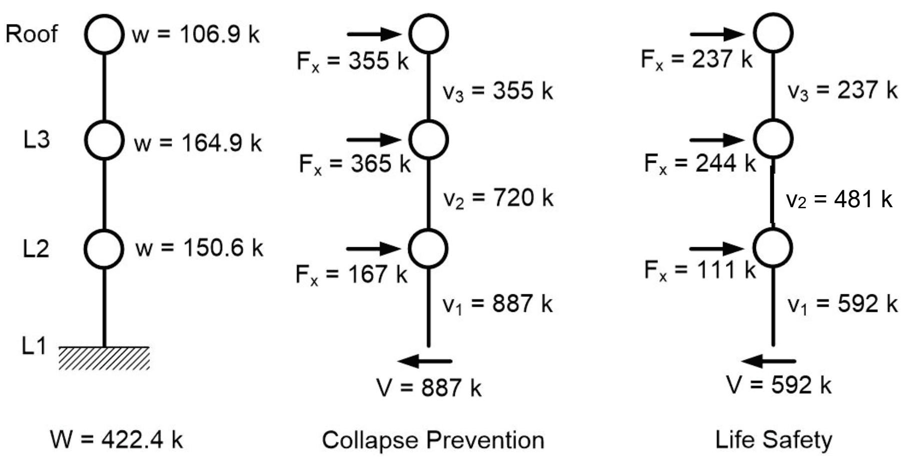


Figure 7-10 Stick model showing diaphragm weights, equivalent static forces and story shears for Collapse Prevention and Life Safety loadings.

Because this structure is better modeled with rigid diaphragms, torsional loads need to be considered. It is worth repeating that the decision to evaluate this building considering rigid diaphragms adds considerable effort compared to the traditional flexible diaphragm method by which wall demands are based only on tributary diaphragm width. Regardless of the pros and cons of each method, given the diaphragm aspect ratios, spacing of shear walls, and presence of lightweight concrete topping, it is unlikely that the diaphragm deflections would meet the criteria for flexible diaphragms in ASCE 41-13 § 7.2.9.1.

Table 7-5 through Table 7-12 contain all the wall locations and stiffnesses and from these, the centroid of the stiffness at each story in each direction can be calculated as the weighted average of each. Because the mass is uniformly distributed, and because the walls are generally symmetric about a north-south centerline, the actual eccentricity between the mass and stiffness is very small in the east west direction. Because of the open front wall, the center of rigidity in the bottom story is found to be 8.5 feet north of the center of mass; asymmetric interior partitions and exterior walls results in a stiffness centroid of 2.8 feet to the south in the second and third stories. Per ASCE 41-13 § 7.2.3, both actual and accidental torsion must be considered, and, depending on the magnitudes of each, the accidental torsion must be amplified. The accidental torsion is 5% of the plan dimension: in the east-west direction 0.05 × 100 ft = 5 ft, while in the north-south direction 0.05 × 36 ft = 1.8 ft.

7.5.7 Torsion

The torsion at each floor is calculated as the story shear multiplied by the eccentricity of that story. Careful attention to sign conventions for rotation is important: shear is considered positive when to the east or north, and rotation is considered positive when counter-clockwise in plan. Table 7-15 and Table 7-16 present the actual and ± 5% accidental eccentricity, and associated BSE-2E applied torsional loads for each story in the north-south and east-west directions, respectively.

To illustrate, consider the first row in Table 7-15:

- Column 1 (Story) is the story level, in this case 3 is second floor to roof.
- Column 2 (Ecc_{ACT}) is the actual eccentricity calculated as the mass center of gravity for the roof (50.0 ft) minus the 3rd story center of resistance (49.95 ft, from Table 7-9, truncated 49.9 feet in the table) = 50 – 49.95 = 0.05 ft (more precisely 0.0543 ft truncated for the table).
- Column 3 (Torsion) is the actual torsion calculated as the third story shear (355 kips from Table 7-14) times the actual eccentricity 0.0543 ft (below) = 355 × 0.0543 = 19 kip-ft.

- Column 4 (+Ecc_{ACC}) is the positive accidental eccentricity based on the prescribed 5% of the north-south dimension = 0.05 × 100 = 5 ft.
- Column 5 (+Torsion) is the positive accidental torsion calculated as the 3rd story shear (355 kips from Table 7-14) times the accidental eccentricity 5 ft = 355 × 5 = 1,775 kip-ft.
- Column 6 (-Ecc_{ACC}) is the negative accidental eccentricity based on the prescribed 5% of the north-south dimension = -0.05 × 100 = -5 ft.
- Column 7 (-Torsion) is the negative accidental torsion calculated as the 3rd story shear (355 kips from Table 7-14) times the accidental eccentricity -5 ft = 355 × -5 = -1,775 kip-ft.

Table 7-15 North-South: Eccentricities and BSE-2E Torsional Moments

Story (1)	Actual Eccentricity		+Accidental		-Accidental	
	Ecc_{ACT} (ft) 2)	Torsion (kip*ft) (3)	+Ecc_{ACC} (ft) (4)	+Torsion (kip*ft) (5)	-Ecc_{ACC} (ft) (6)	-Torsion (kip*ft) (7)
3	0.05	19	5	1,775	-5	-1,775
2	0.05	39	5	3,601	-5	-3,601
1	-0.64	-571	5	4,435	-5	-4,435

Table 7-16 East-West: Eccentricities and BSE-2E Torsional Moments

Story (1)	Actual Eccentricity		+Accidental		-Accidental	
	Ecc_{ACT} (ft) 2)	Torsion (kip*ft) (3)	+Ecc_{ACC} (ft) (4)	+Torsion (kip*ft) (5)	-Ecc_{ACC} (ft) (6)	-Torsion (kip*ft) (7)
3	-2.77	-984	1.8	639	-1.8	-639
2	-2.77	-1,997	1.8	1,297	-1.8	-1,297
1	8.52	7,559	1.8	1,597	-1.8	-1,597

7.5.8 Tier 3 Analysis Results – Existing Structure

Using the loading of Table 7-14, the eccentricities of Table 7-15 and 7-16, and the composite story stiffnesses in Table 7-5 through Table 7-12, the lateral translation and rotation of each rigid floor diaphragm are calculated. Because of the torsion amplification requirements of ASCE 41-13 § 7.2.3.2.2, there are four fundamental load cases that must be considered in each direction: story shear applied through the center of rigidity, actual torsion, positive accidental torsion, and negative accidental torsion. The global loads and resulting lateral displacements and rotations of the rigid diaphragms are provided in Table 7-17 (north-south loading, Load Cases 1 through 4) and Table 7-18 (east-west loading, Load Cases 5 through 8).

Table 7-17 Loads and Displacements for North-South Lateral and Torsional Loads – Existing Building

Story (1)	K_{N-S} (kip/in) (2)	K_θ (kip-ft/rad) (3)	Case 1: Shear to North, No Torsion V_{N-S} (kip) (4)	T (kip-ft) (5)	δ_{N-S} (in) (6)	θ (rad) (7)	Case 2: Actual Torsion V_{N-S} (kip) (8)	T (kip-ft) (9)	δ_{N-S} (in) (10)	θ (rad) (11)
3	150	2,480,049	355	0	2.37	0.000000	0	19	0.00	0.000008
2	150	2,480,049	720	0	4.82	0.000000	0	39	0.00	0.000016
1	113	2,523,628	887	0	7.86	0.000000	0	-571	0.00	-0.000226

Story (1)	K_{N-S} (kip/in) (2)	K_θ (kip-ft/rad) (3)	Case 3: +Accidental Torsion V_{N-S} (kip) (4)	T (kip-ft) (5)	δ_{N-S} (in) (6)	θ (rad) (7)	Case 4: -Accidental Torsion V_{N-S} (kip) (8)	T (kip-ft) (9)	δ_{N-S} (in) (10)	θ (rad) (11)
3	150	2,480,049	0	1,775	0.00	0.000716	0	-1,775	0.00	-0.000716
2	150	2,480,049	0	3,601	0.00	0.001452	0	-3,601	0.00	-0.001452
1	113	2,523,628	0	4,435	0.00	0.001758	0	-4,435	0.00	-0.001758

Table 7-18 Loads and Displacements for East-West Lateral and Torsional Loads – Existing Building

Story (1)	K_{E-W} (kip/in) (2)	K_θ (kip-ft/rad) (3)	Case 5: Shear to East, No Torsion V_{E-W} (kip) (4)	T (kip-ft) (5)	δ_{E-W} (in) (6)	θ (rad) (7)	Case 6: Actual Torsion V_{E-W} (kip) (8)	T (kip-ft) (9)	δ_{E-W} (in) (10)	θ (rad) (11)
3	117	2,480,049	355	0	3.03	0.000000	0	-984	0.00	-0.000397
2	117	2,480,049	720	0	6.15	0.000000	0	-1,997	0.00	-0.000805
1	94	2,523,628	887	0	9.44	0.000000	0	7,559	0.00	0.002995

Story (1)	K_{E-W} (kip/in) (2)	K_θ (kip-ft/rad) (3)	Case 7: +Accidental Torsion V_{E-W} (kip) (4)	T (kip-ft) (5)	δ_{E-W} (in) (6)	θ (rad) (7)	Case 8: -Accidental Torsion V_{E-W} (kip) (8)	T (kip-ft) (9)	δ_{E-W} (in) (10)	θ (rad) (11)
3	117	2,480,049	0	639	0.00	0.000258	0	-639	0.00	-0.000258
2	117	2,480,049	0	1,297	0.00	0.000523	0	-1,297	0.00	-0.000523
1	94	2,523,628	0	1,598	0.00	0.000633	0	-1,597	0.00	-0.000633

In these tables, K_{N-S}, K_{E-W}, and K_Θ are the global stiffnesses in the north-south, east-west and rotation directions, respectively; V_{N-S} and V_{E-W} are the global story shears in the north-south and east-west directions, respectively; T is the global story torsion; and δ_{N-S}, δ_{E-W}, and θ are the global north-south, east-west and rotational displacements, respectively.

To illustrate, consider the first row in Table 7-17:

- Column 1 (Story) is the story level, in this case 3 is second floor to roof.
- Column 2 (K_{N-S}) is the north-south lateral stiffness = 150 kip/in (Table 7-9).

- Column 3 (K_θ) is the rotational stiffness of the 3rd story = 2,480,049 kip-ft /radian (Table 7-9).
- Columns 4 and 8 ($V_{N\text{-}S}$) are the 3rd story shear in the north-south direction = 355 kip (Table 7-14).
- Columns 5 and 9 (T) are the torsion applied to the 3rd story (= 0 by definition for Case 1, = 19 kip-ft for Case 2 from Table 7-15).
- Columns 6 and 10 ($\delta_{N\text{-}S}$) are the north-south translation of the rigid diaphragm = $V_{N\text{-}S}/K_{N\text{-}S}$ = 355/150 = 2.37 in for Case 1, and 0 for Case 2 by definition.
- Columns 7 and 11 (θ) are the rotation of the rigid diaphragm = 0 for Case 1 by definition, and T/K_θ = 19/2,480,049 = 0.000008 radians for Case 2.

The torsional amplifier A_x is calculated per ASCE 41-13 § 7.2.3.2. The amplification is a function of the actual and accidental torsions and rotations. For the existing structure, the torsional amplification was insignificant, and in the retrofit structure, the torsional amplifications were all less than 10%. Therefore, in the interest of brevity and clarity, the details of the torsional amplification calculations will not be presented.

Knowing the displacements (lateral and rotational) for each floor, as well as the wall stiffnesses, the design loads Q_{UD} in each wall are calculated for each load case. The displacement of each wall is the lateral diaphragm (center of rigidity) displacement plus the rotation times the orthogonal distance from the wall to the center of rigidity. It is worth repeating that careful attention to the sign convention is important when calculating the wall drift due to diaphragm rotation.

Four load cases are considered: (1) no torsion; (2) actual torsion; (3) positive accidental torsion; and (4) negative accidental torsion. The largest magnitude absolute value wall load resulting from either positive or negative accidental torsion (when added to the pure translation and actual torsion cases) defines the controlling combination. The demands in each wall are shown in Table 7-19, Table 7-21, Table 7-23, Table 7-25, Table 7-27, and Table 7-29. These wall demands are compared to the expected strengths Q_{CE} times the product of the m-factors and knowledge factor κ determined earlier. In Table 7-20, Table 7-22, Table 7-24, Table 7-26, Table 7-28, and Table 7-30 the acceptance criteria of ASCE 41-13 Equation 7-36 are evaluated for each wall, and shaded red if that criterion is not met.

In Table 7-19, Table 7-21, Table 7-23, Table 7-25, Table 7-27, and Table 7-29, δ_i and Q_i are the displacement and loads in wall i; Q_{UD} is the controlling demand considering torsion in either direction.

To illustrate, consider the first row in Table 7-19:

- Column 1 (Wall) is the wall identifying number from Figure 7-8.
- Column 2 (K) is the wall stiffness, 25.3 kip/in, given in Table 7-9.
- Columns 3 and 4 pertain to pure translation:
    - Column 3 (δ_i) is the displacement due to pure translation, 2.37 inches, from Table 7-17, Column 6, Case 1.
    - Column 4 (Q_i) is the wall load calculated as the product of the stiffness K and the displacement from pure translation, divided by the wall length L.

      $Q_i \quad = K \times \delta_i/L$

      where:

      $K \quad$ = 25.3 kip/in (Column 2)

      $\delta_i \quad$ = 2.37 inches is the wall displacement in pure translation (Column 3)

      $L \quad$ = 36 feet (Table 7-5)

      $Q_i \quad = 25.3 \times 2.37/36 = 1667$ plf (accounting for round-off)
- Columns 5 and 6 pertain to actual torsion:
    - Column 5 (δ_i) is the displacement due to actual torsion, calculated as the rotation times the east-west distance from the wall to the center of stiffness.

      $\delta_i \quad = \theta_i \times X$

      where:

      $\theta_i \quad$ = 0.000008 radians from Table 7-17, Column 11, Case 2.

      $X \quad$ = -49.95 feet (0.00 - 49.95 from Table 7-9)

      $\delta_i \quad = 0.000008 \times -49.95 \times (12 \text{ in/ft}) = -0.0047$ inches. (This is rounded to 0.00 inches the table.)
    - Column 6 (Q_i) is the wall load calculated as the product of the stiffness K and the displacement from actual torsion, divided by the wall length L:

Q_i $= K \times \delta_i/L$

where:

K = 25.3 kip/in (Column 2)

δ_i = -0.0047 inches (Column 5)

L = 36 feet (Table 7-5)

Q_i = 25.3 × -0.0047/36 = -3 plf (accounting for round-off)

- Columns 7 and 8 pertain to positive accidental torsion.
  - Column 7 (δ_i) is the displacement due to positive accidental torsion, calculated as the rotation times the east-west distance from the wall to the center of stiffness:

    δ_i $= \theta_i \times X$

    where:

    θ_i = 0.00072 radians from Table 7-17, Column 7, Case 3

    X = -49.95 feet (0.00 - 49.95 from Table 7-9)

    δ_i = 0.00072 × -49.95 × (12 in/ft) = -0.43 inches

  - Column 8 (Q_i) is the wall load calculated as the product of the stiffness K and the displacement from actual torsion, divided by the wall length L:

    Q_i $= K \times \delta_i/L$

    where:

    K = 25.3 kip/in (Column 2)

    δ_i = 0.-43 inches (Column 7)

    L = 36 feet (Table 7-5)

    Q_i = 25.3 × 0.-43/36 = -301 plf (accounting for round-off)

- Columns 9 and 10 are calculated identically to Columns 7 and 8, respectively, with the rotation direction reversed.
- Column 11 (Q_{UD}) is a candidate wall demand found by summing the demands due to pure translation (Column 4), actual torsion (Column 6) and *positive* accidental torsion (Column 8).

  Q_{UD} = 1,667 – 3 - 301 = 1,362 plf (accounting for round-off)

- Column 12 (Q_{UD}) is a candidate wall demand found by summing the demands due to pure translation (Column 4), actual torsion (Column 6) and *negative* accidental torsion (Column 10).

  Q_{UD} = 1,667 – 3 + 301 = 1,965 plf (accounting for round-off)

- Column 13 (Q_{UD}) is the controlling wall load, taken as the larger of Column 11 and Column 12 (absolute number = 1,965 kips).
- Column 14 (Q_{UD}) is the controlling wall load of Column 13 times the wall length L = 36 feet.

  Q_{UD} = 1,965 × 36 = 70.7 kips (accounting for round-off)

Table 7-19 Demands on Third Story North-South Walls

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	25.3	2.37	1667	0.00	-3	-0.43	-301	0.43	301	1,362	1,965	1,965	70.7
2	4.1	2.37	963	0.00	-2	-0.36	-146	0.36	146	815	1,108	1,108	11.1
3	4.1	2.37	963	0.00	-2	-0.34	-139	0.34	139	822	1,101	1,101	11.0
4	14.6	2.37	963	0.00	-1	-0.26	-104	0.26	104	858	1,066	1,066	38.4
5	4.1	2.37	963	0.00	-1	-0.17	-69	0.17	69	893	1,032	1,032	10.3
6	4.1	2.37	963	0.00	-1	-0.15	-63	0.15	63	900	1,025	1,025	10.2
7	14.6	2.37	963	0.00	0	-0.09	-35	0.09	35	928	997	997	35.9
8	4.1	2.37	963	0.00	0	-0.02	-7	0.02	7	956	970	970	9.7
9	4.1	2.37	963	0.00	0	0.00	0	0.00	0	963	963	963	9.6
10	14.6	2.37	963	0.00	0	0.09	35	-0.09	-35	998	928	998	35.9
11	4.1	2.37	963	0.00	1	0.17	70	-0.17	-70	1,034	894	1,034	10.3
12	4.1	2.37	963	0.00	1	0.19	77	-0.19	-77	1,041	887	1,041	10.4
13	14.6	2.37	963	0.00	1	0.26	105	-0.26	-105	1,069	859	1,069	38.5
14	4.1	2.37	963	0.00	1	0.33	133	-0.33	-133	1,097	832	1,097	11.0
15	4.1	2.37	963	0.00	2	0.34	140	-0.34	-140	1,104	825	1,104	11.0
16	25.3	2.37	1667	0.00	3	0.43	302	-0.43	-302	1,972	1,368	1,972	71.0
Σ			**355**		**0**		**0**		**0**				**395**

Table 7-20, Table 7-22, Table 7-24, Table 7-26, Table 7-28, and Table 7-30 summarize the acceptance criterion check for each wall per ASCE 41-13 Equation 7-36; Q_{CE} is the expected capacity, h/b is the wall aspect ratio; the term $m \times \kappa$ is the m-factor times the knowledge factor as defined in ASCE

41-13 Equation 7-36, and the acceptance ratio of Q_{UD} to ($m \times \kappa \times Q_{CE}$), which is less than 1.0 if the acceptance criterion is met (accepted). To illustrate, consider first row of Table 7-20:

- Column 1 (Wall) is the wall identifying number from Figure 7-8.
- Column 2 (Q_{UD}) is the controlling wall load, 70.7 kips, calculated in Table 7-19, Column 14.
- Column 3 (Q_{CE}) is the wall capacity, 23.8 kips, calculated in Table 7-5, Column 8.
- Column 4 (h/b) is the height to width aspect ratio. The height h = 8 feet (all walls) and wall length b = 36 feet (Table 7-5) so that h/b = 8/36 = 0.2
- Column 5 ($m \times \kappa$) = 4.5 is the product of the component demand modification factor (m) and the knowledge factor (κ). The m-factor is a function of aspect ratio h/b, and per Figure 7-9 is equal to 4.5 for low aspect ratio plywood. Based on the condition assessment described above, κ = 1.0.
- Column 6 ($Q_{CE} \times m \times \kappa$) is the denominator of the acceptance ratio, calculated as the product $Q_{CE} \times (m \times \kappa)$ = 23.8 × 4.5 = 106.9 kips (accounting for round-off)
- Column 7 is the acceptance ratio $Q_{UD}/(m\kappa Q_{CE})$ = 70.7/106.9 = 0.7, which is less than 1.0 and therefore acceptable per ASCE 41-13 Equation 7-36.
- Column 8 highlights the acceptance check.

Table 7-20 Strength Acceptance Criteria Check for Third Story North-South Walls

		Strength Check per ASCE 41-13 Eq. 7-36					
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
1	70.7	23.8	0.2	4.5	106.9	0.7	Yes
2	11.1	4.3	0.8	5.7	24.3	0.5	Yes
3	11.0	4.3	0.8	5.7	24.3	0.5	Yes
4	38.4	15.3	0.2	5.7	87.4	0.4	Yes
5	10.3	4.3	0.8	5.7	24.3	0.4	Yes
6	10.2	4.3	0.8	5.7	24.3	0.4	Yes
7	35.9	15.3	0.2	5.7	87.4	0.4	Yes
8	9.7	4.3	0.8	5.7	24.3	0.4	Yes
9	9.6	4.3	0.8	5.7	24.3	0.4	Yes
10	35.9	15.3	0.2	5.7	87.4	0.4	Yes

Table 7-20 Strength Acceptance Criteria Check for Third Story North-South Walls (continued)

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36					
		Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
11	10.3	4.3	0.8	5.7	24.3	0.4	Yes
12	10.4	4.3	0.8	5.7	24.3	0.4	Yes
13	38.5	15.3	0.2	5.7	87.4	0.4	Yes
14	11.0	4.3	0.8	5.7	24.3	0.5	Yes
15	11.0	4.3	0.8	5.7	24.3	0.5	Yes
16	71.0	23.8	0.2	4.5	106.9	0.7	Yes
					806		

Table 7-21 Demands on Second Story North-South Walls

Wall (1)	K (k/in) (2)	No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
		δ_i (in) (3)	Q_i (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	25.3	4.82	3,382	-0.01	-7	-0.87	-611	0.87	611	2,764	3,986	3,986	143.5
2	4.1	4.82	1,954	-0.01	-3	-0.73	-296	0.73	296	1,654	2,247	2,247	22.5
3	4.1	4.82	1,954	-0.01	-3	-0.70	-282	0.70	282	1,669	2,233	2,233	22.3
4	14.6	4.82	1,954	-0.01	-2	-0.52	-212	0.52	212	1,740	2,163	2,163	77.9
5	4.1	4.82	1,954	0.00	-2	-0.35	-141	0.35	141	1,812	2,093	2,093	20.9
6	4.1	4.82	1,954	0.00	-1	-0.31	-127	0.31	127	1,826	2,080	2,080	20.8
7	14.6	4.82	1,954	0.00	-1	-0.17	-70	0.17	70	1,883	2,024	2,024	72.8
8	4.1	4.82	1,954	0.00	0	-0.03	-14	0.03	14	1,940	1,968	1,968	19.7
9	4.1	4.82	1,954	0.00	0	0.00	0	0.00	0	1,954	1,954	1,954	19.5
10	14.6	4.82	1,954	0.00	1	0.18	71	-0.18	-71	2,026	1,884	2,026	72.9
11	4.1	4.82	1,954	0.00	2	0.35	142	-0.35	-142	2,097	1,814	2,097	21.0
12	4.1	4.82	1,954	0.00	2	0.38	156	-0.38	-156	2,112	1,800	2,112	21.1
13	14.6	4.82	1,954	0.01	2	0.52	212	-0.52	-212	2,169	1,744	2,169	78.1
14	4.1	4.82	1,954	0.01	3	0.66	269	-0.66	-269	2,226	1,688	2,226	22.3
15	4.1	4.82	1,954	0.01	3	0.70	283	-0.70	-283	2,240	1,674	2,240	22.4
16	25.3	4.82	3,382	0.01	7	0.87	612	-0.87	-612	4,001	2,776	4,001	144.0
Σ			**720**		**0**		**0**		**0**				**802**

Table 7-22 **Strength Acceptance Criteria Check for Second Story North-South Walls**

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36					
		Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
1	143.5	23.8	0.2	4.5	106.9	1.3	No
2	22.5	4.3	0.8	5.7	24.3	0.9	Yes
3	22.3	4.3	0.8	5.7	24.3	0.9	Yes
4	77.9	15.3	0.2	5.7	87.4	0.9	Yes
5	20.9	4.3	0.8	5.7	24.3	0.9	Yes
6	20.8	4.3	0.8	5.7	24.3	0.9	Yes
7	72.8	15.3	0.2	5.7	87.4	0.8	Yes
8	19.7	4.3	0.8	5.7	24.3	0.8	Yes
9	19.5	4.3	0.8	5.7	24.3	0.8	Yes
10	72.9	15.3	0.2	5.7	87.4	0.8	Yes
11	21.0	4.3	0.8	5.7	24.3	0.9	Yes
12	21.1	4.3	0.8	5.7	24.3	0.9	Yes
13	78.1	15.3	0.2	5.7	87.4	0.9	Yes
14	22.3	4.3	0.8	5.7	24.3	0.9	Yes
15	22.4	4.3	0.8	5.7	24.3	0.9	Yes
16	144.0	23.8	0.2	4.5	106.9	1.3	No
						806	

Table 7-23 **Demands on First Story North-South Walls**

Wall (1)	K (k/in) (2)	No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
		δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	25.3	7.86	5,51,5	0.14	97	-1.07	-750	1.07	750	4,862	6,362	6,362	229.0
2	4.9	7.86	3,187	0.09	38	-0.73	-296	0.73	296	2,928	3,521	3,521	42.3
3	6.5	7.86	3,187	0.06	25	-0.48	-194	0.48	194	3,018	3,405	3,405	54.5
4	7.7	7.86	3,187	0.01	4	-0.08	-31	0.08	31	3,159	3,222	3,222	61.2
5	32.0	7.86	6,972	-0.01	-6	0.05	44	-0.05	-44	7,011	6,923	7,011	252.4
6	6.5	7.86	3,187	-0.06	-24	0.45	183	-0.45	-183	3,346	2,980	3,346	53.5
7	4.9	7.86	3,187	-0.09	-37	0.70	285	-0.70	-285	3,435	2,865	3,435	41.2
8	25.3	7.86	5,515	-0.13	-94	1.04	731	-1.04	-731	6,152	4,690	6,152	221.5
Σ			**887**		**0**		**0**		**0**				**956**

Table 7-24 Strength Acceptance Criteria Check for First Story North-South Walls

		Strength Check per ASCE 41-13 Eq. 7-36					
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
1	229.0	23.8	0.2	4.5	106.9	2.1	No
2	42.3	5.1	0.7	5.7	29.1	1.5	No
3	54.5	6.8	0.5	5.7	38.9	1.4	No
4	61.2	8.1	0.4	5.7	46.1	1.3	No
5	252.4	24.0	0.2	3.6	86.3	2.9	No
6	53.5	6.8	0.5	5.7	38.9	1.4	No
7	41.2	5.1	0.7	5.7	29.1	1.4	No
8	221.5	23.8	0.2	4.5	106.9	2.1	No
					482		

Table 7-25 Demands on Third Story East-West Walls

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	9.7	3.03	2,09,8	0.10	68	-0.06	-44	0.06	44	2,122	2,210	2,210	30.9
2	9.7	3.03	2,098	0.10	68	-0.06	-44	0.06	44	2,122	2,210	2,210	30.9
3	4.7	3.03	2,048	0.10	67	-0.06	-43	0.06	43	2,072	2,158	2,158	15.1
4	3.2	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	9.9
5	6.5	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	19.8
6	6.5	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	19.8
7	6.5	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	19.8
8	6.5	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	19.8
9	3.2	3.03	1,230	0.01	5	-0.01	-3	0.01	3	1,232	1,239	1,239	9.9
10	3.2	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1,220	1,228	9.8
11	4.1	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1220	1,228	12.3
12	6.5	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1220	1,228	19.6
13	4.1	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1220	1,228	12.3
14	6.5	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1220	1,228	19.6
15	3.2	3.03	1,230	-0.02	-6	0.01	4	-0.01	-4	1,228	1220	1,228	9.8
16	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1931	1,993	10.0
17	2.6	3.03	1,978	-0.07	-47	0.05	31	-0.05	-31	1,962	1900	1,962	7.8
18	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1931	1,993	10.0

Table 7-25 Demands on Third Story East-West Walls (continued)

Wall (1)	K (k/in) (2)	No Torsion δ_i (in) (3)	No Torsion Q_i (plf) (4)	Actual Torsion δ_i (in) (5)	Actual Torsion Q_i (plf) (6)	+ Accidental Torsion δ_i (in) (7)	+ Accidental Torsion Q_i (plf) (8)	- Accidental Torsion δ_i (in) (9)	- Accidental Torsion Q_i (plf) (10)	$+T_{ACC}$ $+Q_{UD}$ (plf) (11)	$-T_{ACC}$ $-Q_{UD}$ (plf) (12)	Demand Q_{UD} (plf) (13)	Demand Q_{UD} (kip) (14)
19	2.6	3.03	1,978	-0.07	-47	0.05	31	-0.05	-31	1,962	1,900	1,962	7.8
20	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1,931	1,993	10.0
21	2.6	3.03	1,978	-0.07	-47	0.05	31	-0.05	-31	1,962	1,900	1,962	7.8
22	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1,931	1,993	10.0
23	2.6	3.03	1,978	-0.07	-47	0.05	31	-0.05	-31	1,962	1,900	1,962	7.8
24	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1,931	1,993	10.0
25	2.6	3.03	1,978	-0.07	-47	0.05	31	-0.05	-31	1,962	1,900	1,962	7.8
26	3.3	3.03	2,010	-0.07	-48	0.05	31	-0.05	-31	1,993	1,931	1,993	10.0
Σ			**355**		**0**		**0**		**0**				**359**

Table 7-26 Strength Acceptance Criteria Check for Third Story East-West Walls

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36 Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
1	30.9	9.2	0.6	4.5	41.6	0.7	Yes
2	30.9	9.2	0.6	4.5	41.6	0.7	Yes
3	15.1	4.6	1.1	4.5	20.8	0.7	Yes
4	9.9	3.4	1.0	5.7	19.4	0.5	Yes
5	19.8	6.8	0.5	5.7	38.9	0.5	Yes
6	19.8	6.8	0.5	5.7	38.9	0.5	Yes
7	19.8	6.8	0.5	5.7	38.9	0.5	Yes
8	19.8	6.8	0.5	5.7	38.9	0.5	Yes
9	9.9	3.4	1.0	5.7	19.4	0.5	Yes
10	9.8	3.4	1.0	5.7	19.4	0.5	Yes
11	12.3	4.3	0.8	5.7	24.3	0.5	Yes
12	19.6	6.8	0.5	5.7	38.9	0.5	Yes
13	12.3	4.3	0.8	5.7	24.3	0.5	Yes
14	19.6	6.8	0.5	5.7	38.9	0.5	Yes
15	9.8	3.4	1.0	5.7	19.4	0.5	Yes
16	10.0	3.3	1.6	4.5	14.9	0.7	Yes
17	7.8	2.6	2.0	4.5	11.9	0.7	Yes
18	10.0	3.3	1.6	4.5	14.9	0.7	Yes

Table 7-26 Strength Acceptance Criteria Check for Third Story East-West Walls (continued)

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36					
		Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
19	7.8	2.6	2.0	4.5	11.9	0.7	Yes
20	10.0	3.3	1.6	4.5	14.9	0.7	Yes
21	7.8	2.6	2.0	4.5	11.9	0.7	Yes
22	10.0	3.3	1.6	4.5	14.9	0.7	Yes
23	7.8	2.6	2.0	4.5	11.9	0.7	Yes
24	10.0	3.3	1.6	4.5	14.9	0.7	Yes
25	7.8	2.6	2.0	4.5	11.9	0.7	Yes
26	10.0	3.3	1.6	4.5	14.9	0.7	Yes
					612		

Table 7-27 Demands on Second Story East-West Walls

Wall (1)	K (k/in) (2)	No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
		δ_i (in) (3)	Q_i (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	9.7	6.15	4,256	0.20	139	-0.13	-90	0.13	90	4,305	4,485	4,485	62.8
2	9.7	6.15	4,256	0.20	139	-0.13	-90	0.13	90	4,305	4,485	4,485	62.8
3	4.7	6.15	4,156	0.20	136	-0.13	-88	0.13	88	4,203	4,379	4,379	30.7
4	3.2	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	20.1
5	6.5	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	40.2
6	6.5	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	40.2
7	6.5	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	40.2
8	6.5	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	40.2
9	3.2	6.15	2,496	0.03	11	-0.02	-7	0.02	7	2,500	2,514	2,514	20.1
10	3.2	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	19.9
11	4.1	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	24.9
12	6.5	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	39.9
13	4.1	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	24.9
14	6.5	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	39.9
15	3.2	6.15	2,496	-0.03	-13	0.02	8	-0.02	-8	2,491	2,475	2,491	19.9
16	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
17	2.6	6.15	4,014	-0.15	-96	0.10	62	-0.10	-62	3,980	3,856	3,980	15.9

Table 7-27 Demands on Second Story East-West Walls (continued)

Wall (1)	K (k/in) (2)	No Torsion δ_i (in) (3)	No Torsion Qi (plf) (4)	Actual Torsion δ_i (in) (5)	Actual Torsion Q_i (plf) (6)	+ Accidental Torsion δ_i (in) (7)	+ Accidental Torsion Q_i (plf) (8)	- Accidental Torsion δ_i (in) (9)	- Accidental Torsion Q_i (plf) (10)	$+T_{ACC}$ $+Q_{UD}$ (plf) (11)	$-T_{ACC}$ $-Q_{UD}$ (plf) (12)	Demand Q_{UD} (plf) (13)	Demand Q_{UD} (kip) (14)
18	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
19	2.6	6.15	4,014	-0.15	-96	0.10	62	-0.10	-62	3,980	3,856	3,980	15.9
20	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
21	2.6	6.15	4,014	-0.15	-96	0.10	62	-0.10	-62	3,980	3,856	3,980	15.9
22	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
23	2.6	6.15	4,014	-0.15	-96	0.10	62	-0.10	-62	3,980	3,856	3,980	15.9
24	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
25	2.6	6.15	4,014	-0.15	-96	0.10	62	-0.10	-62	3,980	3,856	3,980	15.9
26	3.3	6.15	4,079	-0.15	-98	0.10	63	-0.10	-63	4,045	3,918	4,045	20.2
Σ			**720**		**0**		**0**		**0**				**728**

Table 7-28 Strength Acceptance Criteria Check for Second Story East-West Walls

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36 Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
1	62.8	9.2	0.6	4.5	41.6	1.5	No
2	62.8	9.2	0.6	4.5	41.6	1.5	No
3	30.7	4.6	1.1	4.5	20.8	1.5	No
4	20.1	3.4	1.0	5.7	19.4	1.0	No
5	40.2	6.8	0.5	5.7	38.9	1.0	No
6	40.2	6.8	0.5	5.7	38.9	1.0	No
7	40.2	6.8	0.5	5.7	38.9	1.0	No
8	40.2	6.8	0.5	5.7	38.9	1.0	No
9	20.1	3.4	1.0	5.7	19.4	1.0	No
10	19.9	3.4	1.0	5.7	19.4	1.0	No
11	24.9	4.3	0.8	5.7	24.3	1.0	No
12	39.9	6.8	0.5	5.7	38.9	1.0	No
13	24.9	4.3	0.8	5.7	24.3	1.0	No
14	39.9	6.8	0.5	5.7	38.9	1.0	No
15	19.9	3.4	1.0	5.7	19.4	1.0	No
16	20.2	3.3	1.6	4.5	14.9	1.4	No

Table 7-28 Strength Acceptance Criteria Check for Second Story East-West Walls (continued)

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36 Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
17	15.9	2.6	2.0	4.5	11.9	1.3	No
18	20.2	3.3	1.6	4.5	14.9	1.4	No
19	15.9	2.6	2.0	4.5	11.9	1.3	No
20	20.2	3.3	1.6	4.5	14.9	1.4	No
21	15.9	2.6	2.0	4.5	11.9	1.3	No
22	20.2	3.3	1.6	4.5	14.9	1.4	No
23	15.9	2.6	2.0	4.5	11.9	1.3	No
24	20.2	3.3	1.6	4.5	14.9	1.4	No
25	15.9	2.6	2.0	4.5	11.9	1.3	No
26	20.2	3.3	1.6	4.5	14.9	1.4	No
					612		

Table 7-29 Demands on First Story East-West Walls

Wall (1)	K (k/in) (2)	No Torsion δ_i (in) (3)	No Torsion Q_i (plf) (4)	Actual Torsion δ_i (in) (5)	Actual Torsion Q_i (plf) (6)	+ Accidental Torsion δ_i (in) (7)	+ Accidental Torsion Q_i (plf) (8)	- Accidental Torsion δ_i (in) (9)	- Accidental Torsion Q_i (plf) (10)	$+T_{ACC}$ $+Q_{UD}$ (plf) (11)	$-T_{ACC}$ $-Q_{UD}$ (plf) (12)	Demand (Eq. 7-36) Q_{UD} (plf) (13)	Demand (Eq. 7-36) Q_{UD} (kip) (14)
1	4.0	9.44	6,327	-0.34	-228	-0.07	-48	0.07	48	6,050	6,146	6,146	36.9
2	5.4	9.44	6,414	-0.34	-231	-0.07	-49	0.07	49	6,134	6,231	6,231	49.9
3	3.3	9.44	6,258	-0.34	-226	-0.07	-48	0.07	48	5,985	6,080	6,080	30.4
4	2.6	9.44	6,159	-0.34	-222	-0.07	-47	0.07	47	5,890	5,984	5,984	23.9
5	4.0	9.44	6,327	-0.34	-228	-0.07	-48	0.07	48	6,050	6,146	6,146	36.9
6	4.0	9.44	6,327	-0.34	-228	-0.07	-48	0.07	48	6,050	6,146	6,146	36.9
7	2.6	9.44	6,159	-0.34	-222	-0.07	-47	0.07	47	5,890	5,984	5,984	23.9
8	3.3	9.44	6,258	-0.34	-226	-0.07	-48	0.07	48	5,985	6,080	6,080	30.4
9	5.4	9.44	6,414	-0.34	-231	-0.07	-49	0.07	49	6,134	6,231	6,231	49.9
10	4.0	9.44	6,327	-0.34	-228	-0.07	-48	0.07	48	6,050	6,146	6,146	36.9
11	4.9	9.44	3,829	0.09	37	0.02	8	-0.02	-8	3,874	3,858	3,874	46.5
12	3.7	9.44	3,829	0.09	37	0.02	8	-0.02	-8	3,874	3,858	3,874	34.9
13	3.7	9.44	3,829	0.09	37	0.02	8	-0.02	-8	3,874	3,858	3,874	34.9
14	4.9	9.44	3,829	0.09	37	0.02	8	-0.02	-8	3,874	3,858	3,874	46.5
15	19.1	9.44	3,829	0.31	124	0.06	26	-0.06	-26	3,980	3,927	3,980	187.0
16	19.1	9.44	3,829	0.31	124	0.06	26	-0.06	-26	3,980	3,927	3,980	187.0
Σ			**887**		**0**		**0**		**0**				**893**

Table 7-30 Strength Acceptance Criteria Check for First Story East-West Walls

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	$Q_{UD}/(m\kappa Q_{CE})$ (7)	Accept? (8)
		Strength Check per ASCE 41-13 Eq. 7-36					
1	36.9	4.0	1.3	4.5	17.8	2.1	No
2	49.9	5.3	1.0	4.5	23.8	2.1	No
3	30.4	3.3	1.6	4.5	14.9	2.0	No
4	23.9	2.6	2.0	4.5	11.9	2.0	No
5	36.9	4.0	1.3	4.5	17.8	2.1	No
6	36.9	4.0	1.3	4.5	17.8	2.1	No
7	23.9	2.6	2.0	4.5	11.9	2.0	No
8	30.4	3.3	1.6	4.5	14.9	2.0	No
9	49.9	5.3	1.0	4.5	23.8	2.1	No
10	36.9	4.0	1.3	4.5	17.8	2.1	No
11	46.5	5.1	0.7	5.7	29.1	1.6	No
12	34.9	3.8	0.9	5.7	21.9	1.6	No
13	34.9	3.8	0.9	5.7	21.9	1.6	No
14	46.5	5.1	0.7	5.7	29.1	1.6	No
15	187.0	20.0	0.2	5.7	114.1	1.6	No
16	187.0	20.0	0.2	5.7	114.1	1.6	No
					502		

7.6 Schematic Retrofit

An important aspect of the tuck-under retrofit design is that not all nonconforming shear wall elements need to be retrofit to meet the acceptance criteria of ASCE 41-13 Equation 7-36. For wood frame construction, ASCE 41-13 Table 12-3, footnote "a" permits shear walls to be declared secondary components, and states that acceptance criteria need not be checked for these elements. ASCE 41-13 § 7.2.3.3 limits the total lateral stiffness of secondary elements to 25% of the story stiffness. Therefore, the retrofit strategy will be to select elements on each floor, based on location and access, to be classified as primary elements, strengthened with additional wood panel sheathing (OSB) as required to meet the strength acceptance criterion of ASCE 41-13 § 7.5.2.2. (The parking function of the bottom floor (L1) will not allow shear walls to be added along the front wall, and steel moment frames will be used around the openings.) Other elements will be classified as secondary (after checking that their total stiffness is less than 25% of the story stiffness) and not further addressed. In the spreadsheet analysis of the retrofit structure, the secondary elements will be assigned zero stiffness and will therefore not contribute to the building response.)

As seen in Table 7-20 and Table 7-26, the strength acceptance criteria of ASCE 41-13 Equation 7-36 are met in both directions of the top story, and thus no further strengthening is required. However, hold-down devices would be required on several walls to meet overturning requirement of ASCE 41-13 Equation 7-6. Rather than forcing compliance by adding hold-down devices to gypsum wallboard shear walls, a more robust and reliable retrofit was implemented by adding plywood (and hold-downs) to select shear walls on the top story in both directions.

If the shear capacity of the non-plywood walls in the top story of the existing building is discounted, the sum of the plywood wall capacities (Σ ($Q_{CE} \times m \times \kappa$) in Table 7-20, Column 6 from Walls 1 and 16 = 214 k) falls short of the total shear demand (Σ Q_{UD} from Table 7-19, Column 14 = 395 k) by about 181 k in the north-south direction; the associated east-west deficit is 106 k. Retrofit walls with 15/32-inch OSB and 8d nails spaced at 4 inches will add about 1.29 klf (Q_{CE} in Table 7-4), and, assuming an m-factor of 4.5, a preliminary retrofit estimate would require 181 kips/(4.5 × 1.29 klf) = about 31 feet of additional shear wall sheathing in the north-south direction (20 feet in the east-west direction, by the same calculation). Ultimately, it is decided that the addition of additional retrofit shear wall will reduce asymmetry and decrease hold-down requirements in other walls.

Based on similar analysis, the first and second stories will require substantially more strengthening in both directions to meet the CP strength acceptance criteria. Based on strength deficits of 131 k and 106 k, the second story will require approximately 101 feet and 82 feet of additional plywood sheathing in the north-south and east-west directions, respectively. Likewise, the bottom story will require about 128 feet of plywood shear wall in the north-south direction to make up for a strength deficit of 165 kips, and 160 kips of additional strength, using some combination of structural wood panel sheathing and steel frames, in the east-west direction. (The moment frame design is ultimately controlled by stiffness, not strength.)

The spreadsheet developed to evaluate the existing building is modified to remove the secondary shear walls and to add retrofit elements. After a few iterations, the retrofit scheme described in the following section is shown to meet the acceptance criteria of ASCE 41-13 Equation 7-36 for wall strength.

7.7 Tier 3 Retrofit

The retrofit selected for this example uses structural wall panel shear walls and moment frames composed of standard structural sections. It is not presumed that this is the optimum retrofit for all circumstances; the experiences of the engineer and contractor may dictate other methods or materials to achieve the same expected performance. This retrofit does not reference specific proprietary (sub)assemblies that have been qualified by testing as allowed in ASCE 41-13 § 7.6, though such assemblies may be good candidates for this retrofit.

Commentary

The reader is encouraged to research proprietary moment frames that have been developed for soft story retrofits and shown by testing to meet ASCE 41-13 performance requirements.

In some cases, the acceptance ratios of Q_{UD} to $m\kappa Q_{CE}$ are low. While more optimal solutions might be available by reducing the amount of shear wall, in many cases, extra wall was added to reduce torsion (asymmetry) or because it was decided that the incremental strength and stiffness benefit of retrofitting an entire length of wall outweighed the additional cost compared to retrofitting only a portion of a wall. In addition, extra stiffness was added in some cases to draw load from other walls to avoid additional retrofit locations. Such decisions would be dictated by the specific circumstances of any retrofit.

Commentary

Per ASCE 41-13 § 7.2.3.3, all elements must be designated as primary of secondary. In this example, only those elements shown in Figure 7-11 through Figure 7-15 are primary elements. All other elements are secondary or nonstructural.

7.7.1 Retrofit Design Elements

7.7.1.1 Top (Third) Story

In the east-west direction in the top two stories, the exterior plywood walls and north corridor walls are chosen as primary elements; all other east-west walls are considered secondary. In the north-south direction, only the exterior plywood walls and the four full-length interior transverse walls are considered primary elements. Using the same primary elements in the top two stories avoids the introduction of discontinuous shear walls and associated diaphragm demands. (The stiffness of the secondary elements was summed in the spreadsheet and shown to be less than 25% of the story strength in all cases, thereby satisfying ASCE 41-13 § 7.2.3.3.) Figure 7-11 shows the primary elements used for the second and third stories.

In the north-south direction, two interior shear walls (#7 and #10, 72 feet total) will be sheathed full length (one side) with 15/32 OSB using 8d nails spaced at 4 inches on the perimeters and 12 inches in the field. Per SDPWS, this wall has a yield strength of 860 plf, and per ASCE 41-13 § 12.4.4.6.2, the expected capacity Q_{CE} is thus 1.5 × 860 = 1,290 plf. In the east-west direction, the same sheathing is applied to interior bearing Walls #5, #6, #7 and #8 (62 feet total). These retrofit walls in conjunction with exterior plywood walls (not retrofit), as shown in Figure 7-12, provide sufficient strength to meet the BPOE; all other walls are considered secondary elements. The figure shows which walls are to be sheathed, and locations of required hold-downs to resist overturning per ASCE 41-13 Equation 7-6. (Note that calculations indicate that hold-downs are not required for some of the primary shear walls, though the engineer may decide to include them for additional capacity.) Wall types in Figure 7-12, Figure 7-13, and Figure 7-15 correspond to those listed in Table 7-4.

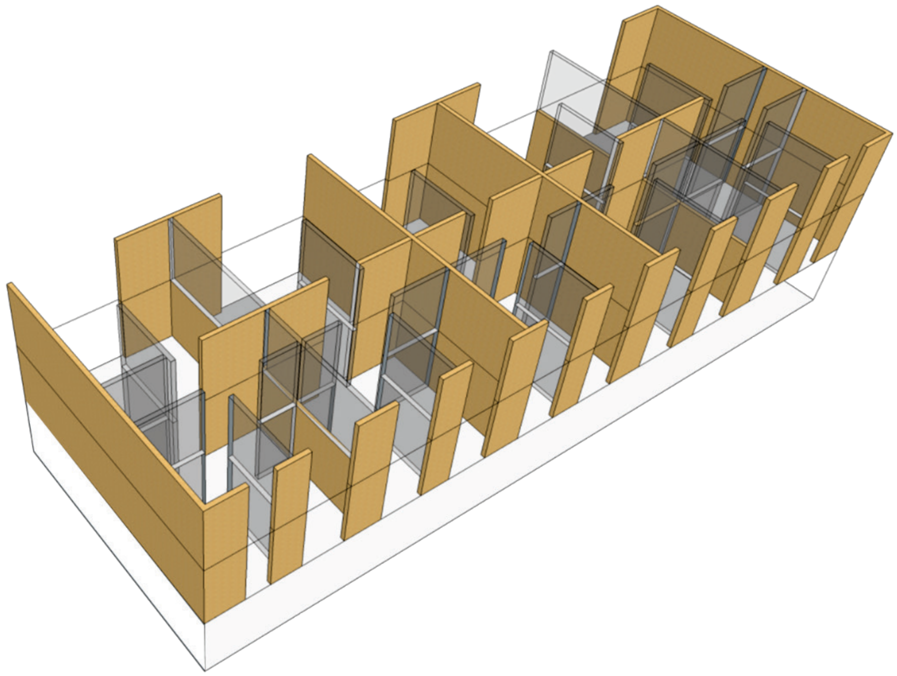


Figure 7-11 Primary elements of the top two stories of retrofit shown with solid brown fill.

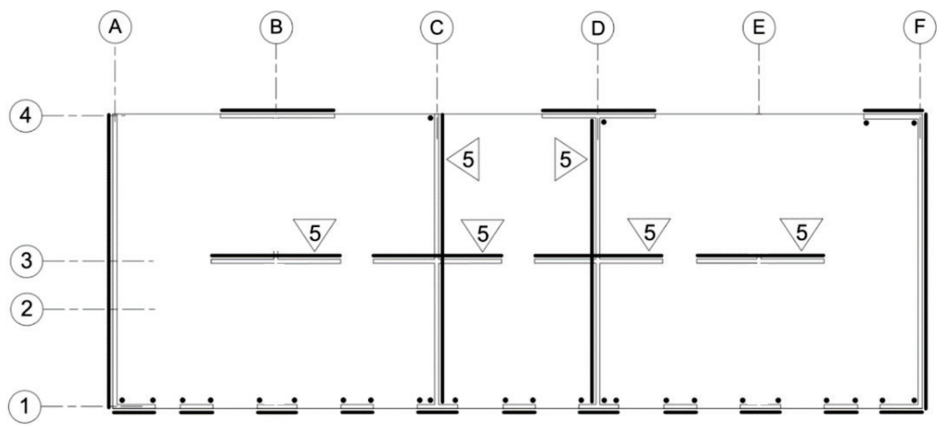


Figure 7-12 Top story primary elements. (Type 5 is structural panel added on one side; existing exterior plywood utilized without retrofit; black dots represent added hold-down locations.)

7.7.1.2 Second Story

As discussed in Section 7.5.6 of this *Example Application Guide*, the second story requires significant additional strength. Figure 7-13 shows only the interior walls that are retrofit with 15/32 OSB using 8d nails apaced at 4 inches on the perimeter and 12 inches in the field. (Several of the walls receive structural panel sheathing on both sides.) These retrofit walls in conjunction with exterior plywood walls (no retrofit) provide sufficient strength to meet the BPOE; all other walls are considered secondary elements.

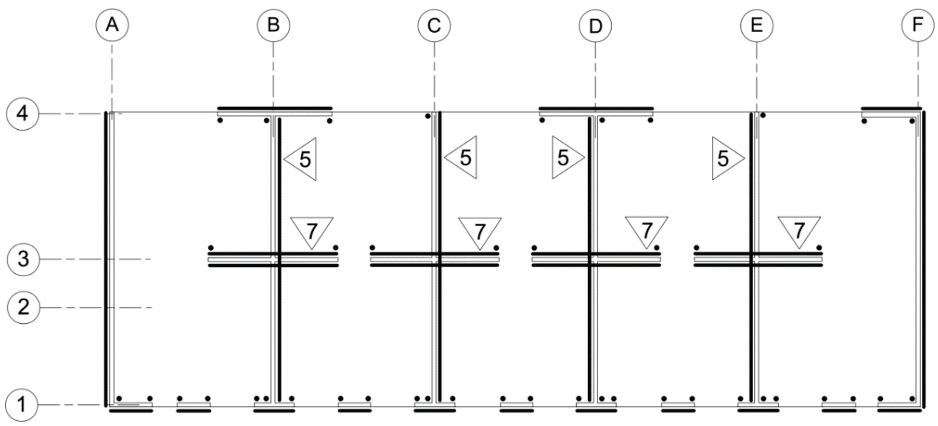


Figure 7-13 Second story primary elements. (Type 5 is structural panel added on one side; Type 7 is two-sided structural panel; existing exterior plywood shear walls utilized; black dots represent hold-down locations.)

7.7.1.3 Bottom Story

The bottom story is significantly deficient and required substantial retrofit. Figure 7-14 and Figure 7-15 show only the primary shear walls and two moment frames. In the north-south direction, exterior Walls #5 and #8 require an additional layer of interior structural wood panel (15/32 OSB using 8d nails spaced at 4 inches on the perimeter and 12 inches in the field). These retrofit walls, in conjunction with exterior plywood walls also require sheathing on the interior for sufficient strength to satisfy the BPOE. As with the other stories, all walls not shown in the retrofit figures are considered secondary elements, and per ASCE 41-13 Table 12-3 Footnote (a) do not need to be checked for compliance.

7.7.2 Tier 3 Analysis Results

The global translations and rotations for the fundamental load combinations are recalculated for the retrofit building; Table 7-31 and Table 7-32 provide the same information in, and are calculated in the same way as, Table 7-17 and Table 7-18 but for the retrofit condition. Note that the model only includes stiffness contributions from the primary elements, and thus overestimates the deflections (and primary element demands).

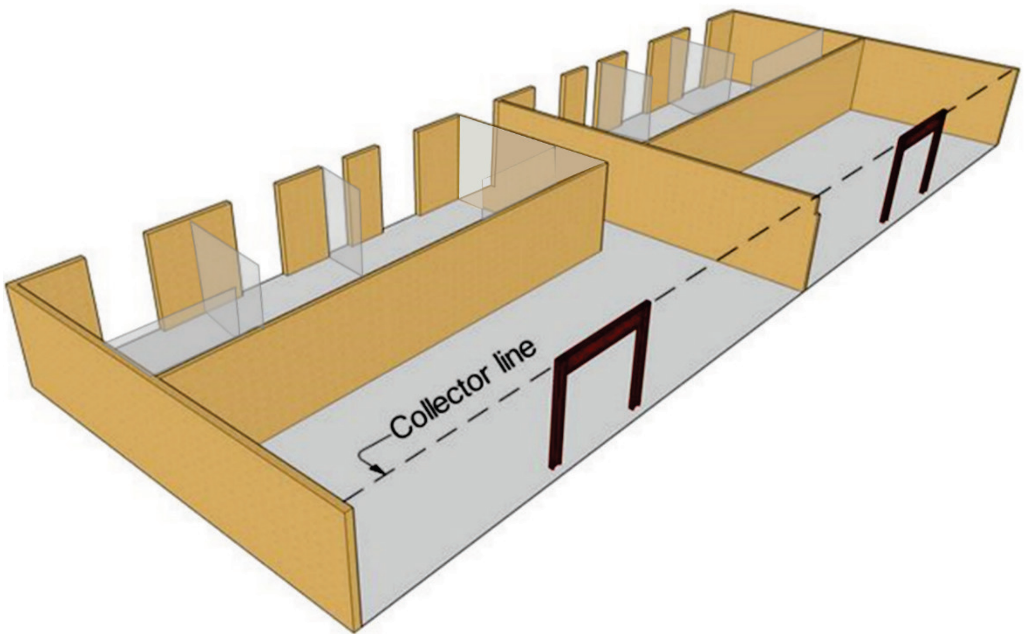


Figure 7-14 Primary elements of the bottom story of the retrofit shown with solid brown fill.

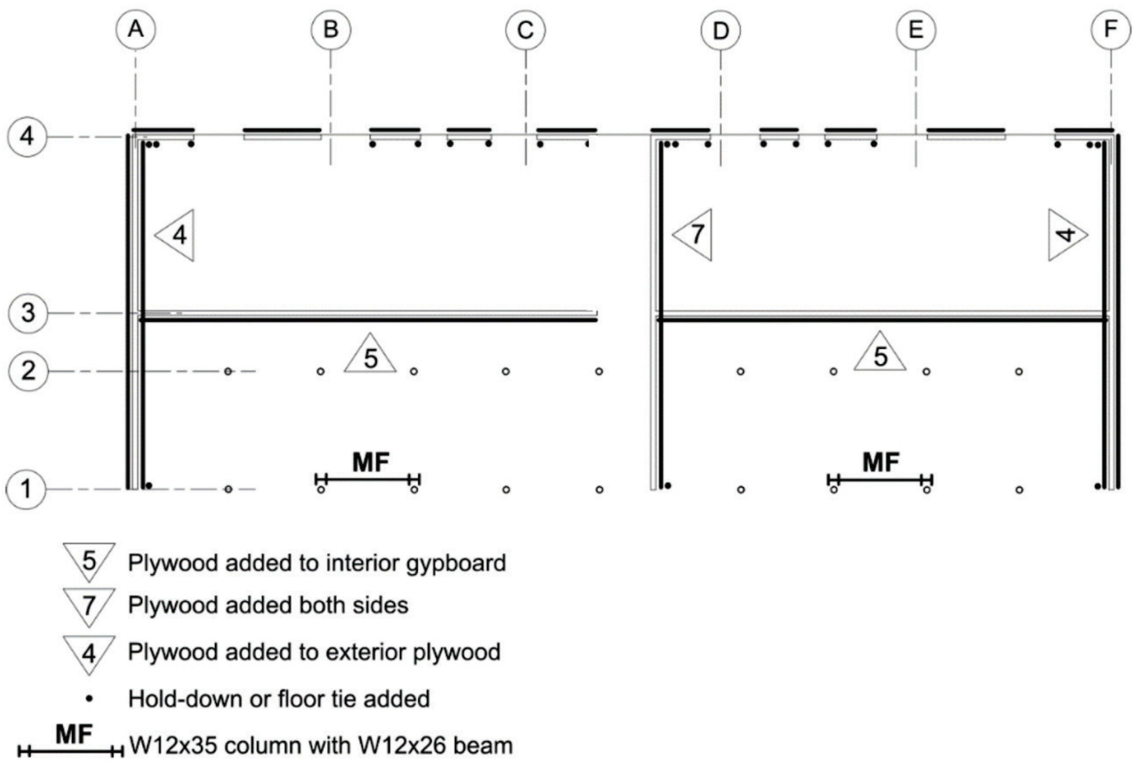


Figure 7-15 Bottom story primary elements. Type 5 is structural panel added on one side; Type 7 is two-sided structural panel; Type 4 is structural panel added on inside face of exterior wall; existing black dots on exterior plywood shear walls represent hold-down locations.

Table 7-31 Loads and Displacements for North-South Lateral and Torsional Loads – Retrofit Building

			Case 1: Shear to North, No Torsion				Case 2: Actual Torsion			
Story (1)	K_{N-S} (kip/in) (2)	K_θ (kip-ft/rad) (3)	V_{N-S} (kip) (4)	T (kip-ft) (5)	δ_{N-S} (in) (6)	θ (rad) (7)	V_{N-S} (kip) (8)	T (kip-ft) (9)	δ_{N-S} (in) (10)	θ (rad) (11)
3	154	1,773,084	355	0	2.31	0.000000	0	0	0.00	0.000000
2	259	2,905,965	720	0	2.79	0.000000	0	0	0.00	0.000000
1	258	5,348,539	887	0	3.44	0.000000	0	-1,071	0.00	-0.000200
			Case 3: +Accidental Torsion				Case 4: -Accidental Torsion			
Story (1)	K_{N-S} (kip/in) (2)	K_θ (kip-ft/rad) (3)	V_{N-S} (kip) (4)	T (kip-ft) (5)	δ_{N-S} (in) (6)	θ (rad) (7)	V_{N-S} (kip) (8)	T (kip-ft) (9)	δ_{N-S} (in) (10)	θ (rad) (11)
3	154	1,773,084	0	1,775	0.00	0.001001	0	-1,775	0.00	-0.001001
2	259	2,905,965	0	3,601	0.00	0.001239	0	-3,601	0.00	-0.001239
1	258	5,348,539	0	4,435	0.00	0.000829	0	-4,435	0.00	-0.000829

Table 7-32 Loads and Displacements for East-West Lateral and Torsional Loads – Retrofit Building

			Case 5: Shear to East, No Torsion				Case 6: Actual Torsion			
Story (1)	K_{E-W} (kip/in) (2)	K_θ (kip-ft/rad) (3)	V_{E-W} (kip) (4)	T (kip-ft) (5)	δ_{E-W} (in) (6)	θ (rad) (7)	V_{E-W} (kip) (8)	T (kip-ft) (9)	δ_{E-W} (in) (10)	θ (rad) (11)
3	132	1,773,084	355	0	2.69	0.000000	0	-241	0.00	-0.000136
2	214	2,905,965	721	0	3.37	0.000000	0	-302	0.00	-0.000104
1	224	5,348,539	887	0	3.97	0.000000	0	-1,488	0.00	-0.000278
			Case 7: +Accidental Torsion				Case 8: -Accidental Torsion			
Story (1)	K_{E-W} (kip/in) (2)	K_θ (kip-ft/rad) (3)	V_{E-W} (kip) (4)	T (kip-ft) (5)	δ_{E-W} (in) (6)	θ (rad) (7)	V_{E-W} (kip) (8)	T (kip-ft) (9)	δ_{E-W} (in) (10)	θ (rad) (11)
3	132	1,773,084	0	639	0.00	0.000360	0	-639	0.00	-0.000360
2	214	2,905,965	0	1,297	0.00	0.000446	0	-1,297	0.00	-0.000446
1	224	5,348,539	0	1,597	0.00	0.000299	0	-1,597	0.00	-0.000299

Given these global translations and displacements, the wall and frame element demands are calculated and summarized in Table 7-33, Table 7-35, Table 7-37, Table 7-39, Table 7-41, and Table 7-43; the calculations and column definitions are identical to those described for Table 7-19. Table 7-34, Table 7-36, Table 7-38, Table 7-40, Table 7-42, and Table 7-44 summarize the acceptance criteria calculations of ASCE 41-13 Equation 7-6 and Equation 7-36 and are identical to Table 7-20 except that they pertain to the retrofit condition and they also summarize hold-down criteria and demands in Columns 9 to 13.

Table 7-33 Demands on Third Story North-South Walls – Retrofit Building

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	24.4	2.31	1,566	0.00	0	-0.60	-407	0.60	407	1,117	2,015	2,015	72.5
7	52.4	2.31	3,364	0.00	0	-0.12	-175	0.12	175	3,189	3,539	3,539	127.4
10	52.4	2.31	3,364	0.00	0	0.12	175	-0.12	-175	3,539	3,189	3,539	127.4
16	24.4	2.31	1,566	0.00	0	0.60	407	-0.60	-407	2,015	1,117	2,015	72.5
Σ			**355**		**0**		**0**		**0**				**400**

Table 7-34 Strength Acceptance Criteria Check for Third Story North-South Walls – Retrofit Building

		Strength Check per ASCE 41-13 Eq. 7-36						Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0)				
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	Accept? (8)	0.9 × M_{ST} (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	HD Req'd? (11)	Q_{UD} (kip) (12)	Q_{UF} (kip) (13)
1	72.5	23.8	0.2	4.5	106.9	0.7	Yes	115.1	41.5	No	0.0	0.0
7	127.4	46.4	0.2	4.5	209.0	0.6	Yes	63.2	72.8	Yes	27.3	8.8
10	127.4	46.4	0.2	4.5	209.0	0.6	Yes	63.2	72.8	Yes	27.3	8.8
16	72.5	23.8	0.2	4.5	106.9	0.7	Yes	115.1	41.5	No	0.0	0.0
					632							

Table 7-35 Demands on Second Story North-South Walls – Retrofit Building

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	24.4	2.79	1,889	0.00	0	-0.74	-504	0.74	504	1,327	2,451	2,451	88.2
4	52.4	2.79	4,058	0.00	0	-0.45	-650	0.45	650	3,408	4,707	4,707	169.5
7	52.4	2.79	4,058	0.00	0	-0.15	-217	0.15	217	3,841	4,274	4,274	153.9
10	52.4	2.79	4,058	0.00	0	0.15	217	-0.15	-217	4,274	3,841	4,274	153.9
13	52.4	2.79	4,058	0.00	0	0.45	650	-0.45	-650	4,707	3,408	4,707	169.5
16	24.4	2.79	1,889	0.00	0	0.74	504	-0.74	-504	2,451	1,327	2,451	88.2
Σ			**720**		**0**		**0**		**0**				**823**

Table 7-36 Strength Acceptance Criteria Check for Second Story North-South Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36) Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	Accept? (8)	Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0) 0.9 × M_{ST} (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	HD Req'd? (11)	Q_{UD} (kip) (12)	Q_{UF} (kip) (13)
1	88.2	23.8	0.2	4.5	106.9	0.8	Yes	115.1	50.4	No	0.0	0.0
4	169.5	46.4	0.2	4.5	209.0	0.8	Yes	63.2	96.8	Yes	36.9	8.8
7	153.9	46.4	0.2	4.5	209.0	0.7	Yes	63.2	87.9	Yes	33.4	8.8
10	153.9	46.4	0.2	4.5	209.0	0.7	Yes	63.2	87.9	Yes	33.4	8.8
13	169.5	46.4	0.2	4.5	209.0	0.8	Yes	63.2	96.8	Yes	36.9	8.8
16	88.2	23.8	0.2	4.5	106.9	0.8	Yes	115.1	50.4	No	0.0	0.0
					1,050							

Table 7-37 Demands on First Story North-South Walls – Retrofit Building

Wall (1)	K (k/in) (2)	No Torsion δ_i (in) (3)	No Torsion Qi (plf) (4)	Actual Torsion δ_i (in) (5)	Actual Torsion Q_i (plf) (6)	+ Accidental Torsion δ_i (in) (7)	+ Accidental Torsion Q_i (plf) (8)	- Accidental Torsion δ_i (in) (9)	- Accidental Torsion Q_i (plf) (10)	$+T_{ACC}$ $+Q_{UD}$ (plf) (11)	$-T_{ACC}$ $-Q_{UD}$ (plf) (12)	Demand (Eq. 7-36) Q_{UD} (plf) (13)	Demand (Eq. 7-36) Q_{UD} (kip) (14)
1	77.1	3.44	7,36,3	0.12	264	-0.51	1,092	0.51	1,092	6,535	8,718	8,718	313.9
5	103.8	3.44	9,915	0.00	-12	0.02	51	-0.02	-51	9,954	9,851	9,954	358.4
8	77.1	3.44	7,363	-0.12	-251	0.49	1,040	-0.49	-1,040	8,152	6,072	8,152	293.5
Σ			**887**		**0**		**0**		**0**				**966**

Table 7-38 Strength Acceptance Criteria Check for First Story North-South Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Strength Check (Eq. 7-36) Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	Accept? (8)	Overturning Check (Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0) 0.9 × M_{ST} (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	HD Req'd? (11)	Q_{UD} (kip) (12)	Q_{UF} (kip) (13)
1	313.9	70.2	0.2	4.5	315.9	1.0	Yes	119.2	179.3	Yes	68.3	12.6
5	358.4	92.9	0.2	4.5	418.0	0.9	Yes	119.2	204.8	Yes	78.5	17.8
8	293.5	70.2	0.2	4.5	315.9	0.9	Yes	119.2	167.7	Yes	63.7	12.6
					1,050							

Table 7-39 Demands on Third Story East-West Walls – Retrofit Building

Wall (1)	K (k/in) (2)	No Torsion δ_i (in) (3)	No Torsion Qi (plf) (4)	Actual Torsion δ_i (in) (5)	Actual Torsion Q_i (plf) (6)	+ Accidental Torsion δ_i (in) (7)	+ Accidental Torsion Q_i (plf) (8)	- Accidental Torsion δ_i (in) (9)	- Accidental Torsion Q_i (plf) (10)	$+T_{ACC}$ $+Q_{UD}$ (plf) (11)	$-T_{ACC}$ $-Q_{UD}$ (plf) (12)	Demand Q_{UD} (plf) (13)	Demand Q_{UD} (kip) (14)
1	8.9	2.69	1,705	0.03	19	-0.08	-51	0.08	51	1,673	1,775	1,775	24.9
2	8.9	2.69	1,705	0.03	19	-0.08	-51	0.08	51	1,673	1,775	1,775	24.9
3	3.3	2.69	1,496	0.03	17	-0.08	-45	0.08	45	1,468	1,558	1,558	9.3
5	22.0	2.69	3,693	0.00	2	0.00	-4	0.00	4	3,690	3,698	3,698	59.2
6	20.5	2.69	3,668	0.00	2	0.00	-4	0.00	4	3,666	3,674	3,674	55.1
7	20.5	2.69	3,668	0.00	2	0.00	-4	0.00	4	3,666	3,674	3,674	55.1
8	22.0	2.69	3,693	0.00	2	0.00	-4	0.00	4	3,690	3,698	3,698	59.2
16	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
17	2.0	2.69	1,351	-0.03	-14	0.07	38	-0.07	-38	1,375	1,300	1,375	5.5
18	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
19	2.0	2.69	1,351	-0.03	-14	0.07	38	-0.07	-38	1,375	1,300	1,375	5.5
20	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
21	2.0	2.69	1,351	-0.03	-14	0.07	38	-0.07	-38	1,375	1,300	1,375	5.5
22	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
23	2.0	2.69	1,351	-0.03	-14	0.07	38	-0.07	-38	1,375	1,300	1,375	5.5
24	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
25	2.0	2.69	1,351	-0.03	-14	0.07	38	-0.07	-38	1,375	1,300	1,375	5.5
26	2.7	2.69	1,435	-0.03	-15	0.07	40	-0.07	-40	1,460	1,380	1,460	7.3
Σ			**355**		**0**		**0**		**0**				**359**

Table 7-40 Strength Acceptance Criteria Check for Third Story East-West Walls – Retrofit Building

		Strength Check per ASCE 41-13 Eq. 7-36						Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0)				
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	h/b (4)	0.9 × M_{ST} (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	Q_{CE} (kip) (3)	h/b (4)	Q_{UF} (kip) (13)
1	24.9	9.2	0.6	4.5	41.6	0.6	Yes	29.9	14.2	No	0.0	0.0
2	24.9	9.2	0.6	4.5	41.6	0.6	Yes	29.9	14.2	No	0.0	0.0
3	9.3	4.0	1.3	4.5	17.8	0.5	Yes	5.5	5.3	No	0.0	0.0
5	59.2	20.6	0.5	4.5	92.9	0.6	Yes	48.5	33.8	No	0.0	0.0
6	55.1	19.4	0.5	4.5	87.1	0.6	Yes	42.6	31.5	No	0.0	0.0
7	55.1	19.4	0.5	4.5	87.1	0.6	Yes	42.6	31.5	No	0.0	0.0
8	59.2	20.6	0.5	4.5	92.9	0.6	Yes	48.5	33.8	No	0.0	0.0

Table 7-40 Strength Acceptance Criteria Check for Third Story East-West Walls – Retrofit Building (continued)

Wall (1)	Q_{UD} (kip) (2)	Strength Check per ASCE 41-13 Eq. 7-36						Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0)				
		Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	h/b (4)	$0.9 \times M_{ST}$ (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	Q_{CE} (kip) (3)	h/b (4)	Q_{UF} (kip) (13)
16	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
17	5.5	2.6	2.0	4.5	11.9	0.5	Yes	2.4	3.1	Yes	13.9	6.2
18	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
19	5.5	2.6	2.0	4.5	11.9	0.5	Yes	2.4	3.1	Yes	13.9	6.2
20	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
21	5.5	2.6	2.0	4.5	11.9	0.5	Yes	2.4	3.1	Yes	13.9	6.2
22	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
23	5.5	2.6	2.0	4.5	11.9	0.5	Yes	2.4	3.1	Yes	13.9	6.2
24	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
25	5.5	2.6	2.0	4.5	11.9	0.5	Yes	2.4	3.1	Yes	13.9	6.2
26	7.3	3.3	1.6	4.5	14.9	0.5	Yes	3.8	4.2	Yes	13.6	5.6
					609							

Table 7-41 Demands on Second Story East-West Walls – Retrofit Building

Wall (1)	K (k/in) (2)	No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
		δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	8.9	3.37	2,141	0.02	15	-0.10	-63	0.10	63	2,093	2,218	2,218	31.0
2	8.9	3.37	2,141	0.02	15	-0.10	-63	0.10	63	2,093	2,218	2,218	31.0
3	3.3	3.37	1,879	0.02	13	-0.10	-55	0.10	55	1837	1,947	1,947	11.7
5	43.1	3.37	9,084	0.00	1	0.00	-6	0.00	6	9,080	9,092	9,092	145.5
6	40.1	3.37	9,013	0.00	1	0.00	-6	0.00	6	9,008	9,020	9,020	135.3
7	40.1	3.37	9,013	0.00	1	0.00	-6	0.00	6	9,008	9,020	9,020	135.3
8	43.1	3.37	9,084	0.00	1	0.00	-6	0.00	6	9,080	9,092	9,092	145.5
16	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
17	2.0	3.37	1,697	-0.02	-11	0.09	47	-0.09	-47	1,733	1,639	1,733	6.9
18	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
19	2.0	3.37	1,697	-0.02	-11	0.09	47	-0.09	-47	1,733	1,639	1,733	6.9
20	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
21	2.0	3.37	1,697	-0.02	-11	0.09	47	-0.09	-47	1,733	1,639	1,733	6.9

Table 7-41 Demands on Second Story East-West Walls – Retrofit Building (continued)

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
22	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
23	2.0	3.37	1,697	-0.02	-11	0.09	47	-0.09	-47	1,733	1,639	1,733	6.9
24	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
25	2.0	3.37	1,697	-0.02	-11	0.09	47	-0.09	-47	1,733	1,639	1,733	6.9
26	2.7	3.37	1,802	-0.02	-12	0.09	50	-0.09	-50	1,840	1,740	1,840	9.2
Σ			**720**		**0**		**0**		**0**				**725**

Table 7-42 Strength Acceptance Criteria Check for Second Story East-West Walls – Retrofit Building

		Strength Check per ASCE 41-13 Eq. 7-36						Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0)				
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	Accept? (8)	$0.9 \times M_{ST}$ (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	HD Req'd? (11)	Q_{UD} (kip) (12)	Q_{UF} (kip) (13)
1	31.0	9.2	0.6	4.5	41.6	0.7	Yes	29.9	17.7	No	0.0	0.0
2	31.0	9.2	0.6	4.5	41.6	0.7	Yes	29.9	17.7	No	0.0	0.0
3	11.7	4.0	1.3	4.5	17.8	0.7	Yes	5.5	6.7	Yes	17.6	5.2
5	145.5	41.3	0.5	4.5	185.8	0.8	Yes	59.6	83.1	Yes	73.6	18.0
6	135.3	38.7	0.5	4.5	174.2	0.8	Yes	52.4	77.3	Yes	73.6	18.4
7	135.3	38.7	0.5	4.5	174.2	0.8	Yes	52.4	77.3	Yes	73.6	18.4
8	145.5	41.3	0.5	4.5	185.8	0.8	Yes	59.6	83.1	Yes	73.6	18.0
16	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
17	6.9	2.6	2.0	4.5	11.9	0.6	Yes	2.4	4.0	Yes	17.7	6.2
18	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
19	6.9	2.6	2.0	4.5	11.9	0.6	Yes	2.4	4.0	Yes	17.7	6.2
20	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
21	6.9	2.6	2.0	4.5	11.9	0.6	Yes	2.4	4.0	Yes	17.7	6.2
22	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
23	6.9	2.6	2.0	4.5	11.9	0.6	Yes	2.4	4.0	Yes	17.7	6.2
24	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
25	6.9	2.6	2.0	4.5	11.9	0.6	Yes	2.4	4.0	Yes	17.7	6.2
26	9.2	3.3	1.6	4.5	14.9	0.6	Yes	3.8	5.3	Yes	17.4	5.6
					969							

Table 7-43 Demands on First Story East-West Walls – Retrofit Building

		No Torsion		Actual Torsion		+ Accidental Torsion		- Accidental Torsion		$+T_{ACC}$	$-T_{ACC}$	Demand	
Wall (1)	K (k/in) (2)	δ_i (in) (3)	Qi (plf) (4)	δ_i (in) (5)	Q_i (plf) (6)	δ_i (in) (7)	Q_i (plf) (8)	δ_i (in) (9)	Q_i (plf) (10)	$+Q_{UD}$ (plf) (11)	$-Q_{UD}$ (plf) (12)	Q_{UD} (plf) (13)	Q_{UD} (kip) (14)
1	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
2	4.7	3.97	2,335	0.07	39	-0.07	-41	0.07	41	2,332	2,415	2,415	19.3
3	2.7	3.97	2,119	0.07	35	-0.07	-38	0.07	38	2,117	2,192	2,192	11.0
4	2.0	3.97	1,996	0.07	33	-0.07	-35	0.07	35	1,994	2,065	2,065	8.3
5	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
6	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
7	2.0	3.97	1,996	0.07	33	-0.07	-35	0.07	35	1,994	2,065	2,065	8.3
1	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
2	4.7	3.97	2,335	0.07	39	-0.07	-41	0.07	41	2,332	2,415	2,415	19.3
3	2.7	3.97	2,119	0.07	35	-0.07	-38	0.07	38	2,117	2,192	2,192	11.0
4	2.0	3.97	1,996	0.07	33	-0.07	-35	0.07	35	1,994	2,065	2,065	8.3
5	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
6	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
7	2.0	3.97	1,996	0.07	33	-0.07	-35	0.07	35	1,994	2,065	2,065	8.3
8	2.7	3.97	2,119	0.07	35	-0.07	-38	0.07	38	2,117	2,192	2,192	11.0
9	4.7	3.97	2,335	0.07	39	-0.07	-41	0.07	41	2,332	2,415	2,415	19.3
10	3.3	3.97	2,210	0.07	37	-0.07	-39	0.07	39	2,207	2,286	2,286	13.7
15	69.2	3.97	5,844	0.01	8	-0.01	-9	0.01	9	5,843	5,861	5,861	275.5
16	69.2	3.97	5,844	0.01	8	-0.01	-9	0.01	9	5,843	5,861	5,861	275.5
19	26.5	3.97	105,104	-0.05	-1443	0.06	1549	-0.06	-1549	105,210	102,112	105,210	105.2
20	26.5	3.97	105,104	-0.05	-1443	0.06	1549	-0.06	-1549	105,210	102,112	105,210	105.2
Σ			**887**		**0**		**0**		**0**				**893**

Table 7-44 Strength Acceptance Criteria Check for First Story East-West Walls – Retrofit Building

		Strength Check per ASCE 41-13 Eq. 7-36						Overturning Check (ASCE 41-13 Eq. 7-6 with C_1C_2=1.4, μ_{OT}=10, J=2.0)				
Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	h/b (4)	$m \times \kappa$ (5)	$m\kappa Q_{CE}$ (kip) (6)	Q_{UD} / $(m\kappa Q_{CE})$ (7)	Accept? (8)	0.9 × M_{ST} (kip ft) (9)	$M_{OT}/\mu_{OT}C_1C_2$ (kip ft) (10)	HD Req'd? (11)	Q_{UD} (kip) (12)	Q_{UF} (kip) (13)
1	13.7	4.0	1.3	4.5	17.8	0.8	Yes	5.5	7.8	Yes	20.8	5.2
2	19.3	5.3	1.0	4.5	23.8	0.8	Yes	9.8	11.0	Yes	20.7	4.6
3	11.0	3.3	1.6	4.5	14.9	0.7	Yes	3.8	6.3	Yes	21.0	5.6
4	8.3	2.6	2.0	4.5	11.9	0.7	Yes	2.4	4.7	Yes	21.2	6.2
5	13.7	4.0	1.3	4.5	17.8	0.8	Yes	5.5	7.8	Yes	20.8	5.2
6	13.7	4.0	1.3	4.5	17.8	0.8	Yes	5.5	7.8	Yes	20.8	5.2
7	8.3	2.6	2.0	4.5	11.9	0.7	Yes	2.4	4.7	Yes	21.2	6.2
8	11.0	3.3	1.6	4.5	14.9	0.7	Yes	3.8	6.3	Yes	21.0	5.6
9	19.3	5.3	1.0	4.5	23.8	0.8	Yes	9.8	11.0	Yes	20.7	4.6
10	13.7	4.0	1.3	4.5	17.8	0.8	Yes	5.5	7.8	Yes	20.8	5.2
15	275.5	60.6	0.2	4.5	272.8	1.0	Yes	418.6	157.4	No	0.0	0.0
16	275.5	60.6	0.2	4.5	272.8	1.0	Yes	418.6	157.4	No	0.0	0.0
19	105.2	30.2	8.0	8.0	241.6	0.4	Yes	Moment Frames				
20	105.2	30.2	8.0	8.0	241.6	0.4	Yes					
					1,201							

Treatment of hold-down devices to resist overturning was covered in detail in Section 4.6 of this *Guide*. That section demonstrates that some components of a hold-down device and connection are to be treated as deformation-controlled while others are treated as force-controlled. Table 7-34 through Table 7-44 summarize calculations of the deformation-controlled demand, Q_{UD}, and the force-controlled demand, Q_{UF}, to be used in hold-down device evaluation per Section 4.6, but those calculations are not replicated in this section.

To illustrate the hold-down calculations, consider the second row (Wall 7) of Table 7-34, Columns 9 through 13. (The second row is selected rather than the first row to illustrate because Wall 7 overturning is not fully resisted by dead load, and thus hold-down devices are required and the demands calculated.)

- Column 9 (0.9×M_{ST} = 0.9 × 70.3 kip-ft = 63.2 kip-ft) is the factored resistance to overturning provided by the wall self-weight and supported seismic weight. Its calculation was described in Table 7-5, Column 11, but is marginally higher than the value for the existing structure (59.9 kip-ft) because of the added plywood. The load factor of 0.9 is prescribed in ASCE 41-13 Equation 7-6, which is the alternate criterion for determining the adequacy of dead loads to resist overturning, as well as in the load combinations (factor for gravity load) in ASCE 41-13 § 7.2.2 Equation 7-2, used to calculate the demand for the deformation-controlled components of the hold-down device.

- Column 10 ($M_{OT}/\mu_{OT}C_1C_2$) is the overturning demand prescribed in ASCE 41-13 Equation 7-6.

  $M_{OT}/\mu_{OT}C_1C_2$

  where:

  M_{OT} = is the wall demand Q_{UD} times the wall height (8 feet) as = 127.4 kips (Column 2) × 8 = 1,019.2 kip-ft

  μ_{OT} = 10 for CP (ASCE 41-13 § 7.2.8.1)

  C_1C_2 = 1.4 for T less than 0.3 seconds and $m_{\max}$ between 2 and 6 (ASCE 41-13 Table 7-3)

  $M_{OT}/\mu_{OT}C_1C_2 = 1{,}019.2/(10 \times 1.4) = 72.7$ kip-ft

- Column 11 applies ASCE 41-13 Equation 7-6 to determine whether a hold-down device is required by comparing 0.9×M_{ST} (Column 9) to $M_{OT}/\mu_{OT}C_1C_2$ (Column 10). In this case, the answer is "Yes" because ASCE 41-13 Equation 7-6 is not satisfied and a hold-down is required.

- Column 12 (Q_{UD}) is the demand on the hold-down components that are to be treated as deformation-controlled elements, calculated as the overturning demand minus the 90% of the dead load resistance, divided by the length of the wall (minus one foot to account for the hold-down reaction being inboard of the wall end).

  $\Sigma M = 0$: $Q_{UD} \times (L - 1.0) + 0.9 \times M_{ST} - M_{OT} = 0$

  $Q_{UD} = (M_{OT} - 0.9 \times M_{ST})/(L - 1)$

  M_{OT} = is the wall demand Q_{UD} (from Column 2) times the wall height (8 feet throughout) = 127.4 kips × 8 ft = 1,019.2 kip-ft.

  $0.9 \times M_{ST}$ = 63.2 from Column 9

  L = 36 feet (Table 7-5)

  $Q_{UD} = (1{,}019.2 - 63.2)/(36\text{-}1.0) = 27.3$ kips

- Column 13 (Q_{UF}) is the demand on the hold-down components that are to be treated as force-controlled elements. Per ASCE 41-13 § 7.5.2.1.2, Q_{UF} can be taken as the maximum action that can be developed in the element based on the expected strength of the elements delivering the force. (In some circumstances, such as this example, the limiting load in a hold-down might be less than the demand M_{OT} predicted from a linear elastic analysis.) The walls deliver the force to the hold-downs, and the maximum overturning moment that can be delivered by the wall is limited to Q_{CE} times the wall height (h = 8 feet). The hold-down demand, Q_{UF}, is based on the net moment (the limiting moment minus 90% of the dead load resistance) divided by the length of the wall (minus one foot to account for the hold-down reaction being inboard of the wall end).

$$\Sigma M = 0: Q_{UF} \times (L - 1\text{ft}) + 0.9 \times M_{ST} - Q_{CE} \times h = 0$$

$$Q_{UF} = (Q_{CE} \times h - 0.9 \times M_{ST})/(L - 1)$$

Q_{CE} = 46.4 kips (Column 3)

h = 8 feet throughout

$0.9 \times M_{ST}$ = 63.2 kip-ft (Column 9)

L = 36 feet (Table 7-5, Column 3)

$$Q_{UF} = (46.4 \times 8 - 63.2)/(36\text{-}1) = 8.8 \text{ kips}$$

Sections 4.6.4.3 and 4.6.4.4 of this *Guide* demonstrated how different components of a hold-down strap are checked for deformation-controlled and force-controlled actions, respectively. The same hold-down strap is used here to demonstrate how those components are likewise evaluated against the demands in Columns 12 and 13.

The capacity of the strap nailing is calculated for deformation-controlled action in Section 4.6.4.3 of this *Guide* and compared here to the demand Q_{UD} from Column 12:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

where:

m = capacity modification factor = 6.0 (Section 4.6.4.3)

κ = knowledge factor = 1.0

Q_{CE} = expected component strength = 7.7 kips (Section 4.6.4.3)

Q_{UD} = 27.3 kips from Column 12

$m\kappa Q_{CE}$ = 6(1.0)(7.7) = 46.2 kips

46.2 kips > 27.3 kips (ASCE 41-13 Eq. 7-36)

Therefore, 13-10d nails at each end of the strap (26 total) are satisfactory.

Likewise, the capacity of the strap itself is calculated for force-controlled actions in Section 4.6.4.4 of this *Guide* and compared here to the demand Q_{UF} from Column 13:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

where:

κ = 1.0

Q_{CL} = lower-bound component strength = 3.6 kips (Section 4.6.4.4)

Q_{UF} = 8.8 kips from Column 13

κQ_{CL} = (1.0)(3.6) = 3.6 kips

3.6 kips < 8.8 kips (ASCE 41-13 Eq. 7-37)

Therefore, the strap is inadequate.

7.7.3 Force-Controlled Elements

There are three force-controlled elements to be considered: (1) front beam supporting discontinuous shear walls; (2) the diaphragm loads in locations where the shear walls are offset; and (3) the bodies of hold-down devices. Treatment of the hold-down bodies is treated in Section 4.6 of this *Guide* and is not addressed in this example, other than calculation of demands in the previous section.

> **Useful Tip**
> Per ASCE 41-13 § 7.4.1.3.4, effects of offset walls need to be considered force-controlled.

7.7.3.1 Glued-Laminated Beam

Hold-down forces from the upper floors will accumulate in stacks of shear walls, resulting in significant overturning demands on the supporting glued-laminated (glulam) beam. Per ASCE 41-13 § 12.3.4.1, beams that support discontinuous shear walls are to be considered force-controlled elements, and therefore must satisfy ASCE 41-13 Equation 7-37 as follows:

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

where:

Q_{UF} = Force-controlled action caused by earthquake and gravity

Q_{CL} = Lower bound strength of the glulam beam

While the lower bound strength is defined in ASCE 41-13 § 12.3.2.3.1 as the mean minus one standard deviation based on testing, there is little such information available for vintage glulam beams. Instead, one can use the default value of 0.85 times the expected strength as provided in ASCE 41-13

§ 12.2.2.5. To calculate Q_{UF}, the accumulation of the hold-down demands on the beam needs to be applied in conjunction with the gravity loads (ASCE 41-13 Equation 7-37). The accumulation of hold-down forces from the front wall shear walls are shown in Figure 7-16 and Figure 7-17 using the deformation- and force-controlled demands in Columns 12 and 13, respectively of Table 7-40 and Table 7-42. These demands should be used in conjunction with acceptance criteria of ASCE 41-13 § 7.2.5.2.2.1 for Deformation-Controlled Actions, and § 7.2.5.2.2.1 for Force-Controlled Actions. While ASCE 41-13 § 7.2.5 indicates that multi-directional effects need not be considered for this building, it is prudent to also consider the overturning loads from the transverse shear walls using, for instance, the 100% - 30% rule of ASCE 41-13 § 7.2.5.1.1. The hold-down forces from the perpendicular shear walls (Walls 4, 7, 10 and 11 in Figure 7-8) are found in Table 7-34 and Table 7-36. Check of the glulam beam is outside the scope of this example, but note that the grading and allowable stresses in glulams have evolved over recent decades, and that expected strength using current standards might be significantly lower than that used in the original design.

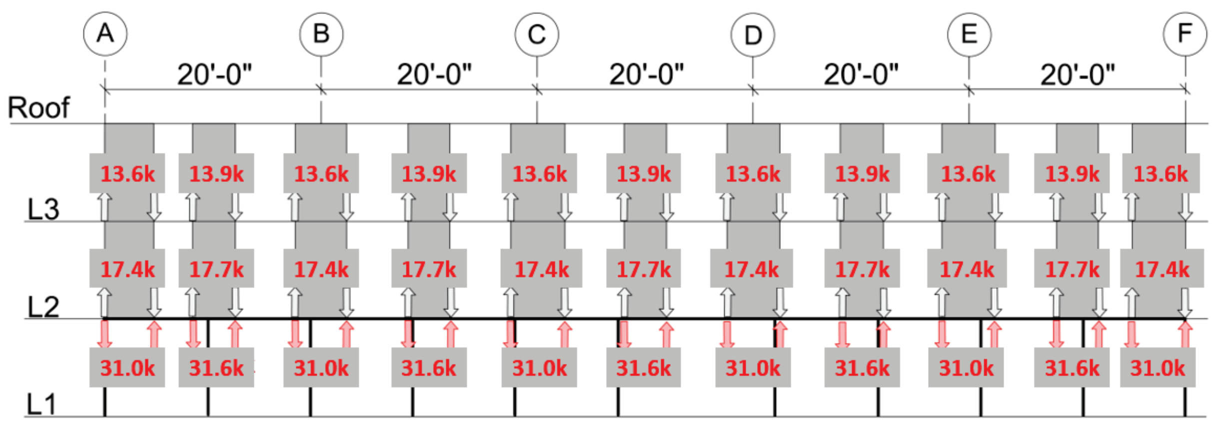


Figure 7-16 Front wall hold-down demands for deformation-controlled elements of the device, Q_{UD}.

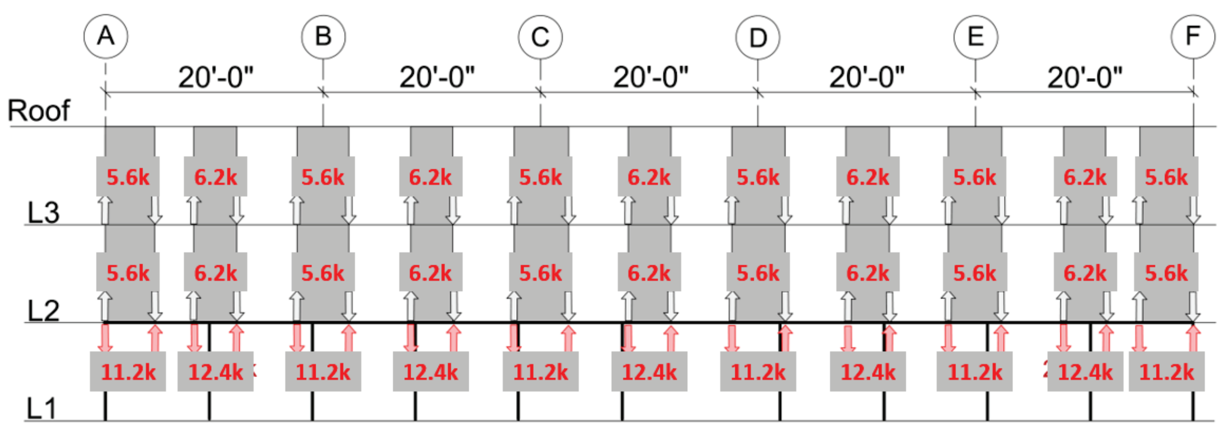


Figure 7-17 Front wall hold-down demands for force-controlled elements of the device, Q_{UF}.

The accumulation of hold-down forces for the other shear wall stacks are not shown (for brevity) but are calculated in the same manner. Some of these overturning forces are small, and some hold-downs might be eliminated if the dead load resisting the uplift is more carefully considered. However, some accumulations of hold-down loads are significant at the foundation level, and testing may be required to demonstrate the capacity of the post-installed anchors and foundation to resist the uplift loads. In this case, the concrete foundation had been found to be in very good condition, and testing was not required.

7.7.3.2 Diaphragms

The last force-controlled element to address is the L2 floor diaphragm (other diaphragms, with continuous shear walls, are deformation-controlled elements). The floors are constructed with 1.5 inches of lightweight topping, which is ignored in this calculation but likely contributes strength and stiffness. The shear in the diaphragm can be found by applying the shear wall forces from above and below in combination with the distributed inertial forces from the diaphragm itself (ASCE 41-13 § 7.4.1.3.4).

For plywood diaphragm strength, ASCE 41-13 refers the user to SDPWS provisions. SDPWS Table 4.2A provides a unit *yield* strength of 510 pounds per foot (15/32 plywood with 8d nails spaced at 6 inches on center along the edges and 12 inches on center in the field). ASCE 41-13 § 12.5.3.6 states the expected diaphragm strength is 1.5 times the tabulated value in that reference, that is, $Q_{CE} = 1.5 \times 510 = 765$ plf. For deformation-controlled diaphragm elements, per ASCE 41-13 Table 12-3, for the Collapse Prevention Performance Level the *m*-factor for low aspect ratio wood panel, blocked, unchorded diaphragms is 3.0. (In this case, the diaphragm span used for length/width ratio is best represented by the 20 feet between transverse walls used as primary elements, and thus $L/b < 1$ in the transverse direction. As such, the diaphragm capacity $m\kappa Q_{CE}$ is 3 (1.0) 765 = 2,295 plf.

The third floor (L3) and roof diaphragms can be treated as deformation-controlled elements using the *m*-factors of ASCE 41-13 Table 12-3. The diaphragm shear diagrams for L3 and the roof are shown in Figure 7-18 and Figure 7-19, respectively. It is seen in Figure 7-18 that, with the exception of negligible exceedance at Walls 7 and 10, the demands on the roof diaphragm are below the 2,295 plf capacity, and therefore no retrofit is required. The Level L3 diaphragm demand exceeds its capacity in two small areas inboard of Walls 4 and 13. If the diaphragm is strengthened in those areas by adding an 8d nail between the existing nails (with care not to split the joists), the spacing would reduce from six inches to three inches, and the strength would be more than sufficient.

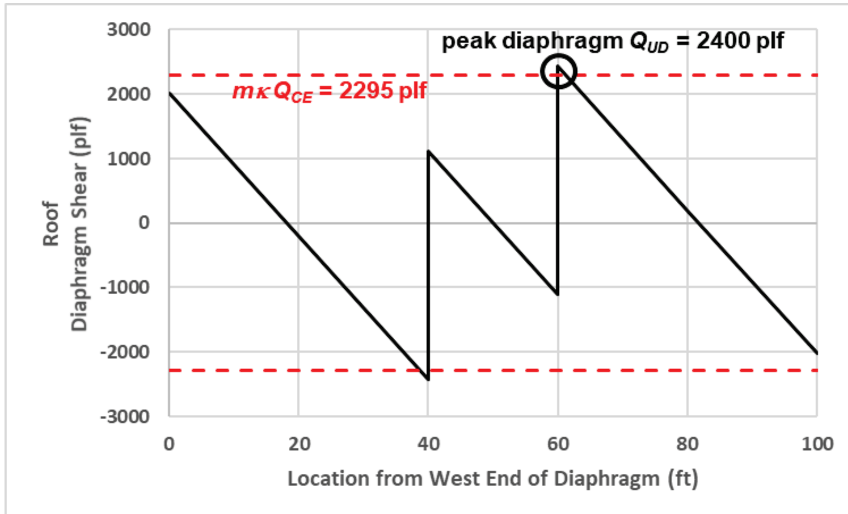


Figure 7-18 Transverse roof diaphragm shear demand, Q_{UD}.

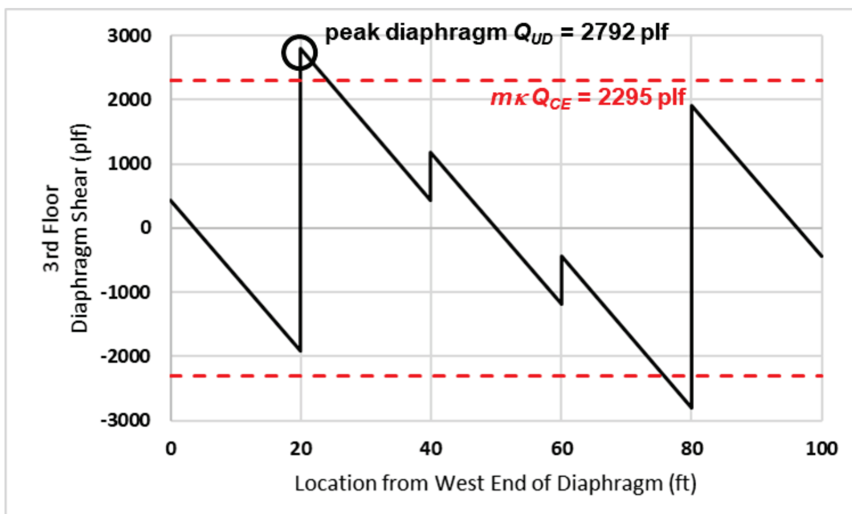


Figure 7-19 Transverse third floor (L3) diaphragm shear demand, Q_{UD}.

Commentary

Per ASCE 41-13 § 7.2.9.5, continuous cross ties should be provided between the chords or diaphragm boundaries. This requirement is intended for diaphragms laterally supporting heavy walls, and need not be applied to light-framed, platform construction of the subject building.

The transverse walls are discontinuous at the second floor (L2). Above the second floor, there are four evenly spaced interior shear walls; below the second floor, there is essentially one, heavily loaded central transverse shear wall. Therefore, the diaphragm is required to transfer considerable horizontal shear. ASCE 41-13 § 7.4.1.3.4 states that diaphragms transferring horizontal forces between offset shear walls shall be treated as force-controlled, and therefore must satisfy:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

The capacity Q_{CL} in the acceptance criterion of ASCE 41-13 Equation 7-37, using the 0.85 default reduction from expected strength to lower bound:

$$Q_{CL} = 0.85 \times Q_{CE} \qquad \text{(ASCE 41-13 § 12.2.2.5)}$$

where:

Q_{CE} = 765 plf

$$Q_{CL} = 0.85 \times 765 = 650 \text{ plf}$$

In this case the demand on the L2 diaphragm is not limited by the strengths delivered by other elements, and the force-controlled demands are calculated using the alternative provision of ASCE 41-13 § 7.5.2.1.2.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

where:

Q_E = earthquake shears demands as calculated in Table 7-33 through Table 7-43.

Q_G = 0.0 kips

C_1C_2 = 1.4 (alternate value from ASCE 41-13 Table 7-3)

J = 2.0 (ASCE 41-13 § 7.5.2.1.2)

Q_{UF} = calculated shear/2.8

To demonstrate the calculation, consider the demand at the west end of L2, where the diaphragm shear is the difference between the shear delivered from the wall above and from the wall below. The load from shear wall above (Wall 1, Table 7-35) is 88.2 kips; the shear in the wall below (Wall 1, Table 7-37) is 313.9 kips. The diaphragm shear is thus (313.9 – 88.2) = 225.7 kips, or 6.27 kips per foot for a 36-foot wide diaphragm (which happens to be the highest shear in the diaphragm). The force-controlled shear Q_{UF} at the west end is therefore 6.27 klf/2.8 = 2,240 plf, which is the value in the shown in the shear diagram of Figure 7-20.

Since the demand Q_{UF} far exceeds Q_{CL}, (κ = 1.0), the diaphragm would need to be strengthened to meet the criterion of ASCE 41-13 Equation 7-37. Options for increasing the diaphragm strength include some combination of adding fasteners, or supplementing the sheathing (perhaps to the bottom of the floor joists). However, it would likely be difficult to add sufficient (lower bound) strength to exceed the 2,240 plf demand. Alternatively, it may be more effective to lower the diaphragm shear span, and thus demand, by sheathing the back north-south walls with OSB and extending collectors to the front opening.

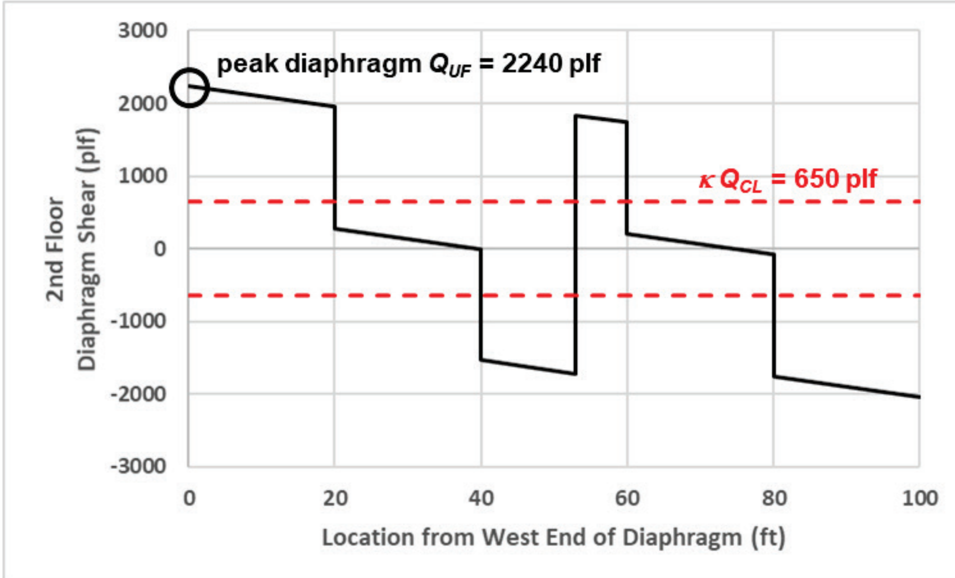


Figure 7-20 Transverse second floor (L2) diaphragm shear demand Q_{UF} and capacity $\kappa \times Q_{CL}$.

7.7.4 Moment Frame

Retrofit of the open front could take several different forms: steel moment frames (as shown in this example), cantilever steel columns, or proprietary solutions. Detailing the moment frames connections and demonstrating compliance with ASCE 41-13 Chapter 9 is beyond the scope of this example. Some aspects that will require attention include:

- Bracing of the moment frame beam back to a wood diaphragm can present challenges in meeting the AISC strength and stiffness requirements.
- The strength and stiffness of the moment frame in this example is based on pinned column bases and a moment resisting beam connection. Depending on conditions at the building, it may be preferable to fix the column base utilizing a stiff grade beam.
- For this example, the column is a W12×35 and the beam is a W12×22. The yield load of 38 k, stiffness of 30 k/in, and yield deflection of 1.25 inches were determined by simple frame analysis.

7.7.5 Verification of the LSP

ASCE 41-13 precludes the linear static procedure (LSP) from being used for structures with significant higher mode effects, significant irregularities, or significant nonlinear excursions. In this section, those criteria are revisited to ensure that the retrofit structure complies.

To use a linear procedure, ASCE 41-13 § 7.3.1.1 allows buildings with irregularities (as defined in ASCE 41-13 § 7.3.1.1.3 or § 7.3.1.1.4) as long as the ratio of deformation-controlled demands Q_{UD} to capacities Q_{CE} (DCRs) are all lower than the lesser of 3.0 and the associated *m*-factor. Table 7-34 through Table 7-44 show these ratios to be greater than three (since many shear wall strengths were evaluated using *m*-factors of 4.5), and therefore it must be shown that both referenced irregularities have been eliminated to justify the use of a linear procedure.

ASCE 41-13 § 7.3.1.1.3 Weak Story Irregularity: A weak story irregularity is present if the average DCR (weighted by shear in the element) of all elements in a story exceeds 125% of the average DCR of an adjacent story. These can be calculated from the values in Table 7-33 through Table 7-44 and are summarized in Table 7-45 through Table 7-50.

$$\overline{\mathrm{DCR}} = \frac{\sum_{1}^{n} \mathrm{DCR}_i V_i}{\sum_{1}^{n} V_i} \qquad \text{(ASCE 41-13 Eq. 7-17)}$$

To illustrate, the first row of calculations is described:

- Column 1 (Wall) is the wall number shown in Figure 7-8
- Column 2 ($V_i = Q_{UD}$) = 72.5 kips is the deformation-controlled demand from Column 14 of Table 7-33 and is used for the value V_i in ASCE 41-13 Equation 7-17.
- Column 3 (Q_{CE}) = 23.8 kips is the expected strength of the element from Column 3 of Table 7-34.
- Column 4 (DCR) = 3.1 is the ratio of Q_{UD} to Q_{CE}. In this case, 72.5/23.8 = 3.1.
- Column 5 (Q_{UD} × DCR) is the product of Q_{UD} and DCR used to calculate the weighted average, in this case 72.5 × 3.1 = 221.5 kips.

At the bottom of the table is the weighted average is calculated as the ratio of the sum of values in Column 5 (ΣV_i) to the sum of values in Column 2 ($\Sigma \mathrm{DCR}_i \times V_i$). In this case, 1,142 kips/399.9 kips = 2.9; 125% of this value (3.6) is also reported. Table 7-45 through Table 7-50 show that the weighted average of the DCRs is always less than 125% of any adjacent story in the same direction, and so there is no weak story irregularity.

Table 7-45 Average DCR for Third Story North-South Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3	
			DCR (4)	Q_{UD} × DCR (kip) (5)
1	72.5	23.8	3.1	221.5
7	127.4	46.4	2.7	349.6
10	127.4	46.4	2.7	349.6
16	72.5	23.8	3.1	221.5
Weighted Average:				**2.9**
125% Weighted Average:				**3.6**

Table 7-46 Average DCR for Second Story North-South Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3	
			DCR (4)	Q_{UD} × DCR (kip) (5)
1	88.2	23.8	3.7	327.6
4	169.5	46.4	3.6	618.4
7	153.9	46.4	3.3	509.8
10	153.9	46.4	3.3	509.8
13	169.5	46.4	3.6	618.4
16	88.2	23.8	3.7	327.6
Weighted Average:				**3.5**
125% Weighted Average:				**4.4**

Table 7-47 Average DCR for First Story North-South Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3	
			DCR (4)	Q_{UD} × DCR (kip) (5)
1	313.9	70.2	4.5	1,403.2
5	358.4	92.9	3.9	1,382.6
8	293.5	70.2	4.2	1,226.9
Weighted Average:				**4.2**
125% Weighted Average:				**5.2**

Table 7-48 Average DCR for Third Story East-West Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3 DCR (4)	ASCE 41-13 § 7.3.1.1.3 Q_{UD} × DCR (kip) (5)
1	24.9	9.2	2.7	66.8
2	24.9	9.2	2.7	66.8
3	9.3	4.0	2.4	22.1
5	59.2	20.6	2.9	169.6
6	55.1	19.4	2.8	156.9
7	55.1	19.4	2.8	156.9
8	59.2	20.6	2.9	169.6
16	7.3	3.3	2.2	16.1
17	5.5	2.6	2.1	11.5
18	7.3	3.3	2.2	16.1
19	5.5	2.6	2.1	11.5
20	7.3	3.3	2.2	16.1
21	5.5	2.6	2.1	11.5
22	7.3	3.3	2.2	16.1
23	5.5	2.6	2.1	11.5
24	7.3	3.3	2.2	16.1
25	5.5	2.6	2.1	11.5
26	7.3	3.3	2.2	16.1
Weighted Average:				**2.7**
125% Weighted Average:				**3.4**

Table 7-49 Average DCR for Second Story East-West Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3 DCR (4)	ASCE 41-13 § 7.3.1.1.3 Q_{UD} × DCR (kip) (5)
1	31.0	9.2	3.4	104.3
2	31.0	9.2	3.4	104.3
3	11.7	4.0	2.9	34.4
5	145.5	41.3	3.5	512.6
6	135.3	38.7	3.5	473.0
7	135.3	38.7	3.5	473.0
8	145.5	41.3	3.5	512.6

Table 7-49 Average DCR for Second Story East-West Walls – Retrofit Building (continued)

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3 DCR (4)	Q_{UD} × DCR (kip) (5)
16	9.2	3.3	2.8	25.7
17	6.9	2.6	2.6	18.2
18	9.2	3.3	2.8	25.7
19	6.9	2.6	2.6	18.2
20	9.2	3.3	2.8	25.7
21	6.9	2.6	2.6	18.2
22	9.2	3.3	2.8	25.7
23	6.9	2.6	2.6	18.2
24	9.2	3.3	2.8	25.7
25	6.9	2.6	2.6	18.2
26	9.2	3.3	2.8	25.7
Weighted Average:				**3.4**
125% Weighted Average:				**4.2**

Table 7-50 Average DCR for First Story East-West Walls – Retrofit Building

Wall (1)	Q_{UD} (kip) (2)	Q_{CE} (kip) (3)	ASCE 41-13 § 7.3.1.1.3 DCR (4)	Q_{UD} × DCR (kip) (5)
1	13.7	4.0	3.5	47.5
2	19.3	5.3	3.7	70.7
3	11.0	3.3	3.3	36.4
4	8.3	2.6	3.1	25.8
5	13.7	4.0	3.5	47.5
6	13.7	4.0	3.5	47.5
7	8.3	2.6	3.1	25.8
8	11.0	3.3	3.3	36.4
9	19.3	5.3	3.7	70.7
10	13.7	4.0	3.5	47.5
15	275.5	60.6	4.5	1,251.5
16	275.5	60.6	4.5	1,251.5
19	105.2	30.2	3.5	366.5
20	105.2	30.2	3.5	366.5
Weighted Average:				**4.1**
125% Weighted Average:				**5.2**

ASCE 41-13 § 7.3.1.1.4 Torsional Strength Irregularity: A torsional strength irregularity is present if the ratio of the DCRs of critical elements on either side of the center of resistance exceeds 1.5 for any story in any direction. As can be seen in Table 7-45 through Table 7-50, none of the DCRs in a single story in a single direction differ by more than 150%, and so this criterion is met and no torsional strength irregularity exists.

Additionally, ASCE 41-13 § 7.3.1.2 prescribes six criteria for the use of linear *static* procedures, all of which must be met.

1. The fundamental period of the building (0.24 sec) shall be less than 3.5 seconds. This criterion is met (Table 7-13).
2. The aspect ratios of the floor plan dimensions cannot differ by more than 1.4 for any adjacent stories, which is met in this case because the floor plan dimensions are identical for all floors.
3. No story shall present a torsional stiffness irregularity defined as a drift along one side of the building greater than 150% of the average drift. By inspection of Table 7-33 through Table 7-43, the controlling story is Level L3 to roof, loaded in the north-south direction (Table 7-33). The average deflection of Level 2, as seen in Column 3, is 2.31 inches. The deflection of the extreme wall (Wall 1) due to pure translation, actual torsion and – accidental torsion is the sum of Columns 3, 5, and 9 of that table = 2.31 + 0.00 + 0.60 = 2.91 inches, which is less than 1.5 × 2.31 inches = 3.5 inches, and therefore no torsional stiffness irregularity exists.
4. No story shall present a vertical stiffness irregularity defined as its average drift being greater than 150% of the average drift in any adjacent story. For north-south loading, the story drifts from top to bottom are 2.31, 2.79, and 3.44 inches, none of which is greater than 150% of its neighbor; likewise, for east-west loading, the story deflections are 2.69, 3.37, and 3.97 inches, which also vary by less than 150%. Therefore, no vertical stiffness irregularity exists.
5. The building shall not have a non-orthogonal lateral force-resisting system. Since all walls in this building are either north-south or east-west oriented, the retrofit also complies with this requirement.

In summary, the retrofit design has eliminated all the irregularities that would preclude the use of a linear static procedure.

Chapter 8

Steel Moment Frame (S1)

8.1 Overview

This chapter provides discussion and example application of the Tier 1, Tier 2, and Tier 3 procedures of ASCE 41-13 (ASCE, 2014) on a five-story "Pre-Northridge" steel moment frame office building located in a high seismic region of Northern California. This building archetype is representative of large office buildings commonly constructed in California prior to the 1994 Northridge earthquake. The example illustrates the use of Tier 1, Tier 2, and Tier 3 evaluation procedures for the Basic Performance Objective for Existing Buildings (BPOE). Relevant Tier 1 Quick Checks with non-conforming items are presented initially, followed by a Tier 2 deficiency-based evaluation using both the linear static procedure (LSP) and the linear dynamic procedure (LDP). Lastly, the Tier 3 nonlinear static procedure (NSP) is summarized for a single frame line to compare and contrast the linear and nonlinear behavior of this type of structure with the ASCE 41-13 modeling and acceptance criteria requirements.

Example Summary

Building Type: S1

Performance Objective: BPOE

Risk Category: II

Location: San Francisco Bay Area, California

Analysis Procedure: LSP, LDP, NSP

Evaluation Procedure: Tier 1, Tier 2, and Tier 3

Reference Documents:

AISC 341

AISC 358

AISC 360

This example is not a complete evaluation of the structure and focuses on selected structural elements of one multi-bay moment frame line. This expanded example is intended for illustrative purposes as it may not be common to perform this many types of analysis in practice. Nonstructural evaluation is required in the BPOE and is an important part of an overall seismic evaluation; however, it is outside the scope of this design example. The building geometry and system layout are described in the following section.

This example illustrates the following:

- **Section 8.2:** Building description
- **Section 8.3:** Tier 1 evaluation
- **Section 8.4**: Tier 2 evaluation
  - Evaluation using linear static procedure (LSP)
  - Comparison of results from LSP and linear dynamic procedure (LDP)
- **Section 8.5:** Tier 3 Summary example evaluation using nonlinear static procedure (NSP)

8.1.1 Pre-Northridge Moment Connections

In the 1980s when the example building was constructed, welded steel moment frames were regarded as being among the most ductile structural systems. Steel moment frame buildings are designed to resist earthquake loads based on the assumption that extensive yielding and plastic deformation will dissipate seismic energy and will occur without strength loss; it was believed that earthquake damage for this system would be limited to ductile yielding of members and connections. However, following the 1994 Northridge Earthquake, fractures were observed in a number of welded steel moment frame buildings, indicating that these connections are susceptible to brittle and sudden fracture at the beam-to-column interface. The damaged buildings were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking.

The typical beam-to-column connection used in steel moment frames during this time is shown in the Figure 8-1 and is commonly referred to as a welded unreinforced flange (WUF) connection or "Pre-Northridge" connection. The connection requires the beam flanges to be welded to the column using complete joint penetration groove welds, and a shear tab that is typically bolted to the beam web and welded to the column. The Northridge Earthquake revealed that brittle fractures could occur within this type of connection at very low levels of plastic demand. The fractures were typically observed to have initiated at the weld between the beam bottom flange and column flange, and these fractures progressed along a number of different paths depending on the specific connection detailing and geometry (FEMA, 2000).

For this example building, which was designed per the 1982 UBC, *Uniform Building Code* (ICBO, 1982), there are critical steel design provisions contained within current building codes that were not in place during the original design. These include more stringent welding procedures and quality assurance procedures, Strong-Column, Weak-Beam (SCWB) requirements, stability bracing to inhibit lateral buckling or lateral-torsional buckling of primary framing members, and improved beam-column panel zone requirements. These will be illustrated in the following sections.

In many existing conditions where the framing supports a concrete floor slab, the bracing is provided—through the diaphragm—for only the top beam flange. Lateral bracing of columns at the floor levels is also needed. This bracing is especially important for deep column sections like those in this example that, although efficient for frame stiffness, are potentially more susceptible to lateral-torsional buckling than heavier W14 column sizes.

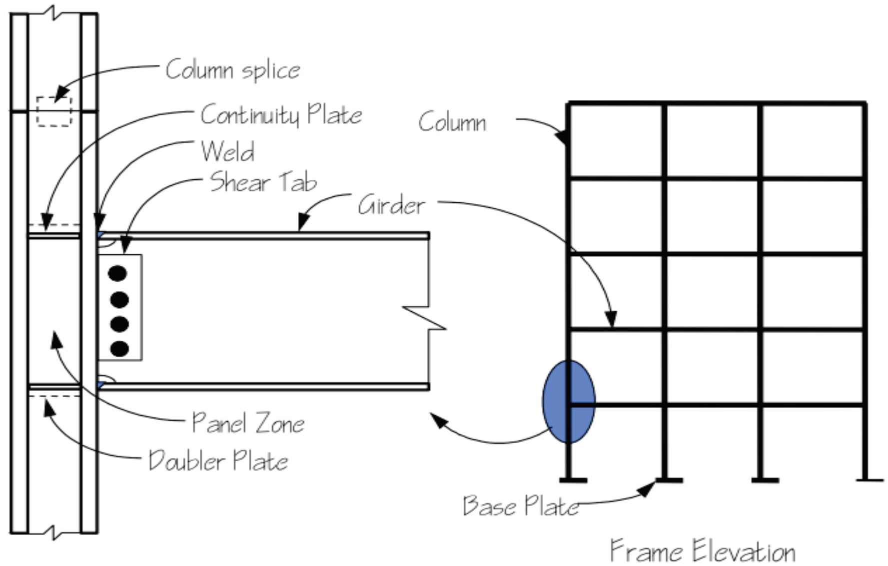


Figure 8-1 A depiction of a typical pre-Northridge WUF connection used in steel moment frames.

8.2 Building Description

The example building was constructed in 1985 using the 1982 Uniform Building Code and has a square 217ft × 217ft footprint. There are four stories of framing above grade level with one story below grade. The below grade portion of the building structure is setback from the exterior basement retaining walls such that, for the mathematical model, the building is five stories above the seismic base. A central full height atrium, 100 feet square in plan, is located within the building and is offset approximately 28 feet to the southwest of the center as shown in Figure 8-2 and Figure 8-3. The first story (basement level) is 16'-0" high, and the remaining stories are 14'-3" high. The primary gravity system is comprised of steel framing supporting metal decks with concrete fill. The base of the structure is founded on spread footings with grade beams connecting the footings around the perimeter.
The seismic force-resisting system consists of steel moment frames around the perimeter with rigid concrete diaphragms. The building is considered a Type S1 building in accordance with ASCE 41-13 Table 3-1, since it is a steel moment-resisting frame building with rigid diaphragms. The building was first checked to see if it could be considered a Benchmark Building per ASCE 41-13 Table 4-6. Type S1 buildings are only benchmarked for Life Safety if designed and detailed per the 1994 or later UBC (including emergency provisions per footnote f). The example building was designed prior to the adoption of the 1994 UBC and does not meet the Benchmark Building criteria; therefore, evaluation is required to determine if the building meets the ASCE 41-13 BPOE standard.

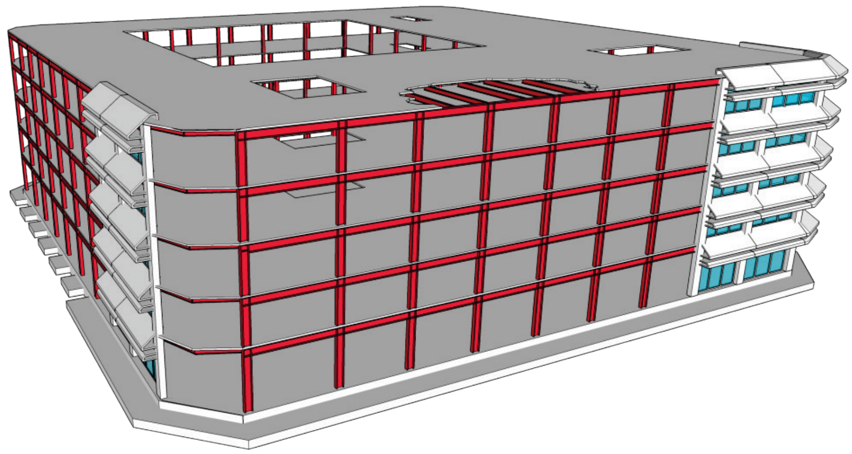


Figure 8-2 Three-dimensional view of the existing building.

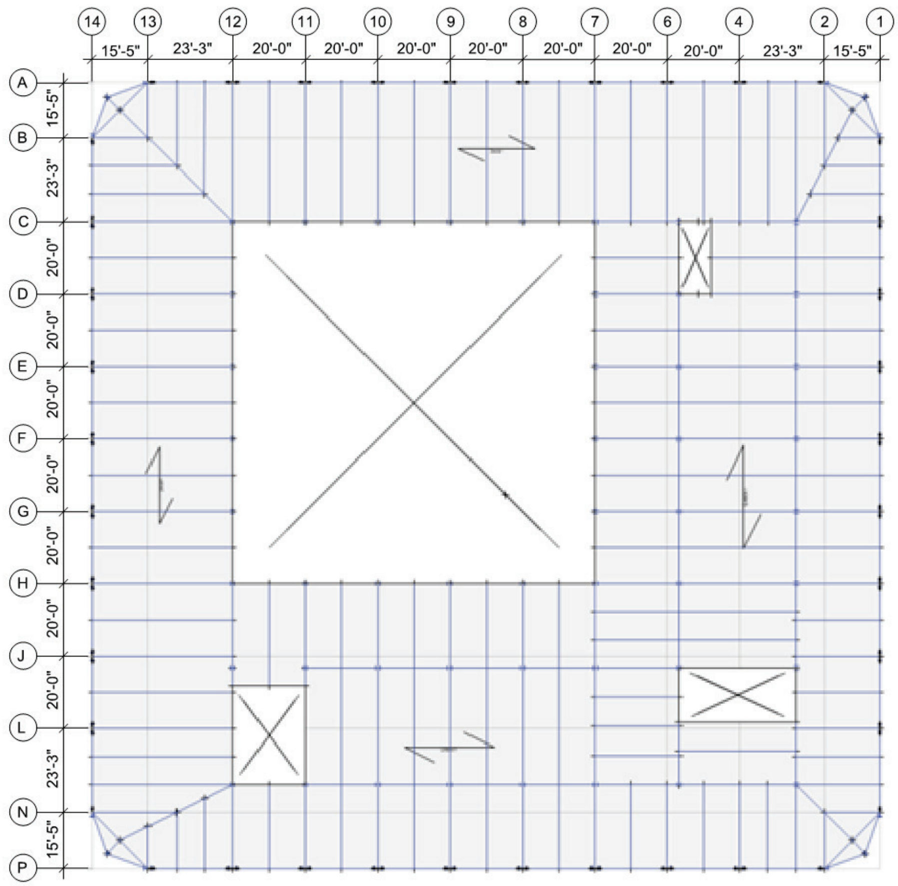


Figure 8-3 Typical floor plan.

8.2.1 Building Use and Risk Category

The building usage is primarily office space. The Authority Having Jurisdiction has no specific policies regarding Risk Category definitions, so per ASCE 41-13 § 2.2.1, ASCE 7-10 (ASCE, 2010) is used to define the

Risk Category. Per ASCE 7-10 Table 1.5-1, this office building is classified as Risk Category II.

8.2.2 Structural System

The structural system is described in detail in the following sections.

8.2.2.1 Roof Framing

12- to 18-inch deep steel beams span up to 39 feet between 18- to 24-inch deep steel girders. The girders typically span between steel columns spaced at 20 feet. The beams support a 2-inch deep steel deck with 3 inches of lightweight concrete (LWC) fill.

8.2.2.2 Typical Floor Framing

12- to 18-inch deep steel beams span up to 39 feet to 18 inch to 24 inch deep steel girders. The girders typically span between steel columns spaced at 20 feet. The beams support a 2-inch deep steel deck with 3 inches of normal weight concrete (NWC) fill.

8.2.2.3 Vertical Load-Bearing Elements

Steel wide flange columns support the horizontal steel framing. The columns are laid out in a grid pattern with 20- to 39-foot spacing. Interior columns are 14 inches deep and perimeter columns are 24 inches deep.

8.2.2.4 Seismic Force-Resisting System (SFRS)

Five-story steel moment frames are located on all four perimeter lines of the building to resist lateral forces. See Figure 8-4 and Table 8-1 for a typical perimeter frame elevation and member sizes. Though it is more common for buildings of this type to use heavier W14 columns shapes, both the moment frame columns and beams are 24 inches deep in this building. The moment connections consist of pre-Northridge full penetration welds. Column splices consist of full penetration welds at both the flanges and the webs and occur at 4 feet above the ground and third floors. Doubler and stiffener plates are installed at the vast majority of the connections. Each column is anchored to the foundation with six 1-1/2” diameter anchor rods embedded 21 inches into concrete. Concrete slabs connected to the framing with embedded steel studs serve as the floor diaphragms, which are determined to be rigid.

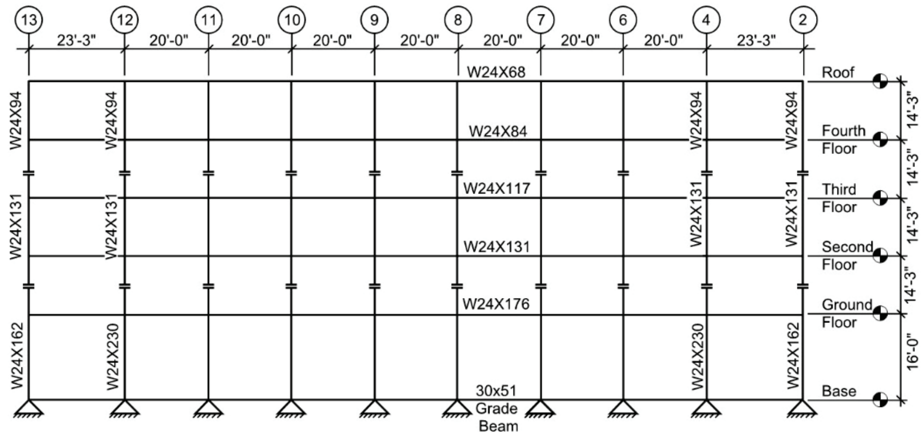


Figure 8-4 Typical north-south moment frame elevation (Gridline P).

Table 8-1 Typical Moment Frame Properties (Gridline 1)

	Floor	Beam W-Shape	Beam Length (in.)	Column W-Shape	Average Story Height (in.)	Stiffener Plate Thickness (in.)
End Bay, End Connection	Ground	W24×176	279	W24×230	193.5	0.50
	2	W24×131	279	W24×131	171	0.50
	3	W24×117	279	W24×131	171	0.50
	4	W24×84	279	W24×94	171	0.50
	Roof	W24×68	279	W24×94	171	0.50
End Bay, Internal Connection	Ground	W24×176	279	W24×162	193.5	0.63
	2	W24×131	279	W24×131	171	0.50
	3	W24×117	279	W24×131	171	0.50
	4	W24×84	279	W24×94	171	0.50
	Roof	W24×68	279	W24×94	171	0.50
Internal Bay, Internal Connection	Ground	W24×176	240	W24×162	193.5	0.63
	2	W24×131	240	W24×131	171	0.50
	3	W24×117	240	W24×131	171	0.50
	4	W24×84	240	W24×94	171	0.50
	Roof	W24×68	240	W24×94	171	0.50

8.2.2.5 Exterior Walls/Cladding

The building exterior consists of glazing and precast concrete spandrel panels. Two lines of spandrel panels occur between each floor level continuously around the building perimeter. A larger 8’-0” tall panel occurs at each floor and roof level, while a smaller 2’-6” tall panel occurs approximately 5 feet below each floor and roof level. Panels are connected to the primary steel structure with steel outriggers at each primary building column.

8.2.2.6 Foundations

Columns are founded on reinforced concrete spread and strip footings. The spread footings are typically sized between 7'-6" square and 13 foot square with footing thicknesses between 18 and 36 inches. Columns on the north and west exterior sides and portions of the east exterior side are supported on strip footings typically 16'-6" wide and 27 inches deep. Reinforced concrete grade beams are located around the entire perimeter and tie the spread footings together. The steel moment frame columns are anchored into the grade beams as described in more detail in Section 8.4.1.4 of this *Guide*.

8.2.2.7 Material Properties

Properties for the structural materials and specifications are based on original construction drawings. The following lower-bound values for the primary structural steel components are taken from ASCE 41-17 Table 9-1 and are utilized in the analysis calculations.

> **ASCE 41-17 Revision**
>
> ASCE 41-13 Table 4-5 has errors that have been corrected in ASCE 41-17. Use of ASCE 41-17 Table 9-1 for structural steel material strengths is recommended.

- Structural steel, ASTM A36

  $F_{y\text{-}LB}$ = 39 ksi (for ground floor frame beams Group 3)

  $F_{y\text{-}LB}$ = 41 ksi (for 2nd floor - roof frame beams – Group 2)

  F_u = 60 ksi (for ground floor frame beams Group 3)

  F_u = 59 ksi (for 2nd floor - roof frame beams – Group 2)

- Structural steel, ASTM A572 GR50

  $F_{y\text{-}LB}$ = 50 ksi (for moment frame columns)

  F_u = 65 ksi (for moment frame columns)

- Concrete (lower bound properties as specified on construction drawings)

  f'_c = 3 ksi (suspended slabs)

  f'_c = 4 ksi (all other concrete)

- Reinforcing steel (lower bound properties as specified on construction drawings)

  f_y = 40 ksi (#3 bars)

  f_y = 40 ksi (#4 bars and larger)

For most steel materials which have been extensively tested and documented in the past, the default lower bound values listed in ASCE 41-13 Table 9-1 are permitted to be used when the design drawings contain explicit ASTM specifications and material test records or reports are available. In this case, special inspection reports for the structural steel framing were available and were deemed to satisfy these conditions. In addition, this information is considered to satisfy usual testing requirements in ASCE 41-13 § 9.2.2.4.1 for usual data collection requirements in ASCE 41-13 § 6.2.2. Thus, in accordance with ASCE 41-13 Table 6-1, a knowledge factor of $\kappa = 1.0$ is permitted for all analysis procedures provided the selected Structural Performance Level is Life Safety or lower. Component actions deemed as force-controlled are analyzed using lower-bound strengths, while deformation-controlled actions are compared to expected strengths. The nominal material properties for the steel designations listed in ASCE 41-13 Table 9-1 are taken as lower-bound values, and corresponding expected strengths are calculated by multiplying the lower-bound values by the appropriate factors in ASCE 41-13 Table 9-3.

See other chapters for further discussion on use of material properties and special inspection data.

8.2.3 Field Verification and Condition Assessment

For the purposes of this example, it is assumed that engineers conducted an on-site investigation to assess the as-built condition of the structure and identify any obvious evidence of deterioration or conditions that do not conform to the available drawings.

It is assumed that based on visual observations, the building was in excellent condition with little sign of deterioration. No structurally significant damage or foundation cracking were observed where foundations were visible, nor was any indication of foundation settlement or lateral movement observed. In addition, the existing conditions observed generally conformed to the construction drawings.

8.2.4 Structural Performance Objective

In the context of ASCE 41-13, a structural Performance Objective consists of a selected target Structural Performance Level in combination with a specific Seismic Hazard Level. For the seismic assessment of the subject building, BPOE is used as outlined in ASCE 41-13 Table 2-1. The BPOE varies with the designated Risk Category, and for Risk Category II, the BPOE is defined by the Life Safety (LS) Structural Performance Level. For this performance objective, the Seismic Hazard Level for Tier 1 and Tier 2 is the BSE-1E per ASCE 41-13 Table C2-2, which is characterized by an earthquake with a 20% probability of exceedance in 50 years (or 225-year return period) per

Table C2-1. For Tier 3 analysis, ASCE 41-13 Table 2-1 requires checks of Structural Performance Levels at the BSE-1E and BSE-2E Seismic Hazard Level. Although Tier 1 and Tier 2 procedures do not explicitly address the Collapse Prevention (CP) Structural Performance Level, they are deemed to comply with the full BPOE based on demonstrated compliance with the Life Safety Structural Performance Level. For this design example, only the BSE-1E Seismic Hazard Level is evaluated for illustrative purposes. It is worth noting that the single performance level evaluation for Tier 1 and Tier 2 in ASCE 41-17 is required to be performed at the CP performance level at the BSE-2E Seismic Hazard Level, which is intended to provide consistency with ASCE 7 provisions. If the structure is demonstrated to be compliant with CP acceptance criteria at BSE-2E, it is deemed to comply with LS at BSE-1E.

The Life Safety Structural Performance Level is defined in ASCE 41-13 as the post-earthquake damage state in which a structure has damaged components but retains a margin against the onset of partial or total collapse. Buildings that satisfy the BPOE requirements are expected to experience little damage from relatively frequent, moderate earthquakes, but the potential exists for significant damage and economic loss from the most severe and infrequent earthquakes.

8.2.5 Spectral Response Acceleration Parameters

Seismic parameters for the building are determined according to the General Procedure requirements of ASCE 41-13 § 2.4.1 for the BSE-1E Seismic Hazard Level. Similarly, per the requirements of ASCE 41-13 § 4.1.2, the BSE-1E is also used for the component evaluations in Tier 1 screening. The site class, latitude, and longitude for the building are as follows.

Site Class D

$F_a = 1.0$

$F_v = 1.5$

The following ground motion parameters are obtained using the online tools described in Chapter 3 of this *Guide.*

$S_{S,BSE\text{-}2N} = 2.363\text{g}$

$S_{1,BSE\text{-}2N} = 0.898\text{g}$

$S_{S,BSE\text{-}1E} = 1.096\text{g}$

$S_{1,BSE\text{-}1E} = 0.377\text{g}$

8.2.6 Level of Seismicity

The level of seismicity is based on the parameters, S_{DS} and S_{D1}, which are based on the ground motion parameters $S_{S,BSE\text{-}2N}$ and $S_{1,BSE\text{-}2N}$, respectively. Calculation of S_{DS} and S_{D1}, per ASCE 41-13 § 2.5, are as follows:

$$S_{DS} = (2/3)(F_a)(S_{S,BSE\text{-}2N})$$
$$= (2/3)(1.0)(2.363) = 1.575\text{g}$$

$$S_{D1} = (2/3)(F_v)(\ S_{1,BSE\text{-}2N})$$
$$= (2/3)(1.5)(0.898) = 0.898\text{g}$$

Based on ASCE 41-13 Table 2-5, for the given S_{DS} and S_{D1} values, the Level of Seismicity for the example site is defined as High. The designated Level of Seismicity determines the required Tier 1 checklists [ASCE 41-13 § 4.1.3] per ASCE 41-13 Chapter 16 and any supplementary Tier 2 calculations. Note that High seismicity sites include all evaluation statements for Low and Moderate seismicity sites as well.

8.2.7 Dead Loads and Seismic Weight

The flat load tables below describe the uniform distributed dead loads and seismic weight tributary to the building rigid diaphragms, such as steel beams, metal deck, concrete topping, MEP, and fireproofing. The weight of the perimeter curtain walls is treated separately, is applied as line loads to the perimeter framing in analysis models, and is not included in the flat loads.

The seismic weights tributary to each level are computed below and summarized in Table 8-2. The seismic weight at each level is the sum of the diaphragm flat weight, the exterior concrete wall panels around the perimeter and the tributary weight of curtain walls and partitions. The flat weight of the roof diaphragm is computed in Table 8-3, and the flat weight of a typical floor is computed in Table 8-6. Exterior concrete panel weights are computed in Table 8-4, and typical curtain wall and partition weights are computed in Table 8-5. The weight of curtain walls and partitions below the ground floor is computed in Table 8-7. The diaphragm seismic weight at the roof is computed as follows:

Roof weight = Flat weight + Exterior concrete wall panel
+ Curtain/Partition walls
= 2,421 k + 913 k + (833 k)/2 = 3,750 k

Note that only half of the curtain/partition weight is tributary to the roof level. For the fourth floor the diaphragm seismic weight is computed as follows:

$$4^{th} \text{ Floor Weight} = \text{Flat weight} + \text{Exterior concrete wall panel} + \text{Curtain/Partition walls}$$

$$= 2{,}382 \text{ k} + 913 \text{ k} + 833 \text{ k} = 4{,}129 \text{ k}$$

The remaining weights are summarized in Table 8-2 through Table 8-7.

Table 8-2 Diaphragm Tributary Weights

Level	Height (ft)	Perimeter (ft)	Area (ft²)	Weight (k)	Weight (psf)	Mass (lb-s²/ft)
Roof	14.25	869	37,232	3,750	100.7	116,583
4th Floor	14.25	869	37,232	4,129	110.9	128,339
3rd Floor	14.25	869	37,232	4,129	110.9	128,339
2nd Floor	14.25	869	37,232	4,129	110.9	128,339
Ground Floor	16	869	37,232	3,183	85.5	98,948
Total	**73.0**		**186,162**	**19,320**		**600,549**

Table 8-3 Roof Flat Weight Take-Off

Roof	Material Weight (psf)	Area (ft²)	Weight (k)	Notes
Roof Assembly	6	37,232	223	5-ply felt and gravel
Rigid Board Insulation	1	37,232	37	
Light Weight Concrete	40	37,232	1,489	3" on top of 2" Metal Deck, 120pcf
Roof Steel Decking	2	37,232	74	2" Galvanized Deck
Steel Framing	4	37,232	149	
Glass/Steel Skylight	15	10,000	150	
MEP Equipment	3	37,232	112	
MEP Systems	3	37,232	112	
Miscellaneous	2	37,232	74	
Total			**2,421**	

Table 8-4 Exterior Concrete Wall Panel Weight

Typical Precast Panels	Panel Weight (plf)	Thickness (in.)	Panel Height (ft)	Perimeter (ft)	Weight (k)	Notes
Upper Panel	800	8	8.0	869	695.5	Assume NWC
Lower Panel	250	8	2.5	869	217.3	Assume NWC
Total	**1050**				**913**	

Table 8-5 Curtain Wall Weight (Each Level)

Typical Walls	Material Weight (psf or plf)	Height (ft)	Length (ft), Number or Area (sf)	Weight (k)	Notes
Exterior Glazing	10	7.1	869	62	
Exterior Tile	20	7.1	869	124	
Exterior Column Covers	87.5	14.3	40	50	Assume 1" thick stucco/plaster
Interior Atrium Glazing	10	14.3	400	57	
Columns	131	14.3	90	168	
Partitions	10		37,232	372	
Total				**833**	

Table 8-6 Floor Flat Weight Take-Off

Typical Floor	Material Weight (psf)	Area (sf)	Weight (k)	Notes
Flooring	1	37,232	37	
Normal Weight Concrete	50	37,232	1,862	3" on top of 2" Metal Deck, 150pcf
Floor Steel Decking	2	37,232	74	2" Galvanized Deck
Steel Framing	5	37,232	186	
MEP Systems	3	37,232	112	
Ceilings	1	37,232	37	
Miscellaneous	2	37,232	74	
Total			**2,382**	

Table 8-7 Weight of Walls Below Grade

Walls Below Grade	Material Weight (psf or plf)	Height (ft)	Perimeter (ft), Number or Area (sf)	Weight (k)	Notes
Interior Atrium Glazing	10	16.0	400	64	
Columns	230	16.0	90	331	Assume W24×230 for all
Partitions	10		37,232	372	
Total				**768**	

8.3 Tier 1 Evaluation

Based on ASCE 41-13 Table 3-2, the Tier 1 and 2 evaluation procedures are permitted for steel moment frame buildings up to 8 stories with a High Level of Seismicity and a Life Safety performance objective. A Tier 1 screening of the building was initially performed. The building is 5-stories tall with 73 ft height, h_n, and with a weight, W, of 19,320 k.

The ASCE 41-13 Tier 1 16.1 Basic Checklist, 16.1.2LS Life Safety Basic Configuration Checklist, and 16.4LS Life Safety Structural Checklist for Building Types S1: Steel Moment Frames with Stiff Diaphragms and S1A: Steel Moment Frames with Flexible Diaphragms indicate that the primary building structure is noncompliant in four areas for Life Safety Performance as described in the following section. Following the Tier 1 evaluation, a Tier 2 Deficiency-Based Evaluation will be performed.

The following pages provide a compilation of select Tier 1 Quick Checks performed with an emphasis on those which typically result in non-compliance for this building type. Completed Tier 1 checklists are not provided for this example but are illustrated in detail in other chapters.

8.3.1 Seismic Hazard (ASCE 41-13 § 2.4)

Seismic hazard levels based on 2008 USGS National Seismic Hazard Maps are obtained for BSE-1E Seismic Hazard Level with Site Class D at the example building.

$S_{S,BSE\text{-}1E} = 1.096\text{g}$

$S_{S,BSE\text{-}1E} = 0.377\text{g}$

$S_{XS,BSE\text{-}1E} = 1.163\text{g}$

$S_{X1,BSE\text{-}1E} = 0.620\text{g}$

8.3.2 Pseudo Seismic Force (ASCE 41-13 § 4.5.2)

Fundamental period of the building:

$$T = Ch_n^{\beta} \quad \text{(ASCE 41-13 Eq. 4-5)}$$

$$C_t = 0.035 \quad \text{(Building Type S1)}$$

$$\beta = 0.80 \quad \text{(Building Type S1)}$$

$$T = (0.035)(73)^{0.80} = 1.083 \text{ seconds}$$

Response spectral acceleration:

$$S_a = \frac{S_{X1}}{T}\text{; shall not exceed } S_{XS} \quad \text{(ASCE 41-13 Eq. 4-4)}$$

$$S_a = \frac{(0.62\text{g})}{(1.083 \text{ seconds})} = 0.572\text{g} \rightarrow 0.572\text{g} < S_{XS} = 1.163\text{g}$$

Horizontal pseudo seismic force:

$$V = CS_aW \quad \text{(ASCE 41-13 Eq. 4-1)}$$

Modification factor $C = 1.0$ is used per ASCE 41-13 Table 4-8 for five-story Building Type S1.

$$V = (1.0)(0.572)(19{,}320 \text{ kips}) = 11{,}058 \text{ kips}$$

8.3.3 Story Shear Forces (ASCE 41-13 § 4.5.2.2)

The vertical distribution of seismic forces for an example frame in the north-south or Y-Direction is calculated.

$$V_{N/S} = 11{,}058 \text{ kips}$$

Seismic forces are vertically distributed in accordance with ASCE 41-13 Equations 4-3a and 4-3b:

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \quad \text{(ASCE 41-13 Eq. 4-3a)}$$

$$V_j = \sum_{x=j}^{n} F_x \quad \text{(ASCE 41-13 Eq. 4-3b)}$$

where:

V_j = Story shear at level j

n = Total number of stories above ground level

j = Number of story levels under consideration

W = Total seismic weight, per ASCE 41-13 Section 4.5.2.1

V = Pseudo seismic force from ASCE 41-13 Eq. 4-1

w_i = Portion of total building weight W located on or assigned to level i

w_x = Portion of total building weight W located on or assigned to level x

h_i = Height from base to floor level i

h_x = Height from base to floor level x

k = 1.3 (using linear interpolation for a period of T = 1.083 seconds).

Example Calculation, Roof Level

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V = C_{vx} V$$

$$w_x h_x^k = (3{,}750 \text{ kips})(73 \text{ ft})^{1.3} = 991{,}633$$

Sum of Previous Element for All Levels

$$\sum_{i=1}^{n} w_i h_i^k = 2{,}852{,}902$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} = \frac{991{,}633}{2{,}852{,}902} = 0.348$$

$$F_x = C_{vx} V = (0.348)(11{,}058 \text{ kips}) = 3{,}848 \text{ kips}$$

Since the total story shear, V_j, is a sum of shear from the current level and all levels above, for the roof level $F_x = V_x$. Table 8-8 presents a summary of story shear forces for all levels.

Table 8-8 Tier 1 - Vertical Distribution of Seismic Forces

	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_j (kips)
Roof	3,750	73.00	991,633	0.348	3,848	3,848
4th Floor	4,129	58.75	823,230	0.289	3,191	7,034
3rd Floor	4,129	44.50	573,692	0.201	2,224	9,263
2nd Floor	4,129	30.25	347,339	0.122	1,346	10,609
Ground Floor	3,183	16.00	117,007	0.041	454	11,063
Total	**19,320**		**2,852,902**	**1.00**	**11,063**	

8.3.4 Soft Story Check (ASCE 41-13 § A2.2.3)

The stiffness of the seismic-force-resisting system in any story is compared to 70% of the stiffness in the adjacent story above or 80% of the average stiffness of the three stories above. Sample calculations are provided for loading in the north-south direction and summarized in Table 8-9.

Notes

1. Assume that the stiffness of each story is based upon the column stiffness of exterior SFRS columns with fixed end conditions:

$$\text{Stiffness} = \frac{12EI}{L^3}$$

2. Columns at each story are all the same member size and oriented to resist loading along the strong axis.
3. This simplified method is valid where beam stiffness reduces in proportion to column stiffness at each story. Other relationships can be employed where this assumption is not satisfied.

Definitions

n_c = Total number of columns in the direction of loading

h_i = L = Story height at each story

I_x = I = Column moment of inertia

E = Elastic modulus = 29,000 ksi

C = Checklist item is Compliant

NC = Checklist item is Noncompliant

Example calculation at the ground story with twenty W24×162 columns:

$$\text{Total Stiffness} = n_c \frac{12EI}{L^3}$$

$$= (20 \text{ columns}) \frac{12(29{,}000 \text{ ksi})(5170 \text{ in.}^4)}{(192 \text{ in.})^3} = 5{,}084 \text{ k/in.}$$

The stiffness of the story above is calculated similarly and is compared to the ground floor stiffness:

$$\%\text{Stiffness} = \frac{\text{Ground Story Stiffness}}{\text{2nd Story Stiffness}} = \frac{5{,}084 \text{ k/in.}}{5{,}596 \text{ k/in.}} = 0.91$$

The ground story stiffness is 91% of the second floor stiffness, which is greater than the 70% required.

The average stiffness of the three stories above the ground story is calculated and compared to the ground stories stiffness.

$$\text{Avg Stiffness Above} = \frac{(5{,}596 \text{ k/in.} + 5{,}596 \text{ k/in.} + 3{,}758 \text{ k/in.})}{3}$$

$$= 4{,}983 \text{ k/in.}$$

$$\%\text{Stiffness} = \frac{\text{Ground Story Stiffness}}{\text{Avg Stiffness Above}} = \frac{5{,}084 \text{ k/in.}}{4{,}983 \text{ k/in.}} = 1.02$$

The ground story stiffness is 102% of the average stiffness for the three stories above, which is greater than the 80% required. Since the ground story stiffness meets both prescribed stiffness requirements, this building story is Compliant for this checklist item.

Table 8-9 Tier 1 - Soft Story Calculations

Level	Column Size	No	*L* (in.)	*I* (in⁴)	Stiffness (k/in.)	% Stiff-ness	% Stiffness (3-stories above)	Check
4th Story	W24×94	20	171	2,700	3,758	N.A.	N.A.	N.A.
3rd Story	W24×94	20	171	2,700	3,758	100%	100%	C
2nd Story	W24×131	20	171	4,020	5,596	149%	149%	C
Ground Story	W24×131	20	171	4,020	5,596	100%	128%	C
Base Story	W24×162	20	192	5,170	5,084	91%	102%	C

All individual levels indicate compliance with the soft story requirements. Therefore, the subject building is Compliant for this checklist item.

8.3.5 Torsion Check (ASCE 41-13 § A2.2.7)

The estimated distance between the story mass and the story center of rigidity is compared to 20% of the building width in either plan dimension.

Building width, BW = 217.3 ft

Rigidity

The structure's lateral force-resisting system is symmetric; therefore, the center of rigidity is located at the center of the building in width.

Center of Rigidity, CR = 108.7 ft

Mass

Table 8-10 presents the necessary information for calculation of center of mass.

Table 8-10 Tier 1 - Center of Mass

	Area (ft²)	ȳ (ft)	Area * ȳ (ft³)
	17,095	108.7	1,857,647
	8,402	108.7	913,001
	3,866	19.3	74,730
	7,866	178.0	1,400,069
Total	**37,229**		**4,245,448**

$$\text{Center of Mass, } CM = \frac{\sum(\text{Area} \times \overline{y})}{\sum \text{Area}} = \frac{4,245,448 \text{ ft}^3}{37,229 \text{ ft}^2} = 114 \text{ ft}$$

$0.2\ BW$ = (0.2)(217.3) = 43.5 ft

$CM - CR$ = 114 ft – 108.7 ft = 5.3 ft << 43.5 ft OK

The distance between Center of Mass and Center of Rigidity does not exceed 20% of the building width. Therefore, the subject building is Compliant for this checklist item.

8.3.6 Drift Quick Check (ASCE 41-13 § A3.1.3.1)

The story drift ratio of steel moment frames is evaluated using the Quick Check procedure. Sample calculations are provided for loading in the north-south direction.

Demand (D_r) = Quick check story drift per ASCE 41-13 § 4.5.3.1 Equation 4-7

Limit (D_A) = 0.025 for Life Safety (LS) per ASCE 41-13 § A3.1.3.1

Definitions

V_c = Shear in the column

h = Story height

I = Moment of inertia

L = Beam length from center-to-center of adjacent columns

k_b = I/L for the representative beam

k_c = I/h for the representative column

E = Elastic modulus = 29,000 ksi

$$D_r = \text{Drift ratio}^* = \left(\frac{(k_b + k_c)}{k_b k_c}\right)\left(\frac{h}{12E}\right)V_c$$

*if first floor columns are pinned at base, D_r at first floor shall be multiplied by 2

C = Checklist item is Compliant

NC = Checklist item is Noncompliant

Table 8-11 presents properties of the moment frames. An example calculation is shown for the roof level, and the results for each level are summarized in Table 8-12.

$$k_b = \frac{I}{L} = \frac{1830 \text{ in.}^4}{240 \text{ in.}} = 7.6 \text{ in.}^3$$

$$k_c = \frac{I}{h} = \frac{2700 \text{ in.}^4}{171 \text{ in.}} = 15.8 \text{ in.}^3$$

$$D_r = \left(\frac{\left(7.6 \text{ in.}^3 + 15.8 \text{ in.}^3\right)}{(7.6 \text{ in.}^3)(15.8 \text{ in.}^3)}\right)\left(\frac{171 \text{ in.}}{12(29000 \text{ ksi})}\right)(192 \text{ kips})$$

$$= 0.018 < 0.025 \text{ Compliant}$$

Table 8-11 Tier 1 - Moment Frame Properties

Level	Column Section	Beam Section	Columns No.	Columns V_c (k)	Column I (in.^{4})	Column h (in.)	Beam I (in.^{4})	Beam L (in.)
Roof	W24×94	W24×68	20	192	2,700	171	1,830	240
4th Floor	W24×94	W24×84	20	351	2,700	171	2,370	240
3rd Floor	W24×131	W24×117	20	462	4,020	171	3,540	240
2nd Floor	W24×131	W24×131	20	530	4,020	171	4,020	240
Base	W24×162	W24×176	20	553	5,170	192	5,680	240

Table 8-12 Tier 1 - Moment Frame Story Drift Ratio

Level	k_c (in^3)	k_b (in^3)	D_r	D_A	Check
Roof	15.8	7.6	0.018	0.025	C
4th Floor	15.8	9.9	0.028	0.025	NC
3rd Floor	23.5	14.8	0.025	0.025	NC
2nd Floor	23.5	16.8	0.027	0.025	NC
Base	26.9	23.7	0.024	0.025	C

Moment frames at the second, third and fourth floors all exceed the allowable story drift ratio. Therefore, the subject building is Noncompliant for this checklist item.

8.3.7 Axial Stress Due to Overturning Quick Check (ASCE 41-13 § A3.1.3.2)

The axial stress in end moment frame columns due to overturning is evaluated using the Quick Check procedure. Sample calculations are provided for a base column with loading in the north-south direction.

Demand = Quick Check axial stress in columns at base per ASCE 41-13 § 4.5.3.6 Equation 4-12

Capacity = $0.3F_y$ or $0.1f_y$ per ASCE 41-13 § A.3.1.3.2. The $0.3F_y$ check is selected here

Definitions

n_f = Total number of frames in the direction of loading

V = Pseudo seismic force (base shear)

h_n = Height above the base to the roof level

L = Total length of the frame

A_{col} = Area of the end column of the frame

M_s = System modification factor; M_s = 2.0 for LS

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right)$$

= Axial stress in moment frame column at the base due to overturning

C = Checklist item is Compliant

NC = Checklist item is Noncompliant

Table 8-13 presents the values for the base column. An example calculation is shown below:

$$p_{ot} = \left(\frac{1}{2}\right)\left(\frac{2}{3}\right)\left(\frac{(11058 \text{ kips})(73 \text{ ft})}{(186.5 \text{ ft})(2)}\right)\left(\frac{1}{47.8 \text{ in.}^2}\right) = 15.1 \text{ ksi}$$

$0.3F_y = 0.3(50 \text{ ksi}) = 15.0 \text{ ksi}$

Table 8-13 Tier 1 - Axial Stress Due to Overturning

Column Size	V (k)	F_y (ksi)	M_s	n_f	h_n (ft)	L (ft)	A_{col} (in.^{2})	p_{ot} (ksi)	$0.3F_y$ (ksi)	Check
W24×162	11,058	50	2.0	2	73.00	186.50	47.80	15.1	15.0	**NC**

The axial stress in the moment frame column due to overturning exceeds the allowable limits of $0.3F_y$. Therefore, the example column is Noncompliant for this checklist item.

8.3.8 Moment Frame Flexural Stress Quick Check (ASCE 41-13 § A3.1.3.3)

Check that the average flexural stresses in moment frame columns and beams using the Quick Check procedure do not exceed allowable values. Sample calculations are provided for loading in the north-south direction summarized in Table 8-14 and Table 8-15.

Demand = Flexural stress per ASCE 41-13 § 4.5.3.9 Equation 4-15.

Capacity = Material yield stress, F_y, per ASCE 41-13 § A3.1.3.3

Definitions

V_j = Story shear at level j

n_c = Total number of frame columns at level j

n_f = Total number of frames in direction of loading at level j

h = Story height

Z = Columns: The sum of plastic section moduli of all the frame columns at level j

= Beams: The sum of plastic section moduli of all frame beams with moment-resisting connections at level j (multiplied by 2 if beam has a moment connection each end)

M_s = System modification factor; $M_s = 8.0$ for LS

$$f_j^{\text{avg}} = V_j \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right)\frac{1}{Z}$$

= Average flexural stress in columns or beams

C = Checklist Item is Compliant

NC = Checklist Item is Noncompliant

Table 8-14 lists the moment frame properties, and Table 8-15 summarizes the results of the calculations.

Example Calculation, Roof Level Columns

$$f_j^{\text{avg}} = (3{,}833 \text{ kips})\left(\frac{1}{8}\right)\left(\frac{20}{20-2}\right)\left(\frac{171 \text{ in.}}{2}\right)\left(\frac{1}{20\left(254 \text{ in.}^3\right)}\right) = 9 \text{ ksi}$$

The average flexural stress in the roof level columns does not exceed F_y.

Example Calculation, Roof Level Beams

$$f_j^{avg} = (3{,}833 \text{ kips})\left(\frac{1}{8}\right)\left(\frac{20}{20-2}\right)\left(\frac{171 \text{ in.}}{2}\right)\left(\frac{1}{(18)(2)(177 \text{ in.}^3)}\right) = 7.1 \text{ ksi}$$

The average flexural stresses in roof level beams do not exceed F_y.

Since the average flexural stresses in roof level beams and columns both fall under required values, the example level is Compliant for this checklist item.

Table 8-14 Tier 1 - Moment Frame Properties

Level	V_j (k)	Column Section	Beam Section	Column F_y (ksi)	Beam F_y (ksi)	M_s	n_c	n_f	h(in.)
Roof	3,833	W24×94	W24×68	50	41	8.0	20	2	171
4th Floor	7,021	W24×94	W24×84	50	41	8.0	20	2	171
3rd Floor	9,247	W24×131	W24×117	50	41	8.0	20	2	171
2nd Floor	10,600	W24×131	W24×131	50	41	8.0	20	2	171
Base	11,058	W24×162	W24×176	50	39	8.0	20	2	192

Table 8-15 Tier 1 - Moment Frame Flexural Stresses

Level	V_j (k)	Column Z (in^3)	Beam Z (in^3)	Column f_j^{avg} (ksi)	Beam f_j^{avg} (ksi)	Check
Roof	3,833	5,080	6,372	9.0	7.1	C
4th Floor	7,021	5,080	8,064	16.4	10.3	C
3rd Floor	9,247	7,400	11,772	14.8	9.3	C
2nd Floor	10,600	7,400	13,320	17.0	9.4	C
Base	11,058	9,360	18,396	15.8	8.0	C

Average flexural stresses in moment frame columns and beams do not exceed allowable values at all levels. Therefore, the subject building is Compliant for this checklist item.

8.3.9 Panel Zone Check (ASCE 41-13 § A3.1.3.5)

All panel zones are checked to determine if they have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.

Definitions

M_p = Expected yielding moment capacity of beam

ΣM_p = Sum of the expected yielding moment capacities of the beams

V_p = Expected shear in panel zone due to beam yielding

$F_{ye,\text{beam}}$ = Expected strength of beams

$F_{ye,\text{column}}$ = Expected strength of columns

$F_{ye,\text{plate}}$ = Expected strength of doubler plate

Z_x = Strong axis plastic modulus

d_{beam} = Beam depth

d_{column} = Column depth

$t_{w,\text{column}}$ = Column web thickness

P_r = Column axial demand

P_c = Column axial capacity

t_p = Doubler plate thickness

V_e = Panel zone expected capacity

C = Checklist item is Compliant

NC = Checklist item is Noncompliant

The expected panel zone demand is:

$$\Sigma M_p = 2Z_x F_{ye,\text{beam}}$$

$$V_p = \frac{\sum M_p}{d_{\text{beam}}}$$

The expected panel zone capacity is conservatively calculated, neglecting the effect of panel zone deformation on frame stability in accordance with AISC 360-10 Section J10.6 Bullet (a) (AISC, 2010b):

(i) For $P_r \leq 0.4P_c$

$$V_e = 0.6\left(F_{ye,\text{column}} t_{w,\text{column}} + F_{ye,\text{plate}} t_p\right) d_{\text{column}}$$

(ii) For $P_r > 0.4P_c$

$$V_e = 0.6\left(F_{ye,\text{column}} t_{w,\text{column}} + F_{ye,\text{plate}} t_p\right) d_{\text{column}} \left(1.4 - \frac{P_r}{P_c}\right)$$

Note that the above equation is based on AISC 360-10 Equations J10-9 and J-10, accounting for potential differing material properties of column and plate. The panel zone capacity could also be computed using AISC 360-10 Section J10.6 Bullet (b) if panel zone deformation is included in the analysis.

Example Calculation, Roof Level Interior Joint

Table 8-16 presents the relevant information.

Table 8-16 Tier 1 – Panel Zone Check

Level	Beam	Z_x (in³)	d_{beam} (in.)	M_p (k-ft)	$0.8V_p$ (k)	Column
Roof (int)	W24×68	177	23.7	1,330	539	W24×94
Roof (ext)	W24×68	177	23.7	665	269	W24×94
4th Fl (int)	W24×84	224	24.1	1,684	671	W24×94
4th Fl (ext)	W24×84	224	24.1	842	335	W24×94
3rd Fl (int)	W24×117	327	24.3	2,458	971	W24×131
3rd Fl (ext)	W24×117	327	24.3	1,229	486	W24×131
2nd Fl (int)	W24×131	370	24.5	2,781	1,090	W24×131
2nd Fl (ext)	W24×131	370	24.5	1,391	545	W24×131
Ground (int)	W24x176	511	25.2	3,654	1,392	BW24x230
Ground (ext)	W24×176	511	25.2	1,827	696	W24×162

Beam = W24×68

Column = W24×94

$F_{y\text{-}LB\text{-beam}}$ = 41 ksi

$F_{ye\text{-beam}}$ = 1.1(41 ksi) = 45.1 ksi

$F_{y\text{-}LB\text{-column}}$ = 50 ksi

$F_{ye\text{-column}}$ = 1.1(50 ksi) = 55 ksi

A_T = (23.25 ft / 2)(20 ft) = 233 ft²

P_D = [(233 ft²)(65 psf) + (20 ft)(1,050 plf)] / (1000 lb/k)
= 36.1 k

P_r = $1.1P_D$
= (1.1)(36.1 k) = 39.7 k (Note that roof live load is zero since it is not combined with dead load for the earthquake case)

r_y = 1.98 in

KL = 1.2(171 in.) = 205 in

KL/r = 104

$$F_e = \frac{\pi^2\left(29{,}000\text{ ksi}\right)}{104^2} = 26.6\text{ ksi} \qquad (F_e > 0.44F_{y\text{-}LB})$$

$$F_{cr} = F_y\left(0.658\right)^{\left(F_y/F_e\right)}$$

F_{cr} = (50 ksi)(0.658)$^{(50/26.6)}$ = 22.8 ksi

P_c = (22.8 ksi)(27.7 in^2) = 632 k

P_r/P_c = 39.7 k/632 = 0.06 ($P_r \leq 0.4P_c$)

ΣM_p = 2(177 in.^{3})(45.1 ksi) = 15,695 k-in.

$$V_p = \frac{15{,}965 \text{ k-in.}}{23.7 \text{ in.}} = 674 \text{ k}$$

$0.8V_p$ = 539 k

V_e = 0.6[(55 ksi)(0.515 in.) + (55 ksi)(0.5 in.)](24.3) = 814 k

$0.8V_p < V_e$ Compliant

Panel zones are Compliant for this checklist item at the roof level. Additional levels are as noted in Table 8-17. The exterior panels zones at the 2nd and ground floors are Noncompliant for this checklist item.

Table 8-17 Tier 1 – Panel Zone Check

Level	$t_{w,\text{column}}$ (in.)	d_{column} (in.)	Doubler Plate, t_p (in.)	P_r/P_c	PZ Capacity, V_e (k)	Check
Roof (int)	0.515	24.3	0.50	0.06	814	C
Roof (ext)	0.515	24.3	0.00	0.08	413	C
4th Fl (int)	0.515	24.3	1.00	0.12	1,215	C
4th Fl (ext)	0.515	24.3	0.00	0.12	413	C
3rd Fl (int)	0.605	24.5	1.25	0.16	1,500	C
3rd Fl (ext)	0.605	24.5	0.00	0.16	489	C
2nd Fl (int)	0.605	24.5	1.50	0.21	1,702	C
2nd Fl (ext)	0.605	24.5	0.00	0.21	489	**NC**
Ground (int)	0.705	25.0	1.75	0.25	2,063	C
Ground (ext)	0.705	25.0	0.00	0.25	582	**NC**

8.3.10 Strong Column/Weak Beam Check (ASCE 41-13 § A3.1.3.7)

The percentage of strong column/weak beam joints in each story of each line of moment frames is compared to 50%. Sample calculations are provided for loading in the north-south direction. See AISC 341-10 Section E3.4a (AISC, 2010a) for additional information.

Definitions

Z_x = Strong axis plastic modulus

P_D = Column axial dead load

P_L = Column axial live load

P_G = Column axial gravity load, ASCE 41-13 Equation 7-1

P_E = Column axial seismic load

V_D = Beam shear dead load

V_L = Beam shear live load

V_G = Beam shear gravity load, ASCE 41-13 Equation 7-1

V_E = Beam shear seismic load

M_{pb} = $Z_x F_{ye,\text{beam}} + M_{uv}$, Nominal yielding moment capacity of beam, AISC 341-10 Equation E3-3a

M_{uv} = Additional moment due to shear amplification from location of plastic hinge to column centerline

M_{pc} = $Z_x(F_{ye,\text{column}} - P/A_g)$, Nominal yielding moment capacity of column, AISC 341-10 Equation E3-2a

$F_{ye,\text{column}}$ = 55 ksi

$F_{ye,\text{beam}}$ = 45.1 ksi (for group 2 beams, 2nd floor to roof)

SC = Indicates a strong column joint condition

SB = Indicates a strong beam joint condition

C = Checklist item is Compliant

NC = Checklist item is Noncompliant

Example Calculation at Second Floor Joint Along Gridline P/6

At an interior floor joint, there are two beams and one column framing into the joint. The tributary width for gravity loads is 11.63' and the beam span is 20'-0". The roof flat weight is 65 psf, the floor flat weight is 64 psf and the cladding is 1,050 plf.

Column = W24×131

Beam = W24×131

A = 38.6 in.^{2}

Z_x = 370 in.^{3}

A_T = (11.63 ft)(20 ft) = 233 ft^2

P_{Dead} = (233 ft^2)[3(0.064 ksf) + 0.065 ksf] + (4)(20 ft)(1.05 klf) = 144 k

P_L = (3)(233 ft^2)(50 psf)/(1000 lb/k) = 35 k

$M_{E\text{-}2}$ = Overturning moment at 2nd floor

$= \sum F_i \left(h_i - h_2\right)$

= (3,848 k)(73 ft – 30.25 ft) + (3,191 k)(58.75 ft – 30.25 ft) + (2,224 k)(44.5 ft – 30.25 ft)

= 287,129 k-ft

I_c = Moment of inertia of column group at second floor on Gridline P

$= \sum A_i d_i^2$ (all columns same size, set A = 1)

= 2[(10 ft)^2 + (30 ft)^2 + (50 ft)^2 + (70 ft)^2 + (93.25 ft)^2]

= 34,191 ft^2

$$P_E = \frac{0.5 M_{E-2} d_{P6}}{I_{C2-P}} = \frac{0.5\left(287{,}129 \text{ k-ft}\right)\left(50 \text{ ft}\right)}{34{,}191 \text{ ft}^2}$$

= 210 k

$$\frac{P_G + P_E}{A_g} = \frac{1.1\left[144 \text{ k} + 0.25\left(35 \text{ k}\right)\right] + 210 \text{ k}}{38.6 \text{ in.}^2} = 9.40 \text{ ksi}$$

ΣM_{pc} = 2(370 in.^{3})(55 ksi – 9.80 ksi) = 33,448 k-in

V_D = [(1.05 k/ft)(20 ft) + (11.63 ft)(20 ft)(0.065 ksf)]/2

= 18.1 k

V_L = (11.63 ft)(20 ft)(0.050 ksf)/2

= 5.82 k

V_G = 1.1[18.1 k + 0.25(5.82 k)]

= 21.5 k

$$V_E = \frac{\sum Z_x F_{ye}}{L_b}$$

$$= \frac{2\left(370 \text{ in.}^3\right)\left(45.1 \text{ ksi}\right)}{\left(20 \text{ ft}\right)\left(12 \text{ in./ft}\right) - 24 \text{ in.}}$$

= 155 k

V_{bL} = $V_E + V_G$

= 176 k

V_{bR} = $V_E - V_G$

= 133 k

ΣM_{uv} = [(176 k)(12 in.) + (133 k)(12 in.)]

= 3,708 k-in.

ΣM_{pb} = 2(370 in.^{3})(45.1 ksi) + 3,708 k-in. = 37,082 k-in.

$$\frac{\sum M_{pc}}{\sum M_{pb}} = \frac{33{,}744 \text{ k-in.}}{37{,}082 \text{ k-in.}} = 0.91 < 1.0 \quad \text{Joint is strong beam (SB)}$$

Table 8-18 summarizes the strong column/weak beam check.

Table 8-18 Tier 1 - Strong Column/Weak Beam Check

Level	End Columns		Field Columns		Total Joints #	SB Joints #	% SB Joints	Check
	#	Joint	#	Joint				
Roof	4	SC	16	SB	20	16	80%	**NC**
4th Floor	4	SC	16	SC	20	0	0%	C
3rd Floor	4	SC	16	SC	20	0	0%	C
2nd Floor	4	SC	16	SB	20	16	80%	**NC**
Ground Floor	4	SC	16	SC	20	0	0%	C

In the above table, "field columns" refers to interior moment frame columns between the two end columns of each frame. The percentage of strong column/weak beam joints at the roof and second floor levels does not meet minimum allowable requirements. Therefore, the subject building is Noncompliant for this checklist item. Strong-column/weak-beam deficiencies are generally not considered an issue at the top story of multistory frames as long as the members are compact.

8.3.11 Compact Members Check (ASCE 41-13 § A3.1.3.8)

Check that moment frame elements meet section requirements set forth by AISC 341-10 Table D1.1 for moderately ductile members. Sample calculations are provided for loading in the north-south direction.

Acceptance Criteria

Flanges $= \lambda_{md}$ shall be greater than $b_f/2t_f$

Webs $= \lambda_{md}$ shall be greater than h_f/t_w

Definitions

b_f = Flange width

t_f = Flange thickness

h = Web depth

t_w = Web thickness

F_y = Yield strength

E = Elastic modulus = 29,000 ksi

$C_a = \dfrac{P_u}{\phi_c P_y}$

P_u = Factored compression load using 1.1(D+L)+E

ϕ_c = 0.85 Resistance Factor

P_y = Axial yield strength = $A_g F_y$

A_g = Column gross area

λ_{md} = Limiting width-to-thickness ratio

For flanges: $\lambda_{md} = 0.38\sqrt{E/F_y}$

For webs: $\lambda_{md} = 3.76\sqrt{E/F_y}\left(1-2.75C_a\right)$

where:

$C_a \le 0.125$

$\lambda_{md} = 1.12\sqrt{E/F_y}\left(2.33-C_a\right) \ge 1.49\sqrt{E/F_y}$

where:

$C_a > 0.125$

C = Checklist Item is Compliant

NC = Checklist Item is Noncompliant

Example Calculation for Second Floor Column at Gridline P-6

Column: W24×131

Flanges:

$$\lambda_{md} = 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 > \frac{b_f}{2t_f} = 6.7 \quad \text{Compliant}$$

Webs:

P_D = 144 k (computed in previous example)

P_L = 35 k (computed in previous example)

P_E = 390 k (computed similar to previous example)

P_u = 1.1(144 k + 35 k)+390 k = 588 k

$$C_a = \frac{P_u}{\phi_c P_y}$$

$$= \frac{588 \text{ k}}{0.85\left(38.6 \text{ in.}^2\right)\left(50 \text{ ksi}\right)} = 0.36 > 0.125$$

$$\lambda_{md} = 1.12\sqrt{E/F_y}\left(2.33-C_a\right) \ge 1.49\sqrt{\frac{E}{F_y}}$$

$$= 1.12\sqrt{\frac{29,000}{50}}(2.33 - 0.36) = 53.1 \geq 1.49\sqrt{\frac{29,000}{50}} = 35.9$$

$$= 53.1$$

$$\lambda_{md} = 53.1 > \frac{h}{t_w} = 35.6 \quad \text{Compliant}$$

Table 8-19 and Table 8-20 present the results from columns and beams, respectively.

Table 8-19 Tier 1 - Compact Member Check - Columns

Section	F_y (ksi)	$b_f/2t_f$	λ_{md} Flange	P_u (k)	A_g (in²)	$\phi_c{*}P_y$ (k)	C_a	h/t_w	λ_{md} Web	Check
W24×94	50	5.2	9.2	198	27.7	1177	0.17	41.9	58.3	C
W24×131	50	6.7	9.2	674	38.6	1641	0.36	35.6	53.1	C
W24×162	50	5.3	9.2	1134	47.8	2032	0.56	30.6	47.8	C
W24×230	50	3.3	9.2	1134	67.8	2879	0.39	33.3	52.3	C

Table 8-20 Tier 1 - Compact Member Check - Beams

Section	F_y (ksi)	$b_f/2t_f$	λ_{md} Flange	C_a	h/t_w	λ_{md} Web	Check
W24×68	36	7.7	10.8	0	52.0	106.7	C
W24×84	36	5.9	10.8	0	45.9	106.7	C
W24×117	36	7.5	10.8	0	39.2	106.7	C
W24×131	36	6.7	10.8	0	35.6	106.7	C
W24×176	36	4.8	10.8	0	28.7	106.7	C

All moment frame columns and beams meet the compact section requirements. Therefore, the subject building is Compliant for this checklist item.

8.4 Tier 2 Evaluation

A Tier 2 evaluation is performed to explore the deficiencies identified in the Tier 1 evaluation in more detail. The building is evaluated per the requirements of ASCE 41-13 § 7.3 to confirm that all linear and nonlinear procedures are permitted. Chapter 4 of this *Guide* provides more information on analysis procedures. For this example, the following analyses are performed: linear static procedure (LSP) and linear dynamic procedure (LDP). In practice, it may not be practical to perform both linear analysis procedures for a Tier 2 evaluation. Many engineers will proceed directly with the LDP analysis, and forego the LSP, since both analysis procedures require similar effort, but the LDP often provides more favorable results.

8.4.1 Linear Static Procedure (LSP)

For the LSP analysis, a 3D model is used consisting of the moment frames along the perimeter of the building as shown in Figure 8-5. The model consists of structural members that exhibit linear elastic properties. The diaphragms satisfy the rigid diaphragm requirements defined in ASCE 41-13 § 9.8.2.2.1 and ASCE 41-13 § 7.2.9.1. Thus, diaphragms were modeled as rigid elements in plane. Torsion is included in the analysis per ASCE 41-13 § 7.2.3.2.

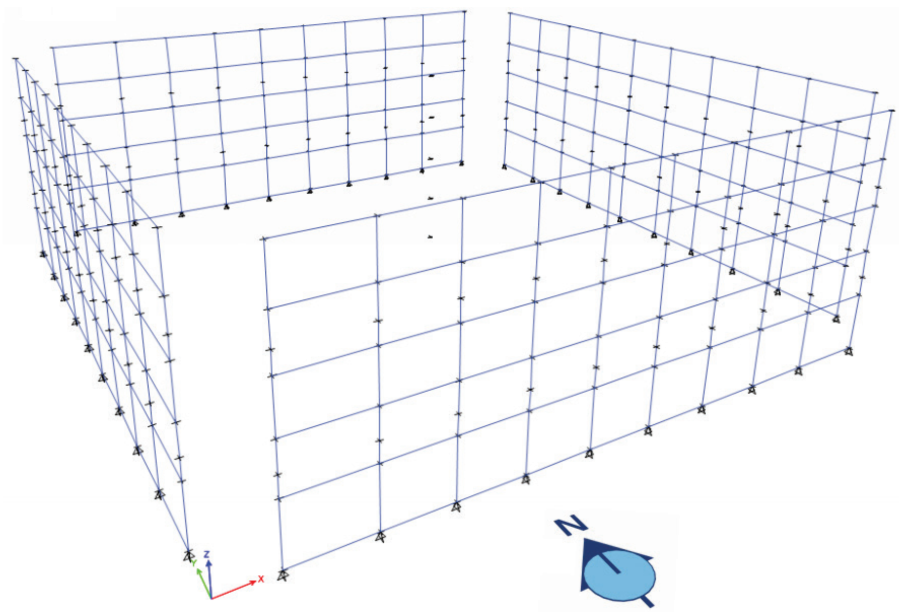


Figure 8-5 Linear model of example building.

The example building has a 100 foot by 100 foot atrium in plan that extends from the basement floor to the roof. The atrium is offset in plan about 28 feet to the southwest. Although the offset exists, the building is symmetric in plan and is expected to perform similarly in both principal directions. Additionally, the moment frames stop one bay in from the corners of the building, so there are no shared columns between orthogonal lateral force-resisting lines and each moment frame primarily resists horizontal forces in a single principal direction. Given these characteristics, the concurrent seismic effects (i.e., bi-directional analysis) of ASCE 41-13 § 7.2.5.1 need not be considered, and the LSP only considers one principal direction.

A vertical distribution of horizontal force based upon the mass and height of each floor level is applied at the center of mass at each level in the model. This type of distribution is approximately triangular. The model is run and demand forces are output and compared to component strengths with the component *m*-factors applied to evaluate the expected performance of the building as shown in the following sections.

8.4.1.1 Modeling Considerations

The analysis model is constructed using single line elements that connect node to node without rigid offsets. This method neglects deformation of the panel zone and increases the length of flexural members producing counteracting inaccuracies and is permitted by ASCE 41-13 § 9.4.2.2.1 Note 3. A rigid panel zone may be used where analysis shows that the stiffness of the panel zone is 10 times larger than the flexural stiffness of the beams. The panel zone stiffness was investigated, and it was found that, for the combinations of flexural members and doubler plates used in this building, the deformations of a centerline model were within 5% of the deformations where a flexible panel is considered. An example of a procedure to evaluate the effect of panel zone deformation on total drift is provided below. The story deformation for a joint is computed based on the three sources shown in Figure 8-6.

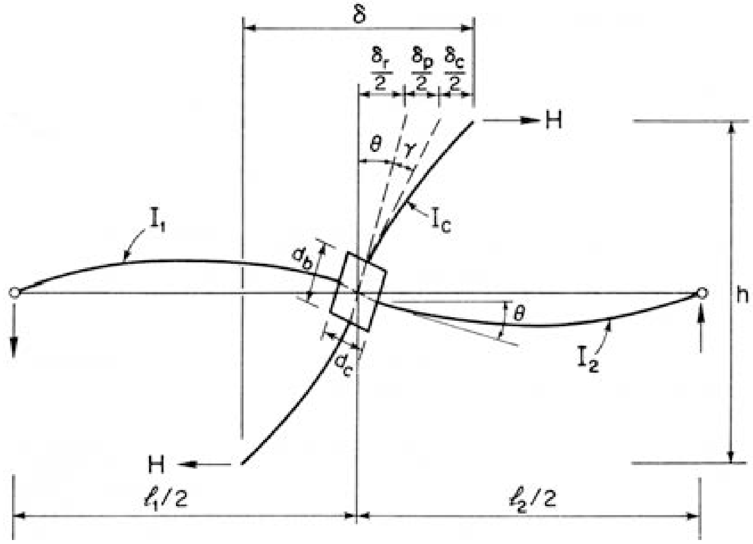


Figure 8-6 Illustration of joint deformations (from NIST, 2009).

The story drift, δ, is the sum of the three components:

$$\delta_r = \frac{h^2\left(1-\frac{2d_c}{l_1+l_2}\right)}{6E\left(\frac{I_1}{l_1-d_c}+\frac{I_2}{l_2-d_c}\right)}V_{\text{col}} \qquad \text{(story drift due to beam flexure)}$$

$$\delta_c = \frac{\left(h-d_b\right)^3}{12EI_c}V_{\text{col}} \qquad \text{(story drift due to column flexure)}$$

$$\delta_p = \frac{\left(h-d_b\right)\left(\frac{h}{d_b}-1\right)}{Gt_pd_c}V_{\text{col}} \qquad \text{(story drift due to panel zone deformation)}$$

The story drift with panel zone deformation included is evaluated as

$\delta_{PZ} = \delta_r + \delta_c + \delta_p$

The story drift using the centerline model is approximated taking $d_b = 0$ and $d_c = 0$ and evaluating as the following expression.

$\delta_{PZ} = \delta_r + \delta_c$

where:

V_{col} = Column shear force, kips

h = story height (centerline dimension), in.

l_1 = beam 1 span, in.

l_2 = beam 2 span, in.

I_1 = beam 1 moment of inertia, in.^{4}

I_2 = beam 2 moment of inertia, in.^{4}

I_c = column moment of inertia, in.^{4}

d_b = depth of beam, in.

d_c = depth of column, in.

t_p = thickness of joint panel zone, in.

E = modulus of elasticity of steel, ksi

The example will demonstrate the checks for an interior joint with W24×131 beams, W24×131 columns and a 1.0” doubler plate.

V_{col} = 300 kips

h =171 in.

l_1 = 240 in.

l_2 = 240 in.

I_1 = 4020 in.^{4}

I_2 = 4020 in.^{4}

I_c = 4020 in.^{4}

d_b = 24.5 in.

d_c = 24.5 in.

t_p = 1.61 in.

$$\delta_r = \frac{171^2\left(1-\frac{2(24.5)}{240+240}\right)}{6(29{,}000)\left(\frac{4020}{240-24.5}+\frac{4020}{240-24.5}\right)}(300)$$

$= 1.213$ in.

$$\delta_c = \frac{(171-24.5)^3}{12(29{,}000)(4{,}020)}(300)$$

$= 0.674$ in.

$$\delta_p = \frac{(171-24.5)\left(\frac{171}{24.5}-1\right)}{11{,}500(1.61)(24.5)}(300)$$

$= 0.579$ in.

$\delta_{PZ} = 1.213 + 0.674 + 0.579 = 2.467$ in

This total is compared to the centerline drift which neglects panel zone deformation by evaluating the same expressions with $d_b = d_c = 0$.

$$\delta_r = \frac{171^2\left(1-\frac{2(0)}{240+240}\right)}{6(29{,}000)\left(\frac{4020}{240-0}+\frac{4020}{240-0}\right)}(300)$$

$= 1.505$ in.

$$\delta_c = \frac{(171-0)^3}{12(29{,}000)(4{,}020)}(300)$$

$= 1.072$ in.

$\delta_p = 0$ (δ_p need not be evaluated for centerline model)

$\delta_{PZ} = 1.505 + 1.072 + 0 = 2.577$ in.

At this floor level, the centerline modeling assumption results in an increase in story drift of 4.4% compared to the connection model with panel zone deformation explicitly included, which was considered a reasonable agreement for this analysis. Therefore, the centerline model is used for this analysis.

8.4.1.2 Summary of Linear Static Procedure (LSP) Forces

In the following section, linear static procedure base shears and story forces are calculated per ASCE 41-13 § 7.4.1. Note that in the Tier 1 analysis the base shear was determined using the approximate building period. For the Tier 2 evaluation, the base shear is determined using the fundamental period based on an analytical Eigenvalue modal analysis of the mathematical model of the building. Soil-Structure Interaction (SSI) reductions per ASCE 41-13 § 8.5, such as kinematic effects, were not used for this example.

$V = C_1 C_2 C_m S_a W$ (ASCE 41-13 Eq. 7-21)

T = 1.68 s (Per analysis model)

$T_s = S_{X1}/S_{XS} = 0.620\text{g}/1.163\text{g} = 0.53$ s (ASCE 41-13 Eq. 2-9)

$T \geq T_S$

$\beta = 0.05$ (ASCE41-13 § 7.2.3.6)

$B_1 = 4/\left[5.6 - \ln\left(100\beta\right)\right]$ (ASCE 41-13 Eq. 2-11)

$= 4/\left[5.6 - \ln\left(100(0.05)\right)\right] = 1.0$

$S_a = S_{X1}/(B_1 T)$ (ASCE 41-13 Eq. 2-7)

$= (0.620\text{g})/[(1.0)(1.68 \text{ s}) = 0.369\text{g}$

k = 1.59(ASCE 41-13 § 7.4.1.3.2: interpolated for 0.50s < T < 2.50s)

Preliminarily assume $2 \leq m < 6$,

$C_1C_2 = 1.0$ (ASCE 41-13 Table 7-3: $T > 1.0$)

$C_m = 1.0$ ($C_m = 1.0$ where T > 1.0 s – see footnote)

V = (1.0)(1.0)(0.369g)(19,320 kips)
= 7,129 kips

Story forces are accordingly calculated per ASCE 41-13 Equation 7-24 and Equation 7-25, using a similar process as shown in Section 8.3.3 of this *Guide*, and are summarized in the following table. For the sake of comparison, these values are compared to the story forces and shears computed using the approximate period as determined in the Tier 1 screening. Note that in this case, the product of $C_1C_2C_m = 1.0$ matches the same coefficient, C, used to determine the horizontal pseudo seismic force from ASCE 41-13 Equation 4-1.

As seen in Table 8-21, the use of the calculated period provides a base shear reduction of approximately 35% compared to the empirical method for this particular structure. It should be noted that ASCE 41-13 does not have a cap on the calculated period used for base shear determination in comparison to ASCE 7-10.

Table 8-21 Story Forces and Shears

Story	h_x (ft)	Method 1 Calculated Period F_x (kips)	Method 1 Calculated Period V_x (kips)	Method 2 Empirical Period F_x (kips)	Method 2 Empirical Period V_x (kips)
Roof	73.00	2,713	2,713	3,833	3,833
4th	58.75	2,114	4,827	3,188	7,021
3rd	44.50	1,360	6,187	2,227	9,248
2nd	30.25	736	6,923	1,352	10,600
Ground	16.00	200	7,123	458	11,058

8.4.1.3 Moment Frame Connections

Acceptance criteria for moment frame connections are determined in accordance with ASCE 41-13 § 9.4.2.4.2. The strength of Fully Restrained (FR) beam-column connections is taken as the plastic moment capacity of the beam.

$M_{CE} = F_{ye}Z_x$

Knowledge factor, $\kappa = 1.0$ (ASCE 41-13 Table 6-1)

Existing beam material properties:

For Group 2 beams:

$F_{y\text{–}LB} = 41$ ksi (ASCE 41-13 Table 9-1, ASTM A36, 1961-1990)

$F_{ye} = (F_{y\text{–}LB})$(Factor from ASCE 41-13, Table 9-3)

$= (41 \text{ ksi})(1.10) = 45.1$ ksi

$F_u = 59$ ksi (ASCE 41-13 Table 9-1, ASTM A36, 1961-1990)

For Group 3 beams:

$F_{y\text{–}LB} = 39$ ksi (ASCE 41-13 Table 9-1, ASTM A36, 1961-1990)

$F_{ye} = (F_{y\text{–}LB})$(Factor from ASCE 41-13, Table 9-3)

$= (39 \text{ ksi})(1.10) = 42.9$ ksi

$F_u = 60$ ksi (ASCE 41-13 Table 9-1, ASTM A36, 1961-1990)

Existing column material properties:

$F_{y\text{–}LB} = 50$ ksi (ASCE 41-13 Table 9-1, ASTM A572, 1961-Present)

$F_{ye} = (F_{y\text{–}LB})$(Factor from ASCE 41-13, Table 9-3)

$= (50 \text{ ksi})(1.10) = 55$ ksi

$F_u = 66$ ksi (ASCE 41-13 Table 9-1, ASTM A572, 1961-Present)

m-Factor Example Calculation

Table 8-22 shows the *m*-factors for a Fully Restrained Moment Connection - Welded Unreinforced Flange (WUF) used as a primary component for a Life Safety Structural Performance Level (excerpted from ASCE 41-13 Table 9-4).

Table 8-22 Acceptance Criteria for Welded Unreinforced Flange

Component/ Action	*m*-Factors for Linear Procedures				
	IO	Primary		Secondary	
		LS	CP	LS	CP
Fully Restrained Moment Connections					
WUF	1.0	4.3 – 0.083d	3.9 – 0.043	4.3 – 0.048d	5.5 – 0.064d

the *m*-factor for:

$m_{LS} = 4.3 - 0.083d$ (ASCE 41-13 Table 9-4)

$= 4.3 - (0.083)(25.2 \text{ in.}) = 2.21$

where d = beam depth = 25.2 in.

Per ASCE 41-13 Table 9-4 Footnote (g), the tabulated *m*-factors for Fully Restrained Moment connections shall be modified as indicated in ASCE 41-13 § 9.4.2.4.2 Item (4) if prescriptive member configuration requirements are not satisfied. The modifiers each focus on an individual element within the moment frame beam to column connection. The items reflect recommended detailing practices from current code documents and are intended to identify connections that will perform in a ductile manner with balanced yielding between elements. When the connection detail does not meet these requirements, the beam-to-column performance is anticipated to perform with reduced ductility, and as such the allowable *m*-factor is reduced; however, it need not be taken as less than 1.0.

For this example connection, the modifiers are calculated for each item and the *m*-factor is adjusted based on the product of all modifiers:

- Item (4.1): Continuity plate modifier is based on FEMA 355F, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings* (FEMA, 2000f), recommendations for the relationship of continuity plate to column flange detailing. The connection shall satisfy one of the following three conditions, or a 0.8 modifier is required:

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

or

$$\frac{b_{bf}}{7} \leq t_{cf} < \frac{b_{bf}}{5.2} \text{ and continuity plates with } t \geq \frac{t_{bf}}{2}$$

or

$$t_{cf} < \frac{b_{bf}}{7} \text{ and continuity plates with } t > t_{bf}$$

where:

t_{cf} = thickness of column flange

b_{bf} = width of bean flange

t = thick of continuity plate

t_{bf} = thickness of beam flange

The three conditions are evaluated for the W24×176 beam to W24×162 column connection.

t_{cf} = 1.22 in.

b_{bf} = 12.9 in.

t = 1.25 in.

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

$$1.22 < \frac{12.9}{5.2} = 2.48 \quad \text{Not Satisfied}$$

$$\frac{b_{bf}}{7} \leq t_{cf} \leq \frac{b_{bf}}{5.2}$$

and

$$t \geq \frac{t_{bf}}{2}$$

$$\frac{12.9}{7} = 1.84 > t_{cf} = 1.22 \quad \text{Not satisfied}$$

$$t_{cf} < \frac{b_{bf}}{7} \text{ AND } t > t_{bf}$$

$$1.22 < \frac{12.9}{7} = 1.84 \text{ AND } t = 1.0 < t_{bf} = 1.22 \quad \text{Not satisfied}$$

- Since none of the three conditions are satisfied an adjustment factor of 0.8 is required.

- Item (4.2): Panel zone modifier is based on FEMA 355F recommendations for panel zone strength related to the flexural strength of the beam. If the following condition is not met, the tabulated values shall be modified by a factor of 0.8:

$$0.6 \le \frac{V_{PZ}}{V_y} \le 0.9$$

where:

$V_y = 0.55F_{ye(\mathrm{col})}d_c t_{cw}$

V_{PZ} is the computed panel zone shear at the critical location of the connection. For M_y at the face of the column:

$$V_{PZ} = \frac{\sum M_{ye(\mathrm{beam})}}{d_b}\left(\frac{L}{L-d_c}\right)\left(\frac{h-d_b}{h}\right)$$

where:

$F_{ye(\mathrm{col})}$ = expected yield strength pf column

d_c = column depth

t_{cw} = thickness of column

$M_{ye(\mathrm{beam})}$ = expected yield moment of beam

d_b = depth of beam

L = length of beam, center-to-center of columns

h = average story height of columns

The condition is evaluated for the W24×176 beam to W24×162 column connection at the end of the ground floor frame.

$F_{ye(\mathrm{col})}$ = 55 ksi

$F_{ye(\mathrm{beam})}$ = 42.9 ksi

d_c = 25.0 in.

t_{cw} = 0.705 in.

d_b = 25.2 in.

Z_{xb} = 511 in.^{3}

L = 23.25 ft
= 279 in.

h = 15.13 ft
= 181.5 in.

There is only one beam framing into the column, thus:

$\Sigma M_{ye(\mathrm{beam})} = F_{ye(\mathrm{beam})}Z_{xb}$ = (42.9 ksi)(511 in.^{3})
= 21,922 k-in.

The panel zone shear, V_{PZ}, is computed

$$V_{PZ} = \frac{21{,}922 \text{ k-in.}}{25.2 \text{ in.}}\left(\frac{279}{279-25}\right)\left(\frac{181.5-25.2}{181.5}\right)$$
$$= 823 \text{ k}$$

The column yield shear is found

$$V_y = 0.55 F_{ye(\text{col})} d_c t_{cw}$$
$$= 0.55(55)(25.0)(0.705) = 533 \text{ k}$$

Note that the above equation is for a column without any doubler plates, as is the case for this example column. When doubler plates are present, they should be included in the calculation of V_y, which takes the form:

$$V_y = 0.55\,(F_{ye(\text{col})} t_{cw} + F_{ye(dp)} t_{dp}) d_c$$

Where $F_{ye(dp)}$ and t_{dp} are the expected strength and thickness of doubler plates respectively.

Since $V_{PZ} > V_y$, the expression $0.6 \le \dfrac{V_{PZ}}{V_y} \le 0.9$ is not satisfied and an adjustment factor of 0.8 is required.

- Item (4.3): Clear span-to-depth modifier reflects the decreased ductility in longer span beams due to increased elastic rotations. If the clear span-to-depth ratio, L_c/d, is greater than 10, the tabulated *m*-factors in Table 9-4 shall be multiplied by:

$$\text{Adjustment factor} = 1.4 - 0.04\frac{L_c}{d}$$

where:

L_c = Length of clear span

d = depth of beam

L_c = 279 – 25 = 254 in.

d = 25.2 in

L_c/d= 10.1 > 10

The adjustment factor is required and is calculated as:

$$\text{Adjustment factor} = 1.4 - 0.04\frac{L_c}{d} = 0.997$$

- Item (4.4): Beam Flange and Web Slenderness Modifiers – reflect the effect of beam section properties on connection performance. If the beam flange and web meet the following conditions, the tabulated

m-factors in ASCE 41-13 Table 9-4 need not be modified for flange and web slenderness:

$$\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}} \text{ and } \frac{h}{t_w} < \frac{418}{\sqrt{F_{ye}}}$$

If the beam flange or web slenderness values exceed either of the following limits, the tabulated m-factors in ASCE 41-13 Table 9-4 shall be multiplied by 0.5.

$$\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}} \text{ and } \frac{h}{t_w} > \frac{640}{\sqrt{F_{ye}}}$$

Linear interpolation, based on the case that results in the lower modifier, shall be used for intermediate values of beam flange or web slenderness.

For a W24×176 beam:

$$\frac{b_f}{2t_f} = 4.81 < \frac{52}{\sqrt{F_{ye}}} = \frac{52}{\sqrt{42.9}} = 7.93$$

$$\frac{h}{t_w} = 28.7 < \frac{418}{\sqrt{F_{ye}}} = \frac{418}{\sqrt{42.9}} = 63.8$$

Therefore, no adjustment is required for the W24×176 beam.

Adjustment factor $= 1.0$

Therefore, the adjusted m-factor is:

$m_{LS\text{–adjusted}} = (2.21)(0.8)(0.8)(0.997)(1.0) = 1.41$

The moment strength of the connection is then calculated with the adjusted m-factor applied and is compared to the moment demand determined from the analysis model. The expected flexural capacity of the connection is:

$M_{CE} = F_{ye}Z_x$

For a W24×176 beam:

F_{ye} = 42.9 ksi

Z_x = 511 in.^{3}

M_{CE} = (42.9 ksi)(511 in.^{3}) / (12 in./ft) = 1,827 k-ft

M_U = 3,418 k-ft (moment demand from the analysis model)

κ = 1.0

m = 1.41

Q_{UD} = 3,418 k-ft

$\kappa m Q_{CE}$ = (1.0)(1.41)(1,827 k-ft) = 2,574 k-ft

$\kappa m Q_{CE} > Q_{UD}$ Connection does not meet Life Safety Performance Level acceptance criteria

In Table 8-23, the demand, Q_{CE}, and the product of the knowledge factor, m-factor and expected capacity, $\kappa m Q_{CE}$, are summarized for interior and exterior connections for one moment frame line at each floor level. During this analysis, all connections except for the roof level failed to meet the acceptance criteria for the Life Safety Performance Level using the LSP analysis.

Table 8-23 Tier 2 - LSP Connection Capacity Summary

		Beam W-Shape	Column W-Shape	m_{LSb}	Q_{UD} (k-ft)	Q_{CE} (Base Material Yield) (k-ft)	$\kappa m Q_{CE}$ (k-ft)	$\kappa m Q_{CE} > Q_{UD}$?
Ground Floor	Ext	W24×176	W24×162	1.41	3,696	1,827	2,574	N
	Int	W24×176	W24×230	1.41	3,669	1,827	2,574	N
2nd Floor	Ext	W24×131	W24×131	1.43	2,948	1,391	1,986	N
	Int	W24×131	W24×131	1.43	2,790	1,391	1,986	N
3rd Floor	Ext	W24×117	W24×131	1.43	2,700	1,229	1,762	N
	Int	W24×117	W24×131	1.43	2,567	1,229	1,762	N
4th Floor	Ext	W24×84	W24×94	1.44	1,595	842	1,211	N
	Int	W24×84	W24×94	1.44	1,589	842	1,211	N
Roof	Ext	W24×68	W24×94	1.81	774	665	1,204	Y
	Int	W24×68	W24×94	1.81	737	665	1,204	Y

[a] Above results are representative of one of the typical transverse moment frames. All transverse frames have similar connections.

[b] m_{LS} includes all applicable adjustment factors.

8.4.1.4 Moment Frame Beams

Steel beams in flexure are evaluated as deformation-controlled actions with acceptance criteria taken from ASCE 41-13 Table 9-4. Where Q_{CE} is less than M_{pCE} because of lateral torsional buckling, the m-factor from ASCE 41-13 Table 9-4 should be modified per ASCE 41-13 Equation 9-9. The W24×176 beam whose connections were evaluated in Section 8.4.1.2 of this *Guide* is evaluated below for flexure. The bottom flange is braced at the midpoint for all frame beams.

M_U = 3,669 k-ft (moment demand from the analysis model)

For a W24×176 beam:

F_{ye} = 42.9 ksi

Z_x = 511 in.^{3}

r_y = 3.04 in.

L_b = 120 in. (beams braced at midpoint)

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_{ye}}}$$

$$= (1.76)(3.04 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{42.9 \text{ ksi}}} = 139 \text{ in.}$$

Since L_b is less than L_p, the moment strength of the beam is not affected by lateral torsional buckling and no modification to the *m*-factor from ASCE 41-13 is required.

$M_e = F_{ye}Z_x$ (per AISC 360-10 § F2)

= 1,827 k-ft (computed previously)

Determine *m*-factor from ASCE 41-13 Table 9-4:

Case a: $m = 6$

Case b: $m = 2$

Flange compactness

$\lambda_f = b_f/2t_f = 4.81$

$\lambda_{fa} = 52/\sqrt{F_{ye}} = 52/\sqrt{42.9} = 7.94$

$\lambda_{fb} = 65/\sqrt{F_{ye}} = 65/\sqrt{42.9} = 9.92$

$\lambda_f < \lambda_{fa}$ (Case a applies for flange)

Web compactness

$\lambda_w = h/t_w = 27.8$

$\lambda_{wa} = 418/\sqrt{F_{ye}} = 418/\sqrt{42.9} = 63.8$

$\lambda_{wb} = 640/\sqrt{F_{ye}} = 640/\sqrt{42.9} = 97.7$

$\lambda_w < \lambda_{wa}$ (Case a applies for web)

$m = 6$

$\kappa = 1.0$

M_U = 3,669 k-ft

$\kappa m M_{CE}$ = (1.0)(6)(1,827 k-ft) = 10,962 k-ft

The same methodology is used to check all beams. All beams meet the acceptance criteria for the Life Safety Performance Level. Note that beam shear does not govern and is not evaluated.

8.4.1.5 Moment Frame Columns

For steel columns under combined axial compression and biaxial flexure (where the axial column load is less than 50% of the lower-bound axial strength), the column is considered deformation-controlled for flexural behavior (ASCE 41-13 Table 9-4 Footnote (b)) and the combined strength is evaluated using one of the following formulas.

Acceptance criteria for columns per ASCE 41-13 § 9.4.2.4.2:

For: $0.2 \le \frac{P_{UF}}{P_{CL}} \le 0.5$

$$\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \qquad \text{(ASCE 41-13 Eq. 9-10)}$$

For: $\frac{P_{UF}}{P_{CL}} < 0.2$

$$\frac{P_{UF}}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \qquad \text{(ASCE 41-13 Eq. 9-11)}$$

where:

P_{UF} = Axial force in the member computed in accordance with ASCE 41-13 § 7.5.2.1.2

P_{CL} = Lower-bound compression strength of the column

M_x = Bending moment in the member for the x -axis computed in accordance with ASCE 41-13 § 7.5.2.1.1

M_y = Bending moment in the member for the y -axis computed in accordance with ASCE 41-13 § 7.5.2.1.1

M_{CEx}= Expected bending strength of the column for the x-axis

M_{CEy} = Expected bending strength of the column for the y-axis

m_x = Value of *m* for the column bending about the x-axis in accordance with ASCE 41-13 Table 9-4

m_y = Value of *m* for the column bending about the y-axis in accordance with ASCE 41-13 Table 9-4

For steel columns with axial compressive forces exceeding 50% of the lower-bound axial strength, flexural and axial demands are considered force-controlled and evaluated using the following formula.

$$\frac{P_{UF}}{P_{CL}} + \frac{M_{UFx}}{M_{CLx}} + \frac{M_{UFy}}{M_{CLy}} \leq 1.0 \qquad \text{(ASCE 41-13 Eq. 9-12)}$$

The *m*-factors used in equations above are determined from ASCE 41-13 Table 9-4.

Note that columns are spliced with full penetration welds at the second and fourth floor levels; see Figure 8-7 for the splice detail from the existing building drawings. Column sections above and below the splice were reviewed at these levels. In cases where the splice connection does not develop the full strength of the column, the splice capacity should be evaluated. Although ASCE 41-13 does not provide explicit provisions for column splices, the following approach is considered reasonable based on AISC 341-10 provisions. NIST GCR 17-917-46v2, *Guidelines for Nonlinear Structural Analysis for Design of Buildings, Part IIa – Steel Moment Frames* (NIST, 2017), also provides guidance on nonlinear modeling of column splices with partial penetration welds.

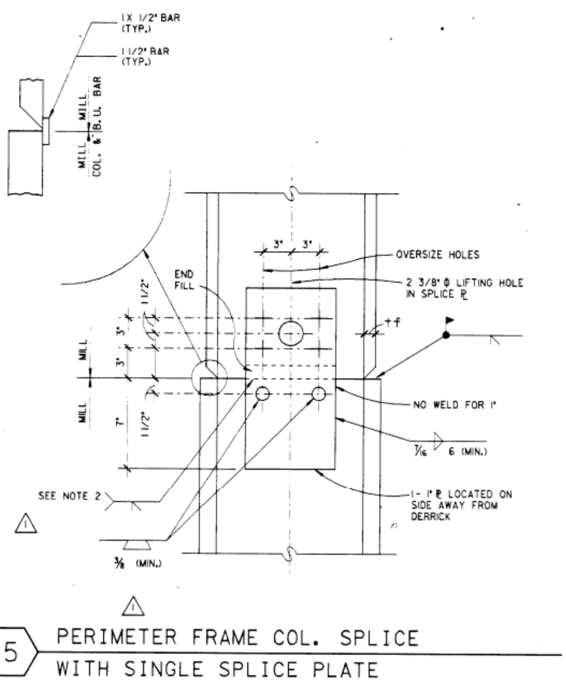


Figure 8-7 Existing column splice detail.

The flexural strength of the splice is evaluated as a force-controlled action with a demand equal to the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building. Alternatively, for linear procedures, the force-controlled action is calculated per ASCE 41-13 Equation 7-35. Note that the flexural demand need not exceed the expected plastic flexural strength of the smaller column ($M_p = F_{ye}Z$). The capacity would be determined as the lower-bound flexural strength of the splice connection considering limit states of welds or bolts as applicable. The shear strength is also evaluated as a force-controlled action, where the shear demand is the sum of the expected plastic flexural strength of the columns above and below the splice divided by the story height.

The following example illustrates a sample calculation for the evaluation of combined axial and bending strength of a second-story interior column. The column being evaluated is illustrated in Figure 8-8.

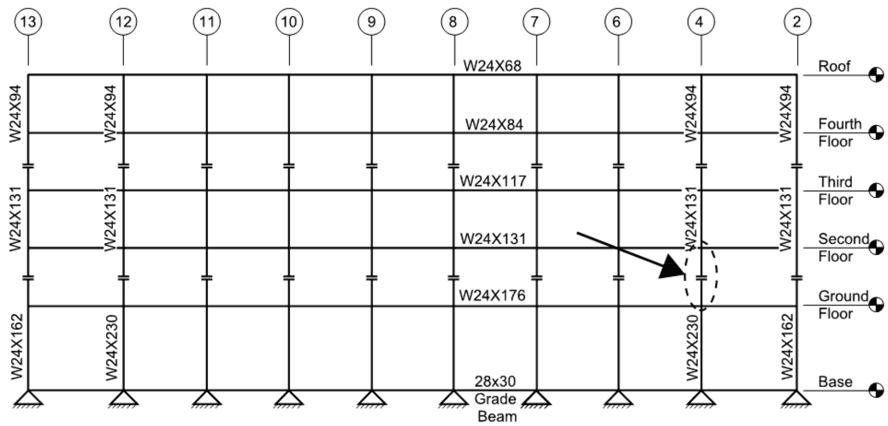


Figure 8-8 Elevation of Gridline P.

Demands

P_{UF} = 363 k (from analysis model)

M_{UDx} = 2,564 k-ft (from analysis model)

M_{UDy} = 7 k-ft (from analysis model)

Second-Order Effects (AISC 360-10 Appendix 8.2.2)

α = 1.00 (LRFD force-level adjustment factor)

$P_{story} = 1.1(P_{D,story} + 0.25P_{L,story}) = 18{,}987$ kips (from analysis model)

$P_{mf} = 1.1(P_{D,mf} + 0.25P_{L,mf}) = 1{,}271$ kips (from analysis model)

$R_M = 1 - 0.15\,(P_{mf} / P_{story}) = 0.99$ (AISC 360-10 Eq. A-8-8)

$H = V = 6{,}923$ kips (story shear from analysis model)

Δ_H = 3.40 in. (story drift from analysis model)

L = 14.25 ft (story height)

$$P_{e,\text{story}} = R_M\left(\frac{HL}{\Delta_H}\right) \quad \text{(AISC 360-10 Eq. A-8-7)}$$

$$= 0.99\left(\frac{(6{,}923 \text{ kips})(14.25 \text{ ft})(12 \text{ in./ft})}{3.40 \text{ in.}}\right)$$

$$= 344{,}704 \text{ kips}$$

$$B_2 = \frac{1}{1-\dfrac{\alpha P_{\text{story}}}{P_{e,\text{story}}}} \quad \text{(AISC 360-10 Eq. A-8-6)}$$

$$= \frac{1}{1-\dfrac{1.00(18{,}987 \text{ kips})}{344{,}704 \text{ kips}}}$$

$$= 1.06$$

Since $B_2 < 1.1$, the exception noted in AISC 360-10 Appendix 7.2.3 is met and it is permitted to take $K = 1.0$ for all columns.

Column Properties for W24×131

A = 38.6 in.^{2}

$\dfrac{b_f}{2t_f}$ = 6.7

$\dfrac{h}{t_w}$ = 35.6

Z_x = 370 in.^{3}

Z_y = 82 in.^{3}

r_{min} = 2.97 in.

L = 171 in.

Compute Lower Bound Strength per AISC 360-10

KL/r = (1.0)(171 in.)/(2.97 in.) = 57.6

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} = 86.3 \text{ ksi} \quad \text{(AISC 360-10 Eq. E3-4)}$$

F_y/F_e = (50 ksi) / (86.3 ksi) = 0.579 < 2.25

$$F_{cr} = \left(0.658^{F_y/F_e}\right)F_y \quad \text{(AISC 360-10 Eq. E3-2)}$$

$$= (0.658^{0.579})(50 \text{ ksi}) = 39.2 \text{ ksi}$$

P_{CL} $= F_{cr}A_g$ (AISC 360-10 Eq. E3-1)

$= (38.6 \text{ in.}^2)(39.2 \text{ ksi}) = 1{,}515 \text{ k}$

$$\frac{P_{UF}}{P_{CL}} = \frac{363 \text{ kips}}{1{,}515 \text{ kips}} = 0.24 \geq 0.2$$ (Use ASCE 41-13 Eq. 9-10)

Note that the above calculations assume that the W24x131 column is spanning the entire story height. In reality, the first quarter of the column is much larger (W24×230) and as such these member capacities are conservative.

Compute Flexural *m*-Factors Based on Slenderness

Check flange slenderness where F_{ye} = 55 ksi:

$$\frac{b_f}{2t_f} = 6.7 < \frac{52}{\sqrt{F_{ye}}} = 7.01$$

therefore:

$$m_f = 9\left(1-\left(\frac{5}{3}\right)\left(\frac{P}{P_{CL}}\right)\right) = 9\left(1-\left(\frac{5}{3}\right)\left(\frac{P}{P_{CL}}\right)\right) = 5.41$$

Check web slenderness:

$$\frac{260}{\sqrt{F_{ye}}} = 35.1 < \frac{h}{t_w} = 35.6 < \frac{400}{\sqrt{F_{ye}}} = 53.9$$

therefore:

m_w is found by interpolation.

The two points for interpolation are the m values at each limit for web slenderness, which is denoted as x below.

$(x_1, m_1) = (35.1, 5.41)$

$(x_2, m_2) = (53.9, 1.25)$

$$m_w = m_1 + (x - x_1)\left(\frac{m_2 - m_1}{x_2 - x_1}\right)$$

$$= 5.41 + (35.6 - 35.1)\left(\frac{1.25 - 5.41}{53.9 - 35.1}\right) = 5.30$$

m $= \min\{ m_f, m_w \}$

$= \min\{ 5.41, 5.30 \}$

$= 5.30$

Compute Flexural Strength

$M_{CEx} = F_{ce}Z_x = (370 \text{ in.}^3)(55 \text{ ksi})/(12 \text{ in./ft}) = 1{,}696 \text{ k-ft}$

M_{CEy} $= F_{ce}Z_y$ = (82 in.^{3})(55 ksi)/(12 in./ft) = 376 k-ft

Evaluate ASCE 41-13 Equation 9-10

$$\frac{P_{UF}}{P_{CL}}+\frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}}+\frac{M_y}{m_y M_{CEy}}\right]\le 1.0$$

$$\frac{363\text{ kips}}{1,515\text{ kips}}+\frac{8}{9}\left[\frac{2,564\text{ kip-ft}}{5.3(1,696\text{ kip-ft})}+\frac{7\text{ kip-ft}}{5.3(376\text{ kip-ft})}\right]=0.50\le 1.0\ \text{ OK}$$

Therefore, this column meets the acceptance criteria for the Life Safety Performance Level.

This calculation is repeated for the end column at Gridline 2 shown in Figure 8-9.

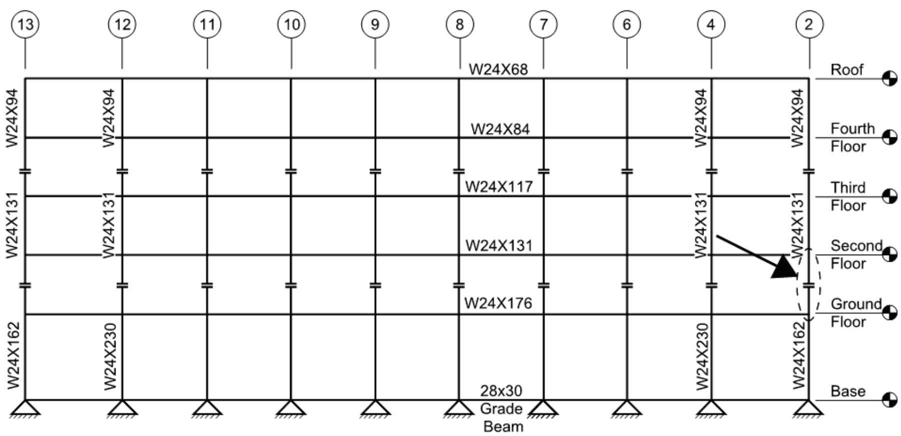


Figure 8-9 Elevation of Gridline P.

Demands

P_{UF}	= 532 k	(From analysis model)
M_{UDx}	= 1,354 k-ft	(From analysis model)
M_{UDy}	= 6 k-ft	(From analysis model)

Compute Lower Bound Strength per AISC 360-10

P_{CL} = 1,515 k (Computed above)

$$\frac{P_{UF}}{P_{CL}} = \frac{532}{1,515}=0.35 \qquad \text{(Use ASCE 41-13 Eq. 9-10)}$$

Compute Flexural *m*-Factors Based on Slenderness

Check flange slenderness where F_{ye} = 55 ksi

$$\frac{b_f}{2t_f} = 6.7<\frac{52}{\sqrt{F_{ye}}}=7.01$$

therefore:

$$m_f = 9\left(1-\left(\frac{5}{3}\right)\left(\frac{P}{P_{CL}}\right)\right) = 9\left(1-\left(\frac{5}{3}\right)\left(\frac{P}{P_{CL}}\right)\right) = 3.73$$

Check web slenderness.

$$\frac{260}{\sqrt{F_{ye}}} = 35.1 < \frac{h}{t_w} = 35.6 < \frac{400}{\sqrt{F_{ye}}} = 53.9$$

therefore:

m_w is found by interpolation

The two points for interpolation are the m-factors values at each limit for web slenderness, which is denoted as x below.

$(x_1, m_1) = (35.1, 3.73)$

$(x_2, m_2) = (53.9, 1.25)$

$$m_w = m_1 + (x - x_1)\left(\frac{m_2 - m_1}{x_2 - x_1}\right)$$

$$= 3.73 + (35.6 - 35.1)\left(\frac{1.25 - 3.73}{53.9 - 35.1}\right) = 3.66$$

$$\begin{aligned} m &= \min\{ m_f, m_w \} \\ &= \min\{ 3.73, 3.66 \} \\ &= 3.66 \end{aligned}$$

Compute Flexural Strength

$M_{CEx} = F_{ce}Z_x = (370\text{ in.}^3)(55\text{ ksi})/(12\text{ in./ft}) = 1{,}696\text{ k-ft}$

$M_{CEy} = F_{ce}Z_y = (82\text{ in.}^3)(55\text{ ksi})/(12\text{ in/ft}) = 376\text{ k-ft}$

Evaluate ASCE 41-13 Equation 9-10

$$\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0$$

$$\frac{532}{1{,}515} + \frac{8}{9}\left[\frac{1{,}354}{3.66(1{,}696)} + \frac{6}{3.66(376)}\right] = 0.55 \le 1.0 \text{ OK}$$

These calculations were performed at all locations and it was found that end column (located at each end of the moment frames) at the ground floor exceeds the Tier 2 allowable capacity. All other columns pass the checks.

Note that column shear does not govern and is not evaluated.

8.4.1.6 Panel Zone Evaluation

The panel zone shear is verified for the W24×176 beam to W24×162 column at the end of the ground floor frame.

The panel zone is evaluated as a deformation-controlled action with plastic shear capacity, Q_{CE}, calculated from ASCE 41-13 Equation 9-5 and with *m*-factors from ASCE 41-13 Table 9-4.

The expected panel zone demand is:

$$Q_{UD} = \frac{\sum M_{UD}}{d_{\text{beam}}}$$

ΣM_{UD} = 3,418 k-ft (moment demand from the analysis model)

Q_{UD} = (3,418 k-ft)(12 in./ft) / (25.2 in.)
= 1628 k

Q_{CE} = $0.55 F_{ye} d_c t_p$ (ASCE 41-13 Eq. 9-5)
= 0.55(55 ksi)(25.0 in.)(0.705 in.)
= 533 k

m = 8.0 (ASCE 41-13 Table 9-4)

Q_{UD} = 1,628 k

$m\kappa Q_{CE}$ = (8)(1.0)(533 k) = 4,264 k

$m\kappa Q_{CE} > Q_{UD}$

This exterior panel zone meets the Life Safety performance level. The process is repeated for an interior panel zone.

The shear is based on the sum of moments from analysis from each side of the column.

ΣM_{UD} = 3,230 k-ft + 2,960 k-ft = 6,190 k-ft (from analysis)

Q_{UD} = (6,190 k-ft)(12 in./ft) / (25.2 in.)
= 2,948 k

The column at interior joints is a W24×230 and is reinforced with 1-½" A572 Gr. 50 doubler plates.

d_c = 25.0 in.

t_{cw} = 0.75 in. (column web thickness)

t_{dpl} = 1.5 in. (doubler plate thickness)

t_p = $t_{cw} + t_{dpl}$ = 2.25 in.

$$Q_{CE} = 0.55F_{ye}d_c t_p$$
$$= 0.55(55 \text{ ksi})(25.0 \text{ in.})(2.25 \text{ in.})$$
$$= 1{,}702 \text{ k}$$

$$m = 8.0 \qquad \text{(ASCE 41-13 Table 9-4)}$$

$$Q_{UD} = 2{,}948 \text{ k}$$

$$m\kappa Q_{CE} = (8)(1.0)(1{,}702 \text{ k}) = 13{,}616 \text{ k}$$

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{LS is OK}$$

This interior panel zone meets the Life Safety performance level. The process is repeated for all panel zones, which are all found to meet the Life Safety Performance Level.

8.4.1.7 Frame Column Bases

Column bases in this example are modeled as pinned beneath the foundation beam with fixity provided by moment connected grade beams at the foundation level. The grade beams at the foundation level are modeled explicitly as fixed to steel columns. Connections between steel and concrete components shall meet the provisions of both steel and concrete sections of ASCE 41-13, Chapter 9 and Chapter 10 respectively, for strength and classification as either force-controlled or deformation-controlled.

For column baseplate yielding, bolt yielding, and weld failure, the use of *m*-factors from ASCE 41-13 Table 9-4, based on the respective limit states for partially restrained end plates, shall be permitted. Column base connection limit states controlled by anchor bolt failure modes governed by the concrete shall be considered force-controlled. The engineer is required to determine the appropriate analytical restraint conditions for column bases and evaluate all potential failure modes. Additional information is provided in ASCE 41-13 § 9.3.2.4. For brevity, this example assumes that the base plate and anchorage are sufficient to develop the moment strength of the columns.

8.4.1.8 LSP Results Summary

Based on the results of the LSP analysis, two deficiencies were identified: (1) the connections between the moment frame beams and the columns do not have adequate capacity, and (2) the columns at each end of moment frames are overstressed.

- An analysis was completed that shows that the beam-to-column connections yield prior to the column panel zones or the beams at nearly all of the locations throughout the building. Of those three mechanisms (connections, panel zones, beam hinges), the connection capacity will control the frame performance, which could be expected based on findings from the 1994 Northridge Earthquake. The capacity of the connections was exceeded at most locations, with calculated overstresses of up to 153% of connection capacities.

- The end columns at the ground floor exceed their capacity. All other columns have an interaction ratio of less than one using ASCE 41-13 Equation 9-10 with *m*-factors for Life Safety. The reason for the disparity in stress levels between interior and end columns is the additional compression forces imposed on the end columns as a result of complete frame overturning from resistance of lateral forces. At end columns, overstresses of up to 131% of column capacities were calculated.

A linear dynamic analysis may be performed to further evaluate the deficiencies identified by the LSP analysis.

8.4.1.9 Applicability of Linear Procedures (LSP and LDP)

Following the completion of the initial linear analysis, it is important to verify that the linear analysis procedures are permitted. ASCE 41-13 § 7.3.1.1 states: "*If a component DCR exceeds the lesser of* 3.0 *and the m-factor for the component action and any irregularity described in Section* 7.3.1.1.3 *or Section* 7.3.1.1.4 *is present, then linear procedures are not applicable and shall not be used.*"

For many of the components found to be deficient, specifically the beam-column connections, the DCR exceeds the lesser of 3.0 and the *m*-factor used to evaluate the component. However, no irregularities described in ASCE 41-13 § 7.3.1.1.3 or § 7.3.1.1.4 are present; therefore, linear procedures are allowed. In addition, ASCE 41-13 § 7.3.1.2 has a set of further limits on use of the LSP for buildings with long periods, major setbacks, torsional or vertical stiffness irregularities, and nonorthogonal seismic force-resisting systems, but the building in this design example does not have any of the features listed, so the linear procedures (LSP and LDP) remain viable.

8.4.2 Linear Dynamic Procedure (LDP)

For the LDP, a modal response spectrum analysis per ASCE 41-13 § 7.4.2.2.3 was performed on the same model built for the LSP analysis. A response spectrum analysis uses the mode shapes and modal participation factors to determine the peak forces and stresses within the structure.

Similar to the LSP analysis, the model is run and demand forces are output and compared to component strengths with the component *m*-factors applied (for deformation-controlled components) to evaluate the expected performance of the building elements. In most cases, the *m*-factors were the same as those used in the LSP analysis; however, there were slight variations for the columns that are dependent on calculated axial load.

The same component analyses for the moment frame connections and moment frame columns previously analyzed using LSP were evaluated with LDP, and the findings from the two procedures are compared.

8.4.2.1 Comparison of LDP and LSP Forces

Table 8-24 shows the story forces and story shear comparisons. The forces from the LDP analysis are slightly lower in magnitude than the LSP analysis, and they have a different distribution, as shown in the story forces.

Table 8-24 Tier 2 – Comparison of LSP and LDP Applied Story Forces

Level	LSP (kips)	LDP (kips)	LDP /LSP (kips)
Roof	2,713	2,520	0.93
4th Floor	2,114	1,450	0.69
3rd Floor	1,360	947	0.70
2nd Floor	736	919	1.25
Ground Floor	200	611	3.05
Sum	7,123	6,447	0.91

The LDP yielded a modest reduction in the base shear; however, energy that was located at the 3rd and 4th floors in the static distribution shifted down to the 2nd and ground floors for the LDP, which reduced the global overturning demands. ASCE 41-13 § 7.4.2.3.2 requires that seismic diaphragm forces calculated by the LDP must be at least 85% of those calculated by the LSP. It is assumed that this is met as the ratio of LDP/LSP for the base shear is 0.91.

8.4.2.2 Moment Frame Connections

Results from the LDP analysis were used to determine connection demands, Q_{UD}, and are compared against the product of the knowledge factor, *m*-factor and expected capacity, $\kappa m Q_{CE}$. Results are summarized in Table 8-25. The procedure for determining the *m*-factors is the same as for the LSP procedure outlined in Section 8.4.1.2 of this *Guide*. During this analysis, the connections were still determined to be overstressed ($Q_{UD} > \kappa m Q_{CE}$), although they are generally less overstressed than the LSP analysis.

Table 8-25 Tier 2 – LDP Moment Frame Connections

		Beam W-Shape	Column W-Shape	m_{LS}[b]	M_{UD} (k-ft)	MCE (Base Material Yield) (k-ft)	$\kappa m M_{CE}$ (k-ft)	$\kappa m Q_{CE} > Q_{UD}$?
Ground Floor	Ext	W24×176	W24×162	1.41	3,221	1,827	2,574	N
	Int	W24×176	BW24×230	1.76	3,028	1,827	2,574	N
2nd Floor	Ext	W24×131	W24×131	1.43	2,023	1,391	1,986	N
	Int	W24×131	W24×131	1.79	2,276	1,391	1,986	N
3rd Floor	Ext	W24×117	W24×131	1.43	2,170	1,229	1,762	N
	Int	W24×117	W24×131	1.79	2,060	1,229	1,762	N
4th Floor	Ext	W24×84	W24×94	1.44	1,350	842	1,211	N
	Int	W24×84	W24×94	1.80	1,340	842	1,211	N
Roof	Ext	W24×68	W24×94	1.81	774	665	1,204	Y
	Int	W24×68	W24×94	1.81	737	665	1,204	Y

[a] Above DCR results are representative of one of the typical transverse moment frames. All transverse frames have similar connections.

[b] m_{LS} includes all applicable adjustment factors.

8.4.2.3 Moment Frame Columns

Columns were evaluated using demands from the LDP analysis. The procedure for determining the *m*-factors is the same as for the LSP procedure outlined in Section 8.4.1.3 of this *Guide*. The LDP forces had a reduced global overturning demand which helped to reduce the overstress at the end columns at the ground floor. The level of overstress reduced from 125% of capacity to 90% of capacity.

8.4.2.4 LDP Results

The following is a summary of the results from the LDP evaluation:

- The capacity of the moment frame beam-to-column connections was again exceeded at nearly all locations. Although the overstresses were less than calculated using LSP analysis, overstresses were as high as 125% of connection capacities.
- Demands at the ground floor end columns have been reduced to 90% of column capacities.

As a result of the LDP, overall member DCRs have been reduced from the LSP analysis, only one deficiency remains: the connections between the moment beams and the columns do not have adequate capacity.

In comparing the results of the Tier 1 analysis with the Tier 2 linear procedures, the LSP Tier 2 analysis reduced the total number of deficiencies from four to two and the LDP Tier 2 analysis reduced the total number of deficiencies from four to one.

8.5 Tier 3 Evaluation

Following the Tier 2 evaluation, the moment frames are analyzed using the Tier 3 Nonlinear Static Procedure (NSP) to assess their performance. The process and results are summarized here to illustrate the difference and potential benefits in performing the nonlinear analysis.

8.5.1 Nonlinear Static Procedure (NSP)

For the NSP analysis, a two-dimensional (2D) nonlinear model was built as shown in Figure 8-10. The model was analyzed in PERFORM-3D (Version 5.0.1) (CSI, 2013). Note that other nonlinear modeling programs are available; use of PERFORM-3D is not an endorsement of the software by FEMA.

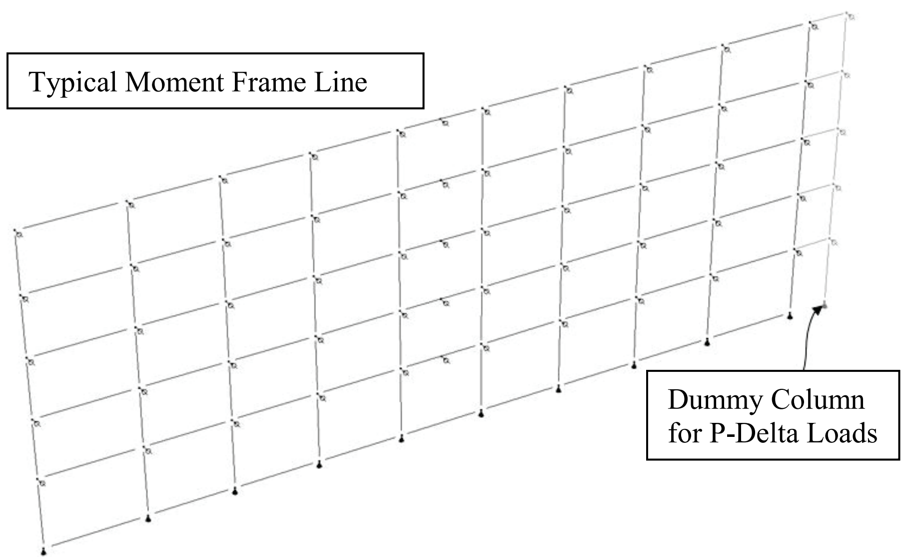


Figure 8-10 Nonlinear moment frame model from example.

Modeling one 2D frame line is a good approximation for a three-dimensional representation of the four moment frame lines since (1) all four moment frame lines consist of the same member sections; (2) all floor and roof levels consist of rigid diaphragms; and (3) the building has adequate torsional rigidity to equally distribute forces to individual frame lines. To account for

P-Delta effects, an 11th 'dummy' column was included in the model to represent the tributary gravity load and mass of the gravity frames that are not included in the 2D model. This column does not exist in the building, but affords the model the ability to accurately include the lateral displacement and force amplifications that occur from P-Delta effects. Alternatively, additional bays of gravity framing with partially restrained beam-column joints can be provided to account for the lateral stiffness of gravity framing while also capturing the P-Delta effects.

ASCE 41-13 § 9.4.2.2.2 Note (3) specifies that nonlinear behavior of panel zones shall be included in the mathematical model of the structure except where analysis indicates that panel zones remain elastic. Where panel zones remain elastic, it is permitted to model panel zones as indicated for linear static procedures in ASCE 41-13 § 9.4.2.2.1 Note (3). It was determined during the linear static procedures that the panel zones at the end connection of frames do not remain elastic and as such, the panel zone non-linearity is explicitly modeled for the NSP model.

It is important to note that in some configurations, the panel zone deformation can be significant even if panel zones remain elastic. When this occurs, the centerline model (used in a linear analysis) will yield an unrealistically low fundamental period of the building, which significantly underestimates the target displacement.

The model consists of structural members with linear elastic properties and hinges at the member ends with nonlinear properties. Three types of nonlinear hinges were incorporated into the model: (1) moment hinges at beam-to-column connections; (2) moment-axial interaction hinges at column ends; and (3) panel-zone hinges at beam-to-column connections, with column bases modeled as pinned at the foundation level with fixity provided by embedded moment connected beams at the foundation level. Each hinge is represented by a trilinear moment/rotation curve with strength degradation as specified in ASCE 41-13 Figure 9-1. Note that beam and column shear hinges do not govern the structural response and are not included in the model.

8.5.1.1 Connection Hinge Modeling Parameters and Acceptance Criteria

The following example derives the hinge backbone curve parameters per ASCE 41-13 Section 9.4.2 for Fully Restrained (FR) frames using modeling parameters and acceptance criteria of ASCE 41-13 Table 9-6. Figure 8-11 shows the location of the joint where hinge properties are calculated.

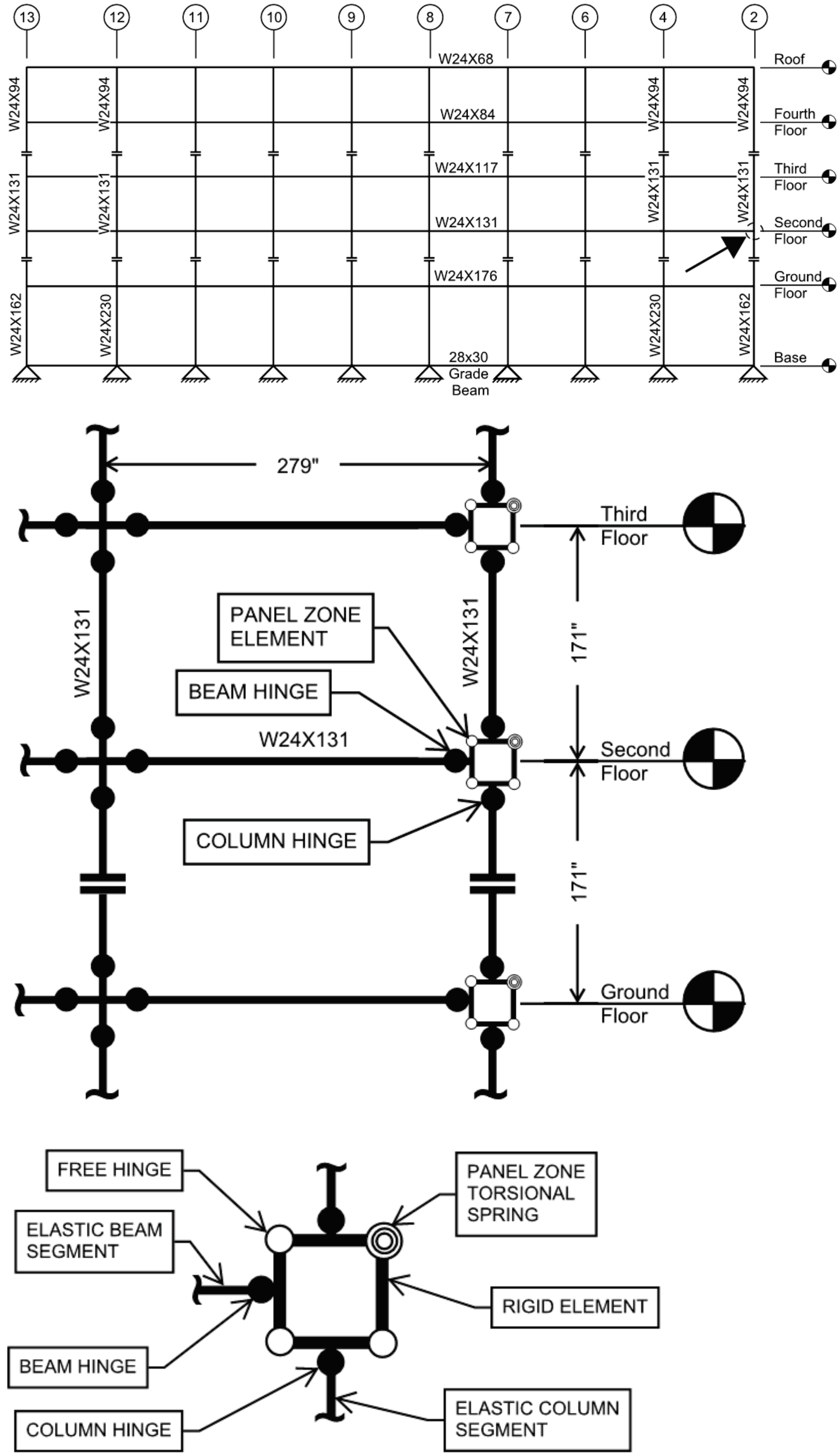


Figure 8-11 Elevation showing location of example hinges.

Beam Hinge

The beam hinge properties are based on the generalized force-deformation relation idealized in ASCE 41-13 Figure 9-1 (duplicated in Figure 8-12).

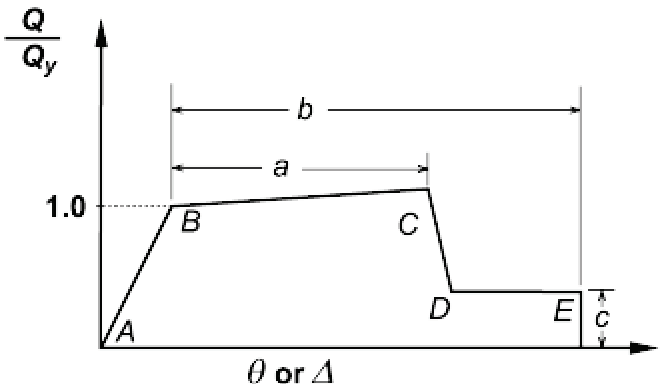


Figure 8-12 Generalized force-deformation relation (ASCE 41-13 Figure 9-1). Printed with permission from ASCE.

The hinges are located at the face of the columns. For the beam hinge, the yield moment of the connection is calculated based on the assumption that the connection is capable of developing the capacity of the beam.

W24×131 beam properties:

d = 24.5 in.

t_f = 0.96 in.

b_f = 12.9 in.

F_{ye} = 45.1 ksi

l_b = 279 in.

I_b = 4,020 in.^{4}

Z_x = 370 in.^{3}

E = 29,000 ksi

Yield strength of connection

$$
\begin{aligned}
M_y &= F_{ye}Z_x \\
&= (45.1\text{ ksi})(370\text{ in.}^3) / (12\text{ in./ft}) \\
&= 1{,}391\text{ k-ft}
\end{aligned}
$$

Yield rotation of connection per ASCE 41-13 Equation 9-1:

$$\theta_y = \frac{M_y l_b}{6EI_b} = \frac{(1{,}391\text{ kip-ft})(12\text{ in./ft})(279\text{ in.})}{6(29{,}000\text{ ksi})(4{,}020\text{ in.}^4)} = 0.0067\text{ radians}$$

Modeling parameters, a, b, and c, for a WUF connection are given in ASCE 41-13 Table 9-6.

$a = 0.051 - 0.0013d = 0.051 - 0.0013(24.5) = 0.0192$ radians

$b = 0.043 - 0.00060d = 0.043 - 0.00060(24.5) = 0.0283$ radians

$c = 0.2$

Acceptance criteria are also given in ASCE 41-13 Table 9-6:

$IO = 0.0260 - 0.00065d = 0.0260 - 0.00065(24.5) = 0.0101$ radians

$LS = 0.0323 - 0.00045d = 0.0323 - 0.00045(24.5) = 0.0213$ radians

$CP = 0.0430 - 0.00060d = 0.0430 - 0.00060(24.5) = 0.0283$ radians

Adjustment Factors

Per ASCE 41-13 Table 9-6 Footnote (f), the tabulated acceptance criteria plastic rotations are modified by the factors given in ASCE 41-13 § 9.4.2.4.3 Items (4.1) – (4.4).

- Item (4.1): Continuity plate modifier –The connection shall satisfy one of the following three conditions, or a 0.8 modifier will be required:

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

or

$$\frac{b_{bf}}{7} \leq t_{cf} < \frac{b_{bf}}{5.2} \text{ and continuity plates with } \frac{t_{bf}}{2}$$

or

$$t_{cf} < \frac{b_{bf}}{7} \text{ and continuity plates with } t > t_{bf}$$

where:

t_{cf} = thickness of column flange;

b_{bf} = width of beam flange;

t = thick of continuity plate; and

t_{bf} = thickness of beam flange.

The three conditions are evaluated for the W24×131 beam to W24×131 column connection below.

t_{cf} = 0.96 in.

b_{bf} = 12.9 in.

t = 0.5 in.

$$t_{cf} \geq \frac{b_{bf}}{5.2}$$

$$0.96 < \frac{12.9}{5.2} = 2.48 \quad \text{Not Satisfied}$$

$$\frac{b_{bf}}{7} \leq t_{cf} \leq \frac{b_{bf}}{5.2} \text{ and } t \geq \frac{t_{bf}}{2}$$

$$\frac{12.9}{7} = 1.84 > t_{cf} = 0.96 \quad \text{Not Satisfied}$$

$$t_{cf} < \frac{b_{bf}}{7} \text{ AND } t > t_{bf}$$

$$0.96 < \frac{12.9}{7} = 1.84 \text{ and } t = 0.5 < t_{bf} = 0.96 \quad \text{Not Satisfied}$$

Since none of the three conditions are satisfied an adjustment factor of 0.8 is required.

- Item (4.2): Panel zone modifier –If the following condition is not met, the tabulated values shall be modified by a factor of 0.8:

$$0.6 \leq \frac{V_{PZ}}{V_y} \leq 0.9$$

where:

$V_y = 0.55F_{ye(\text{col})}d_c t_{cw}$

V_{PZ} is the computed panel zone shear at the critical location of the connection. Note that there is no doubler plate at this location. For M_y at the face of the column:

$$V_{PZ} = \frac{\sum M_{y(\text{beam})}}{d_b}\left(\frac{L}{L-d_c}\right)\left(\frac{h-d_b}{h}\right)$$

where:

$F_{ye(\text{col})}$ = Expected yield strength of column

d_c = Column depth

t_{cw} = Thickness of column web

$M_{y(\text{beam})}$ = Yield moment of beam

d_b = Depth of beam

L = Length of beam, center-to-center of columns

h = Average story height of columns

The condition is evaluated for the W24×131 beam to W24×131 column connection.

$F_{ye(\text{col})}$ = 55 ksi

$F_{ye(\text{beam})}$ = 45.1 ksi

d = 24.5 in.

t_{cw} = 0.605 in.

d_b = 24.5 in.

Z_{xb} = 370 in.^{3}

L = 23.25 ft
= 279 in.

h = 171 in.

$\Sigma M_{y(\text{beam})} = F_{y(\text{beam})} Z_{xb}$ = (45.1 ksi)(370 in.^{3})
= 16,687 k-in.

The panel zone shear, V_{PZ}, is:

$$V_{PZ} = \frac{16{,}687 \text{ k-in.}}{24.5 \text{ in.}}\left(\frac{279 \text{ in.}}{279 \text{ in.} - 24.5 \text{ in.}}\right)\left(\frac{171 \text{ in.} - 24.5 \text{ in.}}{171 \text{ in.}}\right)$$
$$= 640 \text{ k}$$

The column yield shear is:

$$V_y = 0.55(55 \text{ ksi})(24.5 \text{ in.})(0.605 \text{ in.}) = 448 \text{ k}$$

Since $V_{PZ} > V_y$, the expression $0.6 \le \frac{V_{PZ}}{V_y} \le 0.9$ is not satisfied and an adjustment factor of 0.8 is required.

- Item (4.3): Clear span-to-depth modifier – If the clear span-to-depth ratio, L_c/d, is less than 8, the tabulated plastic rotations in ASCE 41-13 Table 9-6 shall be multiplied by:

  Adjustment factor = $(0.5)^{[(8 - L_c/d)/3]}$

  where:

  L_c = Length of clear span

  d = Depth of beam

  L_c = 279 in. – 24.5 in. = 254.5 in.

  d = 24.5 in.

  L_c/d= (254.5 in.)/(24.5 in.) = 10.4 > 8

  Since the clear span to depth ratio is greater than 8, no adjustment factor is required. Therefore, the factor is not calculated.

- Item (4.4): Beam flange and web slenderness modifiers reflect the effect of beam section properties on connection performance. If the beam flange and web meet the following conditions, the tabulated plastic rotations in ASCE 41-13 Table 9-6 need not be modified for flange and web slenderness:

$$\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}} \quad \text{and} \quad \frac{h}{t_w} < \frac{418}{\sqrt{F_{ye}}}$$

If the beam flange or web slenderness values exceed either of the following limits, the tabulated plastic rotations in ASCE 41-13 Table 9-6 shall be multiplied by 0.5.

$$\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}} \quad \text{and} \quad \frac{h}{t_w} > \frac{640}{\sqrt{F_{ye}}}$$

Linear interpolation, based on the case that results in the lower modifier, shall be used for intermediate values of beam flange or web slenderness.

For the W24×131 beam:

$b_f/2t_f = 6.72 < 52/\sqrt{F_{ye}} = 7.74$ Condition Satisfied

$h/t_w = 40.5 < 418/\sqrt{F_{ye}} = 62.2$

Therefore, no adjustment factor is required.

The tabulated acceptance criteria are adjusted by a cumulative adjustment factor, *AF*, of:

$AF = (0.8)\ (0.8)\ (1.0)\ (1.0) = 0.64$

Therefore:

$a = 0.0192\ (0.64) = 0.0123$

$b = 0.0283\ (0.64) = 0.0181$

$c = 0.2\ (0.64) = 0.128$

$IO = 0.0101\ (0.64) = 0.00645$

$LS = 0.0213\ (0.64) = 0.0136$

$CP = 0.0283\ (0.64) = 0.0181$

The strain hardening slope is 3% per ASCE 41-13 § 9.4.2.2.2. Thus, the moment at Point C is given by:

$$M_C = M_y \left(1 + 0.03 \frac{a}{\theta_y}\right) = 1{,}391 \left[1 + 0.03 \left(\frac{0.0123}{0.0067}\right)\right] = 1{,}468 \text{ k-ft}$$

The rotation at Point C is $\theta_c = \theta_y + a = 0.0067 + 0.0123 = 0.0190$ radians. The residual strength at Points D and E, $M_D = M_E = cM_y = 0.128M_y = 178$ k-ft. A gradual slope is recommended along the C-D segment for model stability and is taken as $\theta_D = 1.1\theta_C = 0.0209$ radians. The rotation at point E, $\theta_E = \theta_y + \mathrm{b} = 0.0067 + 0.0181 = 0.0248$ radians.

The beam connection hinge moment-rotation relationship is shown in Figure 8-13 below with a vertical line at the LS limit.

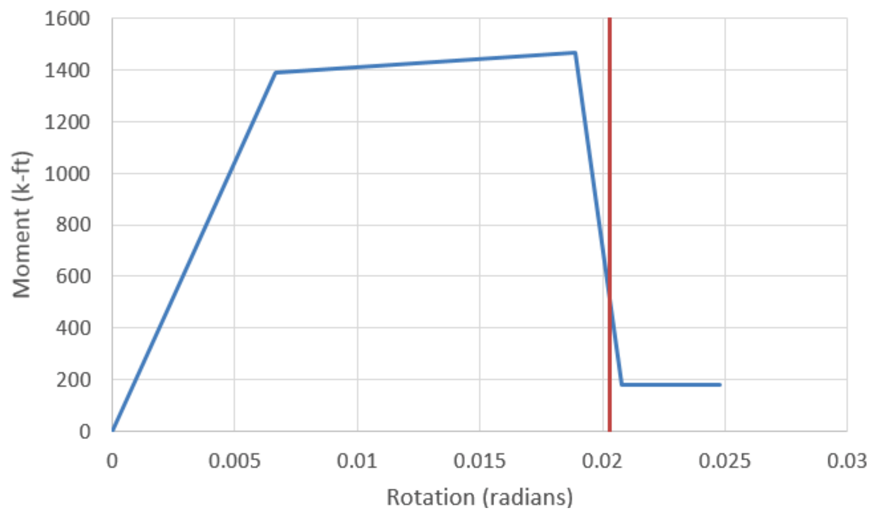


Figure 8-13 Nonlinear beam connection hinge for a W24×131 beam.

8.5.1.2 Column Modeling Parameters and Acceptance Criteria

All nonlinear modeling parameters and acceptance criteria for the columns are a function of the axial load, P, at the target displacement. But since the target displacement depends on the pushover results, an initial estimate must be made and verified later. In this example, the axial load, P, will be taken as the axial loads calculated in the previous LDP.

P_{UF} = 532 kips (Assumed force at target displacement)

P_{CL} = 1,515 kips (Lower bound compressive strength, see Section 8.4.1.3 of this *Guide*)

$P_{ye} = F_{ye}A_g$ (Expected compressive strength)

$= (55 \text{ ksi})(38.6 \text{ in.}^2)$

$= 2{,}123$ kips

$P/P_{ye} = (532 \text{ kips})/(2{,}123 \text{ kips}) = 0.25$

$P/P_{CL} = (532 \text{ kips})/(1{,}515 \text{ kips}) = 0.35$

$0.2 \leq P/P_{CL} \leq 0.5$

Modeling Parameters

F_{ye} = 55 ksi (Expected column yield strength)

$Z = Z_x = 370$ in.^{3} (W24×131 column plastic section modulus)

$I = I_x = 4{,}020$ in.^{4} (W24×131 column moment of inertia)

L = 14.25 ft (Column length)

= 171 in.

$M_y = M_{CE} = \min\{1.18ZF_{ye}(1 - P/P_{ye}),\ ZF_{ye}\}$ (ASCE 41-13 Eq. 9-4)

$= \min\{1.18(370 \text{ in.}^3)(55 \text{ ksi})(1 - 0.25)\ ,\ (370 \text{ in.}^3)(55 \text{ ksi})\}$

= min{18,010 k-in., 20,350 k-in.} / (12 in./ft)

= 1,501 k-ft

$$\theta_y = \frac{ZF_{ye}L}{6EI}\left(1 - \frac{P}{P_{ye}}\right)$$ (ASCE 41-13 Eq. 9-2)

$$= \frac{(370 \text{ in.}^3)(55 \text{ ksi})(171 \text{ in.})}{6(29000 \text{ ksi})(4020 \text{ in.}^4)}\left(1 - 0.25\right)$$

= 0.0037 rad

Flange compactness:

$\lambda_f = b_f/2t_f = 6.72$

$\lambda_{fa} = 52/\sqrt{F_{ye}} = 52/\sqrt{55} = 7.01$

$\lambda_{fb} = 65/\sqrt{F_{ye}} = 65/\sqrt{55} = 8.76$

$\lambda_f < \lambda_{fb}$

Web compactness:

$\lambda_w = h/t_w = 35.6$

$\lambda_{wa} = 260/\sqrt{F_{ye}} = 260/\sqrt{55} = 35.1$

$\lambda_{wb} = 400/\sqrt{F_{ye}} = 400/\sqrt{55} = 53.9$

$\lambda_{wa} < \lambda_w < \lambda_{wb}$ (Interpolation is required)

$(\lambda_w - \lambda_{wa}) / (\lambda_{wb} - \lambda_{wa}) = (35.6 - 35.1) / (53.9 - 35.1) = 0.0266$

Modeling parameters (ASCE 41-13 Table 9-6)

$a_a/\theta_y = 11(1 - 5/3P/P_{CL}) = 11(1 - 5/3(0.35)) = 4.58$

$a_b/\theta_y = 1.00$

$a_f/\theta_y = 4.58$ (flange is compact, use case a)

$a_w/\theta_y = 4.58 + 0.0266(1 - 4.58) = 4.48$ (web is non-compact, use case c)

a/θ_y = min{a_f/θ_y, a_w/θ_y} = 4.48 (use min of flange/web)

b_a/θ_y = 17(1 –5/3P/P_{CL}) =17(1 – 5/3(0.35)) = 7.08

b_b/θ_y = 1.50

b_f/θ_y = 7.08 (flange is compact, use case a)

b_w/θ_y = 7.08 + 0.0266(1.50 – 7.08) = 6.93 (web is non-compact, case c)

b/θ_y = min{b_f/θ_y, b_w/θ_y} = 6.93 (use min of flange/web)

c_a/θ_y = c_b/θ_y = 0.20

Acceptance criteria

IO_a/θ_y = IO_b/θ_y = 0.25

IO/θ_y = 0.25

LS_a/θ_y = 14(1 –5/3P/P_{CL}) =14(1 – 5/3(0.35)) = 5.83

LS_b/θ_y = 1.2

LS_f/θ_y = 5.83 (flange is compact, use case a)

LS_w/θ_y = 5.83 + 0.0266(1.2 – 5.83) = 5.71 (web is non-compact, case c)

LS/θ_y = min{ LS_f/θ_y , LS_w/θ_y } = 5.71 (use min of flange/web)

CP_a/θ_y = 17(1 –5/3P/P_{CL}) =17(1 – 5/3(0.35)) = 7.08

CP_b/θ_y = 1.2

CP_f/θ_y = 7.08 (flange is compact, use case a)

CP_w/θ_y = 7.08 + 0.0266(1.2 – 7.08) = 6.92 (web is non-compact, case c)

CP/θ_y = min{ CP_f/θ_y , CP_w/θ_y } = 6.92 (use min of flange/web)

The beam column hinge moment-rotation relationship is shown in Figure 8-14 with a vertical line at the LS limit.

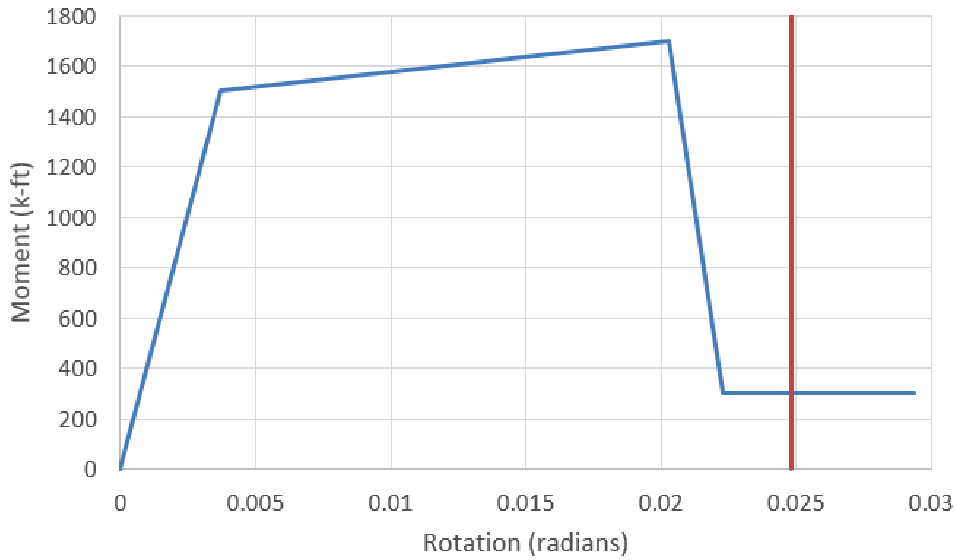


Figure 8-14 Nonlinear column hinge for a W24×131 column.

8.5.1.3 Panel Zone Hinge Modeling Parameters and Acceptance Criteria

The beam hinge properties are based on the generalized force-deformation relation idealized in ASCE 41-13 Figure 9-1 (duplicated in Figure 8-12). The yield strength, Q_{CE}, is defined in ASCE 41-13 Equation 9-5 as follows:

$$Q_{CE} = V_{CE} = 0.55F_{ye}d_c t_p$$

No guidance for yield rotation of the panel zones is provided in ASCE 41-13. This example uses ASCE 41-17 Equation 9-3 to compute the yield rotation.

$$\theta_y \equiv \gamma_y = \frac{F_{ye}}{G\sqrt{3}}\sqrt{1-\left(\frac{|P|}{P_{ye}}\right)^2}$$

where:

d_c = Depth of column

γ_y = Panel zone shear yield strain

F_{ye} = Expected yield strength of panel zone (inclusive of doubler plates)

t_p = Thickness of panel zone (inclusive of doubler plates)

G = Shear modulus of steel

F_{ye} = Expected yield strength of steel

P = Axial force in column (tension or compression)

P_{ye} = Expected axial yield capacity of column (= A_gF_{ye})

The remaining factors that define the generalized force-deformation relationship are provided in ASCE 41-13 Table 9-6 are summarized:

a = $12\theta_y$

b = $12\theta_y$

c = 1.0

IO = $1\theta_y$

LS = $12\theta_y$

CP = $12\theta_y$

As noted in ASCE 41-13 § 9.4.2.2.2, the post-yield strain-hardening slope is permitted to be taken as 6%. The following example derives the force-deformation relationship for the ground level, exterior W24×176 beam to W24×162 column connection. To develop the initial hinge properties, the axial load in the column at the target displacement is estimated and should be verified following the development of the pushover curve and determination of the target displacement.

Column Parameters:

d_c = 25 in.

t_p = 0.705 in. (no doubler plate)

F_{ye} = 55 ksi

A_g = 47.8 in.^{2}

G = 11,500 ksi

P = 895 k (estimated max at target displacement)

P_{ye} = (47.8 in.^{2})(45.1 ksi) = 2,629 k

Yield strength and rotation – Point B

$$Q_{CE} = V_{CE} = 0.55F_{ye}d_ct_p$$
$$= 0.55(55\text{ ksi})(25.0\text{ in.})(0.705\text{ in.})$$
$$= 533\text{ k}$$

$$\theta_y = \gamma_y = \frac{55\text{ ksi}}{(11{,}500\text{ ksi})\sqrt{3}}\sqrt{1-\left(\frac{|895\text{ k}|}{2{,}629\text{ k}}\right)^2}$$
$$= 0.002596$$

Strength and rotation at Point C

$$\theta_C = \theta_y + \theta_a = 0.002596 + 12(0.002596)$$
$$= 0.0338$$

$$Q_C = Q_{CE}[1+12(0.06)] = (533\text{ k})[1+12(0.06)]$$
$$= 917\text{ k}$$

Because $a = b$, and $c = 1$; the generalized force deformation relationship is effectively reduced to a bilinear curve. To convert to a rotational spring for the mechanism described in Figure 8-11, the shear is multiplied by the distance between beam flange centroids.

d_b = 25.2 in. (Beam depth)

t_f = 1.34 in. (flange thickness)

$$M_{CE} = V_{CE}(d_b - t_{fb})$$
$$= (533 \text{ k})(25.2 \text{ in.} - 1.34 \text{ in.}) / (12 \text{ in./ft})$$
$$= 1{,}060 \text{ k-ft}$$

$$M_B = M_{CE}[1+12(0.06)]$$
$$= (1{,}060 \text{ k-ft})[1+12(0.06)]$$
$$= 1{,}823 \text{ k-ft}$$

The bilinear force-deformation relationship is shown in Figure 8-15 below with a vertical line at the LS limit.

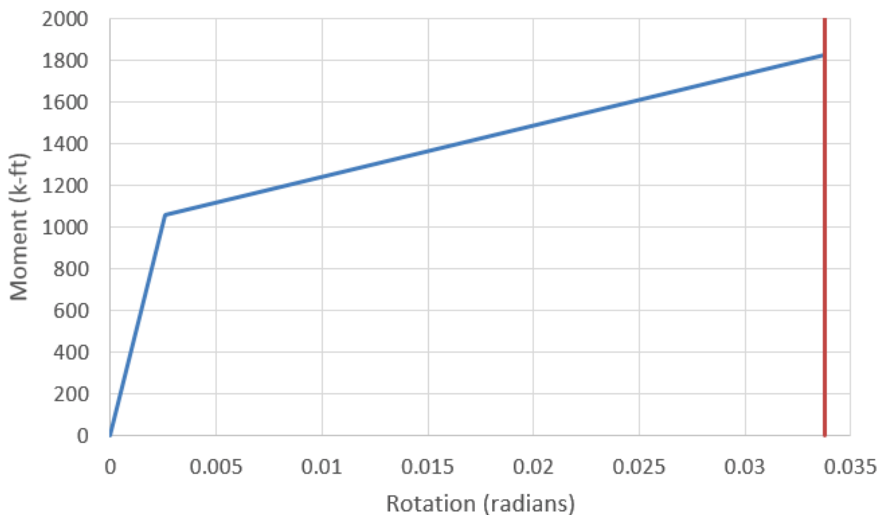


Figure 8-15 Nonlinear panel zone hinge for a W24×176 beam to W24×162 column.

Once the analysis is complete and the target displacement is determined, the assumed axial load used for these calculations may be verified or iterated to modify the panel zone hinge based on the actual target displacement.

8.5.1.4 Pushover Curve and Target Displacement

For the NSP, the structure is first loaded with all tributary gravity loads as specified in ASCE 41-13 § 7.2.2, and then incrementally loaded laterally until the structure reaches a point of failure. Unlike linear analyses, this method shows progressively what elements yield in the structure and how forces are redistributed to other members as those elements yield. In accordance with ASCE 41-13, the vertical distribution of lateral load follows a pattern proportional to the primary modal shape (ASCE 41-13 § 7.4.3.2.3 and FEMA 440 [FEMA, 2005]).

For a given Seismic Hazard Level (20% in 50 years, 5% in 50 years, etc.), a building is expected to drift laterally to a target (maximum) displacement during the earthquake. ASCE 41-13 provides a procedure to determine the target displacement based upon previous building performance during prior earthquakes and analytical research. For this example, the seismic hazard level being evaluated is the BSE-1E (20% in 50 years). If all of the yielding members do not exceed plastic rotation limits at that target displacement, then the building is expected to meet that performance level. For the example building frame model (per Figure 8-10), the structure was pushed using the prescribed modal distribution, a target displacement, δ_t, at the roof level control node was calculated, and then the plastic rotations checked at each nonlinear hinge to see if acceptance criteria were met for the Life Safety Structural Performance Level.

The NSP target displacement is calculated per ASCE 41-13 § 7.4.3.2. An idealized pushover curve is derived from the "actual" pushover curve determined from the nonlinear analysis using the graphical procedure outlined in ASCE 41-13 Figure 7-3. The first iteration of the pushover is shown in Figure 8-15.

V_y = 2,490 kips

K_i = 327 k/in.

K_e = 327 k/in.

T_i = 1.68 sec (From PERFORM-3D modal analysis)

$T_e = T_i\sqrt{K_i / K_e}$ (ASCE 41-13 Eq. 7-27)

$= (1.68\text{ s})\sqrt{(327\text{ k/in.})/(327\text{ k/in.})}$

$= 1.68$ sec

S_{X1} = 0.62 (For BSE-1E)

$S_a = S_{X1}/T_e = 0.620/1.68 = 0.369g$

W = 9,660 kips (per frame)

C_m = 0.90 (ASCE 41-13 Table 7-4)

$\mu_{\text{strength}} = S_aC_m/(V/W)$ (ASCE 41-13 Eq. 7-3)

$= [(0.369)(0.90)] / (2{,}490 / 9{,}660)$

$= 1.29$

a = 60 (Site Class D)

C_0 = 1.4 (ASCE 41-13 Table 7-5: 5-story other building)

$$C_1 = 1+\frac{\mu_{\text{strength}}-1}{aT_e^2} \quad \text{(ASCE 41-13 Eq. 7-29)}$$

$$= 1+\frac{1.29-1}{60(1.68)^2}$$

$$= 1.002$$

$$C_2 = 1+\frac{1}{800}\left(\frac{1-\mu_{\text{strength}}}{T_e}\right)^2 \quad \text{(ASCE 41-13 Eq. 7-30)}$$

$$= 1+\frac{1}{800}\left(\frac{1-1.43}{1.68}\right)^2$$

$$= 1.00$$

$$\delta_t = C_0C_1C_2S_a\frac{T_e^2}{4\pi^2}g \quad \text{(ASCE 41-13 Eq. 7-28)}$$

$$= 1.4(1.002)(1.00)(0.369)\frac{(1.68)^2}{4\pi^2}(386.1)$$

= 14.3 in.

= 14.3 in. / (73 ft building height × 12 in./ft) = 1.63% average story drift

It is worth noting that explicitly calculating the C_0 coefficient per ASCE 41-13 § C7.4.3.3.2 instead of using tabulated values would likely reduce the target displacement. Per ASCE 41-13 § 7.4.3.2.1, the relationship between base shear and control node displacement shall be established for control node displacements ranging between 0% and 150% of the target displacement.

$\delta_{max} = 1.5\delta_t$

= 1.5(14.3 in.)

= 21.4 in.

= 21.4 in. / (73 ft building height × 12 in./ft) = 2.45% average story drift

In this example, the building is being evaluated for Life Safety at the BSE-1E hazard level. The target displacement has been found for the BSE-1E hazard level, and acceptance criteria have been established for individual components.

As shown in Figure 8-16, the moment frame beam-to-column connections exceed the associated Life Safety Structural Performance Level acceptance limits prior to the structure reaching the target displacement for a BSE-1E seismic hazard level. At this limit, the moment frame connections have yielded, strain hardened, and have begun to lose strength (indicating fracture of the welds). Furthermore, the analysis model became unstable for it reached 150% of the target displacement, highlighting that retrofit is needed.

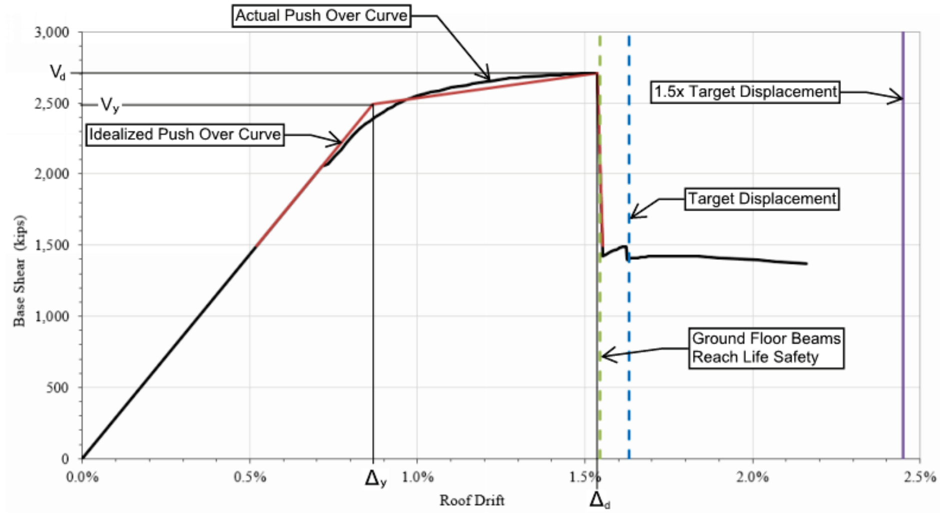


Figure 8-16 Pushover curve.

Figure 8-17 shows the connections that exceed the limits at the target displacement. The output indicates hinges at the ground floor have exceeded the Life Safety limit. The figure illustrates the ratio of hinge rotation to the Life Safety limit with hinges changing color as they exceed user specified thresholds, which are specified as 40%, 60% and 80% of the Life Safety limit. The moment frame connections at the ground floor are therefore considered deficient. Additionally, since all hinge failures at the target displacement occur in beam hinges, the column capacity is no longer considered a deficiency as the column hinges do not exceed the Life Safety limits.

As a result of the NSP, only the connection deficiency between the ground floor moment beams and the columns remains. This deficiency could potentially be deemed adequate with the inclusion of partially-restrained gravity connections in the nonlinear model as the initial stiffness will help to reduce the target displacement (see Section 8.5.1.6 of this *Guide*). It is also worth noting that while one deficiency remains, retrofit would require global reassessment with revised component actions as new deficiencies can occur when strengthening individual components.

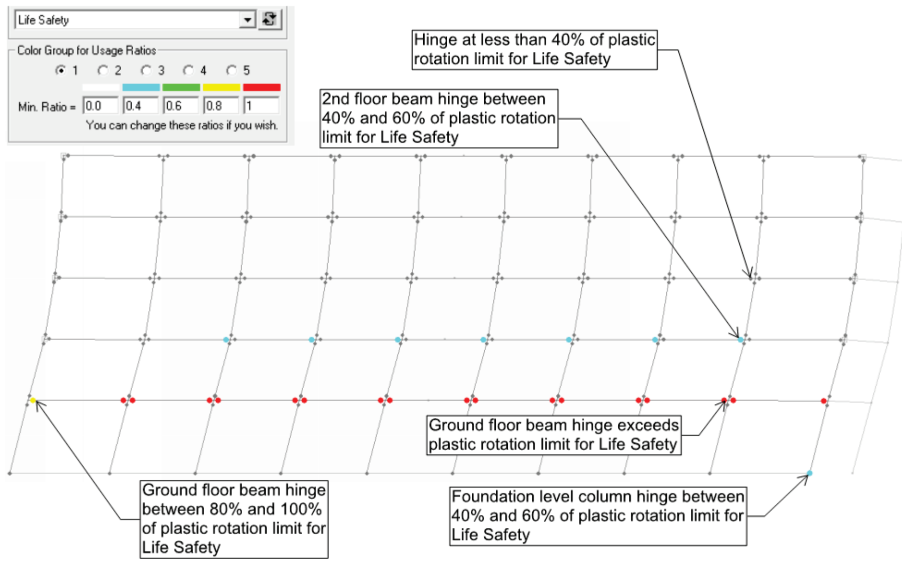


Figure 8-17 Hinges at target displacement of 14.3 inches at roof. Column hinge at ground floor indicates life-safety limits are exceeded.

8.5.1.5 *Applicability of the NSP Procedure*

As outlined in ASCE 41-13 § 7.3.2.1, the NSP is not permitted for structures where significant structural degradation occurs or higher mode effects are significant.

Significant degradation is characterized by the strength ratio, $\mu_{strength}$, exceeding, μ_{max}, as defined by ASCE 41-13 Equations 7-31 and 7-32.

$\mu_{strength} = S_a C_m/(V/W) = 1.29$ (previously determined)

$$\mu_{max} = \frac{\Delta_d}{\Delta_y} + \frac{|\alpha_e|^{-h}}{4} \qquad \text{(ASCE 41-13 Eq. 7-32)}$$

where:

Δ_d = Lesser of the target displacement, δ_t, or displacement corresponding to the maximum base shear defined in ASCE 41-13 Figure 7-3

Δ_y = Displacement at effective yield strength as defined in ASCE 41-13 Figure 7-3

h = $1 + 0.15\ln(T_e)$

α_e = Effective negative post-yield slope ratio

$$= \alpha_{P\text{-}\Delta} + \lambda(\alpha_2 - \alpha_{P\text{-}\Delta}) \qquad \text{(ASCE 41-13 Eq. 7-33)}$$

α_2 = Negative post-yield slope ratio defined in ASCE 41-13 Figure 7-3. This ratio includes P-Δ effects, in -cycle degradation, and cyclic degradation

$\alpha_{P\text{-}\Delta}$ = Negative slope ratio caused by P-Δ effects;

λ = Near-field effect factor:

= 0.8 if $S_{X1} \geq 0.6$ for BSE-2N

= 0.2 if $S_{X1} \leq 0.6$ for BSE-2N

S_{X1} = 0.898 (S_{X1} is at the BSE-2N level for determination of λ below)

λ = 0.8

Δ_d = 1.53%

Δ_y = 0.87%

$$\frac{\Delta_d}{\Delta_y} = \frac{1.53\%}{0.87\%} = 1.76$$

Since the expression, $\frac{|\alpha_e|^{-h}}{4}$, is always positive, μ_{max} must be greater than 1.76. Therefore,

$\mu_{strength} = 1.28 < \mu_{max}$

The significant structural degradation limit is not exceeded.

To determine if higher modes are significant, a modal response spectrum analysis (MRSA) is performed for the structure using sufficient modes to produce 90% mass participation. A second response spectrum analysis is also performed, considering only the first mode participation. Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response.

If higher mode effects are significant, then the NSP shall be permitted if an LDP analysis is also performed to supplement the NSP. Buildings with significant higher mode effects must meet the acceptance criteria of this standard for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled actions (m-factors) provided in Chapter 8 through Chapter 12 of ASCE 41-13. A building analyzed using the NSP, with or without a supplementary LDP evaluation, shall meet the acceptance criteria for nonlinear procedures specified in ASCE 41-13 § 7.5.3.

A MRSA was performed in the Tier 2 LDP evaluation using 12 modes to capture 90% mass participation. A supplementary MSRA is performed using the first mode only. The story forces are compared in Table 8-26.

Table 8-26 Comparison of Multi-Mode to Single-Mode MRSA Story Forces

Level	Story Shear 12-Mode MRSA	Story Shear 1-Mode MRSA	Ratio
Roof	2,520 k	1,776 k	1.42
4th floor	3,970 k	3,479 k	1.14
3rd Floor	4,917 k	4,803 k	1.02
2nd Floor	5,836 k	5,700 k	1.02
Ground Floor	6,447 k	6,072 k	1.06

At the roof, the ratio exceeds 130%, thus higher mode effects are considered significant and the NSP alone is not permitted. The NSP procedure is permitted when supplemented with an LDP. An LDP was performed in Section 8.4.2 of this *Guide* and it was found that beam to column connections were the only deficiency. The demand capacity ratios for connections were reported in Table 8-25 for the determined acceptance criteria. When the LDP is evaluated as a supplement to the NSP, the acceptance criteria are permitted to be increased by a factor of 1.33 for deformation-controlled actions. Table 8-27 shows the connection DCRs with the 1.33 adjustment factor. The NSP acceptance criteria remains unchanged regardless of whether or not a supplementary LDP is performed. When an LDP is required to supplement an NSP, the acceptance criteria are required to be evaluated for both procedures. If the acceptance criteria are exceeded for either procedure, the component is considered deficient.

Higher mode effects are significant and an NSP analysis alone is not permitted. The results of an LDP analysis with increased acceptance criteria is used to supplement the results of the NSP. The beam hinges that did not pass acceptance criteria in the linear procedures do meet the relaxed acceptance criteria used to supplement the NSP. However, the ground floor beam hinges do not meet the acceptance criteria for the NSP therefore they remain deficient regardless of the results of the supplemental LDP.

Table 8-27 Tier 2 – LDP Moment Frame Connections Adjusted

	Beam W-Shape	Column W-Shape	m_{LSb}	Q_{UD} (kip-ft)	MCE (Base Material Yield) (kip-ft)	$1.33(\kappa m M_{CE})$ (k-ft)		$1.33(\kappa m Q_{CE}) > Q_{UD}$?
Ground Floor	Ext	W24×176	W24×162	1.41	3,221	1,827	3,423	Y
	Int	W24×176	BW24×230	1.41	3,028	1,827	3,423	Y
2nd Floor	Ext	W24×131	W24×131	1.43	2,023	1,391	2,641	Y
	Int	W24×131	W24×131	1.45	2,276	1,391	2,641	Y
3rd Floor	Ext	W24×117	W24×131	1.43	2,170	1,229	2,343	Y
	Int	W24×117	W24×131	1.46	2,060	1,229	2,343	Y
4th Floor	Ext	W24×84	W24×94	1.44	1,350	842	1,610	Y
	Int	W24×84	W24×94	1.47	1,340	842	1,610	Y
Roof	Ext	W24×68	W24×94	1.81	774	665	1,602	Y
	Int	W24×68	W24×94	1.49	737	665	1,602	Y

8.5.1.6 *Deformation Compatibility of Gravity Framing*

Tier 3 analysis requires that the entire structure be evaluated for adequacy, including elements not designated as part of the SFRS. These secondary components contribute to the structure's overall initial stiffness but for the purpose of this NSP example they were not included in the overall building model. This exclusion is conservative for the purpose of evaluating the SFRS because the target displacement derived excludes the initial stiffness of the gravity system. A 3D analysis model with all SFRS and gravity frames may be utilized in a Tier 3 analysis. For this example, a separate deformation compatibility analysis is performed on the gravity framing to assess if the Life Safety Performance Level criteria are met. The critical elements to be evaluated are shear tab connections as failure of these elements would compromise the gravity carrying capacity of the floor.

The deformation compatibility analysis consists of first calculating the modeling parameters and acceptance criteria for all gravity frame connections. Once the parameters have been computed, the connections are evaluated by assuming all interstory deformation occurs in the connections. That is, the columns and beams are assumed perfectly rigid and the connection rotation demand is a function of the interstory drift only, which is derived from the NSP results. This analysis is approximate and is generally conservative, since some of the deformation would be expected to occur in the frame members.

The inelastic behavior of the gravity beam connections to the columns are modeled as partially restrained moment frames per ASCE 41-13 § 9.4.3 and FEMA 355D, *State of the Art Report on Connection Performance* (FEMA, 2000e). As shown in FEMA 355D Figure 6-2 and Figure 6-5 (duplicated in Figure 8-19 and Figure 8-20), the moment-rotation response of shear tab connections varies depending on the direction of the applied moment. For negative moment actions, the slab is assumed to be ineffective in tension, so a force couple develops between the top and bottom bolts in the shear tab. For positive moment actions, a force couple develops between the shear tab bolts and compression in the deck slab.

The following example illustrates calculation of modeling parameters and acceptance criteria for a typical W18×40 gravity beam to W14×68 gravity column connection, as shown in Figure 8-18 and Figure 8-19.

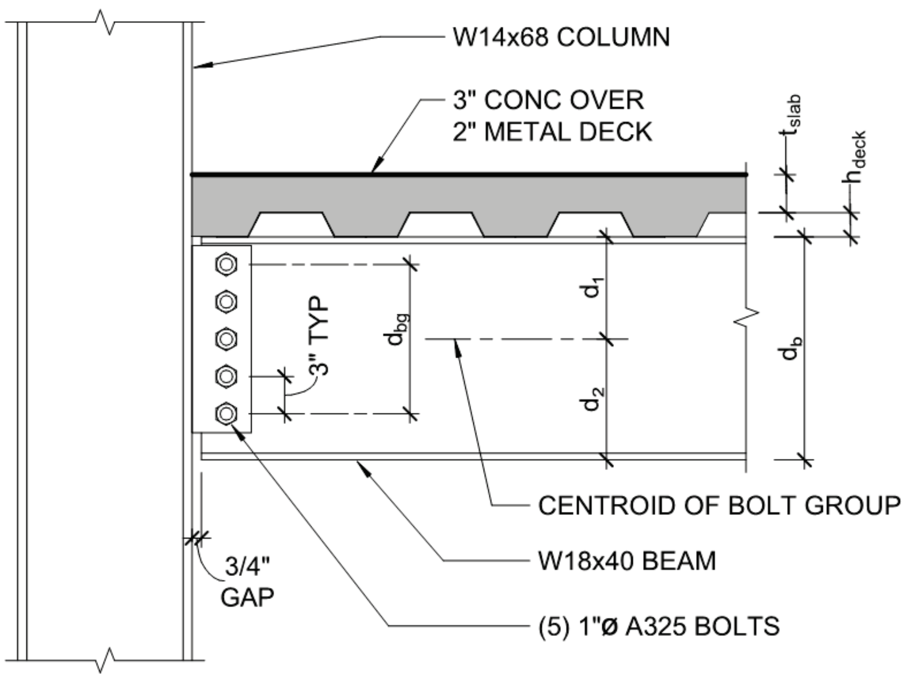


Figure 8-18 Gravity beam connection detail.

d_b = 17.9 in. (beam depth)

d_{bg} = 12 in. (depth of bolt group)

d_1 = 8.5 in. (distance from bolt centroid to top of beam)

d_2 = 9.2 in. (distance from bolt centroid to bottom of beam)

d_{max} $= \max(d_1, d_2)$

$= \max(9.2 \text{ in.}, 8.5 \text{ in.})$

$= 9.2 \text{ in.}$

t_{slab} $= 3$ in. (thickness of concrete topping)

h_{deck} $= 2$ in. (height of metal deck)

F_{ube} $= 1.1(90 \text{ ksi}) = 99 \text{ ksi}$ (Expected bolt tensile strength for A325)

F_{nve} $= 0.563\, F_{ube}$ (per AISC 360-10 Table J3.2)

$= 0.563(99 \text{ ksi})$

$= 55.7 \text{ ksi}$

A_b $= 0.785 \text{ in.}^2$ (for 1" diameter bolt)

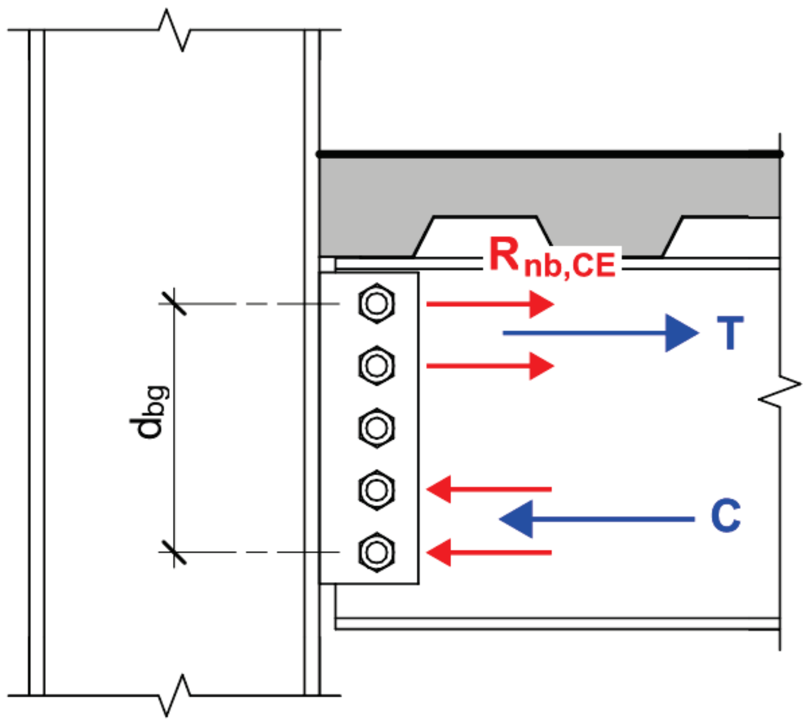


Figure 8-19 Negative moment at gravity beam connection.

The bolted connection and connecting elements were evaluated using AISC 360-10 Sections J3 and J4, and bolt shear strength was determined to be the governing mechanism.

$R_{nb,CE}$ = Expected shear strength of bolt

$= F_{nve} A_b = (55.7 \text{ ksi})(0.785 \text{ in.}^2) = 43.8$ kips/bolt

s_b $= 3$ in. (bolt spacing)

N_b $= 5$ bolts (number of bolts)

d_{bg} $= (N_b - 1)s_b = (5\text{-}1)(3 \text{ in.}) = 12 \text{ in.}$

T $= C = [(N_b\text{-}1)/2)R_{nb,CE}$
$= (4/2)(43.8 \text{ kips})$
$= 87.6 \text{ kips}$

$$M_{CE} = T\left[d_{bg} - \left(\frac{(N_b - 1)s_b}{4}\right)\right]$$
$$= (87.6 \text{ k})\left[12 \text{ in.} - \left(\frac{(5-1)(3 \text{ in.})}{4}\right)\right]$$
$= 788 \text{ k-in.} = 65.7 \text{ k-ft}$

K_s $= 28{,}000(d_{bg} - 5.6)$ (FEMA 355D Eq. 5-19)
$= 28{,}000(12 - 5.6)$
$= 179{,}200 \text{ k-in./rad} = 14{,}933 \text{ k-ft/rad}$

θ_y $= M_{CE} / K_s = (788 \text{ k-in.})/(179{,}200 \text{ k-in./rad})$
$= 0.0044 \text{ rad}$

θ_{binding} $= \text{gap}/(d_{\max}) - 0.02$ (FEMA 355D Eq. 6-3)
$= (0.75 \text{ in.})/(9.2 \text{ in.}) - 0.02$
$= 0.0615 \text{ rad}$

$\theta_{b\text{-}pl}$ $= \theta_{\text{binding}} - \theta_y$
$= 0.0615 - 0.0044$
$= 0.0571 \text{ rad}$

Modeling parameters and acceptance criteria are taken from ASCE 41-13 Table 9-6 for shear connection without slab. However, it is noted in FEMA 355D that the gravity connection loses gravity carrying capacity soon after reaching the binding rotation, θ_{binding}; thus, the plastic rotation limits computed using ASCE 41-13 Table 9-6 are limited to the value, $\theta_{b\text{-}pl}$, the plastic rotation that occurs prior to binding.

$$a = \min\begin{bmatrix} \theta_{b-pl} \\ 0.15 - 0.0036 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0571 \text{ rad} \\ 0.1068 \text{ rad} \end{bmatrix} = 0.0571 \text{ rad}$$

θ_a $= \theta_y + a$
$= 0.0571 + 0.0044$
$= 0.0615 \text{ rad}$

$$b = \min\begin{bmatrix} \theta_{b-pl} \\ 0.15 - 0.0036 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0571 \text{ rad} \\ 0.1068 \text{ rad} \end{bmatrix} = 0.0571 \text{ rad}$$

θ_b $= \theta_y + b$
$= 0.0571 + 0.0044$
$= 0.0615 \text{ rad}$

$c \quad = 0.4$

$$\text{IO} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.075 - 0.0018 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0571 \text{ rad} \\ 0.0534 \text{ rad} \end{bmatrix} = 0.0534 \text{ rad}$$

$$\begin{aligned} \theta_{IO} &= \theta_y + \text{IO} \\ &= 0.0534 + 0.0044 \\ &= 0.0578 \text{ rad} \end{aligned}$$

$$\text{LS} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.1125 - 0.0027 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0571 \text{ rad} \\ 0.0801 \text{ rad} \end{bmatrix} = 0.0571 \text{ rad}$$

$$\begin{aligned} \theta_{LS} &= \theta_y + \text{LS} \\ &= 0.0571 + 0.0044 \\ &= 0.0615 \text{ rad} \end{aligned}$$

$$\text{CP} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.15 - 0.0036 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0571 \text{ rad} \\ 0.1068 \text{ rad} \end{bmatrix} = 0.0571 \text{ rad}$$

$$\begin{aligned} \theta_{CP} &= \theta_y + \text{CP} \\ &= 0.0571 + 0.0044 \\ &= 0.0615 \text{ rad} \end{aligned}$$

Positive Moment (ASCE 41-13 Table 9-6, FEMA 355D)

Figure 8-20 shows the calculation of positive moment at gravity beam connection.

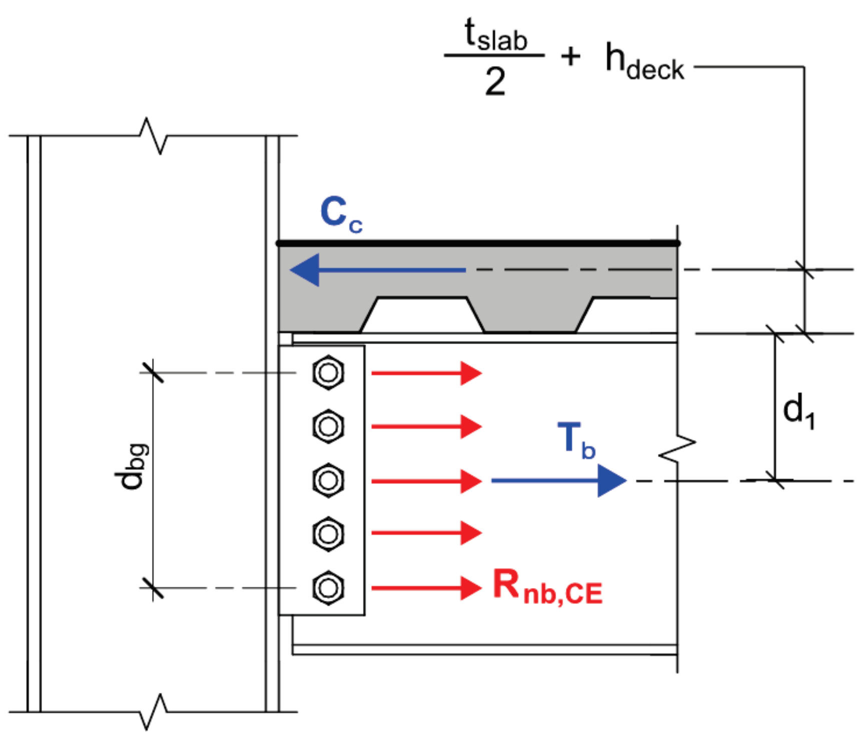


Figure 8-20 Positive moment at gravity beam connection.

f'_{ce} = 4.5 ksi (Expected slab compressive strength)

b_{cf} = 10 in. (W14×68 column flange width)

t_{slab} = 3 in. (Slab thickness)

C_c $= 0.85 f'_{ce} b_{cf} t_{\text{slab}}$ (Slab compression force)

$= 0.85(4.5 \text{ ksi})(10 \text{ in.})(3 \text{ in.})$

$= 115$ kips

T_b $= N_b R_{nb,CE}$ (Tension force resisted by bolt shear)

$= 5(43.8 \text{ kips})$

$= 219$ kips

T $= C = \min\{T_b, C_c\}$

$= \min\{115 \text{ kips} , 219 \text{ kips}\}$

$= 115$ kips

Note that the shear studs connecting the beam top flange to the concrete over metal deck have adequate capacity to resist this force.

M_{CE} $= T(t_{\text{slab}}/2 + h_{\text{deck}} + d_{\text{beam}}/2)$

$= (115 \text{ kips})(3 \text{ in.}/2 + 3 \text{ in.} + 17.9 \text{ in.}/2)$

$= 1{,}773$ k-in. $= 148$ k-ft

k_s $= 28{,}000(d_{bg} - 3.3)$ (FEMA 355D Eq. 6-4)

$= 28{,}000(12 - 3.3)$

$= 243{,}600$ k-in./rad $= 20{,}300$ k-ft/rad

θ_y $= M_{CE}/k_s = (1{,}773 \text{ k-in.})/(243{,}600 \text{ k-in./rad})$

$= 0.00728$ rad

θ_g $= \theta_{\text{binding}} = \text{gap}/(d_{\text{max}}) - 0.02$ (FEMA 355D Eq. 6-3)

$= (0.75 \text{ in.})(8.9 \text{ in.}) - 0.02$

$= 0.0615$ rad

$\theta_{b\text{-}pl}$ $= \theta_{\text{binding}} - \theta_y$

$= 0.0615 - 0.00728$

$= 0.0542$ rad

$$a = \min\begin{bmatrix} \theta_{b-pl} \\ 0.029 - 0.0020 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0542 \text{ rad} \\ 0.0266 \text{ rad} \end{bmatrix} = 0.0266 \text{ rad}$$

θ_a $= \theta_y + a$

$= 0.0073 + 0.0266$

$= 0.0339$ rad

$$b = \min\begin{bmatrix} \theta_{b-pl} \\ 0.15 - 0.0036d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0542 \text{ rad} \\ 0.1068 \text{ rad} \end{bmatrix} = 0.0542 \text{ rad}$$

$$\begin{aligned} \theta_b &= \theta_y + b \\ &= 0.0542 + 0.0073 \\ &= 0.0615 \text{ rad} \end{aligned}$$

$$c = 0.4$$

$$\text{IO} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.014 - 0.00010d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0542 \text{ rad} \\ 0.0128 \text{ rad} \end{bmatrix} = 0.0128 \text{ rad}$$

$$\begin{aligned} \theta_{IO} &= \theta_y + \text{IO} \\ &= 0.0073 + 0.0128 \\ &= 0.0201 \text{ rad} \end{aligned}$$

$$\text{LS} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.1125 - 0.0027d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0542 \text{ rad} \\ 0.0801 \text{ rad} \end{bmatrix} = 0.0542 \text{ rad}$$

$$\begin{aligned} \theta_{LS} &= \theta_y + \text{LS} \\ &= 0.\ 0073 + 0.0542 \\ &= 0.0615 \text{ rad} \end{aligned}$$

$$\text{CP} = \min\begin{bmatrix} \theta_{b-pl} \\ 0.15 - 0.0036d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.0542 \text{ rad} \\ 0.1068 \text{ rad} \end{bmatrix} = 0.0542 \text{ rad}$$

$$\begin{aligned} \theta_{CP} &= \theta_y + \text{CP} \\ &= 0.\ 0073 + 0.0542 \\ &= 0.0615 \text{ rad} \end{aligned}$$

For the above example, the connection rotations do not exceed the Life Safety limit state (shown as the solid line in Figure 8-21) at the target displacement (shown as dashed line in Figure 8-21), where the connection rotation is approximated as the maximum interstory drift that occurs in the building at the target displacement. It is worth noting that the connection is able to deliver about 40% of the beam's plastic moment strength in the positive condition prior to yield and around 15% of the plastic strength in the negative condition and post-yield positive condition.

In order to satisfy the Tier 3 requirement that all primary and secondary components are evaluated, this example utilized an approximate approach to evaluate the secondary gravity framing components without explicitly modeling the framing in a three-dimensional model. This approach minimized the complexity of the mathematical modeling by using a two-dimensional model for the moment frames and was deemed reasonable for this building given the regular plan and symmetry of the building, and the fact that there were no orthogonal loading concerns such as a corner column.

It is important to note that this approach may yield conservative results because the initial stiffness and strength of the gravity frames is not included in the analysis. Alternatively, an additional two-dimensional bay could be included in the 2D model to approximate the initial stiffness of the gravity framing or all gravity framing could be included in a three-dimensional model with plastic hinges modeled at each gravity connection, with modeling parameters and acceptance criteria determined as shown in this example.

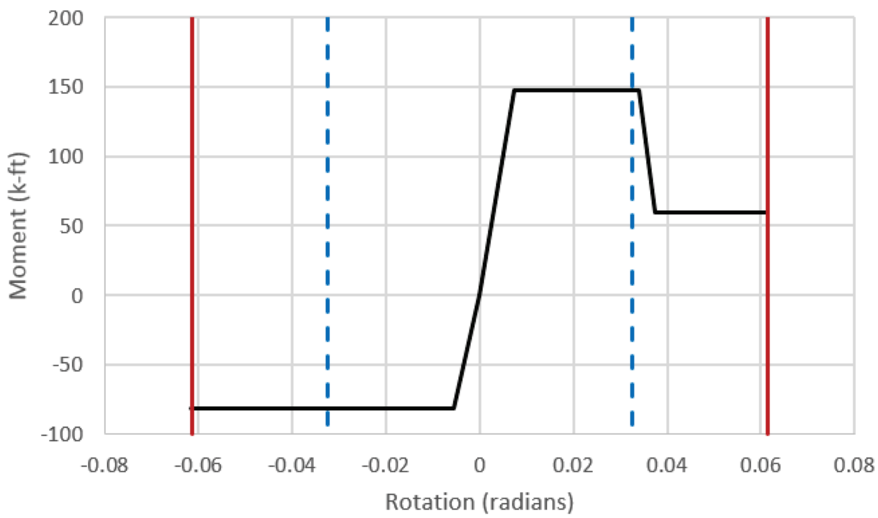


Figure 8-21 Typical gravity connection hinge.

8.5.2 Retrofit (ASCE 41-13 § C9.1)

Techniques for retrofit of deficient steel moment frames are not included in ASCE 41-13. The design professional is referred to SAC joint venture publications FEMA 350, FEMA 351, FEMA 352, and FEMA 353 (FEMA, 2000a,b,c,d), for information on design, evaluation, and repair of damaged or deficient steel moment-resisting frame structures. Retrofit measures for steel moment frame structures include, but are not limited to, the following:

- Retrofit of connections with welded bottom haunch, welded top and bottom haunch or welded cover plated flange (FEMA, 2000b).
- Increase global stiffness of the structure by providing new seismic force-resisting elements such as new steel braces in existing frames or new concrete or masonry shear walls. The new components would be designed using modeling parameters and acceptance criteria from the respective material chapters in ASCE 41-13.
- Add energy dissipation devices (friction or viscous dampers) to reduce deformations, and therefore rotations, of existing connection.

For any retrofit measure, the designer should verify that the modifications do not introduce other deficiencies or irregularities to the structure.

Chapter 9

Steel Braced Frame (S2)

9.1 Overview

This chapter provides discussion and example application of the Tier 1 screening and Tier 3 systematic evaluation and retrofit procedures of ASCE 41-13 (ASCE, 2014) on a 1980s three-story ordinary braced frame building in a region with a moderate Level of Seismicity. The Tier 3 evaluation uses an Enhanced Performance Objective of the Immediate Occupancy Performance Level at the BSE-1N Seismic Hazard Level. The linear static procedure (LSP) is presented initially, followed by the nonlinear static procedure (NSP) for one of the building braced bays to compare and contrast the linear and nonlinear behavior of this type of structure with the ASCE 41-13 modeling and acceptance criteria requirements.

This example is not a complete evaluation of the structure, and focuses on selected structural elements to demonstrate use of ASCE 41-13. Nonstructural evaluation was omitted to keep the example relatively concise, but it is an important part of an overall seismic evaluation especially for buildings requiring a higher seismic performance level. The building geometry and system layout are similar to that of the braced-frame example in *2006 IBC Structural/Seismic Design Manual* (SEAOC, 2006), with detailing adjusted to the 1980s era.

This example illustrates the following:

- **Section 9.2:** Building description
- **Section 9.3:** Gravity loads and seismic weights
- **Section 9.4:** Seismic design parameters
- **Section 9.5:** Deficiencies identified from Tier 1 screening (ASCE 41-13 § 4.5)
- **Section 9.6:** Data collection requirements (ASCE 41-13 § 6.2, § 9.2, and § 10.2)
- **Section 9.7:** Tier 3 evaluation using linear static procedure (LSP)
    - o Determination of demands on components (ASCE 41-13 § 7.4.1 and § 9.5)

Example Summary

Building Type: S2

Performance Objective: Immediate Occupancy at BSE-1N

Risk Category: II

Location: Charlotte, North Carolina

Level of Seismicity: Moderate

Analysis Procedures: Linear Static (LSP), Nonlinear Static (NSP)

Evaluation Procedures: Tier 1 and Tier 3

Reference Documents:

ACI 318-11

AISC 341-10

AISC 360-10

2013 AISC Seismic Design Manual

2011 AISC Construction Manual

  - Determination of element acceptance criteria (ASCE 41-13 § 7.5.2 and § 9.5.2.3)
  - Foundation evaluation (ASCE 41-13 § 8.4.2)
  - Confirmation of applicability of linear static procedure (ASCE 41-13 § 7.3.1.1)
- **Section 9.8:** Tier 3 evaluation using nonlinear static procedure (NSP)
  - Modeling and acceptance criteria of nonlinear steel braces, beams, and columns. (ASCE 41-13 § 9.5.2.2.2 and § 9.5.2.4.3)
  - Modeling and acceptance criteria of partially restrained gravity beam connections (ASCE 41-13 § 9.4.3.2.2)
  - Target displacement (ASCE 41-13 § 7.4.3.3.2)
  - Nonlinear foundation evaluation (ASCE 41-13 § 8.4.2 and § 8.4.2.3.3)

9.2 Building Description

The building is a three-story laboratory building located in Charlotte, North Carolina that is presumed to be built to the 1979 SBC, *Standard Building Code* (SBCCI, 1979). The building is considered a Type S2 building per ASCE 41-13 Table 3-1 because it is a steel concentrically braced frame building with rigid diaphragms. Since this building is being evaluated for BSE-1N Seismic Hazard Level, the Benchmark Buildings in ASCE 41-13 Table 4-6 do not apply. If the building were evaluated for the BSE-1E Seismic Hazard Level, it could be checked to see if it was considered a Benchmark Building per ASCE 41-13 Table 4-6. Buildings designed to the SBC are only Benchmarked for Life Safety per footnote LS, and there are no Benchmarks (LS or IO) for Type S2 buildings designed to SBC per footnote (e); therefore, a seismic evaluation is required. The seismic evaluation and upgrade are being performed voluntarily at the request of the owner to meet the Immediate Occupancy Performance Level at the BSE-1N Seismic Hazard Level. The building is structurally regular. An overall building view, typical floor plan, and typical braced frame elevation are shown in Figure 9-1 through Figure 9-3, respectively.

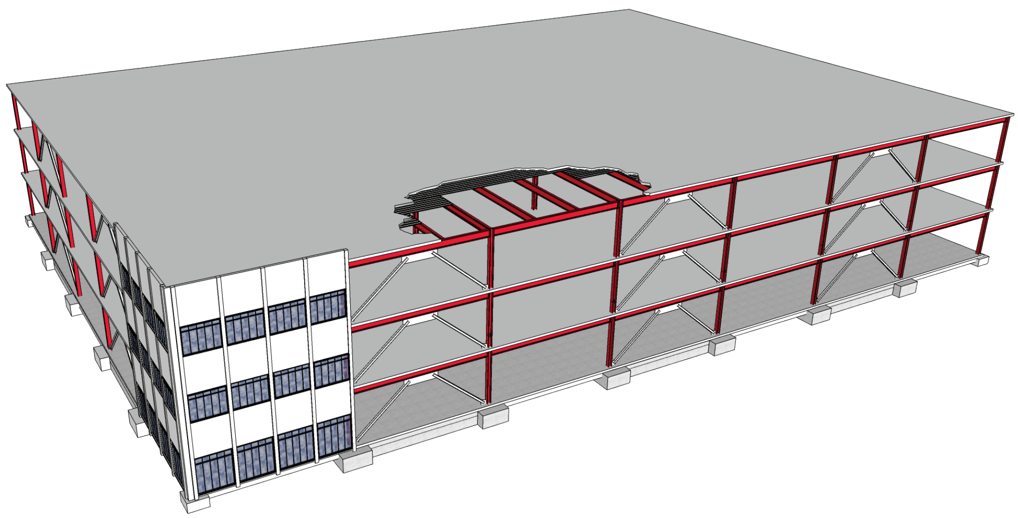


Figure 9-1 Isometric rendering.

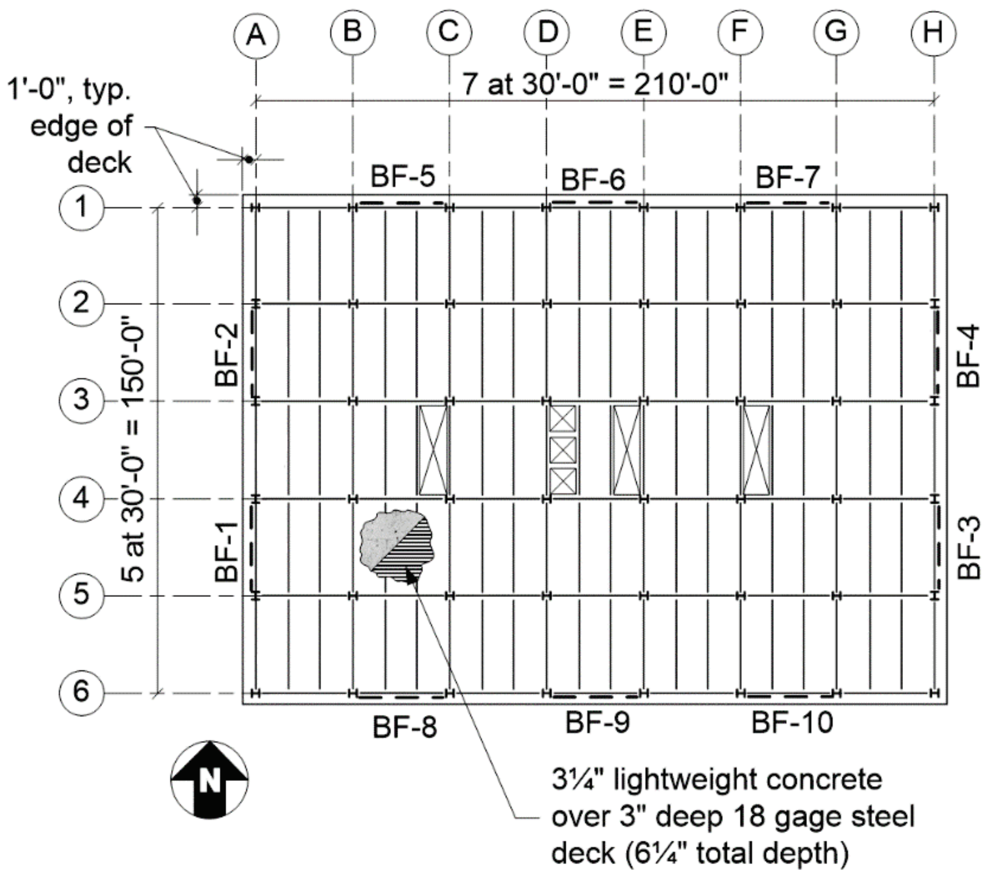


Figure 9-2 Typical floor plan.

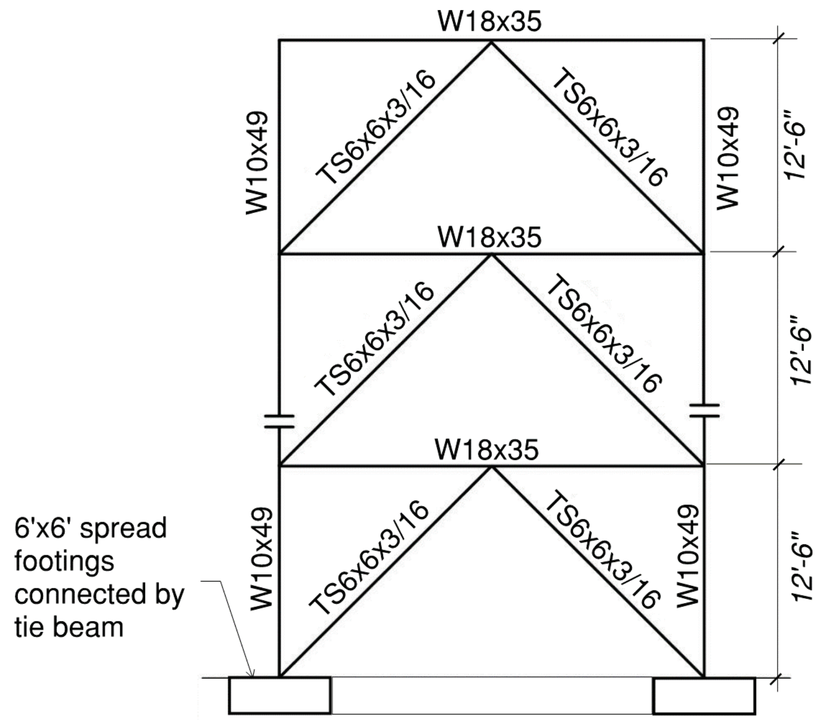


Figure 9-3 Typical elevation of steel braced frames.

9.3 Dead Loads and Seismic Weights

Dead loads and seismic weights are given in Table 9-1 for the roof and Table 9-2 for the typical floor. Live loads are 20 lb/ft^2 for the roof and 50 lb/ft^2 for the typical floor. The exterior wall system is assumed to be 20 lb/ft^2 over the elevation for the seismic weight takeoff.

Table 9-1 Flat Loads on Roof

Component	Dead Load (lb/ft^2)	Seismic Weight (lb/ft^2)
Roofing	6.0	6.0
Insulation	3.0	3.0
Steel deck and concrete fill	47.0	47.0
Steel framing (beams and columns)	8.0	8.0
Ceiling	3.0	3.0
Partitions	0.0	5.0
MEP/miscellaneous components	2.0	2.0
Total	69.0	74.0

Table 9-2 Flat Loads on Floor

Component	Dead Load (lb/ft^2)	Seismic Weight (lb/ft^2)
Floor covering	1.0	1.0
Steel deck and concrete fill	47.0	47.0
Steel framing (beams and columns)	13.0	13.0
Ceiling	3.0	3.0
Partitions	10.0	10.0
MEP/miscellaneous components	2.0	2.0
Total	76.0	76.0

9.4 Seismic Design Parameters

The site class, latitude, and longitude for the building which are needed to determine seismic design parameters are as follows.

- Location: Charlotte, North Carolina
- Latitude: 35.22 N
- Longitude: 80.84 W
- Site Class: D

The following ground motion parameters are obtained for the BSE-1N Seismic Hazard Level using the online tools described in Chapter 3 of this *Guide:*

$S_{XS,\ BSE\text{-}2N} = 0.383$

$S_{X1,\ BSE\text{-}2N} = 0.246$

$S_{XS,\ BSE\text{-}1N} = 0.256$

$S_{X1,\ BSE\text{-}1N} = 0.164$

Determine the Level of Seismicity per ASCE 41-13 Table 2-5:

The design short-period response acceleration is:

$$\begin{aligned} S_{DS} &= 2/3 S_{XS,\ BSE\text{-}2N} \\ &= S_{XS,\ BSE\text{-}1N} \\ &= 0.256 \end{aligned}$$

$0.167 \le S_{DS} < 0.33$

Therefore, Level of Seismicity = "Low" based on S_{DS}

The 1-second period response acceleration is:

$S_{D1} = 2/3 S_{X1,\ BSE\text{-}2N}$

$= S_{X1,\ BSE\text{-}1N}$

$= 0.164$

$0.133 \le S_{D1} < 0.20$

Therefore, Level of Seismicity = "Moderate" based on S_{D1} (governs)

Therefore, the site is in a Moderate Level of Seismicity per ASCE 41-13 or Seismic Design Category C as defined in ASCE 7-10 Section 11.6.

9.5 Deficiencies Identified from Tier 1 Screening (ASCE 41-13 § 4.5)

A Tier 1 screening was conducted with ASCE 41-13 § 16.5 IO checklist and the deficiencies identified were evaluated.

9.5.1 Column Axial Stress

The following is a sample calculation (ASCE 41-13 § 4.5.3.6 Quick Check) for first story column, north-south direction.

$A_{col} = 14.4$ in.^{2} for W10×49

$F_{yc} = 41$ ksi (ASCE 41-13 Table 9-1: ASTM A36 Group 2)

$P_D = [(74 \text{ psf} + 2(76 \text{ psf}))(30 \text{ ft}/2 + 1 \text{ ft}) + 20 \text{ psf}(2)(12.5 \text{ ft})](30 \text{ ft})$

$= 123$ kips

Commentary

The flat roof snow load is less than 30 lb/ft^2. Therefore, per ASCE 41-13 § 7.2.2, the effective snow load is taken to be zero.

Live loads are reduced per ASCE 7-10 (Equation 4.7-1 and Equation 4.8-1) as permitted by ASCE 41-13 § 4.5.2.1 with a tributary area of 480 ft^2.

$P_L = [0.72(20 \text{ psf}) + 2(0.65)(50 \text{ psf})](30 \text{ ft})(30 \text{ ft}/2 + 1 \text{ ft})$

$= 38$ kips

$P_S = 0$ kips

$P_G = P_D + P_L + P_S = 161$ kips

$p_G = P_G / A_{col} = 11.2$ ksi

$p_G /(0.1F_{yc}) = 2.73 > 1.0$

The column's axial stress for gravity loads is not low enough to pass the quick check. Therefore, the alternative method per ASCE 41-13 § 4.5.3.6 is used to check column axial stress due to seismic overturning alone.

$M_s = 1.3$ system modification factor for IO Performance Level

Base shear, V, is calculated as follows:

$V = CS_aW$ (ASCE 41-13 Eq. 4-1)

where:

C = 1.1, per ASCE 41-13 Table 4-8 for 3-story S2 building

$S_a = \min\{S_{X1}/T, S_{XS}\}$, per ASCE 41-13 § 4.5.2.3

$T = C_t h_n^{\beta}$ (ASCE 41-13 Eq. 4-5)

h_n = 3(12.5 ft) = 37.5 ft (Building height)

Per ASCE 41-13 § 4.5.2.4:

C_t = 0.020, for "all other framing systems"

β = 0.75, for "all other framing systems"

$T = 0.020(37.5)^{0.75} = 0.303$ sec

$S_{X1} = S_{X1,\, BSE\text{-}1N} = 0.164\text{g}$

$S_{XS} = S_{XS,\, BSE\text{-}1N} = 0.256\text{g}$

$S_a = \min\{0.164/0.303, 0.256\}$

$= \min\{0.541, 0.256\}$

$= 0.256\text{g}$

Story seismic weight = Flat weight + Exterior wall

Roof weight = 74psf (32,224sf) + 20psf (720 linear feet * 6.25 tributary height) = 2,475 k

Floor weight = 76psf (32,224sf) + 20psf (720 linear feet * 12.5 tributary height) = 2,629 k

W = 2,475 kips + 2(2,629 kips) = 7,733 kips

V = 1.1(0.256g)(7,733 kips) = 2,178 kips

Axial stress of columns at the base subjected to overturning, p_{ot}, is calculated as follows:

$$p_{ot} = \frac{1}{M_S}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{n_f L}\right)\left(\frac{1}{A_{\text{col}}}\right) \quad \text{(ASCE 41-13 Eq. 4-12)}$$

where:

h_n = 3(12.5 ft) = 37.5 ft (Building height)

n_f = 4 frames (Number of braced frames, N-S direction)

L = 30 ft (Typical braced frame bay width)

$$p_{ot} = \frac{1}{1.3}\left(\frac{2}{3}\right)\left(\frac{2{,}178 \text{ kips}(37.5 \text{ ft})}{4 \text{ frames}(30 \text{ ft})}\right)\left(\frac{1}{14.4 \text{ in.}^2}\right)$$

$= 24.2$ ksi

This is compared against the $0.30F_y$ threshold established in the Quick Check procedure of ASCE 41-13 § 4.5.3.6.

$p_{ot} /(0.3F_{yc}) = 1.97 > 1.0$

Therefore, the column axial stress does not satisfy the ASCE 41-13 Tier 1 Quick Check.

The gravity and seismic axial stresses in the columns at the lower levels exceed the $0.10F_y$ and $0.30F_y$ thresholds, respectively, established in the Quick Check procedure of ASCE 41-13 § 4.5.3.6 and Equation 4-12. Table 9-3 and Table 9-4 provide a summary of the Quick Check results for column axial stress.

Table 9-3 Column Stress Check (East-West Direction)

Story	Column Section	P_G (kips)	P_{ot} (kips)	$\frac{p_G}{0.1F_{yc}}$	$\frac{p_{ot}}{0.3F_{yc}}$	Quick Check
3rd	W10×49	42	56	0.72	0.32	OK
2nd	W10×49	102	153	1.73	0.86	OK
1st	W10×49	161	233	2.73	1.31	NG

Table 9-4 Column Stress Check (North-South Direction)

Story	Column Section	P_G (kips)	P_{ot} (kips)	$\frac{p_G}{0.1F_{yc}}$	$\frac{p_{ot}}{0.3F_{yc}}$	Quick Check
3rd	W10×49	42	85	0.72	0.48	OK
2nd	W10×49	102	229	1.73	1.29	NG
1st	W10×49	161	349	2.73	1.97	NG

9.5.2 *Brace Axial Stress*

The following is a sample calculation (ASCE 41-13 § 4.5.3.4 Quick Check) for first story brace, north-south direction.

V_j = V = 2,178 kips, per previous calculations

L_{br} $= \sqrt{(30\text{ ft}/2)^2 + (12.5\text{ ft})^2} = 19.53\text{ ft}$

s = $L_{br,x}$ = 30 ft/2 = 15 ft

N_{br} = 8 braces (4 tension braces, 4 compression braces)

A_{br} = 3.98 in.^{2} for TS6×6×3/16

d/t = b/t = 31.5 for TS6×6×3/16

F_{ybr} = 46 ksi for ASTM A500 Grade B

F_{yebr} = $1.25F_{ybr}$ = 57.5 ksi (ASCE 41-13 Table 4-10)

$$\frac{190}{\sqrt{F_{yebr}}} = 25.1$$

$d/t \quad > 190\sqrt{F_{yebr}}$

$M_S \quad = 1.50$ (ASCE 41-13 Table 4-10)

Determine f_j^{avg}, average flexural stress in the diagonal bracing elements in accordance with ASCE 41-13 Equation 4-10:

$$f_j^{\text{avg}} = \frac{1}{M_S}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) = \frac{1}{1.5}\left(\frac{2{,}178 \text{ kips}}{15 \text{ ft}(8)}\right)\left(\frac{19.53 \text{ ft}}{3.98 \text{ in}^2}\right)$$

$$= 59.4 \text{ ksi}$$

This is compared against the $0.50F_y$ threshold established in the Quick Check procedure of ASCE 41-13 § 4.5.3.4.

$$\frac{f_j^{\text{avg}}}{0.5F_{ybr}} = 2.58 > 1.0$$

Therefore, the brace axial stress does not satisfy the ASCE 41-13 Tier 1 Quick Check.

In the east-west direction, brace stresses at the lower two stories are above the $0.50F_y$ threshold established in the Quick Check procedure of ASCE 41-13 § 4.5.3.4. In the north-south direction, brace stresses at all three stories exceed the $0.50F_y$ threshold. Table 9-5 and Table 9-6 provide a summary.

> **Commentary**
>
> As shown in the Tier 3 LSP calculations later in this chapter, the brace connections are able to develop the expected compressive capacity of the braces but not their expected tensile capacity.

Table 9-5 Brace Stress Check (East-West Direction)

Story	Brace Section	f_j^{avg} (ksi)	$\frac{f_j^{avg}}{0.5F_{ybr}}$	Quick Check
3rd	TS6×6×3/16	19.2	0.83	OK
2nd	TS6×6×3/16	32.8	*1.42*	NG
1st	TS6×6×3/16	39.6	*1.72*	NG

Table 9-6 Brace Stress Check (North-South Direction)

Story	Brace Section	f_j^{avg} (ksi)	$\frac{f_j^{avg}}{0.5F_{ybr}}$	Quick Check
3rd	TS6×6×3/16	28.8	*1.25*	NG
2nd	TS6×6×3/16	49.2	*2.14*	NG
1st	TS6×6×3/16	59.4	*2.58*	NG

9.5.3 Other Deficiencies

Completion of the Tier 1 checklist identified the following additional deficiencies:

- **Redundancy:** The number of braced bays in each line of braced frames is not greater than three in either direction.
- **Column Splices:** Braced frame column splices are not detailed to develop 100% of the column tensile strength.
- **Out-of-Plane Bracing:** In the north-south direction, the bottom flange of the chevron braced frame beams are not braced out-of-plane.
- **Compact Members:** Braces are not "highly ductile" in accordance with AISC 341-10 Table D1.1 (AISC, 2010a).

$$b/t = 31.5$$

$$\lambda_{hd} = 0.55\sqrt{E / F_{yLBbr}} = 0.55\sqrt{29{,}000 \text{ ksi} / 46 \text{ ksi}} = 13.8$$

$$b/t > \lambda_{hd}$$

  Therefore, the braces are not "highly ductile" per AISC 341 and do not satisfy the ASCE 41-13 Tier 1 Quick Check.

- **Chevron Bracing:** The braced frame beams are incapable of resisting the vertical load resulting from simultaneous yielding and buckling of the brace pairs.

Figure 9-4 illustrates the locations of the Tier 1 deficiencies on the 3D view of the example building.

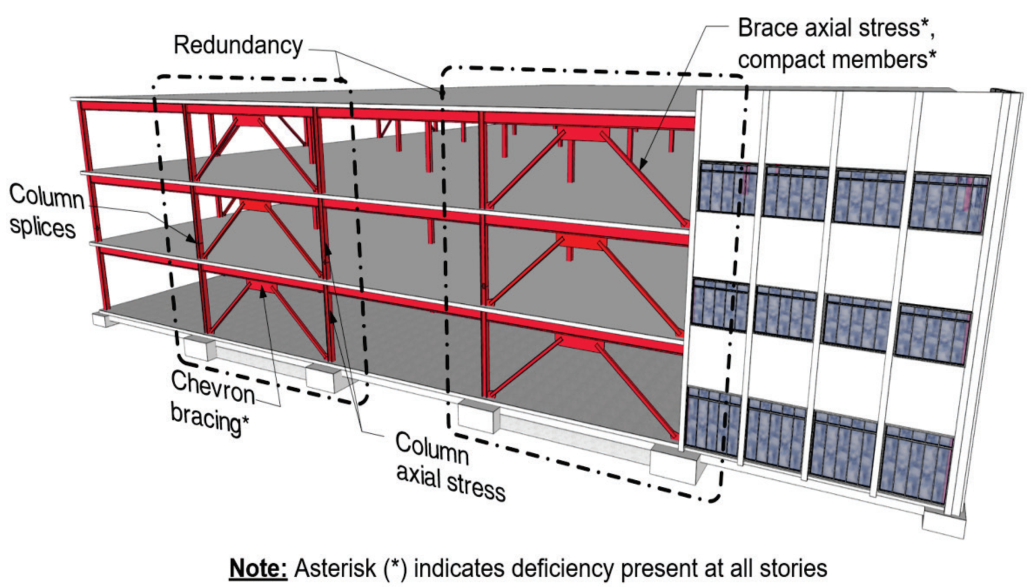


Figure 9-4 Tier 1 screening deficiencies identified (N-S direction).

9.6 Data Collection Requirements (ASCE 41-13 § 6.2, § 9.2, § 10.2)

This section summarizes the data collection requirements for a comprehensive data collection program. Per ASCE 41-13 Table 6-1, Performance Levels greater than Life Safety require a comprehensive data collection program along with a knowledge factor $\kappa = 1.0$. The requirements are listed in ASCE 41-13 § 6.2, 9.2, and 10.2. The three primary considerations are what documentation is available, what testing is required, and how much condition assessment is required.

9.6.1 Available Documentation

This project has complete design drawings, but no test data or specifications. Although there are no specifications available, the design drawings have all of the necessary material properties listed.

9.6.2 Required Testing

Comprehensive materials testing is required since there are no test records per ASCE 41-13 § 6.2.3 Item 3. Below is a list of the minimum number of samples required for each component that is part of the lateral system:

> **Commentary**
>
> Commonly, engineers will use assumed or default properties during the evaluation with confirmation prior to final evaluation of start of retrofit construction documents. This is discussed in ASCE 41-13 § 6.2.4.4.

- **Steel Shapes:** Per ASCE 41-13 § 9.2.2.4 and § 9.2.2.4.2, if documents defining material properties are available but test records are not, then 3 coupons per grade are needed. The results of these coupon tests should be higher than the default values in Table 9-1; otherwise, the number of coupons should be doubled.
  - Three coupons tested for braces (ASTM A500 Grade B)
  - Three coupons tested for beams (ASTM A36 Group 1)
  - Three coupons tested for columns (ASTM A36 Group 2)
- **Welds:** Per ASCE 41-13 § 9.2.2.4.2, if there are no records indicating the filler metal or processes used, then at least one weld per component type should be tested to determine if the weld or base metal limits the connection capacity. Because there are limited standard test methods for existing welds, the approach to confirming the weld quality and strength will likely vary project by project. AISC 360-10 (AISC, 2010b) Appendix 5 discusses using a combination of mechanical and chemical testing depending on the importance of the existing weld. Performing mechanical testing or using a non-destructive test method to determine hardness are two possible approaches to estimating the tensile strength of existing welds. ASTM E10 (ASTM, 2017) provides the standard test method for Brinell Hardness. Chemical properties of the base metals

may be required to determine weldability with or without preheat. Chemical testing for presence of aluminum may also be important for existing demand critical welds to determine if they were performed with low toughness fillers like T4 filler metals common in pre-Northridge moment frame connections. ASTM A751 (ASTM, 2008) provides a standard test method for chemical analysis of steel products.

  - The tensile strength of one existing weld was determined through nondestructive testing following ASTM E10 (ASTM, 2017). The results used in this example were considered the lower-bound value. Non-destructive only testing approaches or more mechanical samples may be required based on the testing program approved by the Authority Having Jurisdiction (AHJ). Three brace connections and columns splices were also visually inspected to confirm the existing fillet weld size and consistency. While this example only has fillet welds, connections with groove welds could be inspected with methods such as ultrasonic testing to confirm the quality and consistency of the existing welds.

- **Concrete:** Per ASCE 41-13 § 10.2.2.4.2.2 the minimum number of cores is based on 3 cores per 400 cubic yards or 10,000 square feet of surface area. This would require 30 concrete cores since the building is just under 100,000 square feet. Since concrete strength is unlikely to be an issue and such a large number of tests appears excessive and unnecessary, a conversation with the AHJ should be able to reduce the number of samples to a level appropriate for the project.

- **Rebar:** Per ASCE 41-13 § 10.2.2.4.2.3 Three slab rebar samples are required.

Commentary

There is no distinction of concrete testing for solid concrete slabs or concrete over metal deck slabs. For this building, proposing an approach to the AHJ to not do testing to the concrete slab while taking a κ of 0.75 for concrete may be considered reasonable since the actual strength of the slab will have limited effects on the evaluation.

9.6.3 Required Condition Assessment

- Site visit to observe general existing condition per ASCE 41-13 § 3.2 and § 9.2.3.1:
  - Observe physical condition of all primary components for presence of degradation, alteration, or damage
  - Confirm configuration and detailing matches the design drawings
- Visual condition assessment per ASCE 41-13 § 9.2.3.2.1. Expose the following connections to confirm detailing matched design drawings:
  - One base plate connection to foundation with brace gusset connection
  - One chevron connection (all brace connections are the same)

ASCE 41-17 Revision

ASCE 41-17 Chapter 6 has been updated to clarify when comprehensive condition assessment is required. This example only requires visual condition assessment since the design drawings are available.

- One frame column splice connection
- One beam-column connection with brace gusset connection (all brace connections are the same)
- One collector connection for each unique detail
- One frame foundation geometry and ground penetrating radar (GPR) scan of reinforcing for consistency with design drawings

Default values and testing results are shown below in Table 9-7, with the test values used for the remaining example material properties.

Table 9-7 Material Properties

Component (ASTM)	Property	Based on Specified Properties: Lower-Bound (Specified)	Based on Specified Properties: $\frac{\text{Expected}}{\text{Lower-Bound}}$	Based on Specified Properties: Expected	Based on Test Results: Lower-Bound $(\mu - \sigma)$	Based on Test Results: Expected (μ)
Braces (A500 Grade B)	F_y	46	1.10	50.6	55.90	57.00
	F_u	58	1.10	63.8	63.44	65.27
Beams, Plates (A36 Group 1)	F_y	44	1.10	48.4	52.21	54.95
	F_u	62	1.10	68.2	72.11	73.26
Columns (A36 Group 2)	F_y	41	1.10	45.1	51.16	53.10
	F_u	59	1.10	64.9	71.53	73.80
Welds	F_{EXX}	70	NA	NA	71.8	NA
Bolts (A325)	F_u	120	1.10	132	120.1	120.2
Rebar (A615)	f_y	60	1.25	75	68.65	70.50
Concrete	f'_c	4	1.50	6	5.185	5.940

9.7 Tier 3 Evaluation using Linear Static Procedure (LSP)

In this section, a Tier 3 evaluation using the linear static procedure (LSP) is conducted for seismic analysis of the building. The 3-dimensional ETABS analysis model used in this analysis is shown in Figure 9-5. Pseudo seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements will be determined using linearly elastic, static analysis in accordance with ASCE 41-13 § 7.4.1.3 and compared against the acceptance criteria of ASCE 41-13 § 7.5.2.

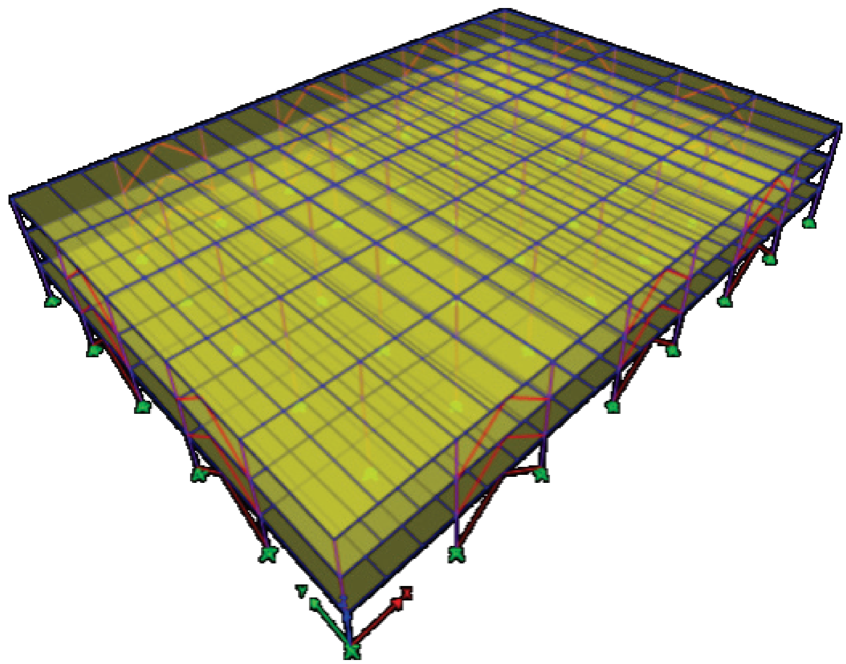


Figure 9-5 Three-dimensional analysis model for linear analysis.

9.7.1 *Summary of Linear Static Procedure (LSP) Forces (ASCE 41-13 § 7.4.1)*

In the following section, linear static procedure base shears and story forces are calculated per ASCE 41-13 § 7.4.1 below.

9.7.1.1 East-West (X) Direction

$V = C_1 C_2 C_m S_a W$ (ASCE 41-13 Eq. 7-21)

where:

T = 0.646 s, per analysis model

$T_s = S_{X1}/S_{XS}$ = 0.164g/0.256g = 0.64 s (ASCE 41-13 Eq. 2-9)

$T \geq T_S$

B_1 = 1.0, for 5% effective viscous damping per ASCE 41-13 § 2.4.1.7

$S_a = S_{X1}/(B_1 T)$ = 0.254g (ASCE 41-13 Eq. 2-7)

k = 1.07, interpolated for 0.50s < T < 2.50s per ASCE 41-13 § 7.4.1.3.2

Per ASCE 41-13 Table 9-4, the largest permissible *m*-factor is at the IO performance level is:

- 2.0 for steel beam flexure
- 2.0 for column flexure

- 1.25 for steel brace tension and compression
- 1.25 for topped metal deck diaphragm shear yielding
- 1.25 for diaphragm chords and collectors

Therefore, m_{max}, the maximum m-factor for all primary elements of the building, is taken equal to 2.0.

Since $2 \le m_{max} < 6$,

$C_1C_2 = 1.1$ for $0.3 < T \le 1.0$ (ASCE 41-13 Table 7-3)

$C_m = 0.9$ for 3-story steel CBF (ASCE 41-13 Table 7-4)

$V = C_1C_2C_mS_aW$ (ASCE 41-13 Eq. 7-21)
$= (1.1)(0.9)(0.254\text{g})(7{,}733 \text{ kips})$
$= 1{,}945$ kips

Story forces are accordingly calculated per ASCE 41-13 Equation 7-24 and Equation 7-25 and summarized in the following table.

Table 9-8 Story Forces and Shears (X-Direction)

Story	h_x (ft)	w_x (kips)	F_x (kips)	V_x (kips)
3rd	37.5	2,475	965	965
2nd	25.0	2,629	664	1,629
1st	12.5	2,629	315	1,945

9.7.1.2 North-South (Y) Direction

$T = 0.802$ s, per analysis model

$T \ge T_S = 0.64$ s

$B_1 = 1.0$, for 5% effective viscous damping per ASCE 41-13 § 2.4.1.7

$S_a = S_{X1}/(B_1T) = 0.204$ g (ASCE 41-13 Eq. 2-7)

$k = 1.15$, per ASCE 41-13 § 7.4.1.3.2

$C_1C_2 = 1.1$ for $0.3 < T \le 1.0$ (ASCE 41-13 Table 7-3)

$C_m = 0.9$ (ASCE 41-13 Table 7-4)

$V = C_1C_2C_mS_aW$ (ASCE 41-13 Eq. 7-21)
$= (1.1)(0.9)(0.204)(7{,}733 \text{ kips})$
$= 1{,}562$ kips

In the following sections, components from a typical north-south frame are evaluated using the LSP. In general, detailed calculations are provided for a specific component in the braced frame, followed by a summary of results for the remaining components throughout the height of the structure. Calculations for the specific component are provided in Section 9.7.X.1 of this *Guide*, where "X" is identified accordingly in Figure 9-6.

Table 9-9 Story Forces and Shears (Y-Direction)

Story	h_x (ft)	w_x (kips)	F_x (kips)	V_x (kips)
3rd	37.5	2,475	794	794
2nd	25.0	2,629	529	1,323
1st	12.5	2,629	238	1,562

9.7.2 Brace Compression Capacity (ASCE 41-13 § 9.5.2.3.2)

In the following section, brace compression acceptance criteria are analyzed as deformation-controlled action in accordance with ASCE 41-13 § 9.5.2.3.2. A detailed calculation for a typical first-story north-south brace is provided along with a summary table of acceptance criteria for north-south braces at all stories.

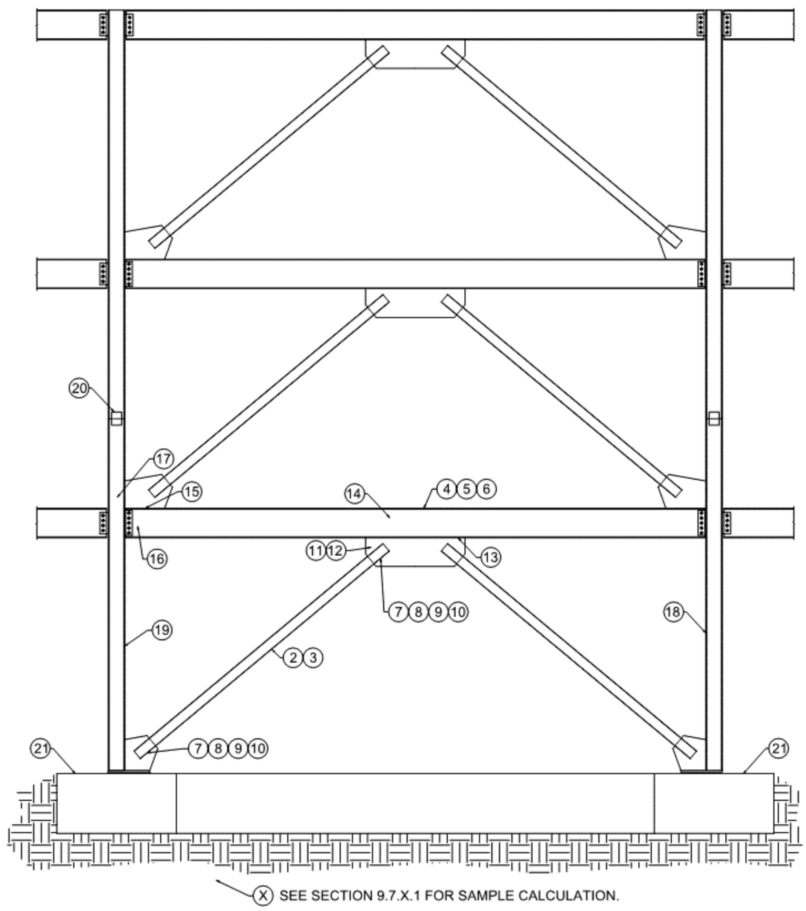


Figure 9-6 LSP sample calculation key.

9.7.2.1 First Story Brace, North-South (Y) Direction

Brace Axial Demands per Analysis Model

P_D = Dead load
= 10.2 kips

P_L = Live load
= 3.7 kips

P_E = Earthquake load
= 268.4 kips

Second-Order Effects (AISC 360-10 Appendix 8.2.2)

B_2 = Multiplier for each story and each direction of lateral translation

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{\text{story}}}{P_{e,\text{story}}}} \qquad \text{(AISC 360-10 Eq. A-8-6)}$$

where:

α = LRFD force-level adjustment factor = 1.00

P_{story} = Total vertical load supported by the story
= $1.1(P_{D,\text{story}} + 0.25P_{L,\text{story}})$ = 9,445 kips (per analysis model)

$P_{e\ \text{story}}$ = Elastic critical buckling strength for the story

$$= R_M \left(\frac{HL}{\Delta_H}\right) \qquad \text{(AISC 360-10 Eq. A-8-7)}$$

where:

R_M = 1.0 (AISC 360-10 Eq. A-8-8: Braced frame)

H = Story shear
= 1,562 kips (per analysis model and slightly larger than hand calculation shown in table 9-9)

Δ_H = First-order interstory drift
= 0.72 in

L = Story height
= 12.5 ft

$$P_{e\ \text{story}} = 1.0\left(\frac{1{,}562\text{ kips}(12.5\text{ ft})(12\text{ in/ft})}{0.72\text{ in}}\right)$$

= 325,417 kips

$$B_2 = \frac{1}{1-\frac{1.00(9,445 \text{ kips})}{325,417 \text{ kips}}}$$

$= 1.03$

Factored Demands (ASCE 41-13 § 7.2.2 and § 7.5.2.1.1)

P_{UD} = Deformation-controlled action caused by gravity loads and earthquake forces, factored load

$= P_G + B_2P_E$

where:

$P_G = 1.1(P_D + 0.25P_L)$

$= 1.1(10.2 \text{ kips} + 0.25(3.7 \text{ kips}))$

$= 12.2 \text{ kips}$

$P_{UD} = P_G + B_2P_E = 12.2 + 1.03(268.4) = 289 \text{ kips}$

Local Buckling (AISC 360-10 Table B4.1a)

For a TS6×6×$^3/_{16}$ section

$b/t = 31.5$

λ_r = Limiting width-to-thickness ratio

$= 1.40\sqrt{E / F_{ye,br}} = 1.40\sqrt{29{,}000 \text{ ksi} / 57 \text{ ksi}} = 31.6$

$b/t < \lambda_r$, therefore, braces are "nonslender" for compression

Flexural Buckling Capacity (AISC 360-10 § E3)

$P_{CE} = \phi F_{cre} A_g$ (AISC 360-10 Eq. E3-1)

The critical stress, F_{cre}, is determined as follows:

$K = 1.0$ (AISC 360-10 Table C-A-7.1: pinned-pinned)

$L = 16.27$ ft, brace end to brace end

$r = 2.37$ in, for TS6×6×$^3/_{16}$

$KL/r = 82.4 \le 4.71\sqrt{E / F_{ye,br}} = 106$ (Slenderness ratio)

Therefore, AISC 360-10 Equation E3-2 governs.

$F_{cre} = \left[0.658^{F_{ye,br}/F_e}\right] F_{ye,br}$ (AISC 360-10 Eq. E3-2)

where:

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29{,}000)}{(82.4)^2} = 42.2 \text{ ksi (AISC 360-10 Eq. E3-4)}$$

$$F_{cre} = \left[0.658^{57/42.2}\right](57)$$

$$= 32.4 \text{ ksi}$$

A_g = 3.98 in.^{2} (TS6×6×$^3/_{16}$)

ϕ = 1.0 (ASCE 41-13 § 9.5.2.3.2)

$P_{CE} = \phi F_{cre} A_g$ = 129 kips (AISC 360-10 Eq. E3-1)

Component Modification Factor (ASCE 41-13 Table 9-4)

The component modification factor for braces in compression is a function of the brace slenderness ratio:

$KL/r = 82.4$

$\rho_a = 4.2\sqrt{E/F_{ye,br}} = 94.7$ (Slender brace limit)

$\rho_b = 2.1\sqrt{E/F_{ye,br}} = 47.4$ (Stocky brace limit)

$\rho_a > KL/r > \rho_b$

Per ASCE 41-13 Table 9-4 footnote (m), which applies to slender and stocky braces, the tabulated *m*-factors (m_a and m_b, respectively) must be multiplied by 0.8, since it is assumed that the brace connections do not satisfy the requirements of AISC 341-10 Section F2.6.

Per ASCE 41-13 Table 9-4 footnote (o), which applies to stocky braces, the tabulated *m*-factor, m_b, must be multiplied by 0.5, since the braces are "noncompact" per AISC 360-10 Table B4.1b (b/t is between λ_p and λ_r), as shown below:

b/t = 31.5

λ_p = Limiting slenderness for a compact web

$$= 1.12\sqrt{E/F_{ye,br}} = 1.12\sqrt{29{,}000 \text{ ksi}/57 \text{ ksi}} = 25.3$$

λ_r = Limiting slenderness for a non-compact web

$$= 1.40\sqrt{E/F_{ye,br}} = 1.40\sqrt{29{,}000 \text{ ksi}/57 \text{ ksi}} = 31.6$$

$\lambda_p < b/t < \lambda_r$

Per ASCE 41-13 Table 9-4 footnote (w), regardless of the modifiers applied, the component modification factor never needs to be taken less than 1.0:

Commentary

The first story braces are slightly longer than the braces at the second and third stories due slightly different bottom gusset geometries, resulting in smaller brace capacities at the first story. It should also be noted that the capacity-based (limit state) analysis demands calculated in the proceeding sections are based on these inadequate brace capacities, which are utilized for the purposes of this example; since the braces require strengthening, limit state analysis demands would ultimately need to correspond to the strengthened condition.

The factored component modifications are accordingly calculated as:

m_a = max[0.8(1.25),1.00] = max(1.00,1.00) = 1.00

m_b = max[0.8(0.5)(1.25),1.0] = max[0.50,1.00] = 1.00

The final *m*-factor is calculated by linear interpolation (for illustrative purposes):

$$m = m_a + \left(\frac{m_b - m_a}{\rho_b - \rho_a}\right)\left((KL/r) - \rho_a\right) = 1.00$$

Brace Compression Acceptance Criteria (ASCE 41-13 § 9.5.2.4)

κ = 1.0 for comprehensive data collection

P_{CE} = 129 kips, per previous calculations

$m\kappa P_{CE}$ = (1.0)(1.0)(129 kips) =129 kips

P_{UD} = 289 kips

$m\kappa P_{CE} < P_{UD}$

$Q_{UD}/(\kappa Q_{CE}) = P_{UD}/(\kappa P_{CE}) = 2.24 > m = 1.00$

Therefore, the brace is inadequate for compression.

9.7.2.2 Brace Compression Summary, North-South (Y) Direction

The following table summarizes the brace compression acceptance criteria in the north-south direction:

Table 9-10 Brace Compression Acceptance Criteria

Story	P_{UD} (kips)	κP_{CE} (kips)	$Q_{UD}/(\kappa Q_{CE})$	m	$Q_{UD}/(\kappa Q_{CE}) \le m$
3rd	148	137	1.08	1.00	NG
2nd	242	137	1.76	1.00	NG
1st	289	129	2.24	1.00	NG

9.7.3 Brace Tension Capacity (ASCE 41-13 § 9.5.2.3.2)

In the following section, brace tension acceptance criteria are analyzed as deformation-controlled action in accordance with ASCE 41-13 § 9.5.2.3.2. A detailed calculation for a typical first-story north-south brace is provided along with a summary table of acceptance criteria for north-south braces at all stories.

9.7.3.1 First Story Brace, North-South (Y) Direction

Factored Demands (ASCE 41-13 § 7.2.2 and § 7.5.2.1.1)

$P_G = 0.9P_D = 9.2$ kips (Minimum gravity load)

$T_{UD} = |P_G - P_E| = |9.2 \text{ kips} - 268 \text{ kips}| = 259$ kips

Tension Capacity (AISC 360-10 Section D2)

$T_{CE} = \phi F_{ye,br} A_g$

where:

$F_{ye,br} = 57$ ksi (Expected yield strength per testing)

$A_g = 3.98 \text{ in.}^2$ for TS6×6×$^3/_{16}$

$\phi = 1.0$ (ASCE 41-13 § 9.5.2.3.2)

$T_{CE} = (1.0)(57 \text{ ksi})(3.98 \text{ in.}^2) = 227$ kips

Component Modification Factor (ASCE 41-13 Table 9-4)

ASCE 41-13 Table 9-4 footnote (p) only applies to tension-only braces, so no reductions in the *m*-factor are required for IO.

$m = 1.25$

Brace Tension Acceptance Criteria (ASCE 41-13 § 9.5.2.4)

$\kappa = 1.0$ for comprehensive data collection

$m\kappa T_{CE} = (1.25)(1.0)(227 \text{ kips}) = 284$ kips

$T_{UD} = 259$ kips

$m\kappa T_{CE} \geq T_{UD}$

$Q_{UD}/(\kappa Q_{CE}) = T_{UD}/\kappa T_{CE} = 1.14 < m = 1.25$

Therefore, the brace is adequate for tension.

9.7.3.2 Brace Tension Summary, North-South (Y) Direction

The following table summarizes the brace tension acceptance criteria in the north-south direction:

Table 9-11 Brace Tension Acceptance Criteria

Story	T_{UD} (kips)	κT_{CE} (kips)	$Q_{UD}/\kappa(Q_{CE})$	m	$Q_{UD}/\kappa(Q_{CE}) \leq m$
3rd	133	227	0.59	1.25	OK
2nd	216	227	0.95	1.25	OK
1st	259	227	1.14	1.25	OK

9.7.4 Beam Flexural Capacity (ASCE 41-13 § 9.5.2.4.2)

In the following section, braced frame beam flexural acceptance criteria are analyzed as a force-controlled action in accordance with ASCE 41-13 § 9.5.2.4.2. A detailed calculation for a typical second floor north-south beam is provided along with a summary table of acceptance criteria for north-south beams at all stories.

9.7.4.1 Second Floor Beam, North-South (Y) Direction

Beam Flexural Demands (ASCE 41-13 § 9.5.2.4.2)

Per ASCE 41-13 § 9.5.2.4.2, the braced frames beams must be evaluated as force-controlled and resist the vertical load resulting from simultaneous yielding and buckling of the brace pairs. The unbalanced load effects are accordingly calculated using the expected tensile yield capacity of one brace and 30% of the expected compression capacity of the brace (limit state analysis, noted as "LSA" throughout this chapter); see Figure 9-7:

$M_{UF} = M_G + M_{LSA}$ (factored beam moment)

where:

$M_G = w_G L_{\text{beam}}^2/8 = (859 \text{ lb/ft})(30 \text{ ft})^2/8$ (Beam gravity moment)

$= 97$ kip-ft

where:

$$w_G = 1.1(76 \text{ psf} + 0.25(50 \text{ psf}))\left(\frac{10\,ft}{2} + 1\,ft\right) + 1.1(20 \text{ psf})(12.5 \text{ ft})$$

$= 859$ lb/ft

$M_{LSA} = R_{LSA} L_{\text{beam}}/4$ (Beam moment due to unbalanced brace forces)

$= 121 \text{ kips}(30 \text{ ft})/4$

$= 908$ kip-ft

where:

$R_{LSA} = (T_{CE} - 0.3P_{CE})(L_{br,z}/L_{br})$(Net downward brace reaction on beam)

$$= (227 \text{ kips} - 0.3(129 \text{ kips}))\left(\frac{12.5 \text{ ft}}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$= 121$ kips

$M_{UF} = M_G + M_{LSA} = 97 \text{ kip-ft} + 908 \text{ kip-ft}$

$= 1{,}005$ kip-ft

Beam Flange Local Buckling (AISC 360-10 Table B4.1b)

$$\frac{b_f}{2t_f} = 7.06 \qquad \text{(W18×35)}$$

$$\lambda_p = 0.38\sqrt{E / F_{yLB,bm}} = 0.38\sqrt{29{,}000 \text{ ksi} / 52.21 \text{ ksi}} = 9.0$$

$\frac{b_f}{2t_f} < \lambda_p$, therefore, beam flanges are "compact" for flexure

Beam Web Local Buckling (AISC 360-10 Table B4.1b)

$$\frac{h}{t_w} = 53.5 \qquad \text{(W18×35)}$$

$$\lambda_p = 3.76\sqrt{E / F_{yLB,bm}} = 3.76\sqrt{29{,}000 \text{ ksi} / 52.21 \text{ ksi}} = 88.6$$

$\frac{h}{t_w} < \lambda_p$, therefore, beam web is "compact" for flexure

Beam Flexural Capacity (AISC 360-10 Section F2)

Since the unbalanced brace forces will only act in a net downward direction on the beam, regardless of the earthquake direction, the beam will always be in positive bending with the compression flange continuously braced by the slab. Therefore, lateral-torsional buckling does not apply, and the beam flexural capacity may be calculated per AISC 360-10 Equation F2-1. Note, however, that the lower-bound yield strength of the beam must be used since this action must be evaluated as force-controlled in accordance with ASCE 41-13 § 9.5.2.4.2:

$F_{yLB,bm}$ = 52.21 ksi (Lower-bound per testing)

Z_x = 66.5 in^3 (W18×35)

ϕ = 1.0 (ASCE 41-13 § 9.5.2.3.2)

$M_{CL} = \phi F_{yLB,bm} Z_x = (1.0)(52.21 \text{ ksi})(66.5 \text{ in}^3)(1 \text{ ft} / 12 \text{ in.}) = 289$ kip-ft

Beam Flexural Acceptance Criteria (ASCE 41-13 § 9.5.2.4)

κ = 1.0 (Knowledge factor, comprehensive data collection)

$\kappa M_{CL} = (1.0)(289 \text{ kip-ft}) = 289$ kip-ft

M_{UF} = 1,005 kip-ft

$\kappa M_{CL} < M_{UF}$

$Q_{UF} / (\kappa Q_{CL}) = M_{UF} / (\kappa M_{CL}) = 3.48 > 1.00$

Therefore, the beam is inadequate for flexure.

Commentary

The first story braces are slightly longer than the braces at the second and third stories due slightly different bottom gusset geometries, resulting in smaller brace compression capacities and thus larger limit state analysis demands at the first story beams.

9.7.4.2 Beam Flexure Summary, North-South (Y) Direction

The following table summarizes the beam flexure acceptance criteria in the north-south direction:

Table 9-12 Beam Flexure Acceptance Criteria

Story	M_{UF} (k-ft)	κM_{CL} (k-ft)	Q_{UF} /(κQ_{CL})	Q_{UF} /(κQ_{CL}) ≤ 1.0
3rd	950	289	3.28	*NG*
2nd	988	289	3.41	*NG*
1st	1005	289	3.48	*NG*

9.7.5 *Beam Compression Capacity (ASCE 41-13 § 9.5)*

In the following section, braced frame beam compression acceptance criteria are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.2. A detailed calculation for a typical second floor north-south beam is provided along with a summary table of acceptance criteria for north-south beams at all stories.

9.7.5.1 Second Floor Beam, North-South (Y) Direction

Beam Compression Demands (ASCE 41-13 § 9.5.2.4.2)

Commentary

Assuming that the beam axial force is distributed equally in tension and compression is an approximation that holds true if the beam does not yield in compression. If the beam yields in compression, axial load will redistribute in beam; the beam's tension demand will continue to increase as its compression demand plateaus and eventually decreases to its post-buckling residual strength. Nevertheless, this equal axial load distribution assumption is always a valid upper bound estimate of the beam's compression demand.

The horizontal component of the unbalanced brace forces must be resolved in the beam axially. Assuming that the unbalanced force places half the beam in tension and the other half in compression:

$$P_{LSA} = (T_{CE} + 0.3P_{CE})(L_{br,y}/L_{br})/2$$

$$= \frac{1}{2}\left(227 \text{ kips} + 0.3(129 \text{ kips})\right)\left(\frac{(30 \text{ ft}/2)}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 102 \text{ kips}$$

$$P_G = 0 \text{ kips}$$

$$P_{UF} = P_G + P_{LSA} = 0 \text{ kips} + 102 \text{ kips} = 102 \text{ kips}$$

Beam Flange Local Buckling (AISC 360-10 Table B4.1a)

$$\frac{b_f}{2t_f} = 7.06 \qquad \text{(W18×35)}$$

$$\lambda_r = 0.56\sqrt{E/F_{yLB,bm}} = 0.56\sqrt{29{,}000 \text{ ksi}/52.2 \text{ ksi}} = 13.2$$

$\frac{b_f}{2t_f} < \lambda_r$, therefore beam flanges are "nonslender" for compression

Beam Web Local Buckling (AISC 360-10 Table B4.1a)

$\dfrac{h}{t_w}$ = 53.5 (W18×35)

$\lambda_r = 1.49\sqrt{E / F_{yLB,bm}} = 1.49\sqrt{29{,}000 \text{ ksi} / 52.21 \text{ ksi}} = 35.1$

$\dfrac{h}{t_w} > \lambda_r$, therefore beam web is "slender" for compression

Beam Compression Capacity with Q = 1 (AISC 360-10 § E7)

L_x = 30 ft, flexural buckling about local x-axis

L_y = 0 ft, flexural buckling about local y-axis

L_z = 30 ft, flexural-torsional buckling

K_x = 1.0, flexural buckling about beam local x-axis

K_y = 1.0, flexural buckling about beam local y-axis

K_z = 1.0, flexural-torsional buckling

r_x = 7.04 in, radius of gyration about beam local x-axis

r_y = 1.22 in, radius of gyration about beam local y-axis

- Critical Flexural Buckling Stress

$$KL/r = \max\left\{\frac{K_xL_x}{r_x}, \frac{K_yL_y}{r_y}\right\} = \frac{K_xL_x}{r_x} = \frac{1.0(30 \text{ ft})(12 \text{ in/ft})}{7.04 \text{ in}} = 51.1$$

$$KL/r = 51.1 \le 4.71\sqrt{E / F_{yLB,bm}} = 111$$

$$F_{e,FB} = \frac{\pi^2 E}{(KL/r)^2} = 110 \text{ ksi}$$

$$F_{cr,FB} = \left[0.658^{F_{yLB,bm}/F_{e,FB}}\right]F_{yLB,bm} = 42.8 \text{ ksi} \qquad \text{(AISC 360-10 Eq. E7-2)}$$

- Critical Constrained-Axis Flexural-Torsional Buckling Stress

Per AISC *Seismic Design Manual* (AISC, 2012) Equation 8-2, the constrained-axis flexural-torsional elastic buckling stress is calculated as:

$$F_{e,FTB} = \left(\frac{\pi^2 E\left(C_w + I_y(d/2)^2\right)}{(K_zL_z)^2} + GJ\right)\left(\frac{1}{I_x + I_y + (d/2)^2 A_g}\right)$$

where:

E = 29,000 ksi

G = 11,200 ksi

K_zL_z = (1.0)(30 ft)(12in/ft) = 360 in

d = 17.7 in for W18×35

A_g = 10.3 in.^{2} for W18×35

I_x = 510 in.^{4} for W18×35

I_y = 15.3 in.^{4} for W18×35

C_w = 1,140 in.^{6} for W18×35

J = 0.506 in^4 for W18×35

$F_{e,FTB}$ = 8.13 ksi

$$\frac{F_{yLB,bm}}{F_{e,FTB}} = (52.21 \text{ ksi} / 8.13 \text{ ksi}) = 6.42 > 2.25$$

$F_{cr,FTB}$ = $0.877F_{e,FTB}$ = 7.13 ksi (AISC 360-10 Eq. E7-3)

Slender Element Reduction Factor, *Q* (AISC 360-10 Section E7)

Q_s = 1.0 for nonslender flanges

f = $\min\{F_{cr,FB}, F_{cr,FTB}\} = F_{cr,FTB}$ = 7.13 ksi

b = $h = d - 2k_{des}$ = 17.7 in – 2(0.827 in.) = 16.0 in

t = t_w = 0.300 in

$$b_e = \min\left\{b, 1.92t\sqrt{\frac{E}{f}}\left[1-\frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right]\right\} = \min\{16.0 \text{ in}, 21.8 \text{ in}\}$$

= 16.0 in

A_g = 10.3 in.^{2} for W18×35

A_e = $A_g - t_w(h - b_e)$ = 10.3 – (0.300)(16.0 – 16.0) = 10.3 in.^{2}

Q_a = A_e/A_g = 1.00

Q = Q_sQ_a = 1.00

Beam Compression Capacity with $Q = Q_a\, Q_s$ (AISC 360-10 Section E7)

$$\frac{QF_{yLB,bm}}{F_{e,FTB}} = (1.0)(52.21 \text{ ksi} / 8.13 \text{ ksi}) = 6.42 > 2.25$$

$F_{cr,LB}$ = $0.877F_{e,FTB}$ = (0.877)(8.13 ksi) = 7.13 ksi

A_g = 10.3 in.^{2} for W18×35

ϕ = 1.0

P_{CL} = $\phi F_{cr,LB}A_g$ = (1.0)(7.13 ksi)(10.3 in.^{2}) = 73 kips

Beam Compression Acceptance Criteria (ASCE 41-13 § 9.5.2.4)

κ = 1.0

κP_{CL} = (1.0)(73 kips) = 73 kips

P_{UF} = 102 kips

$\kappa P_{CL} \leq P_{UF}$

$P_{UF}/(\kappa P_{CL}) = 1.40 > 1.00$

Therefore, the beam is inadequate for compression.

9.7.5.2 Beam Compression Summary, North-South (Y) Direction

The following table summarizes the beam compression acceptance criteria in the north-south direction:

Table 9-13 Beam Compression Acceptance Criteria

Story	P_{UF} (kips)	κP_{CL} (kips)	$Q_{UF}/(\kappa Q_{CL})$	$Q_{UF}/(\kappa Q_{CL})=\leq 1.0$
3rd	103	73	1.40	NG
2nd	103	73	1.40	NG
1st	102	73	1.40	NG

9.7.6 Beam PM-Interaction Acceptance Criteria (ASCE 41-13 § 9.5.2.4.2)

In the following section, braced frame beam PM-interaction acceptance criteria are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.2. A detailed calculation for the typical second floor north-south beam (from the previous sections) is provided along with a summary table of acceptance criteria for north-south beams at all stories.

9.7.6.1 Second Floor Beam, North-South (Y) Direction

Limit State Analysis Case 1

This case is shown in Figure 9-7.

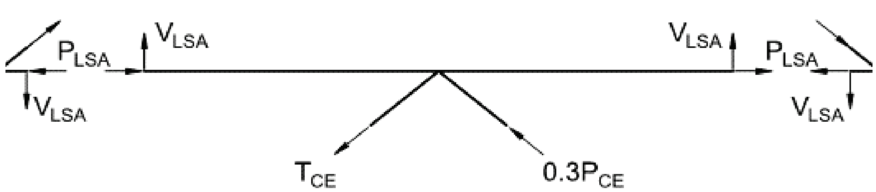


Figure 9-7 Beam Limit State Analysis: Case 1.

Per ASCE 41-13 § 9.5.2.3.2, beams with axial loads exceeding 10% of their axial strength are required to be calculated in accordance with the requirements for fully-restrained (FR) frame columns per ASCE 41-13 § 9.4.2.4.2.2; the axial ratio of 1.40 exceeds the 10% threshold, so the latter beam-column requirements apply. Accordingly, PM-interaction ASCE 41-13 Equation 9-12 governs, since the axial ratio of 1.40 exceeds 0.50. Note, however, that PM-interaction ASCE 41-13 Equation 9-12 actually governs, regardless of the axial ratio, since the ASCE 41-13 § 9.5.2.4.2 requires beams in chevron-braced frames to be evaluated as force-controlled:

$$\frac{P_{UF}}{\kappa P_{CL}} + \frac{M_{UFx}}{\kappa M_{CLx}} + \frac{M_{UFy}}{\kappa M_{CLy}} = 1.40 + 3.48 + 0.00 = 4.87 > 1.00$$

Therefore, with an acceptance ratio (see Chapter 2 of this *Guide*) of 4.87, the beam is inadequate for PM-interaction.

Limit State Analysis Case 2 (Figure 9-8)

This case is shown in Figure 9-8.

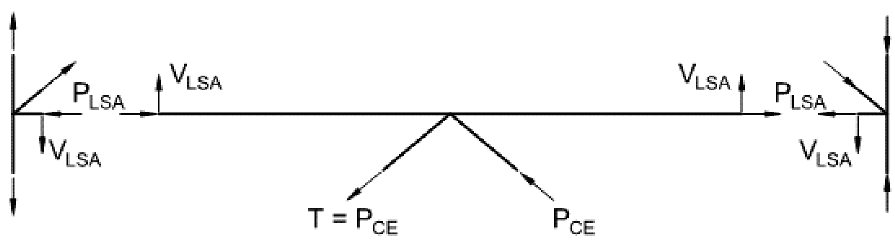


Figure 9-8 Beam Limit State Analysis: Case 2.

Forces corresponding to the compressive brace first reaching its expected compressive strength should be also considered; the adjacent tension brace would have a corresponding tensile force equal to the expected brace compressive strength for this limit state. This condition results in smaller flexural demands but may result in larger compressive demands in the beam:

$$R_{LSA} = (P_{CE} - (T = P_{CE}))(L_{br,z}/L_{br})$$

$$= (129 \text{ kips} - 129 \text{ kips})\left(\frac{12.5 \text{ ft}}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 0 \text{ kips}$$

$$M_{LSA} = R_{LSA}L_{\text{beam}}/4 = 0 \text{ kips}(30 \text{ ft})/4 = 0 \text{ kip-ft}$$

$$M_{UF} = M_G + M_{LSA} = 97 \text{ kip-ft} + 0 \text{ kip-ft} = 97 \text{ kip-ft}$$

$$P_{LSA} = \frac{1}{2}\,(P_{CE} + (\mathrm{T} = P_{CE}))\,(L_{br,y}/L_{br})$$

$$= \frac{1}{2}(129 \text{ kips} + 129 \text{ kips})\left(\frac{(30 \text{ ft}/2)}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 99 \text{ kips}$$

$$P_{UF} = P_G + P_{LSA} = 0 \text{ kips} + 99 \text{ kips} = 99 \text{ kips}$$

$$\frac{P_{UF}}{\kappa P_{CL}} + \frac{M_{UFx}}{\kappa M_{CLx}} + \frac{M_{UFy}}{\kappa M_{CLy}} = 1.36 + 0.34 + 0.00 = 1.70 > 1.00$$

Since the Case 1 PM-interaction acceptance ratio (4.87) exceeds the Case 2 PM-interaction acceptance ratio (1.70), Case 1 governs.

9.7.6.2 Beam PM-Interaction Summary, North-South (Y) Direction

The following table summarizes the beam PM-interaction acceptance criteria in the north-south direction:

Table 9-14 Beam PM-Interaction Acceptance Criteria

	PM-Interaction Acceptance Ratio			
Story	Case 1	Case 2	Max	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	4.69	1.64	4.69	NG
2nd	4.82	1.77	4.82	NG
1st	4.87	1.70	4.87	NG

9.7.7 Brace Connection Demands (ASCE 41-13 § 9.5.2.4.1)

In the following section, force-controlled brace connection demands are calculated per ASCE 41-13 § 9.5.2.4.1. Per ASCE 41-13 § 7.5.2.1.2, the force-controlled demands, Q_{UF}, may be calculated using limit state analysis or the alternative procedure involving the J-factor. All connections are evaluated using limit state analysis in this example. A detailed calculation for a typical first-story north-south brace connection is provided along with a summary table of the connection demands for north-south braces at all stories. Connection details and a photo are presented in Figure 9-9 through Figure 9-12, while the connection in Figure 9-9 is the detail used for the detailed calculations in this example.

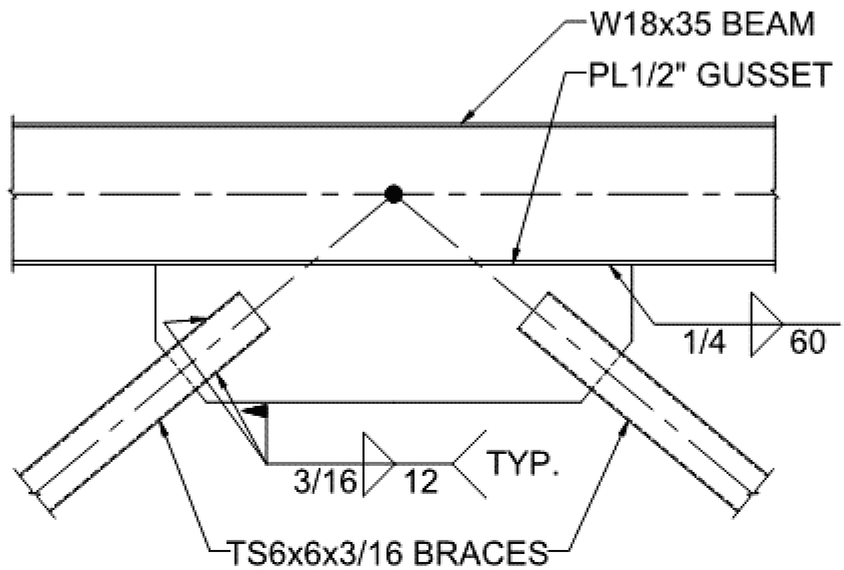


Figure 9-9 Brace-to-beam connection detail.

9.7.7.1 First Story Braces, North-South (Y) Direction

Expected brace tensile and compressive strengths are determined as follows:

$T_{UF} = T_{LSA} = T_{CE} = 227$ kips

$P_{UF} = P_{LSA} = P_{CE} = 129$ kips

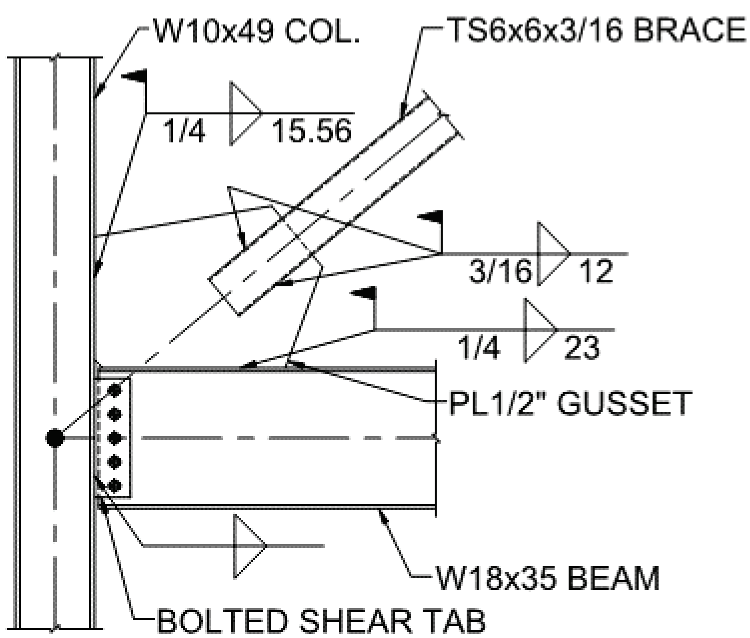


Figure 9-10 Brace-to-beam/column connection detail.

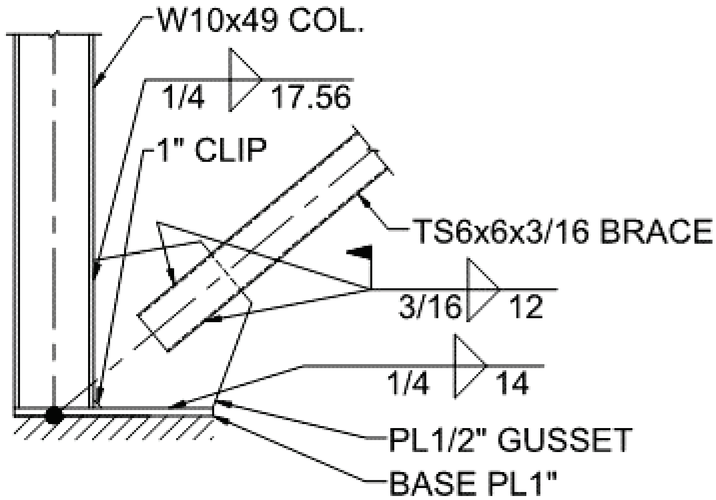


Figure 9-11 Brace-to-column/base plate connection detail.

9.7.7.2 Brace Connection Demand Summary, North-South (Y) Direction

The following table summarizes the brace connection demands in the north-south direction; also see Table 9-10 and Table 9-11:

Table 9-15 Brace Connection Demands

Story	T_{UF} (kips)	P_{UF} (kips)
3rd	227	137
2nd	227	137
1st	227	129

Figure 9-12 Photo of chevron connection used in example.

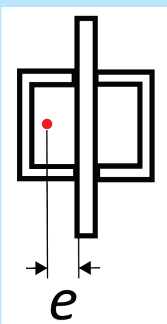
9.7.8 Brace-to-Gusset Weld Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for the brace-to-gusset welds are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace connection is provided along with a summary table for north-south brace connections at all stories.

9.7.8.1 First Story Braces, North-South (Y) Direction

Commentary

The line of action of the brace force will coincide with the centroids of two half tube sections on each side of the gusset plate. This eccentricity will induce a moment demand into the welds.



Fillet Weld Shear Capacity (AISC 360-10 Section J2.4)

L_{weld} = 12 in, fillet weld length

S_{weld} = $L_w^2/6 = (12\text{ in.})^2/6 = 24\text{ in.}^2$, fillet weld section modulus

e = Eccentricity per AISC 360-10 Table D3.1, Case 6

$$= \frac{B^2 + 2BH}{4(B+H)}$$

$$= \frac{(6\text{ in.})^2 + 2(6\text{ in.})(6\text{ in.})}{4(6\text{ in.} + 6\text{ in.})}$$

= 2.25 in

$$f_{\text{peak},T} = \sqrt{f_{v,T}^2 + (f_{a,T} + f_{b,T})^2}$$

$f_{a,T}$ = 0 k/in.

$f_{v,T}$ = $(T_{UF}/4)/L_{weld}$ = (227 k / 4)/12 in = 4.73 k/in.

$f_{b,T}$ = $(T_{UF}/4)\,\bar{x}/S_{weld}$ = (227 k / 4)(2.25)/(24 in.^{2}) = 5.32 k/in.

$$f_{\text{peak},T} = \sqrt{f_{v,T}^2 + (f_{a,T} + f_{b,T})^2} = [4.72^2 + (0+5.32^2)]^{0.5} = 7.11\text{ k/in.}$$

$$f_{\text{avg},T} = \text{avg}\left\{\sqrt{f_{v,T}^2 + \max\{0,(f_{a,T} \pm f_{b,T})\}^2}\right\}$$

= avg{7.11 k/in. , 4.73 k/in.}

= 5.92 k/in.

$f_{UF,T}$ = max{$f_{\text{peak},T}$, 1.25$f_{\text{avg},T}$} (AISC Construction Manual, pg. 13-11)

= 1.25$f_{\text{avg},T}$

= 1.25 (5.92 k/in.)

= 7.40 k/in.

$f_{UF,P}$ = $f_{UF,T}(P_{UF}/T_{UF})$ = 7.40 k/in. (129 k)/(227 k) = 4.21 k/in.

F_{nwLB} = $0.60F_{EXX}$ = (0.60)(71.8 ksi) = 43.1 ksi (AISC 360-10 Eq. J2-5)

F_{EXX} = Filler metal classification strength (lower-bound per testing)

= 71.8 ksi

A_{we}/L_w=Effective weld area per length

$$= \frac{\sqrt{2}}{2}\left(\frac{3}{16}\text{ in.}\right) = 0.1326\text{ in.}^2/\text{in.}$$

ϕ = 1.0

r_{wCL} = $\phi F_{nwLB}A_{we}/L_w$ = (1.0)(43.1 ksi)(0.1326 in.^{2}/in.) = 5.71 k/in.

κ = 1.0

κr_{wCL} = (1.0)(5.71 k/in.) = 5.71 k/in.

$\kappa r_{wCL} < f_{UF,T}$

Acceptance Ratio = $f_{UF,T}$ / (κr_{wCL}) = 1.30 > 1.0

$\kappa r_{wCL} \geq f_{UF,P}$

Acceptance Ratio = P_{UF} / (κR_{wCL}) = 0.74 ≤ 1.0

Therefore, the fillet welds can adequately develop the brace compression capacity but not its tensile capacity.

Commentary

Per AISC 341 Section A.3.2, expected material properties are used to calculate the brace's shear yielding and shear rupture capacities since these capacities are being compared to the expected brace tensile yielding strength. Lower-bound properties should be used if force-controlled demands are calculated with *J*-factors per ASCE 41-13 § 7.5.2.1.2 Method 2.

Brace Base Metal Shear Yielding (AISC 360-10 § J4.2)

$F_{ye,br}$ = Expected brace yield strength per testing
= 57 ksi

A_{gv}/L_w = Gross shear area per length = brace thickness
= t_{br} = 0.174 in.

ϕ = 1.0

r_{ybrCE} = $\phi(0.60F_{ye,br}A_{gv}/L_w)$ = (1.0)(0.60)(57 ksi)(0.174 in.) = 5.95 k/in.

κ = 1.0

κr_{ybrCE} = 5.95 k/in.

$\kappa r_{ybrCE} < f_{UF,T}$

Acceptance Ratio = $f_{UF,T}$ / (κr_{ybrCE}) = 1.24 > 1.0

$\kappa r_{ybrCE} \geq f_{UF,P}$

Acceptance Ratio = $f_{UF,P}$ / (κr_{ybrCE}) = 0.71 ≤ 1.0

Therefore, the brace base metal can develop the expected brace compression strength but not its tensile strength in shear yielding.

Brace Base Metal Shear Rupture (AISC 360-10 Section J4.2)

$F_{ue,br}$ = Expected brace tensile strength per testing
= 65.27 ksi

A_{nv}/L_w = Net shear area per length = brace thickness

$= t_{br} = 0.174$ in.

$\phi = 1.0$

$r_{ubrCE} = \phi(0.60F_{ue,br}A_{gv}/L_w)$

$= (1.0)(0.60)(65.27 \text{ ksi})(0.174 \text{ in.})$

$= 6.81$ k/in.

$\kappa = 1.0$

$\kappa r_{ubrCE} = 6.81$ k/in.

$\kappa r_{ubrCE} < f_{UF,T}$

Acceptance Ratio $= f_{UF,T} / (\kappa r_{ubrCE}) = 1.09 > 1.0$

$\kappa r_{ubrCE} \geq f_{UF,P}$

Acceptance Ratio $= f_{UF,P} / (\kappa r_{ubrCE}) = 0.62 \leq 1.0$

Therefore, the base metal shear rupture strength of the brace can adequately develop the expected brace compressive strength but not its tensile yield strength.

9.7.8.2 Brace Weld Summary, North-South (Y) Direction

The following tables summarize the brace weld acceptance ratios:

Commentary

Since similar gusset plates and brace-to-gusset welds are used at each end of the brace, these acceptance ratios are applicable to the connections at each end of the braces.

Table 9-16 Brace Weld Tension Acceptance Ratios

Story	Weld	Brace Yielding	Shear Rupture
3rd	1.30	1.24	1.09
2nd	1.30	1.24	1.09
1st	1.30	1.24	1.09

Table 9-17 Brace Weld Compression Acceptance Ratios

Story	Weld	Shear Yielding	Shear Rupture
3rd	0.79	0.75	0.66
2nd	0.79	0.75	0.66
1st	0.74	0.71	0.62

9.7.9 Brace Tensile Rupture Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for tensile rupture of the braces are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace connection is provided along with a summary table for north-south brace connections at all stories.

9.7.9.1 First Story Braces, North-South (Y) Direction

Brace Effective Net Tensile Area (AISC 360-10 Section D3)

A_g = 3.98 in.^{2} for HSS6×6×3/16

A_n = Net tensile area

= $A_g - 2t_{br}(t_g + {}^1/_8$ in.)

= 3.98 in.^{2} – 2(0.174 in.)(0.50 in + $^1/_8$ in.)

= 3.76 in.^{2}

B = Brace depth and width

= H = 6 in

l = Weld length

= L_{weld} = 12 in ≥ H

U = Shear lag factor

= $1 - \bar{x}\,/\,l$ (AISC 360-10 Table D3.1, Case 6)

where:

$\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$ (“e” used in Section 9.7.8.1 of this *Guide*, while $\bar{x}$ is a AISC symbol)

= [(6 in.)^2 + 2(6 in.)(6 in.)]/[(4(6 in + 6 in.)]

= 2.25 in

U = 1 – (2.25 in / 12 in.) = 0.813

A_e = UA_n= (0.813)(3.76 in.^{2}) = 3.06 in.^{2}

Brace Tensile Rupture Capacity (AISC 360-10 Section J4.1)

$F_{ue,br}$ = 65.27 ksi

ϕ = 1.0

$R_{rbrCE} = \phi(F_{ue,br}A_e)$ = 200 kips

Brace Tensile Rupture Acceptance Criteria (ASCE 41-13 § 9.5.2.4):

κ = 1.0

κR_{rbrCE} = (1.0)(200 kips) = 200 kips

T_{UF} = 227 kips

$\kappa R_{rbrCE} < T_{UF}$

> **Commentary**
>
> Per AISC 341 Section A.3.2, expected material properties are used to calculate the brace's tensile rupture capacity of its net section since this capacity is being compared to the expected brace tensile yielding strength.

Acceptance Ratio = $T_{UF}/(\kappa R_{rbrCE})$ = 1.14 > 1.00

Therefore, the net brace area is inadequate for tensile rupture at the gusset plate connection.

9.7.9.2 Brace Tensile Rupture Summary, North-South (Y) Direction

The following table summarizes the brace tensile rupture acceptance ratios in the north-south direction:

Table 9-18 Brace Tensile Rupture Acceptance Ratios

Story	T_{UF} (kips)	κR_{rbrCL} (kips)	$Q_{UF}/(\kappa Q_{CL})$	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	227	200	1.14	NG
2nd	227	200	1.14	NG
1st	227	200	1.14	NG

9.7.10 Gusset Plate Block Shear Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for block shear of the gusset plates are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace connection is provided along with a summary table for north-south brace connections at all stories.

9.7.10.1 First Story Braces, North-South (Y) Direction

Shear Yielding (AISC 360-10 Section J4.3)

$F_{yLB,g}$ = 52.21 ksi

$A_{gv} = 2t_g L_{\text{weld}}$ = 2(0.50 in.)(12 in.) = 12.0 in.^{2}

$0.60F_{yLB,g}A_{gv}$ = (0.60)(52.21 ksi)(12.0 in.^{2}) = 376 kips

Shear Rupture (AISC 360-10 Section J4.3)

$F_{uLB,g}$ = 72.1 ksi

$A_{nv} = 2t_g L_{\text{weld}}$ = 2(0.50 in.)(12 in.) = 12.0 in.^{2}

$0.60F_{uLB,g}A_{nv}$ = (0.60)(72.1 ksi)(12.0 in.^{2}) = 519 kips

Tensile Rupture (AISC 360-10 Section J4.3)

U_{bs} = 1.0

$A_{nt} = H_{br}t_g$ = (6 in.)(0.50 in.) = 3.0 in.^{2}

$U_{bs}F_{uLB,g}A_{nt}$ = (1.0)(72.1 ksi)(3.0 in.^{2}) = 216 kips

Block Shear Capacity (AISC 360-10 Section J4.3)

ϕ = 1.0

$R_{bsCL} = \phi[\min\{0.60F_{uLB,g}A_{nv}, 0.60F_{yLB,g}A_{gv}\} + U_{bs}F_{uLB,g}A_{nt}]$

= 1.0[min{519 kips, 376 kips} + 216 kips]

= 1.0[376 kips + 216 kips]

= 592 kips

Block Shear Acceptance Criteria (ASCE 41-13 § 9.5.2.4):

κ = 1.0

κR_{bsCL} = (1.0)(592 kips) = 592 kips

T_{UF} = 227 kips

$\kappa R_{bsCL} \geq T_{UF}$

Acceptance Ratio = $T_{UF}/(\kappa R_{bsCL}) = 0.38 \leq 1.00$

Therefore, the gusset plate has adequate block shear capacity.

9.7.10.2 Gusset Plate Block Shear Summary, North-South (Y) Direction

The following table summarizes the gusset plate block shear acceptance ratios in the north-south direction:

Table 9-19 Gusset Plate Block Shear Acceptance Criteria

Story	T_{UF} (kips)	κR_{bsCL} (kips)	$Q_{UF}/(\kappa Q_{CL})$	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	227	592	0.38	OK
2nd	227	592	0.38	OK
1st	227	592	0.38	OK

9.7.11 Whitmore Section Tensile Yielding Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for tensile yielding of the gusset plate Whitmore section, as shown in Figure 9-13, are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace top connection to the second-floor beam is provided along with a summary table for north-south brace connections at all stories.

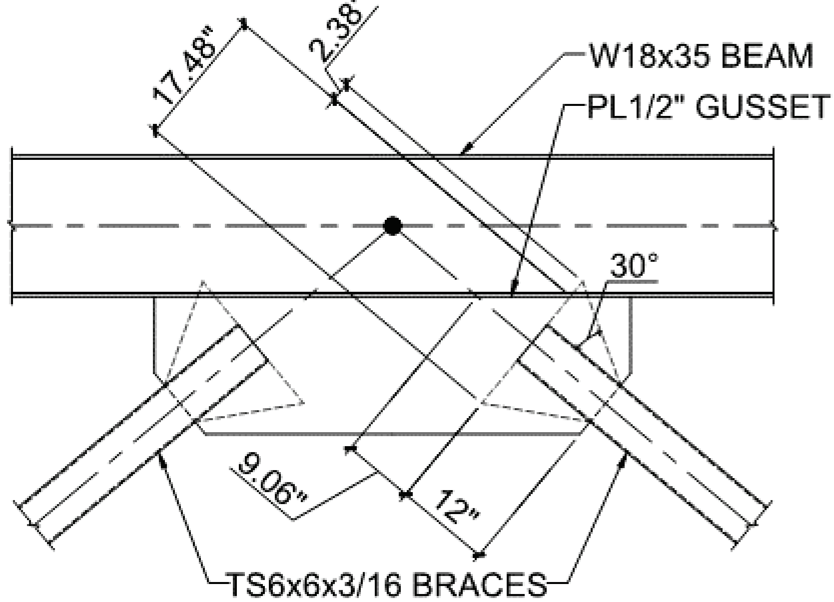


Figure 9-13 Whitmore section at brace-to-beam connection.

9.7.11.1 First Story Brace-to-Beam Connection, North-South (Y) Direction

Whitmore Section (2011 AISC Construction Manual Part 9):

L_W = Whitmore length

$= H_{br} + 2L_{\text{weld}}\tan 30° = 6 \text{ in} + 2(12 \text{ in.})\tan 30°$

$= 19.86$ in

As shown in Figure 9-13, a portion of the Whitmore section is within the gusset plate and the remainder is in the beam web:

$L_{W,g}$ = Gusset plate length

$= 17.48$ in

$L_{W,bm}$ = Beam web length

$= L_w - L_{w,g} = 19.96 \text{ in} - 17.48 \text{ in} = 2.38 \text{ in}$

t_g = Gusset plate thickness

$= 0.50$ in

$t_{bm,\text{web}}$ = Beam web thickness

$= 0.30$ in

Whitmore Section Tensile Yielding Capacity (AISC 360-10 Section J4.1)

$F_{yLB,g}$ = 52.21 ksi

$F_{yLB,bm}$ = 52.21 ksi

ϕ = 1.0

R_{WyCL} $= \phi(F_{yLB,g}L_{W,g}t_g + F_{yLB,bm}L_{W,bm}t_{bm,\text{web}})$

$= (1.0)((52.21 \text{ ksi})(17.48 \text{ in.})(0.50 \text{ in.}) + (52.21 \text{ ksi})(2.38 \text{ in.})(0.30))$

$= 494$ kips

Whitmore Section Tensile Yielding Acceptance Criteria

κ $= 1.0$

$\kappa R_{WyCL} = (1.0)(494 \text{ kips}) = 494$ kips

T_{UF} $= 227$ kips

$\kappa R_{WyCL} \geq T_{UF}$

Acceptance Ratio $= T_{UF} / (\kappa R_{WyCL}) = 0.46 \leq 1.00$

Therefore, the Whitmore section tensile yielding capacity is adequate.

9.7.11.2 Whitmore Section Tension Summary, North-South (Y) Direction

The following table summarizes the Whitmore section tensile yielding ratios in the north-south direction. Note that the Whitmore section is completely within the gusset plates at each brace bottom connection.

Table 9-20 Whitmore Section Tensile Yielding Acceptance Criteria

Story	Brace Conn.	T_{UF} (kips)	κR_{WyCL} (kips)	Q_{UF} /(κQ_{CL})	Q_{UF} /(κQ_{CL}) ≤ 1.0
3rd	Top	227	494	0.46	OK
	Bottom	227	518	0.44	OK
2nd	Top	227	494	0.46	OK
	Bottom	227	518	0.44	OK
1st	Top	227	494	0.46	OK
	Bottom	227	518	0.44	OK

9.7.12 Whitmore Section Compression Buckling Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for compression buckling of the gusset plate Whitmore section are analyzed as a force-controlled action in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace top connection to the second-floor beam is provided along with a summary table for north-south brace connections at all stories.

9.7.12.1 First Story Brace-to-Beam Connection, North-South (Y) Direction

Whitmore Section (2011 AISC Construction Manual Part 9)

L_W = 19.86

$L_{W,g}$ = 17.48 in.

$L_{W,bm}$ = 2.38 in.

t_g = 0.50 in.

$t_{bm,\text{web}}$ = 0.30 in.

$$I_W = \frac{1}{12}(L_{W,g}t_g^3 + L_{W,bm}t_{bm,\text{web}}^3)$$

$= (1/12)((17.48 \text{ in.})(0.50 \text{ in.})^3 + (2.38 \text{ in.})(0.30 \text{ in.})^3)$

$= 0.19 \text{ in.}^4$

$A_W = L_{W,g}t_g + L_{W,bm}t_{bm,\text{web}}$

$= (17.48 \text{ in.})(0.50 \text{ in.}) + (2.38 \text{ in.})(0.30 \text{ in.})$

$= 9.45 \text{ in.}^2$

$r_W = \sqrt{I_W / A_W} = 0.14$ in.

Whitmore Section Compression Capacity (AISC 360-10 Section J4.4)

$F_{yLB,W} = \min\{F_{yLB,g}, F_{yLB,bm}\} = 52.21$ ksi

K_W = 0.65

l_W = 9.06 in from Figure 9-13

$\dfrac{K_W l_W}{r_W}$ = 41.8 > 25 (AISC 360-10 Section E3 applies)

$= 41.8 \leq 4.71\sqrt{E / F_{yLB,W}} = 111$

$$F_{e,W} = \frac{\pi^2 E}{(K_W l_W / r_W)^2} = \frac{\pi^2 (29{,}000)}{(41.8)^2} = 164 \text{ ksi}$$

$F_{cr,W} = \left[0.658^{F_{yLB,W}/F_{e,W}}\right]F_{yLB,W} = 45.7$ ksi (AISC 360-10 Eq. E3-2)

ϕ = 1.0

$R_{WcCL} = \phi F_{cr,W}A_W = 432$ kips

Whitmore Section Compressive Strength Acceptance Criteria

κ = 1.0

κR_{WcCL} = 432 kips

P_{UF} = 129 kips

$\kappa R_{WcCL} \geq P_{UF}$

Acceptance Ratio = P_{UF} / (κR_{WcCL}) = 0.30 ≤ 1.00

Therefore, the Whitmore section can develop the expected compression capacity of the brace.

9.7.12.2 Whitmore Section Compression Summary, North-South (Y) Direction

The following table summarizes the Whitmore section compression buckling ratios in the north-south direction. Note that the Whitmore section is completely within the gusset plates at each brace bottom connection.

Table 9-21 Whitmore Section Compression Buckling Acceptance Criteria

Story	Brace Conn.	P_{UF} (kips)	κR_{WcCL} (kips)	$Q_{UF}/(\kappa Q_{CL})$	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	Top	137	433	0.32	OK
	Bottom	137	385	0.36	OK
2nd	Top	137	433	0.32	OK
	Bottom	137	385	0.36	OK
1st	Top	129	432	0.30	OK
	Bottom	129	448	0.29	OK

9.7.13 Top Gusset Plate Connection Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for the brace top connections (at the gusset-to-beam chevron connection) are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace top connection to the second-floor beam is provided along with a summary table for north-south brace connections at all stories.

9.7.13.1 First Story Gusset-to-Beam Connection, North-South (Y) Direction

Similar to the beam PM-interaction capacity check in Section 9.7.6 of this *Guide*, two limit state analysis load cases, as illustrated in Figure 9-14 and Figure 9-15, are considered.

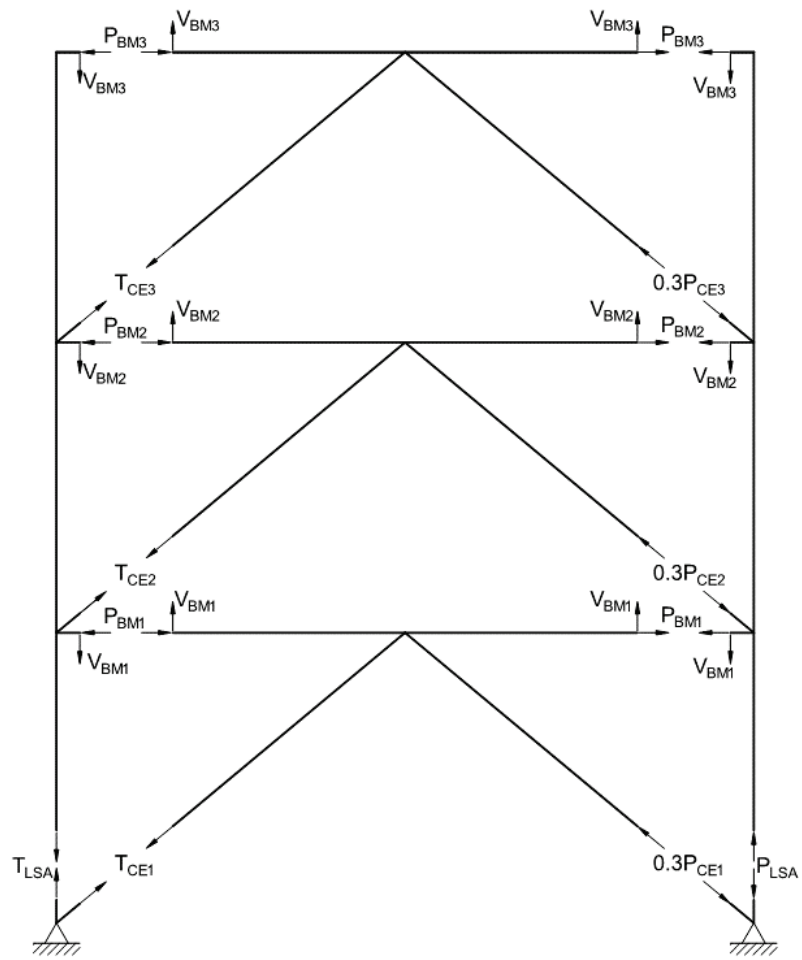


Figure 9-14 Column axial loads: Limit State Analysis Case 1.

Connection Demands

- Limit State Analysis Case 1

  Brace forces corresponding to simultaneous expected yielding in one brace and expected post-compression buckling in the other brace are assumed. See Figure 9-14.

$$P_{UF} = P_{LSA} = (T_{CE} - 0.3P_{CE})(L_{br,z}/L_{br})$$

$$= (227 \text{ kips} - 0.3(129 \text{ kips}))\left(\frac{12.5 \text{ ft}}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 121 \text{ kips}$$

$$V_{UF} = V_{LSA} = (T_{CE} + 0.3P_{CE})(L_{br,y}/L_{br})$$

$$= (227 \text{ kips} + 0.3(129 \text{ kips}))\left(\frac{(30 \text{ ft}/2)}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 204 \text{ kips}$$

$$M_{UF} = V_{UF}(d_{bm}/2) = 1805 \text{ k-in.}$$

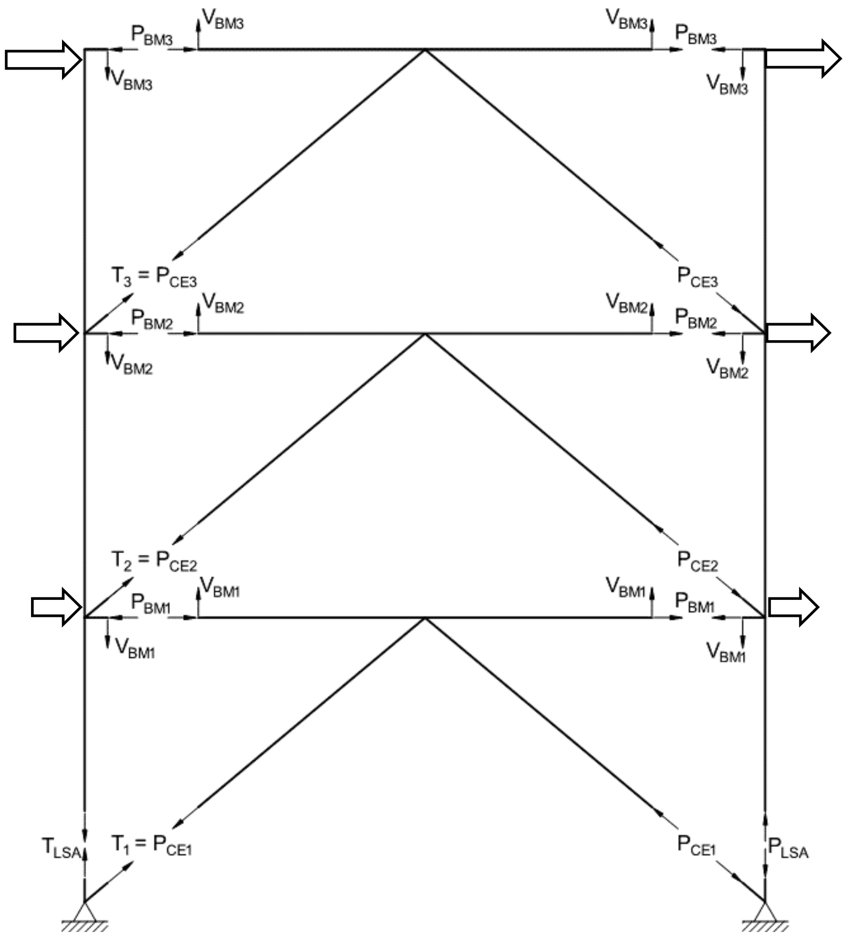


Figure 9-15 Column axial loads: Limit State Analysis Case 2.

- Limit State Analysis Case 2

  Brace forces corresponding to expected compression buckling of one brace and an equal and opposite tensile force in the other brace are assumed. See Figure 9-15.

$$P_{UF} = P_{LSA} = (P_{CE} - (T = P_{CE}))(L_{br,z}/L_{br})$$

$$= (129 \text{ kips} - 129 \text{ kips})\left(\frac{12.5 \text{ ft}}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 0 \text{ kips}$$

$$V_{UF} = V_{LSA} = (P_{CE} + (T = P_{CE}))(L_{br,y}/L_{br})$$

$$= (129 \text{ kips} + 129 \text{ kips})\left(\frac{(30 \text{ ft}/2)}{\sqrt{(12.5 \text{ ft})^2 + (30 \text{ ft}/2)^2}}\right)$$

$$= 198 \text{ kips}$$

$$M_{UF} = V_{UF}(d_{bm}/2) = 1752 \text{ k in}$$

Weld Demands (2011 AISC Construction Manual Part 8)

L_{weld} = 60 in

S_{weld} = (60 in.)^2/6 = 600 in.^{2}

- Limit State Analysis Case 1

  f_a = P_{UF}/L_{weld} = 120 k / 60 in = 2.01 k/in.

  f_v = V_{UF}/L_{weld} = 204 k / 60 in = 3.40 k/in.

  f_b = M_{UF}/S_{weld} = 1805 k-in. / 600 in.^{2} = 3.01 k/in.

  $$f_{\text{peak}} = \sqrt{f_v^2 + (f_a + f_b)^2}$$
  $$= \sqrt{(3.40)^2 + (2.01 + 3.01)^2}$$
  $$= 6.06 \text{ k/in.}$$

  $$f_{\text{avg}} = \text{avg}\left\{\sqrt{f_v^2 + \max\{0,(f_a \pm f_b)\}^2}\right\}$$
  = avg { 6.06 k/in., 3.40 k/in. }
  = 4.73 k/in.

  $f_{UF} = \max\left\{f_{peak},\, 1.25 f_{avg}\right\} = f_{\text{peak}}$ = 6.06 k/in.

- Limit State Analysis Case 2

  f_a = P_{UF}/L_{weld} = 0 k/in.

  f_v = V_{UF}/L_{weld} = 3.30 k/in.

  f_b = M_{UF}/S_{weld} = 2.92 k/in.

  $f_{\text{peak}} = \sqrt{f_v^2 + (f_a + f_b)^2}$ = 4.41 k/in.

  $f_{\text{avg}} = \text{avg}\left\{\sqrt{f_v^2 + \max\{0,(f_a \pm f_b)\}^2}\right\}$ = 3.85 k/in.

  $f_{UF} = \max\left\{f_{peak}, 1.25 f_{avg}\right\} = 1.25 f_{\text{avg}}$ = 4.82 k/in. < 6.06 k/in.

Therefore, Limit State Analysis Case 1 governs.

Fillet Weld Shear Capacity (AISC 360-10 Section J2.4):

F_{EXX} = 71.8 ksi

F_{nwLB} = Filler metal strength

= 0.60F_{EXX} = (0.60)(71.8 ksi) = 43.1 ksi

$\dfrac{A_{we}}{L_w}$ = Effective weld area per length

$$= \frac{\sqrt{2}}{2}\left(\frac{1}{4}\text{ in}\right)(2) = 0.3536\text{ in}$$

ϕ =1.0

r_{wCL} = $\phi F_{nwLB}A_{we}/L_w$ = 15.2 k/in.

κ = 1.0

κr_{wCL} = 15.2 k/in.

$\kappa r_{wCL} \geq f_{UF}$

Acceptance Ratio = $f_{UF}/(\kappa r_{wCL})$ = 0.40 ≤ 1.0

Therefore, the gusset-to-beam fillet welds are adequate.

Gusset Plate Shear Yielding (AISC 360-10 Section J4.2)

$F_{yLB,g}$ = Lower-bound gusset plate yield strength
= 52.21 ksi

t_g = Gusset plate thickness
= 0.50 in

ϕ = 1.0

r_{vygCL} = $\phi(0.60F_{yLB,g}t_g)$
= (1.0)(0.60)(52.21 ksi)(0.50 in.)
= 15.7 k/in.

κ = 1.0

κr_{vygCL} = (1.0)(15.7 k/in.) = 15.7 k/in.

f_{vUF} = max$\{f_{v1}, f_{v2}\}$ = max{3.40 k/in., 3.30 k/in.} = 3.40 k/in.

$\kappa r_{vygCL} \geq f_{vUF}$

Acceptance Ratio = $f_{vUF}/(\kappa r_{vygC})$ = 0.22 ≤ 1.0

Therefore, the gusset plate is adequate for shear yielding. By inspection, the gusset plate is also adequate for shear rupture, since ϕ = 1.0 for both shear yielding and rupture and $A_{nv} = A_{gv}$.

Gusset Plate Tensile Yielding (AISC 360-10 Section J4.1)

$F_{yLB,g}$ = 52.21 ksi

ϕ =1.0

r_{tygCL} = $\phi(F_{yLB,g}t_g)$ = 26.1 k/in.

κ = 1.0

κr_{tygCL}= 26.1 k/in.

f_{tUF} $= \max\{ f_{a1} + f_{b1}, f_{a2} + f_{b2} \}$

$= \max \{ 2.01 + 3.01 , 0.00 + 2.92 \}$

$= 5.02$ k/in.

$\kappa r_{tygCL} \geq f_{tUF}$

Acceptance Ratio = $f_{tUF}/(\kappa r_{tygCL}) = 0.19 \leq 1.0$

Therefore, the gusset plate is adequate for tensile yielding. By inspection, the gusset plate is also adequate for tensile rupture, since $\phi = 1.0$ for both tensile yielding and rupture and $A_e = A_g$.

9.7.13.2 Top Gusset Plate Summary, North-South (Y) Direction

The following table summarizes the top gusset plate-to-beam weld acceptance ratios in the north-south direction.

Table 9-22 Top Gusset Plate Connection Acceptance Ratios

Story	f_{vUF} (k/in.)	f_{tUF} (k/in.)	f_{UF} (k/in.)	Weld	Gusset Shear	Gusset Tension
3rd	3.52	5.02	6.08	0.40	0.22	0.19
2nd	3.52	5.02	6.08	0.40	0.22	0.19
1st	3.40	5.02	6.06	0.40	0.22	0.19

9.7.14 Beam Web Local Yielding and Crippling Capacity at Top Gusset Plate (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for local yielding and crippling in the beam web at the brace top gusset (chevron) connection are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical first-story north-south brace top connection to the second-floor beam is provided along with a summary table for north-south brace connections at all stories.

9.7.14.1 First Story Gusset-to-Beam Connection, North-South (Y) Direction

Beam Demands

- Limit State Analysis Case 1 (unbalanced force, see Figure 9-14)

P_{UF} = 121 kips

M_{UF} = 1,805 k in

$N_{UF} = P_{UF} + 4M_{UF} / L_g$ = 121 kips + 4(1,805 k in.)/60 in = 241 kips

- Limit State Analysis Case 2 (equal tension and compression, see Figure 9-15)

  P_{UF} = 0 kips

  M_{UF} = 1,752 k in

  $N_{UF} = P_{UF} + 4M_{UF} / L_g$ = 0 kips + 4(1,752 k in.)/60 in = 117 kips

  Therefore, Limit State Analysis Case 1 governs.

Beam Web Local Yielding Capacity (AISC 360-10 Section J10.2)

$F_{yLB,bm}$ = Lower-bound beam yield strength
= 52.21 ksi

t_w = 0.300 in for W18×35

k = k_{des} = 0.827 in, W18×35 k-region depth

l_b = 60 in, bearing length = gusset plate length

ϕ =1.0

$R_{wyCL} = \phi(F_{yLB,bm}t_w)(5k + l_b)$
= (1.0)(52.21 ksi)(0.300 in.)(5(0.827 in.) + 60 in.)
= 1,005 kips

κ = 1.0

κR_{wyCL} = 1.0(1,005 kips) = 1,005 kips

$\kappa R_{wyCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wyCL}) = 0.24 \leq 1.0$

Therefore, the beam has adequate capacity for web local yielding.

Beam Web Local Crippling Capacity (AISC 360-10 Section J10.3)

t_f = 0.425 in, flange thickness of W18×35

d = 17.7 in, beam depth of W18×35

ϕ = 1.0

$$R_{wyCL} = \phi(0.80t_w^{\ 2})\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yLB,bm}t_f}{t_w}}$$

$$= (1.0)(0.80(0.300)^2)\left[1+3\left(\frac{60}{17.7}\right)\left(\frac{0.300}{0.425}\right)^{1.5}\right]\sqrt{\frac{29{,}000(52.2)(0.425)}{0.300}}$$

= 741 kips

κ = 1.0

κR_{wyCL} = 1.0(741 kips) = 741 kips

$\kappa R_{wyCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wyCL}) = 0.33 \leq 1.0$

Therefore, the beam has adequate capacity for web local crippling.

9.7.14.2 Beam Web Summary at Top Gusset Plate, North-South (Y) Direction

The following table summarizes the beam web local yielding and crippling acceptance ratios in the north-south direction at the top gusset plate connections.

Table 9-23 Beam Web Local Yielding/Crippling at Top Gusset Acceptance Ratios

Story	N_{UF} (kips)		Web Yielding	Web Crippling	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
	Case 1	Case 2			
3rd	240	125	0.24	0.32	OK
2nd	240	125	0.24	0.32	OK
1st	241	117	0.24	0.33	OK

9.7.15 Bottom Gusset Plate Connection Capacity (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for brace bottom gusset plate connection are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical second-story north-south brace bottom connection to the second-floor beam/column joint is provided along with a summary table for north-south brace connections at all stories.

9.7.15.1 Second Story Gusset-to-Beam/Column Connection, North-South (Y) Direction

Connection Demands (Expected Second Story Brace Capacities)

$T_{UF} = T_{LSA} = T_{CE} = 227$ kips

$P_{UF} = P_{LSA} = P_{CE} = 137$ kips

Connection Geometry in accordance with Figure 9-10

Per 2011 AISC Construction Manual Part 13, "Analysis of Existing Diagonal Bracing Connections":

$e_b = d_{bm}/2 = 17.7 \text{ in.}/2 = 8.85 \text{ in.}$

$e_c = d_{\text{col}}/2 = 10.0 \text{ in.}/2 = 5.00 \text{ in.}$

L_{wb} = 23 in.

L_{wc} = 15.56 in.

$\bar{\alpha}$ = 1 in. + L_{wb}/2 = 1 in. + 23 in./2 = 12.5 in.

$\bar{\beta}$ = 1 in. + L_{wc}/2 = 8.78 in

θ = $\tan^{-1}(L_{br,y}/L_{br,z})$ = $\tan^{-1}$(15 ft/12.5 ft) = 0.88 rad = 50.2°

K = $e_b\tan\theta - e_c$ = (8.85 in.)(tan(0.88)) – 5.00 in. = 5.62 in.

$$K' = \bar{\alpha}\left(\tan\theta + \frac{\bar{\alpha}}{\bar{\beta}}\right) = (12.5\text{ in})\left(\tan(0.88) + \frac{12.5\text{ in.}}{8.78\text{ in.}}\right) = 32.8\text{ in.}$$

$$D = \tan^2\theta + \left(\frac{\bar{\alpha}}{\bar{\beta}}\right)^2 = \tan^2(0.88) + \left(\frac{12.5\text{ in.}}{8.78\text{ in.}}\right)^2 = 3.47$$

$$\alpha = \frac{K'\tan\theta + K\left(\bar{\alpha}/\bar{\beta}\right)^2}{D}$$

$$= \frac{(32.8\text{ in.})(\tan(0.88)) + (5.62\text{ in.})\left(12.5\text{ in.}/8.78\text{ in.}\right)^2}{3.47}$$

= 14.64 in.

$$\beta = \frac{K' - K\tan\theta}{D} = \frac{32.8\text{ in.} - (5.62\text{ in.})(\tan(0.88))}{3.47} = 7.51\text{ in.}$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

$$= \sqrt{(14.64\text{ in.} + 5.00\text{ in.})^2 + (7.51\text{ in.} + 8.85\text{ in.})^2}$$

= 25.56 in.

Gusset/Beam/Column Interface Forces

V_c = $(\beta/r)T_{UF}$ = (7.51 in.)/(25.56 in.)(227 kips) = 67 kips

H_c = $(e_c/r)T_{UF}$ = (5.00 in.)/(25.56 in.)(227 kips) = 44 kips

M_c = $H_c(\beta - \bar{\beta})$ = (44 kips)(7.51 in. – 8.78 in.) = -56 k-in.

V_b = $(e_b/r)T_{UF}$ = (8.85 in.)/(25.56 in.)(227 kips) = 79 kips

H_b = $(\alpha/r)T_{UF}$ = (14.64 in.)/(25.56 in.)(227 kips) = 130 kips

M_b = $V_b(\alpha - \bar{\alpha})$ = (79 kips)(14.64 – 12.5) = 169 k-in.

Gusset Plate-to-Column Weld

- Weld Demands

  L_{weld} = Gusset plate-to-column weld length

  = L_{wc} = 15.56 in.

S_{weld} = Weld section modulus

$= L_{\text{weld}}^2/6 = 40.4$ in.^{2}

$f_a = H_c/L_{\text{weld}}$ = (44 kips)/(15.56 in.) = 2.83 k/in.

$f_v = V_c/L_{\text{weld}}$ = (67 kips)/(15.56 in.) = 4.31 k/in.

$f_b = M_c/\text{S}_{\text{weld}}$ = (56 k-in.)/(40.4 in.^{2}) = 1.39 k/in.

$f_{\text{peak}} = \sqrt{f_v^2 + (f_a + f_b)^2}$ = 6.03 k/in.

$f_{\text{avg}} = \text{avg}\left\{\sqrt{f_v^2 + \max\{0,(f_a \pm f_b)\}^2}\right\}$ = 5.29 k/in.

$f_{UF} = \max\left\{f_{\text{peak}}, 1.25 f_{\text{avg}}\right\} = 1.25 f_{\text{avg}}$ = 6.61 k/in.

- Fillet Weld Shear Capacity (AISC 360-10 Section J2.4)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of a 1/4" double-sided fillet weld is:

  $\kappa r_{wCL} = \kappa\phi F_{nwLB} A_{we}/L_w$ = 15.2 k/in.

  $\kappa r_{wCL} \geq f_{UF}$

  Acceptance Ratio = $f_{UF}/(\kappa r_{wCL}) = 0.43 \leq 1.0$

  Therefore, the gusset plate-to-column weld is adequate.

- Gusset Plate Shear Yielding (AISC 360-10 Section J4.2)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of the 1/2” gusset plate is:

  $\kappa r_{vygCL} = \kappa\phi(0.60 F_{yLB,g} t_g)$ = 15.7 k/in.

  $f_{vUF} = f_v$ = 4.31 k/in.

  $\kappa r_{vygCL} \geq f_{UF}$

  Acceptance Ratio = $f_{UF}/(\kappa r_{vygCL}) = 0.27 \leq 1.0$

  Therefore, the gusset plate is adequate for shear yielding.

- Gusset Plate Tensile Yielding (AISC 360-10 Section J4.1)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of the 1/2” gusset plate is:

  $\kappa r_{tygCL} = \kappa\phi(F_{yLB,g} t_g)$ = 26.1 k/in.

  $f_{tUF} = f_a + f_b$ = 2.83 k/in. + 1.39 k/in. = 4.22 k/in.

  $\kappa r_{tygCL} \geq f_{tUF}$

  Acceptance Ratio = $f_{tUF}/\kappa r_{tygCL} = 0.16 \leq 1.0$

Therefore, the gusset plate is adequate for tensile yielding.

Gusset Plate-to-Beam Weld

- Weld Demands

  $L_{\text{weld}} = L_{wb} = 23$ in.

  $S_{\text{weld}} = L^2_{\text{weld}}/6 = 88.2$ in.^{2}

  $f_a \quad = V_b/L_{\text{weld}} = (79 \text{ kips})/(23 \text{ in.}) = 3.43$ k/in.

  $f_v \quad = H_b/L_{\text{weld}} = (130 \text{ kips})/(23 \text{ in.}) = 5.65$ k/in.

  $f_b \quad = M_b/S_{\text{weld}} = (169 \text{ k-in.})/(88.2 \text{ in.}^2) = 1.92$ k/in.

  $f_{\text{peak}} = \sqrt{f_v^{\,2} + (f_a + f_b)^2} = 7.78$ k/in.

  $f_{\text{avg}} = \text{avg}\left\{\sqrt{f_v^{\,2} + \max\{0,(f_a \pm f_b)\}^2}\right\} = 6.81$ k/in.

  $f_{UF} = \max\left\{f_{\text{peak}}, 1.25 f_{\text{avg}}\right\} = 1.25 f_{\text{avg}} = 8.51$ k/in.

- Fillet Weld Shear Capacity (AISC 360-10 Section J2.4)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of a 1/4" double-sided fillet weld is:

  $\kappa r_{wCL} \quad = \kappa\phi F_{nwLB} A_{we}/L_w = 15.2$ k/in.

  $\kappa r_{wCL} \quad \geq f_{UF}$

  Acceptance Ratio $= f_{UF}/(\kappa r_{wCL}) = 0.56 \leq 1.0$

  Therefore, the gusset plate-to-beam weld is adequate.

- Gusset Plate Shear Yielding (AISC 360-10 Section J4.2)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of the 1/2" gusset plate is:

  $\kappa r_{vygCL} = \kappa\phi(0.60 F_{yLB,g} t_g) = 15.7$ k/in.

  $f_{tUF} \quad = f_v = 5.65$ k/in.

  $\kappa r_{vygCL} \geq f_{vUF}$

  Acceptance Ratio $= f_{vUF}/(\kappa r_{vygCL}) = 0.36 \leq 1.0$

  Therefore, the gusset plate is adequate for shear yielding.

- Gusset Plate Tensile Yielding (AISC 360-10 Section J4.1)

  Per the previous calculations for gusset plate-to-beam weld, the capacity of the 1/2" gusset plate is:

$\kappa r_{tygCL} = \kappa\phi(F_{yLB,g}t_g) = 26.1$ k/in.

$f_{tUF} \quad = f_a + f_b = 3.43$ k/in. + 1.92 k/in. = 5.35 k/in.

$\kappa r_{tygCL} \geq f_{tUF}$

Acceptance Ratio = $f_{tUF}/\kappa r_{tygCL} = 0.20 \leq 1.0$

Therefore, the gusset plate is adequate for tensile yielding.

9.7.15.2 Bottom Gusset Plate Connection Summary, North-South (Y) Direction

The following table summarizes the gusset plate-to-beam/column/base plate connection acceptance ratios in the north-south direction.

Table 9-24 Bottom Gusset Plate Connection Acceptance Ratios

Story	Gusset Conn.	T_{UF} (kips)	Weld	Gusset Shear	Gusset Tension	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	Column	227	0.43	0.27	0.16	OK
	Beam	227	0.56	0.36	0.20	OK
2nd	Column	227	0.43	0.27	0.16	OK
	Beam	227	0.56	0.36	0.20	OK
1st	Column	227	0.72	0.50	0.17	OK
	Base PL	227	0.62	0.48	0.02	OK

9.7.16 Beam Web Local Yielding and Crippling Capacity at Bottom Gusset Plate (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for local yielding and crippling within the beam web at the brace bottom gusset connection to the beam/column joint is analyzed as a force-controlled action in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical second-story north-south brace bottom connection to the second-floor beam/column joint is provided along with a summary table for north-south brace connections at all stories.

9.7.16.1 Second Story Gusset-to-Beam Connection, North-South (Y) Direction

Beam Demands

P_{UF} = Vertical connection force at beam

$= V_b = 79$ kips

M_{UF} = Connection moment at beam

$= M_b = 168$ k-in.

N_{UF} = Effective vertical load at beam web

$= P_b + 4M_{UF}/L_{wb}$

$= 79$ kips + 4(168 k in.)/23 in

$= 108$ kips

Beam Web Local Yielding Capacity (AISC 360-10 Section J10.2)

$F_{yLB,bm}$ = 52.21 ksi

t_w = 0.300 in., beam thickness for W18×35

k = beam k-region depth W18×35

$= k_{des} = 0.827$ in.

l_b = Bearing length = gusset plate length along beam

$= L_{wb} = 23$ in.

ϕ =1.0

$R_{wyCL} = \phi(F_{yLB,bm}t_w)(2.5k + l_b)$

= (1.0)(52.21 ksi)(0.300 in.)(2.5(0.827 in.) + 23 in.)

= 393 kips

κ = 1.0

κR_{wyCL} = (1.0)(393 kips) = 393 kips

$\kappa R_{wyCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wyCL}) = 0.27 \leq 1.0$

Therefore, the beam has adequate capacity for web local yielding.

Beam Web Local Crippling Capacity (AISC 360-10 Section J10.3)

t_f = 0.425 in, beam flange thickness for W18×35

d = 17.7 in, beam web depth for W18×35

ϕ = 1.0

$$R_{wcCL} = \phi(0.80t_w{}^2)\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yLB,bm}t_f}{t_w}}$$

= 349 kips

κ = 1.0

κR_{wcCL} = 349 kips

$\kappa R_{wcCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wcCL}) = 0.31 \leq 1.0$

Therefore, the beam has adequate capacity for web local crippling.

9.7.16.2 Beam Web Summary at Bottom Gusset Plate, North-South (Y) Direction

The following table summarizes the beam web local yielding and crippling acceptance ratios in the north-south direction at the bottom gusset plate connections.

Table 9-25 Beam Web Local Yielding/Crippling at Bottom Gusset Acceptance Ratios

Story	N_{UF} (kips)	Web Yielding	Web Crippling	$Q_{UF}/(\kappa Q_{CL}) \le 1.0$
3rd	108	0.27	0.31	OK
2nd	108	0.27	0.31	OK
1st	NA	NA	NA	NA

9.7.17 Column Web Local Yielding and Crippling Capacity at Bottom Gusset Plate (ASCE 41-13 § 9.5.2.4.1)

In the following section, acceptance criteria for local yielding and crippling within the column web at the brace bottom gusset connection to the beam/column joint are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.5.2.4.1. A detailed calculation for the typical second-story north-south brace bottom connection to the second-floor beam/column joint is provided along with a summary table for north-south brace connections at all stories.

9.7.17.1 Second Story Gusset-to-Column Connection, North-South (Y) Direction

Column Demands

P_{UF} = Horizontal connection force at column

= H_c = 44 kips

M_{UF} = Connection moment at column

= M_c = 56 k/in.

N_{UF} = Effective horizontal force at column

= $P_{UF} + 4M_{UF}/L_{wc}$

= 44 kips + 4(56 k in.)/(15.56 in.)

= 58 kips

Column Web Local Yielding Capacity (AISC 360-10 Section J10.2)

$F_{yLB,c}$ = 51.2 ksi

t_w = 0.340 in, column web thickness for W10×49

k = k_{des} = 1.06 in, column k-region depth for W10×49

l_b = Bearing length = gusset length along column
= L_{wc} = 15.56 in

ϕ =1.0

R_{wyCL} = $\phi(F_{yLB,c}t_w)(5k + l_b)$
= (1.0)(51.2 ksi)(0.340 in.)(5(1.06 in.) + 15.56 in.)
= 363 kips

κ = 1.0

κR_{wyCL} = 363 kips

$\kappa R_{wyCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wyCL})$ = 0.16 ≤ 1.0

Therefore, the column has adequate capacity for web local yielding.

Column Web Local Crippling Capacity (AISC 360-10 Section J10.3)

t_f = 0.560 in, column flange thickness for W10×49

d = 10.0 in, column depth for W10×49

ϕ = 1.0

$$R_{wcCL} = \phi(0.80t_w{}^2)\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yLB,c}t_f}{t_w}} = 464 \text{ kips}$$

κ = 1.0

κR_{wcCL} = 464 kips

$\kappa R_{wcCL} \geq N_{UF}$

Acceptance Ratio = $N_{UF}/(\kappa R_{wcCL})$ = 0.13 ≤ 1.0

Therefore, the column has adequate capacity for web local crippling.

9.7.17.2 Column Web Summary at Bottom Gusset Plate, North-South (Y) Direction

The following table summarizes the column web local yielding and crippling acceptance ratios in the north-south direction at the bottom gusset plate connections.

Story	N_{UF} (kips)	Web Yielding	Web Crippling	$Q_{UF}/(\kappa Q_{CL}) \leq 1.0$
3rd	59	0.16	0.13	OK
2nd	59	0.16	0.13	OK
1st	74	0.21	0.15	OK

Table 9-26 Column Web Local Yielding/Crippling Acceptance Ratios

9.7.18 Column Compression Capacity (ASCE 41-13 § 9.4.2.4.2)

In the following section, column compression acceptance criteria are analyzed as force-controlled actions in accordance with ASCE 41-13 § 9.4.2.4.2 and § 9.5.2.4.2. A detailed calculation for a typical first-story north-south column is provided along with a summary table of acceptance criteria for north-south columns at all stories.

9.7.18.1 First Story Column, North-South (Y) Direction

Column Demands

Column compression is a force-controlled action per ASCE 41-13 § 9.5.2.3.2 and § 9.4.2.4.2.2.

- Gravity Demands: Per analysis model

P_D = Column axial dead load
= 117 kips

P_L = Column axial live load
= 52 kips

P_S = Column axial snow load
= 0 kips

$P_{G,\max} = 1.1(P_D + 0.25\,P_L + P_S) = 143$ kips (ASCE 41-13 Eq. 7-1)

$P_{G,\min} = 0.9P_D = 105$ kips (ASCE 41-13 Eq. 7-2)

- Limit State Analysis Case 1: See Figure 9-14

$T_{CE1} = T_{CE2} = T_{CE3}$ = Brace tension force at stories 1-3
= 227 kips

$0.3P_{CE1}$ = Brace compression at story 1
= 0.3(129 kips)
= 39 kips

$0.3P_{CE2} = 0.3P_{CE3}$ = Brace compression at stories 2-3
= 0.3(137 kips)
= 41 kips

ASCE 41-17 Revision

ASCE 41-17 Chapter 9 has updated acceptance criteria for columns for both linear and nonlinear procedures. The nonlinear modeling parameters were also updated to typically be less conservative for higher axial forces. These updates also apply to frame beams that are treated as beam-columns. Use of ASCE 41-17 for columns checks is recommended.

Commentary

Per ASCE 41-13 Equations 9-10, 9-11, and 9-12, column compression must be evaluated as a force-controlled action.

V_{BM1} = Beam shear at story 1

$= \frac{1}{2}(T_{CE1} - 0.3P_{CE1})(L_{br,z}/L_{br})$

$= \frac{1}{2}(227 \text{ kips} - 39 \text{ kips})(12.5 \text{ ft}) / (19.53 \text{ ft})$

= 60 kips

V_{BM2} = V_{BM3} = Beam shear at stories 2-3

$= \frac{1}{2}(T_{CE3} - 0.3P_{CE3})(L_{br,z}/L_{br})$

$= \frac{1}{2}(227 \text{ kips} - 41 \text{ kips})(12.5 \text{ ft}) / (19.53 \text{ ft})$

= 60 kips

P_{LSA} = $(0.3P_{CE3} + 0.3P_{CE2})(L_{br,z}/L_{br})+(V_{BM3} + V_{BM2} + V_{BM1})$

= (41 k + 41 k)(12.5 ft)/(19.53 ft) + 60 k + 60 k + 60 k

= 232 kips

- Limit State Analysis Case 2: See Figure 9-15

T_1 = Brace tension at story 1

= P_{CE1} = 129 kips

T_2 = T_3 = P_{CE2} = P_{CE3} = Brace tension at stories 2-3

= 137 kips

P_{CE1} = Brace compression at story 1

= 129 kips

P_{CE2} = P_{CE3} = Brace compression at stories 2-3

= 137 kips

$V_{BM1} = \frac{1}{2}(T_1 - P_{CE1})(L_{br,z}/L_{br}) = 0$ kips

$V_{BM2} = V_{BM3} = \frac{1}{2}(T_3 - P_{CE3})(L_{br,z}/L_{br}) = 0$ kips

P_{LSA} = $(P_{CE3} + P_{CE2})(L_{br,z}/L_{br})+(V_{BM3} + V_{BM2} + V_{BM1})$

= (137 k + 137 k)(12.5 ft)/(19.53 ft) + 0 k + 0 k + 0 k

= 175 kips

- Factored Force-Controlled Column Demands

Limit State Analysis Case 1 governs.

P_{LSA} = 232 kips

$P_{UF} = P_{G,\max} + P_{LSA}$ = 375 kips

Second-order effects are not considered, since the column demands are derived using limit state analysis procedures. (Refer to 2011 *AISC Seismic Design Manual* (AISC, 2011) pg. 5-102)

Column Flange Local Buckling (AISC 360-10 Table B4.1a)

$\frac{b_f}{2t_f}$ = 8.93 for W10×49

λ_r $= 0.56\sqrt{E / F_{yLB,c}} = 0.56\sqrt{29{,}000 \text{ ksi} / 51.2 \text{ ksi}}$ = 13.3

$\frac{b_f}{2t_f} < \lambda_r$, therefore column flanges are "nonslender" for compression

Column Web Local Buckling (AISC 360-10 Table B4.1a)

$\frac{h}{t_w}$ = 23.1 for W10×49

λ_r $= 1.49\sqrt{E / F_{yLB,c}} = 1.49\sqrt{29{,}000 \text{ ksi} / 51.16 \text{ ksi}}$ = 35.5

$\frac{h}{t_w} < \lambda_r$, therefore column web is "nonslender" for compression

Column Compression Capacity (AISC 360-10 Section E3 and Section E4)

L_x = 12.5 ft, flexural buckling about local x-axis)

L_y = 12.5 ft, flexural buckling about local y-axis)

L_z = 12.5 ft, flexural-torsional buckling)

K_x = 1.0, flexural buckling about local x-axis)

K_y = 1.0, flexural buckling about local y-axis)

K_z = 1.0, flexural-torsional buckling)

Critical Flexural Buckling Stress

$$\frac{KL}{r} = \max\left\{\frac{K_x L_x}{r_x}, \frac{K_y L_y}{r_y}\right\} = \frac{K_y L_y}{r_y} = \frac{1.0(12.5 \text{ ft})(12 \text{ in/ft})}{2.54 \text{ in}}$$

$$= 59.1 \le 4.71\sqrt{E / F_{yLB,c}} = 112$$

$$F_{e,FB} = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29{,}000)}{(59.1)^2} = 82.1 \text{ ksi} \qquad \text{(AISC 360-10 Eq. E3-4)}$$

$$F_{cr,FB} = \left[0.658^{F_{yLB,c}/F_{e,FB}}\right] F_{yLB,c} = 39.41 \text{ ksi} \qquad \text{(AISC 360-10 Eq. E3-2)}$$

Critical Flexural-Torsional Buckling Stress

$$F_{e,FB} = \left(\frac{\pi^2 EC_w}{(K_z L_z)^2} + GJ\right)\left(\frac{1}{I_x + I_y}\right) = 115 \text{ ksi}$$

E = 29,000 ksi

G = 11,200 ksi

K_zL_z = (1.0)(12.5 ft)(12in / ft) = 150 in

I_x = 272 in^4 for W10×49

I_y = 93.4 in^4 for W10×49

C_w = 2070 in^6 for W10×49

J = 1.39 in^4 for W10×49

$$\frac{F_{yLB,c}}{F_{e,FTB}} = \frac{51.16 \text{ ksi}}{114.7 \text{ ksi}} = 0.446 \le 2.25$$

$$F_{cr,FTB} = \left[0.658^{F_{yLB,c}/F_{e,FTB}}\right] F_{yLB,c} = 42.45 \text{ ksi}$$

Column Compression Capacity

$F_{cr,LB}$= min{ $F_{cr,FB}$, $F_{cr,FTB}$ } = $F_{cr,FB}$ = 39.41 ksi

A_g = 14.4 in.^{2} for W10×49

ϕ = 1.0

P_{CL} = $\phi F_{cr,LB} A_g$ = (1.0)(39.41 ksi)(14.4 in.^{2}) = 568 kips

Column Compression Acceptance Criteria (ASCE 41-13 § 9.5.2.4)

κ = 1.0

κP_{CL} = (1.0)(568 kips) = 568 kips

P_{UF} = 375 kips

$\kappa P_{CL} \ge P_{UF}$

Acceptance Ratio = $P_{UF}/(\kappa P_{CL})$ = 0.66 ≤ 1.00

Therefore, the column is adequate for compression.

> **Commentary**
>
> Braced frame column moments due to inter-story drifts should also be evaluated in accordance with ASCE 41-13 Section 9.5.2.3.2 but is neglected in this example for brevity.

9.7.18.1 Column Compression Summary, North-South (Y) Direction

Table 9-27 summarizes the column compression acceptance ratios:

Table 9-27 Column Compression Acceptance Ratios

Story	P_{UF} (kips)		Acceptance Ratios		$Q_{UF}/(\kappa Q_{CL}) \le 1.0$
	Case 1	Case 2	Case 1	Case 2	
3rd	96	37	0.17	0.06	OK
2nd	234	176	0.41	0.31	OK
1st	375	319	0.66	0.56	OK

Commentary

Per ASCE 41-13 Equation 9-13, column tension may be evaluated as a deformation-controlled action.

9.7.19 Column Tension Capacity (ASCE 41-13 § 9.4.2.4.2)

In the following section, column tension acceptance criteria are analyzed as deformation-controlled actions in accordance with ASCE 41-13 § 9.4.2.4.2.2. A detailed calculation for a typical first-story north-south column is provided along with a summary table of acceptance criteria for north-south columns at all stories.

9.7.19.1 First Story Column, North-South (Y) Direction

Column Demands

Column tension is a deformation-controlled action per ASCE 41-13 § 9.5.2.3.2 and § 9.4.2.4.2.2.

- Demands per Analysis Model

P_D = Column axial dead load
= 117 kips

$P_{G,\min} = 0.9P_D$ (ASCE 41-13 Eq. 7-2)
= (0.9)(117 kips) = 105 kips

P_E = Column axial earthquake load
= 233 kips

- Factored Deformation-Controlled Demand

T_{UD} = Factored column tension
$= | P_{G,\min} - P_E | = 128$ kips

- Column Tension Capacity (ASCE 41-13 § 9.4.2.3.2.2)

$F_{ye,c}$ = Expected column yield strength per testing
= 53.1 ksi

A_g = 14.4 in.^{2} for W10x49

$T_{CE} = A_g F_{ye,c} = (53.1 \text{ ksi})(14.4 \text{ in.}^2) = 765$ kips

- Component Modification Factor (ASCE 41-13 Table 9-4)

Per ASCE 41-13 Table 9-4, for columns in tension at the IO performance level,

$m = 1.25$

- Column Tension Acceptance Criteria (ASCE 41-13 § 9.5.2.4):

$\kappa = 1.0$

$m\kappa T_{CE} = (1.25)(1.0)(765 \text{ kips}) = 956 \text{ kips}$

$m\kappa T_{CE} \geq T_{UD}$

$Q_{UD}/(\kappa Q_{CE}) = T_{UD}/(\kappa T_{CE}) = 0.17 \leq m = 1.25$

Therefore, the column is adequate for tension.

9.7.19.2 Column Tension Summary, North-South (Y) Direction

The following table summarizes the column tension acceptance criteria:

Table 9-28 Column Tension Acceptance Criteria

Story	T_{UD} (kips)	κT_{CE} (kips)	$Q_{UD}/(\kappa Q_{CE})$	m	$Q_{UD}/(\kappa Q_{CE}) \leq m$
3rd	0	765	0.00	1.25	OK
2nd	23	765	0.03	1.25	OK
1st	128	765	0.17	1.25	OK

9.7.20 Column Splice Tension Capacity (ASCE 41-13 § 9.5.2.4.1)

9.7.20.1 Column Splice at Second Story, North-South (Y) Direction

Column Demands

Column splice connections are evaluated as force-controlled per ASCE 41-13 § 9.5.2.4.1 for two limit state analysis cases, as shown in Figure 9-16 and Figure 9-17. The column splice detail is shown in Figure 9-18.

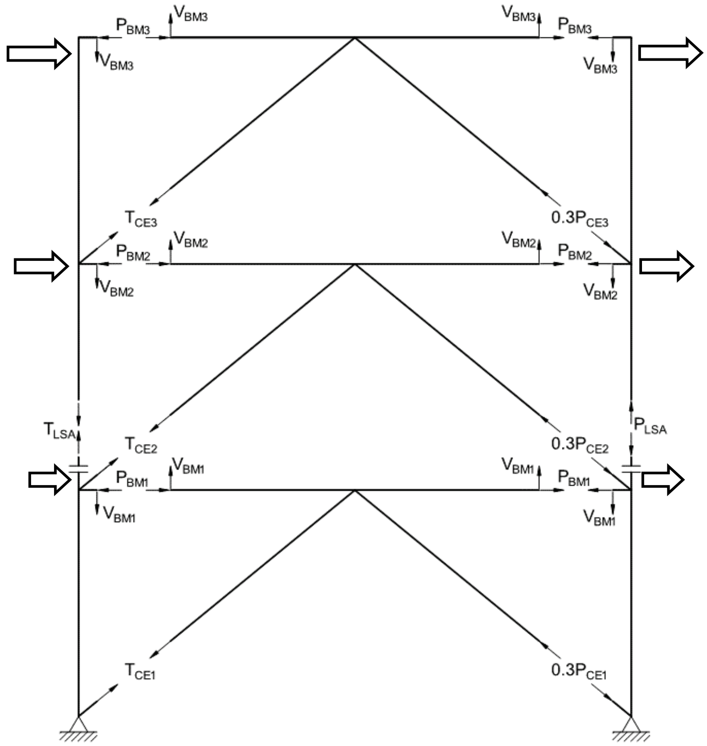


Figure 9-16 Column splice demands: Limit State Analysis Case 1.

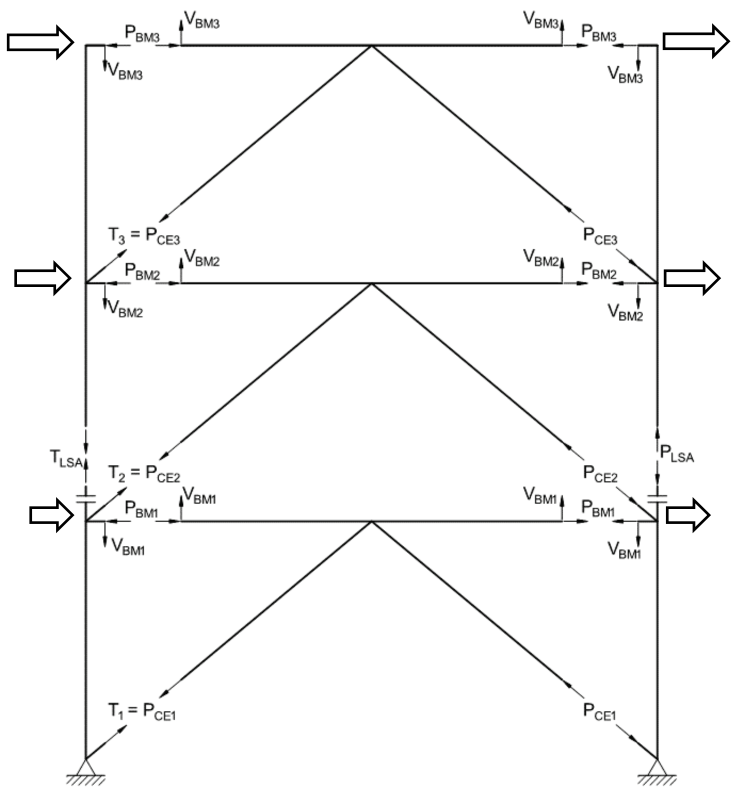


Figure 9-17 Column splice demands: Limit State Analysis Case 2.

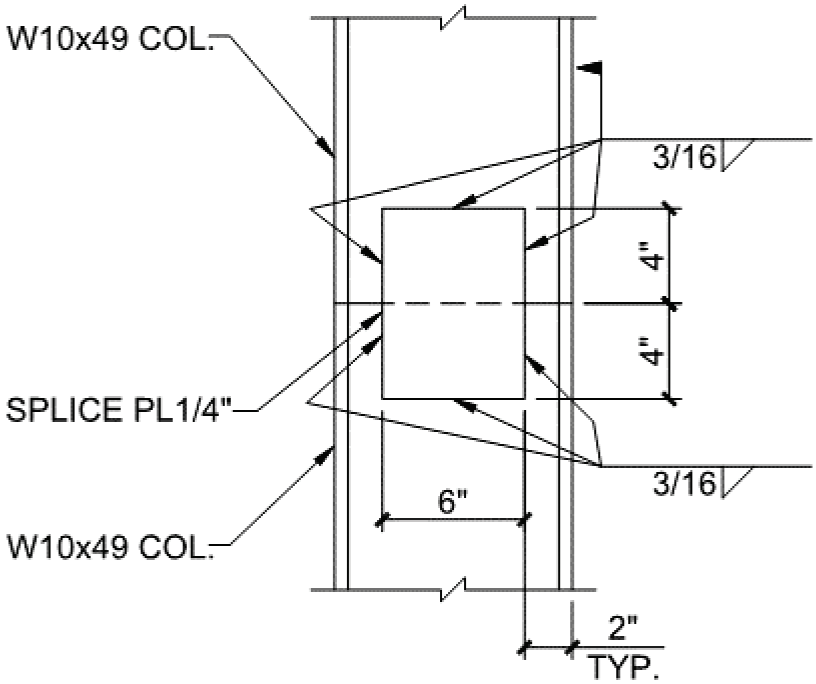


Figure 9-18 Column splice detail.

- Gravity Demands: Per Analysis Model

P_D = 73 kips

$P_{G,\min} = 0.9P_D = 66$ kips

- Limit State Analysis Case 1: See Figure 9-16

T_{CE3} = T_{CE2} = Brace tension at Stories 2-3
= 227 kips

$0.3P_{CE3} = 0.3P_{CE2}$ = Brace compression at Stories 2-3
= 0.3(137 kips)
= 41 kips

V_{BM3} = V_{BM2} = Beam shear at Stories 2-3

$= \frac{1}{2}(T_{CE3} - 0.3P_{CE3})(L_{br,z}/L_{br})$

$= \frac{1}{2}$(227 kips – 41 kips)(12.5 ft)/(19.53 ft)

= 60 k

$T_{LSA} = T_{CE3}(L_{br,z}/L_{br}) - V_{BM3} - V_{BM2}$
= (227 kips)(12.5 ft)/(19.53 ft) – 60 kips – 60 kips
= 25 kips

- Limit State Analysis Case 2: See Figure 9-17

  T_3 = T_2 = Brace tension at Stories 2-3
  = 137 kips

  P_{CE3} = P_{CE2} = Brace compression at Stories 2-3
  = 137 kips

  V_{BM3} = V_{BM2} = Beam shear at Stories 2-3
  $= \frac{1}{2}(T_3 - P_{CE3})(L_{br,z}/L_{br}) = 0$ k

  T_{LSA} $= T_3(L_{cr,z}/L_{br}) - V_{BM3} - V_{BM2}$
  = (137 kips)(12.5 ft)/(19.53 ft) – 2(0 kips)
  = 88 kips

- Factored Force-Controlled Column Splice Demands

  Limit State Analysis Case 2 governs.

  T_{LSA}= 88 kips

  $T_{UF} = | P_{G,\min} - T_{LSA} |$ = | 66 kips – 88 kips | = 22 kips

Splice Weld Capacity (AISC 360-10 Section J2.4) See Figure 9-18

F_{EXX} = 71.8 ksi, lower-bound per testing

$F_{nwLB} = 0.60F_{EXX}$ = 43.1 ksi

L_{wl} = Longitudinal welds
= (4 in/weld)(2 welds) = 8 in

L_{wt} = Transverse weld
= 6 in

$$R_{nwl} = 43.1\text{ ksi}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{16}\text{ in}\right)(8\text{ in}) = 45.7\text{ kips}$$

$$R_{nwt} = 43.1\text{ ksi}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{3}{16}\text{ in}\right)(6\text{ in}) = 34.3\text{ kips}$$

R_{nw} $= \max\{R_{nwl} + R_{nwt}, 0.85R_{nwl} + 1.5R_{nwt}\}$
= max{45.7 kips + 34.3 kips, 0.85(45.7 kips) + 1.5(34.3 kips)}
= max{80 kips , 90 kips}
= 90 kips

ϕ = 1.0

$R_{wCL} = \phi R_{nw}$ = (1.0)(90 kips) = 90 kips

κ = 1.0

κR_{wCL} = (1.0)(90 kips) = 90 kips

$\kappa R_{wCL} \geq T_{UF}$

Acceptance Ratio = $T_{UF}/(\kappa R_{wCL})$ = 0.24 ≤ 1.0

Therefore, the splice weld is adequate.

Splice Plate Tensile Yielding Capacity (AISC 360-10 Section J4.1)

$F_{yLB,p}$ = 52.21 ksi, splice plate lower-bound yield strength

A_g = Gross plate area
= (0.25 in.)(6 in.) = 1.5 in.^{2}

ϕ = 1.0

$R_{ypCL} = \phi(F_{yLB,p}A_g)$ = (1.0)(52.21 ksi)(1.5 in.^{2}) = 78 kips

κ = 1.0

κR_{ypCL} = (1.0)(78 kips) = 78 kips

$\kappa R_{ypCL} \geq T_{UF}$

Acceptance Ratio = $T_{UF}/(\kappa R_{ypCL})$ = 0.28 ≤ 1.0

Therefore, the splice plate is adequate for tensile yielding.

Column Web Block Shear Capacity (AISC 360-10 Section J4.3)

- Shear Yielding

$0.60F_{yLB,c}A_{gv}$ = Shear yielding

where:

$F_{yLB,c}$ = 51.16 ksi

t_w = 0.340 in

A_{gv} = 2(4 in.)(0.340 in.) = 2.72 in.^{2}

$0.60F_{yLB,c}A_{gv}$ = 83 kips

- Shear Rupture

$0.60F_{uLB,c}A_{nv}$ = Shear rupture

where:

$F_{uLB,c}$ = 71.53 ksi

A_{nv} = 2(4 in.)(0.340 in.) = 2.72 in.^{2}

$0.60F_{uLB,c}A_{nv}$ = 117 kips

- Tensile Rupture

$U_{bs}F_{uLB,c}A_{nt}$ = Tensile rupture

where:

U_{bs} = 1.0

A_{nt} = (6 in.)(0.340 in.) = 2.04 in.^{2}

$U_{bs}F_{uLB,c}A_{nt}$ = 146 kips

- Block Shear Capacity

ϕ = 1.0

$R_{bs,CL} = \phi[\min\{0.60F_{uLB,c}A_{nv}, 0.60F_{uLB,c}A_{gv}\} + U_{bs}F_{uLB,c}A_{nt}]$
= 1.0[min{117 kips, 83 kips} + 146 kips]
= 1.0[83 kips + 146 kips]
= 229 kips

κ = 1.0

$\kappa R_{bs,CL}$ = (1.0)(229 kips) = 229 kips

$\kappa R_{bs,CL} \geq T_{UF}$

Acceptance Ratio = $T_{UF}/(\kappa R_{bs,CL}) = 0.10 \leq 1.00$

Therefore, the column web has adequate block shear capacity.

9.7.21 Foundation Capacity and Acceptance Criteria

Chapter 5 of this *Guide* presents detailed discussion of foundations. Assuming the building is not sensitive to base rotations or other types of foundation movement that would cause the structural components to exceed their acceptance criteria, the foundation is modelled as a fixed-base, thus the provisions of ASCE 41-13 § 8.4.2.3.2.1 apply. The following calculations evaluate the typical 6'-0" square, 3'-0" deep spread footings below the north-south braced frame columns. As there are no data collection requirements for Chapter 8 of ASCE 41-13, κ is not included in the evaluation. Refer to Figure 9-19 for the foundation layout.

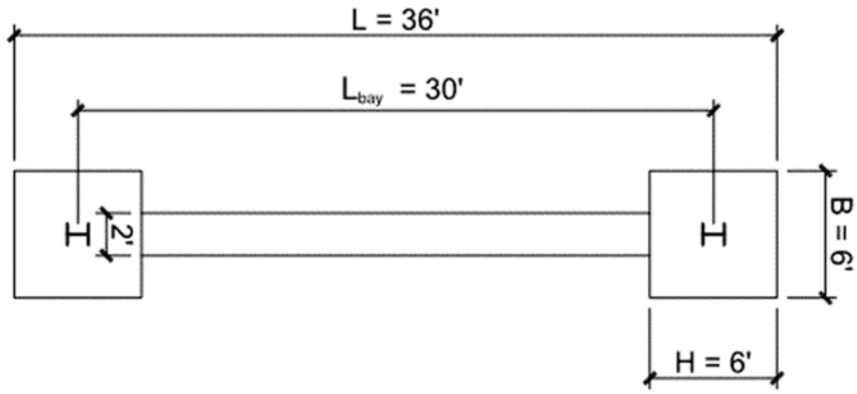


Figure 9-19 Foundation diagram.

9.7.21.1 Typical North-South (Y) Direction Frame

Foundation Demands (Per Analysis Model)

$P_{D,\text{col}}$ = Structure dead load
= 123 kips

$P_{D,SOG}$ = Slab on grade dead load
= 150 pcf(4ft)(6 ft)(6 ft) = 2 kips

$P_{D,\text{soil}}$ = Soil dead load
= 120 pcf(8 ft)(6 ft)(6 ft) = 3 kips

$P_{D,\text{footing}}$ = Footing dead load
= 150 pcf(6 ft)(6 ft)(3 ft) = 16 kips

P_D = Total dead load
= $P_{D,\text{col}} + P_{D,SOG} + P_{D,\text{soil}} + P_{D,\text{footing}}$ = 144 kips

P_L = Live load
= 54 kips

P_S = Snow load
= 0 kips

$P_{G,\max}$ = Gravity loads
= $1.1(P_D + 0.25P_L + P_S)$ = 173 kips (ASCE 41-13 Eq. 7-1)

$P_{G,\min}$ = $0.9P_D$ = 130 kips (ASCE 41-13 Eq. 7-2)

P_E = Total vertical earthquake load at foundation
= 405 kips

Foundation Uplift (ASCE 41-13 § 8.4.2.3.2.1)

m = 1.5 for IO Performance Level

$mP_D = 216$ kips

$mP_D < P_E$

$P_E/P_D = 2.81 > m = 1.5$

Therefore, there is potential for increased deformation at the soil-foundation interface that could affect the gravity framing system or through transfer of load to other seismic-force resisting elements. In lieu of repeating the linear analysis with foundation flexibility, foundation flexibility is considered in the Nonlinear Static Procedure analysis in the following section. To finalize the evaluation at this stage, a flexible-base should be considered for Immediate Occupancy Performance Level, and is required if the system is sensitive to base rotations per ASCE 41-13 § 8.4.2.3.2.1. Although a low-rise braced frame is not expected to be as sensitive to base rotations, a parametric study to understand the impact from a flexible-base is suggested.

Foundation Bearing Pressure (ASCE 41-13 § 8.4.1.1 and § 8.4.2.3.2.1)

The as-built drawings indicate that the allowable dead plus live bearing pressure of 3,000 psf for the 6-foot square spread footings. Therefore, the prescriptive expected bearing pressure per ASCE 41-13 § 8.4.1.1 can be calculated as follows:

$q_{CE} = 3q_{allow} = 3(3{,}000 \text{ psf}) = 9{,}000 \text{ psf}$

$m = 1.5$ for IO Performance Level

$mq_{CE} = 13{,}500$ psf

$B = H =$ Footing length and width

$= 6$ ft

$$q_{UD} = \frac{P_{UD}}{BH} = \frac{P_{G,\max} + P_E}{BH}$$

$$= \frac{(173 \text{ kips} + 405 \text{ kips})(1{,}000 \text{ lb/kip})}{(6 \text{ ft})(6 \text{ ft})}$$

$= 16{,}056$ psf

$mq_{CE} < q_{UD}$

$q_{UD}/q_{CE} = 1.78 > m = 1.5$

Therefore, the foundation bearing pressure exceeds the acceptance criteria established for fixed-base linear analysis. As described above, further analysis is required.

Overturning Stability (ASCE 41-13 §7.2.8.1)

The global overturning moment on each frame is back-calculated from the vertical earthquake reaction at the base each frame column:

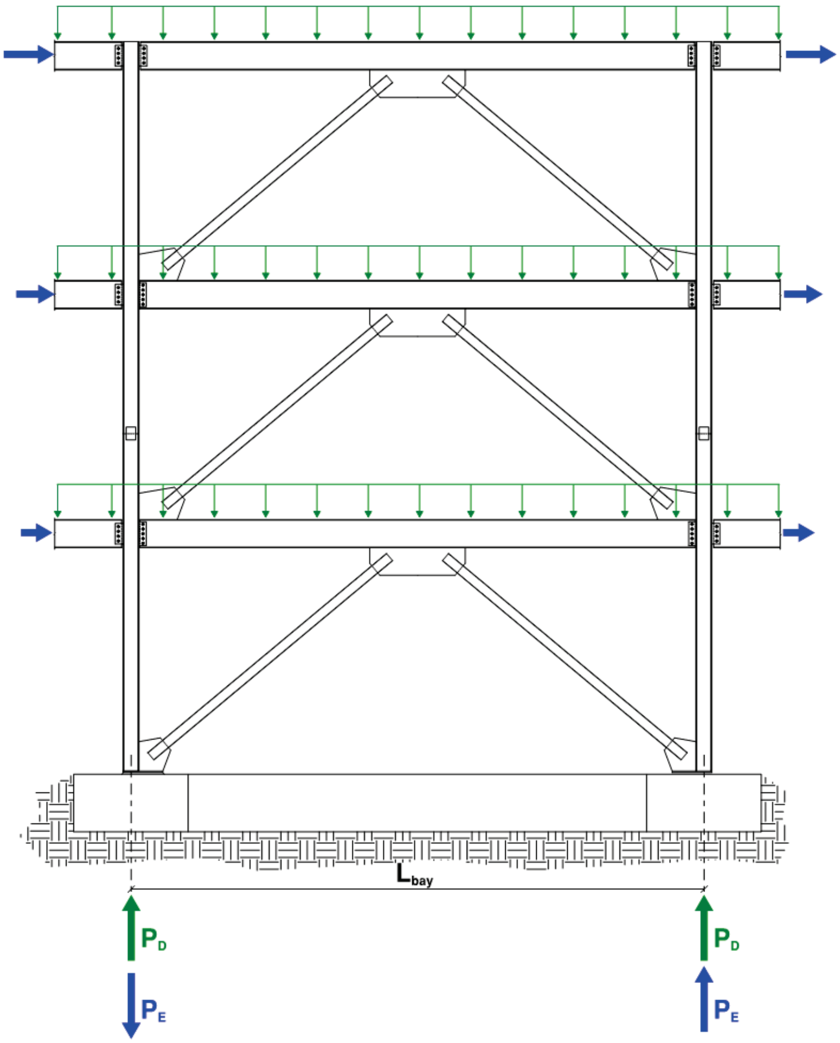


Figure 9-20 Foundation overturning.

$\sum$ Moments about column base vertical reactions = $M_{OT} - P_E L_{\text{bay}} = 0$

L_{bay} = Braced frame bay width
= 30 ft

M_{OT} = Overturning moment
= $P_E L_{\text{bay}}$ = 12,150 k ft

C_1C_2= 1.1 for $0.3 < T \leq 1.0$, $2 \leq m_{\max} < 6$ (ASCE 41-13 Table 7-3)

μ_{OT} = 4.0 for IO Performance Level

$M_{OT}/(C_1C_2\mu_{OT})$ = 2,761 k ft

$M_{ST} = P_D L_{bay} = 4{,}320\text{k ft}$

$0.9M_{ST} = 3{,}888 \text{ k ft}$

$0.9M_{ST} \geq M_{OT}/(C_1 C_2 \mu_{OT})$

Acceptance Ratio = $[M_{OT}/(C_1 C_2 \mu_{OT})]/[\,0.9M_{ST}] = 0.71 \leq 1.0$

Therefore, the overturning stability acceptance criterion is satisfied.

Table 9-29 Foundation Acceptance Criteria Summary

Action	Q_{UD}/Q_{CE}	m	$Q_{UD}/Q_{CE} \leq m$
Uplift	2.81	1.5	NG
Bearing Pressure	1.78	1.5	NG
Overturning Stability	0.71	NA	OK

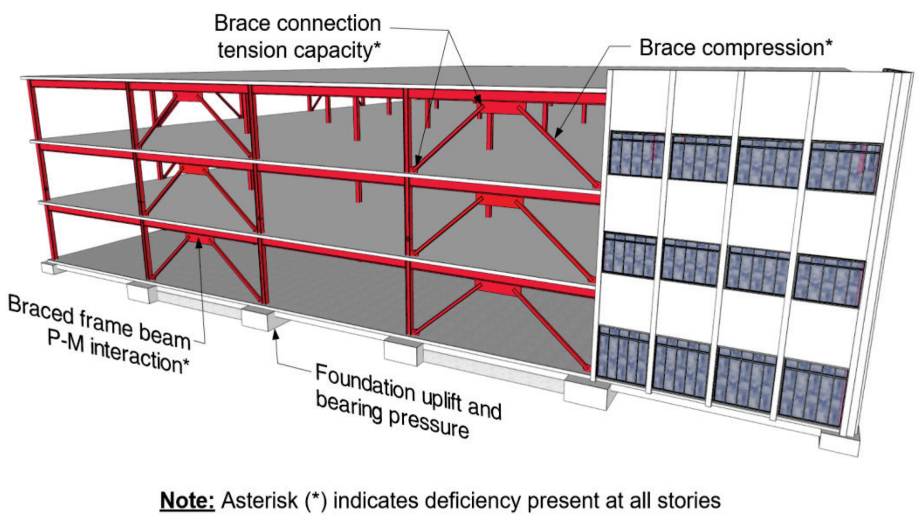


Figure 9-21 Tier 3 LSP deficiencies identified.

9.7.22 Confirm Applicability of Linear Procedure

> **Commentary**
>
> The "weak story" irregularity definition per ASCE 41-13 differs from the related definition in ASCE 7-10, where the story strengths rather than story DCRs are compared.

ASCE 41-13 § 7.3.1.1 states: "*If a component DCR exceeds the lesser of 3.0 and the m-factor for the component action and any irregularity described in Section 7.3.1.1.3 or Section 7.3.1.1.4 is present, then linear procedures are not applicable and shall not be used.*" DCR is calculated without using *m*-values per ASCE 41-13 § 7.3.1.1.

For this building, there are some DCR values greater than the specified limit (the lower of 3.0 or the *m*-factor). Due to the symmetric layout of the braces in plan, the building does not have a torsional strength irregularity per ASCE 41-13 § 7.3.1.1.4. However, based on the brace compression DCRs per Table 9-10 and since the same brace size exists at all three stories, the average element DCR at the second story exceeds the average element DCR at the third story by more than the 25% threshold defined in ASCE 41-13 § 7.3.1.1.3; the average first story element DCR also exceeds the average second story element DCR by more than 25%. Accordingly, the building has a weak story irregularity per ASCE 41-13 § 7.3.1.1.3. Therefore, linear analysis procedures may not be used.

9.8 Tier 3 Evaluation using Nonlinear Static Procedure (NSP)

9.8.1 General

In the following section, the building is evaluated using the nonlinear static procedure (NSP). Analysis is conducted using a three-dimensional PERFORM-3D model. Per ASCE 41-13 § 7.4.3.2.3, the vertical distribution of forces is proportional to the shape of the fundamental mode in the direction under consideration. Gravity loads are defined with distributed line loads on beam elements and point loads on columns; gravity load combinations per ASCE 41-13 Equation 7-3 are used. For brevity, this example only focuses on a typical north-south braced frame line. Figure 9-22 identifies components of the nonlinear analysis model as follows:

- **Braces** are modeled using inelastic bar elements. Brace lengths are defined based on the actual brace end-to-end lengths between gusset plates. Brace modeling parameters are defined based on the nonlinear response of the braces, irrespective of connection behavior.
- **Brace connections** are modeled using compound frame elements, consisting of an inelastic axial hinge representing the strength of the connection and a rigid (end-zone) frame segment spanning from the brace ends to the frame centerline joints per ASCE 41-13 § 9.5.2.2.
- **Braced frame beams** are modeled with compound frame elements, consisting of an inelastic PMM hinge at the chevron connection, an elastic frame segment along the beam clear span, and a rigid end-zone along half the column depth.
- **Columns** are modeled with compound frame elements, consisting of an inelastic PMM hinge at each end, an elastic frame segment along the column clear span, and rigid end-zones along half the beam depth.
- **Gravity beams** are modeled with compound frame elements, consisting of an elastic frame segment along the beam clear span, a semi-rigid inelastic flexural hinge at the beam-column connection, and a rigid end-zone along half the column depth.

Commentary

The modeling and inelastic behavior of the beam-column connection is limited in this example for brevity and the fact that there are limited data available for proper modeling and acceptance criteria for rotation behavior of gusset connections. For systems with more drift (BRBs or EBFs), the rotational limitations of this connection become important to consider. When demands in the connection exceed the beam plastic moment, a conservative approach of treating the connection as a force-controlled component should be used unless test data are available to justify the inelastic behavior of the gusset connection.

Commentary

Although beams and columns are modelled with PMM hinges, only "PM" hinges are required, where the moment, "M", corresponds to actions within the plane of the braces.

- **Foundations** are modeled with inelastic vertical spring elements representing the soil response. The column and braces are assumed to be rigidly anchored to the foundation for simplicity; the strength of this anchorage would need to be verified in a full analysis.

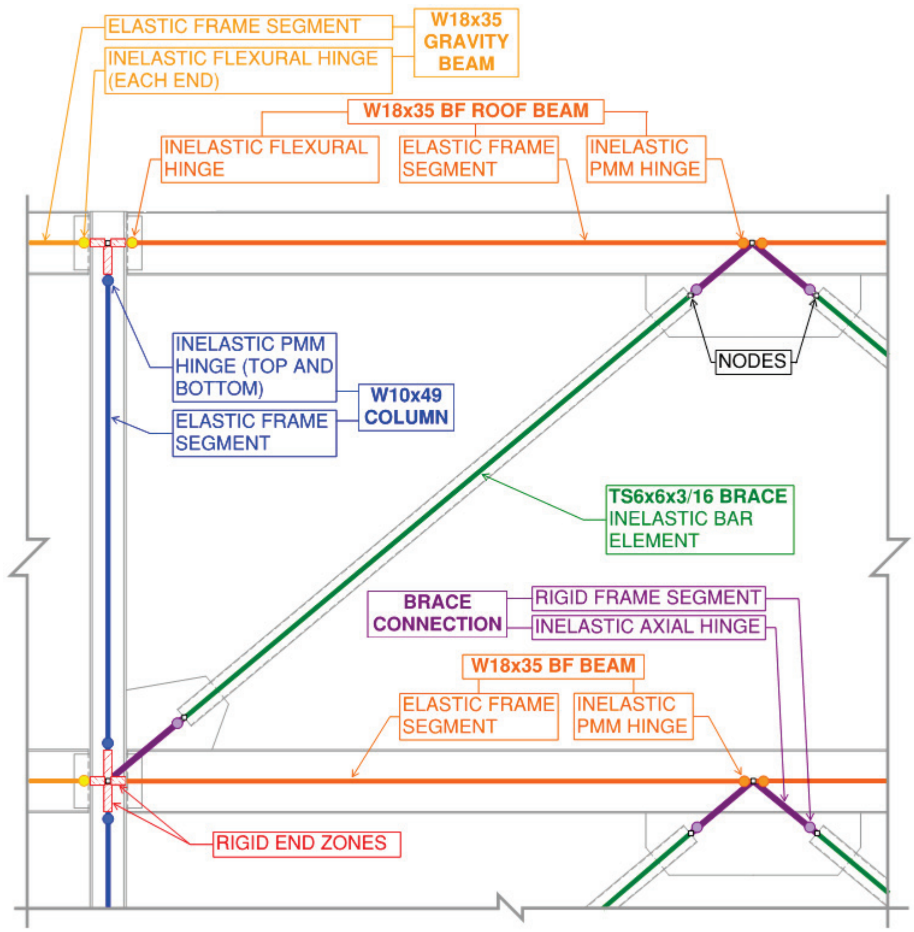


Figure 9-22 Nonlinear analysis model.

9.8.2 *Brace Modeling and Acceptance Criteria (ASCE 41-13 § 9.5.2.2.2 and § 9.5.2.4.3)*

9.8.2.1 Second Story Brace

A summary of brace connection acceptance ratios at the second-story is provided in Table 9-30 with reference to summary tables within this chapter; these acceptance ratios are taken directly from the previous LSP calculations that were based on limit state analysis procedures (with demands equal to the expected brace tensile and/or compressive strengths). It can therefore be seen that the inelastic response of the braces is governed by the brace-to-gusset weld in tension and the brace strength in compression. In other words, the brace connections can develop the expected compressive strength of the braces but not their expected tensile strength.

Table 9-30 **Brace Connection Tension/Compression Acceptance Ratio Summary**

Component	Limit State	Tension	Comp.	Table
Brace	Expected brace capacity	227 kips	137 kips	9-8
Brace Connection to Top and Bottom Gusset Plates	Brace-to-gusset weld and base metal yielding/rupture	1.30	0.79	9-9a 9-9b
	Brace tensile rupture	1.14	NA	9-10
	Gusset plate block shear	0.38	NA	9-11
	Gusset Whitmore section tensile yielding	0.46	NA	9-12a
	Gusset Whitmore section compression buckling	NA	0.30	9-12b
Top Gusset Plate Connection to Beam	Gusset-to-beam weld	0.40		9-13
	Gusset plate shear yielding	0.22		9-13
	Gusset tensile yielding	0.19		9-13
	Beam web local yielding	0.24		9-14
	Beam web local crippling	0.32		9-14
Bottom Gusset Plate Connection to Beam and Column	Gusset-to-beam/column weld	0.56	0.28	9-15
	Gusset plate shear yielding	0.36	0.22	9-15
	Gusset tensile yielding	0.20	NA	9-15
	Beam web local yielding	0.27	0.17	9-16
	Beam web local crippling	0.31	0.19	9-16
	Column web local yielding	0.16	0.10	9-17
	Column web local crippling	0.13	0.08	9-17
	Maximum Acceptance Ratio	1.30	0.79	

Commentary

In this example, braces and brace connections are modelled as separate elements to distinguish between inelastic actions occurring in the braces themselves and their connections. A combined inelastic response enveloping the behavior of the braces and their connections could alternatively be implemented. In this example, the "combined" curve would have deformation-controlled behavior corresponding to the brace yielding in compression and force-controlled behavior corresponding to the brace connection yielding in tension.

Brace Modeling Parameters (ASCE 41-13 § 9.5.2.2.2)

- Brace Tension (ASCE 41-13 Table 9-7)

$$T_y = T_{CE} = 227 \text{ kips}$$

$$\Delta_T = \Delta_{Ty} = \frac{T_y L}{AE} = \frac{227 \text{ kips}(15.31 \text{ ft})(12 \text{ in./ft})}{3.98 \text{ in.}^2(29{,}000 \text{ ksi})} = 0.36 \text{ in}$$

Note that ASCE 41-13 Table 9-7 footnote (f) is not applicable, since the braces are not tension only. Footnote (g) is also inapplicable because the Performance Level is Immediate Occupancy and the brace connection behavior is modeled explicitly in this example.

Modeling parameters from ASCE 41-13 Table 9-7 are calculated below:

$a/\Delta_T = 9$

$b/\Delta_T = 11$

$c = 0.6$

- Brace Compression (ASCE 41-13 Table 9-7)

$P_y = P_{CE} = 137$ kips

$$\Delta_C = \Delta_{Py} = \frac{P_y L}{AE} = \frac{137 \text{ kips}(15.31 \text{ ft})(12 \text{ in./ft})}{3.98 \text{ in.}^2 (29{,}000 \text{ ksi})} = 0.22 \text{ in}$$

Note that ASCE 41-13 Table 9-7 footnote (b) is not applicable, since the brace connection behavior is modeled explicitly in this example. Footnote (d), however, which applies to "stocky" braces, applies since the brace is classified as "noncompact" per AISC 360; modeling parameters and acceptance criteria for the "stocky" condition must accordingly be multiplied by 0.5.

- Brace Slenderness

$KL/r = 77.5$

ρ_a = Slender brace limit

$= 4.2\sqrt{E / F_{ye,br}} = 94.7$

ρ_b = Stocky brace limit

$= 2.1\sqrt{E / F_{ye,br}} = 47.4$

$\rho_b > KL/r > \rho_b$

Therefore, linear interpolation between the "slender" and "stocky" modeling parameters is required.

- Slender Brace Modeling Parameters

$a_a/\Delta_C = 0.5$

$b_a/\Delta_C = 9$

$c_a = 0.3$

- Stocky Brace Modeling Parameters

$a_b/\Delta_C = 0.5(1.0) = 0.5$

$b_b/\Delta_C = 0.5(7) = 3.5$

$c_b/\Delta_C = 0.5(0.5) = 0.25$

- Linear Interpolation

$$a/\Delta_C = 0.5 + \frac{(0.5-0.5)}{(47.4-94.7)}(77.5-94.7) = 0.5$$

$$b/\Delta_C = 9 + \frac{(3.5-9)}{(47.4-94.7)}(77.5-94.7) = 7.00$$

$$c = 0.3 + \frac{(0.25-0.3)}{(47.4-94.7)}(77.5-94.7) = 0.28$$

Brace Acceptance Criteria (ASCE 41-13 § 9.5.2.2.2)

- Brace Tension (ASCE 41-13 Table 9-7)

$\mathrm{IO}/\Delta_T = 0.5$

- Brace Compression (ASCE 41-13 Table 9-7)

$\mathrm{IO}_a/\Delta_C = 0.5$

$\mathrm{IO}_b/\Delta_C = 0.5(0.5) = 0.25$

$$\mathrm{IO}/\Delta_C = 0.5 + \frac{(0.25-0.5)}{(47.4-94.7)}(77.5-94.7) = 0.41$$

Brace Backbone Curve

As shown in Figure 9-23, the brace backbone curve is developed per ASCE 41-13 Figure 9-1 using the previously calculated modelling parameters. The previously calculated Immediate Occupancy acceptance criteria are shown with a red marker.

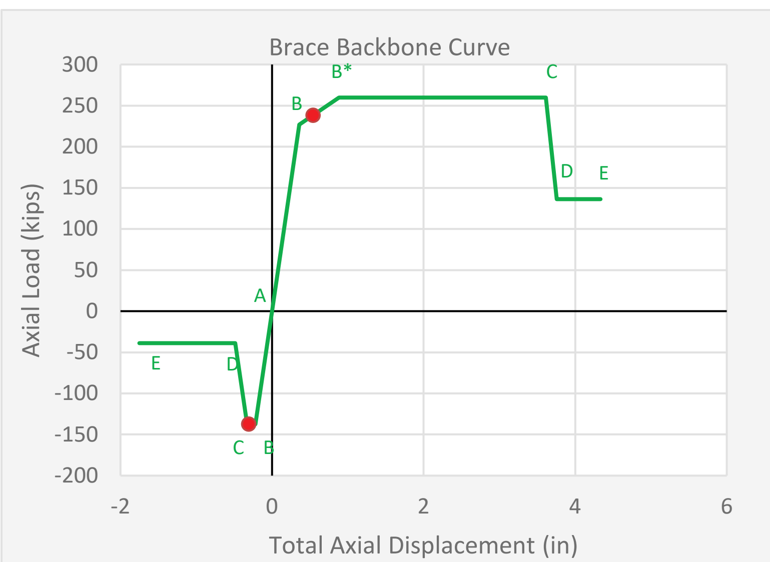


Figure 9-23 Brace backbone curve (with IO acceptance criteria shown in red).

Commentary

Per ASCE 41-13 § C9.5.2.3.3, a strain-hardening slope equal to 10% of the elastic slope is used (along the segment from Point B to B*) in tension until the expected ultimate tensile strength of the brace ($F_{ue,br}\ A_g$) is reached. No strain-hardening is modeled in compression. In both tension and compression, the strength degradation slope is assumed to match the initial elastic slope; the slope from Point C to D matches the slope from Point A to B, and the slope from Point C′ to D′ matches the slope from Point A to B′.

The points on the brace backbone curve are calculated as follows:

- Brace Tension

$T_B \quad = T_y = T_{CE} = 227$ kips

$T_{B*} \quad = F_{ue,br} A_g = 65.27$ ksi $(3.98 \text{ in.}^2) = 260$ kips

$T_C \quad = T_{B*} = 260$ kips

$T_D \quad = c\ T_{CE} = 0.60$ (260 kips) = 136 kips

$T_E \quad = T_D = 136$ kips

$\Delta_B \quad = \Delta_{Ty} = \Delta_T = 0.36$ in

$\Delta_{B*} \quad = \Delta_B + (T_{B*} - T_B)/(0.1 T_B/\Delta_B)$
$= 0.36$ in. + (260 kips – 237 kips)/(0.1(237 kips)/(0.36 in.))
$= 0.89$ in.

$\Delta_C \quad = \Delta_B + (a/\Delta_T)\Delta_T$
$= 0.36$ in. + 9(0.36 in.)
$= 3.60$ in.

$\Delta_D \quad = \Delta_C + (1 - c)\Delta_T$
$= 3.60$ in. + (1 – 0.6)(0.36 in.)
$= 3.74$ in.

$\Delta_E \quad = \Delta_B + (b/\Delta_T)\Delta_T$
$= 0.36$ in. + 11(0.36 in.)
$= 4.32$ in.

$\Delta_{IO} \quad = \Delta_B + (IO/\Delta_T)\Delta_T$
$= 0.36$ in. + 0.50(0.36 in.)
$= 0.54$ in.

- Brace Compression

$P_{B'} \quad = P_y = P_{CE} = 137$ kips

$P_{C'} \quad = P_{B'} = 137$ kips

$P_{D'} \quad = c\ P_{CE} = 0.28$ (137 kips) = 38 kips

$P_{E'} \quad = P_{D'} = 38$ kips

$\Delta_{B'} \quad = \Delta_{Py} = \Delta_C = 0.22$ in.

$\Delta_{C'} \quad = \Delta_{B'} + (a/\Delta_C)\Delta_C$
$= 0.22$ in. + 0.50(0.22 in.)
$= 0.33$ in.

$$\Delta_{D'} = \Delta_{C'} + (1 - c)\Delta_C$$
$$= 0.33 \text{ in.} + (1 - 0.28)(0.22 \text{ in.})$$
$$= 0.49 \text{ in.}$$

$$\Delta_{E'} = \Delta_{B'} + (b/\Delta_C)\Delta_C$$
$$= 0.22 \text{ in.} + 7(0.22 \text{ in.})$$
$$= 1.76 \text{ in.}$$

$$\Delta_{IO} = \Delta_{B'} + (IO/\Delta_C)\Delta_C$$
$$= 0.22 \text{ in.} + 0.41(0.22 \text{ in.})$$
$$= 0.31 \text{ in.}$$

Brace Connections

Brace connections are modeled using inelastic axial hinges with Type 3 (force-controlled) load-deformation curves per ASCE 41-13 Figure 7-4. Note the post-yield slope of the connection load-deformation curves shown in Figure 9-24 are estimated with a gradual slope for model stability, understanding the connection is flagged as deficient once the connection yields.

- Brace Connection Tension (Table 9-30)

  In the following calculation, the maximum brace tensile force that the brace connection can accommodate is back-calculated from the maximum calculated connection acceptance ratio. As shown in Table 9-30, tension in the brace connection is governed by the brace-to-gusset weld capacity.

  $$T_{CE,br} = T_{UF,\text{connection}} = 227 \text{ kips}$$

  Max Tension Acceptance Ratio = 1.30

  $$T_{y,\text{connection}} = T_{CE,br}/1.30 = 175 \text{ kips}$$

- Brace Connection Compression (Table 9-30)

  Similarly, the maximum brace compressive force that the brace connection can accommodate is back-calculated from the maximum calculated connection acceptance ratio. As shown in Table 9-30, compression in the brace connection is also governed by the brace-to-gusset weld capacity.

  $$P_{CE,br} = P_{UF,\text{connection}} = 137 \text{ kips}$$

  Max Compression Acceptance Ratio = 0.785

  $$P_{y,\text{connection}} = P_{CE,br}/0.785 = 175 \text{ kips}$$

- Brace Connection Backbone Curve and Acceptance Criteria

Commentary

The brace-to-gusset weld max acceptance ratio for compression is 0.785, which was rounded to 0.79 in the previous calculations. The more precise ratio of 0.785 is used here, because the connection's compression capacity should match its tension capacity (175 kips); tension and compression are both governed by the brace-to-gusset weld capacity.

For both tension and compression, the brace connections are modelled with a (force-controlled) Type 3 curve per ASCE 41-13 Figure 7-4. IO acceptance criteria are accordingly defined as the point at which the connection is required to resist an axial load of 175 kips, corresponding to failure of the brace-to-gusset weld.

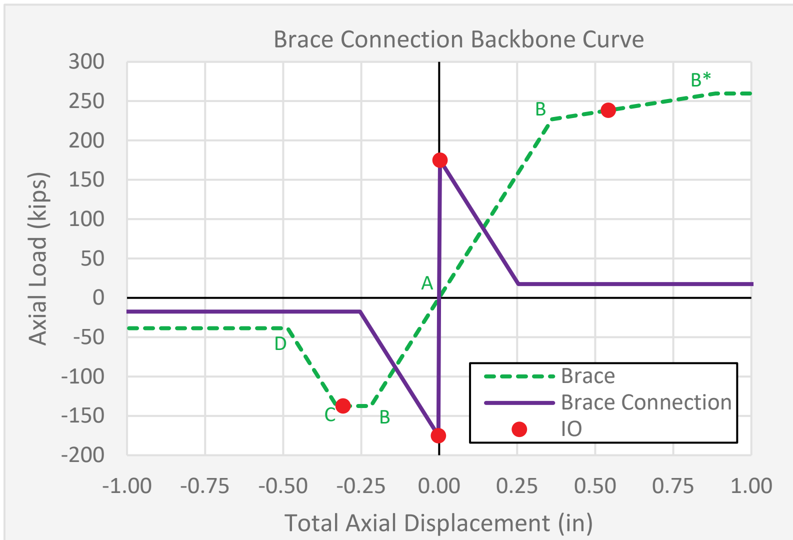


Figure 9-24 Brace connection backbone curve.

Brace/Connection Modeling Parameter and Acceptance Criteria Summary

Table 9-31 Brace Modeling Parameter and Acceptance Criteria Summary (Tension)

Story	T_y (kips)	Δ_T (in.)	a / Δ_T	b / Δ_T	c	IO / Δ_T
3rd	227	0.36	9.00	11.00	0.60	0.50
2nd	227	0.36	9.00	11.00	0.60	0.50
1st	227	0.38	9.00	11.00	0.60	0.50

Table 9-32 Brace Modeling Parameter and Acceptance Criteria (Compression)

Story	P_y (kips)	Δ_C (in.)	a / Δ_C	b / Δ_C	c	IO / Δ_C
3rd	137	0.22	0.50	7.00	0.28	0.41
2nd	137	0.22	0.50	7.00	0.28	0.41
1st	129	0.22	0.50	7.57	0.29	0.43

Table 9-33 Brace Connection Modeling Parameters

Story	$T_{y,connection}$ (kips)	$P_{y,connection}$ (kips)
3rd	175	175
2nd	175	175
1st	175	175

9.8.3 Beam Modeling and Acceptance Criteria (ASCE 41-13 § 9.4.3.2.2)

9.8.3.1 Second Story Beams

Beam Axial Loads

As shown in Table 9-14, the LSP analysis revealed very large flexural and axial compression acceptance ratios in the existing beams. By inspection, the NSP is unlikely to mitigate this deficiency. Therefore, for the NSP analysis, a partial retrofit of the existing beam involving the addition of bottom flange (torsional) kicker braces at beam quarter points is assumed; this increases the lower-bound beam compression capacity from 76 kips to 364 kips and accordingly decreases the axial compression acceptance ratio in the beams to from 1.40 to 0.29 (based on the same limit state analysis demands corresponding to the expected brace axial strengths). Per ASCE 41-13 § 9.4.2.3.2.2, the beams must be evaluated as beam-columns since the compression acceptance ratio exceeds 10%. All nonlinear modeling parameters and acceptance criteria are a function of the axial load, P, at the target displacement. But since the target displacement depends on the pushover results, an initial estimate must be made and verified later. In this example, the axial load, P, will conservatively be taken as the limit state analysis axial loads calculated in the previous LSP.

P = Assumed force at target displacement

= $P_{UF} = P_{LSA}$ = 106 kips

P_{ye} = Expected beam yield strength

= $T_{ye} = F_{ye}A_g$

= 54.95 ksi(10.3 in.^{2})

= 566 kips

P_{CL} = Retrofitted beam lower-bound compressive strength

= $F_{cr}A_g$

= 364 k

P/P_{ye} = (106 kips)/(566 kips) = 0.19

P/P_{CL} = (106 kips)/(364 kips) = 0.29

Beam Modeling Parameters (ASCE 41-13 Table 9-6)

F_{ye} = Expected beam yield strength

= 54.95 ksi

Z = Z_x = Beam plastic section modulus

= 66.5 in.^{3} for W18×35

I = I_x = Beam moment of inertia

= 510 in.^{4} for W18×35

L = Effective beam length for hinge at midspan

= 30 ft/2 = 15 ft

$M_y = M_{CE} = \min\{1.18ZF_{ye}(1 - P/P_{ye}), ZF_{ye}\}$ (ASCE 41-13 Eq. 9-4)

$= \min\{1.18(66.5 \text{ in.}^3)(54.95 \text{ ksi})(1 - 0.19), (66.5 \text{ in.}^3)(54.95 \text{ ksi})\}$

= min{3493 k-in. , 3654 k-in. }

= 3493 k in.

$$\theta_y = \frac{ZF_{ye}L}{6EI}\left(1 - \frac{P}{P_{ye}}\right) \quad \text{(ASCE 41-13 Eq. 9-2)}$$

$$= \frac{(66.5 \text{ in.}^3)(54.95 \text{ ksi})(15 \text{ ft})(12 \text{ in./ft})}{6(29{,}000 \text{ ksi})(510 \text{ in.}^4)}(1 - 0.19)$$

= 0.0060 rad

- Flange Compactness (ASCE 41-13 Table 9-6)

  $\lambda_f = b_f/2t_f = 7.06$

  $\lambda_{fa} = 52/\sqrt{F_{ye}} = 52/\sqrt{54.95} = 7.01$

  $\lambda_{fb} = 65/\sqrt{F_{ye}} = 65/\sqrt{54.95} = 8.77$

  $\lambda_{fa} \leq \lambda_f \leq \lambda_{fb}$

- Web Compactness (ASCE 41-13 Table 9-6)

  $\lambda_w = h/t_w = 53.5$

  $\lambda_{wa} = 260/\sqrt{F_{ye}} = 260/\sqrt{54.95} = 35.1$

  $\lambda_{wb} = 400/\sqrt{F_{ye}} = 400/\sqrt{54.95} = 54.0$

  $\lambda_{wa} \leq \lambda_w \leq \lambda_{wb}$

- Modeling Parameters (ASCE 41-13 Table 9-6)

  $a_a/\theta_y = 11(1 - 5/3P/P_{CL}) = 11(1 - 5/3(0.29)) = 5.68$

  $a_b/\theta_y = 1.00$

$$a_f/\theta_y = 5.68 + \frac{(1.00 - 5.68)}{(8.77 - 7.01)}(7.06 - 7.01) = 5.55$$

$$a_w/\theta_y = 5.68 + \frac{(1.00 - 5.68)}{(54.0 - 35.1)}(53.5 - 35.1) = 1.12$$

$a/\theta_y \ = \min\{a_f/\theta_y,\ a_w/\theta_y\} = 1.12$

$b_a/\theta_y = 17(1 - 5/3P/P_{CL}) = 17(1 - 5/3(0.29)) = 8.78$

$b_b/\theta_y = 1.50$

$$b_f/\theta_y = 8.78 + \frac{(1.50 - 8.78)}{(8.77 - 7.01)}(7.06 - 7.01) = 8.57$$

$$b_w/\theta_y = 8.78 + \frac{(1.50 - 8.78)}{(54.0 - 35.1)}(53.5 - 35.1) = 1.69$$

$b/\theta_y \ = \min\{b_f/\theta_y,\ b_w/\theta_y\} = 1.69$

$c_a/\theta_y = 0.20$

$c_b/\theta_y = 0.20$

$c \quad = 0.20$

Beam Acceptance Criteria (ASCE 41-13 Table 9-6)

$\mathrm{IO}_a/\theta_y = 0.25$

$\mathrm{IO}_b/\theta_y = 0.25$

$\mathrm{IO}/\theta_y \ = 0.25$

Beam Backbone Curve

As shown in Figure 9-25, the beam midspan hinge backbone curve is developed per ASCE 41-13 Figure 9-1 using the previously calculated modelling parameters. The previously calculated Immediate Occupancy acceptance criteria are shown with a red marker.

The points on the beam backbone curve are calculated as follows:

$\theta_B \quad = \theta_y = 0.0060$ rad

$\theta_C \quad = \theta_B + (a/\theta_y)\theta_y$

$\quad = 0.0060$ rad $+ 1.12(0.0060$ rad$)$

$\quad = 0.0127$ rad

$\theta_D \quad = (\theta_C + \theta_E)/2$

$\quad = (0.0127$ rad $+ 0.0161$ rad$)$

$\quad = 0.0144$ rad

$$\theta_E = \theta_B + (b/\theta_y)\theta_y$$
$$= 0.0060 \text{ rad} + 1.69(0.0060 \text{ rad})$$
$$= 0.0161 \text{ rad}$$

$$\theta_{IO} = \theta_B + (IO/\theta_y)\theta_y$$
$$= 0.0060 \text{ rad} + 0.25(0.0060 \text{ rad})$$
$$= 0.0075 \text{ rad}$$

$$\theta_{B'} = \theta_B = 0.0060 \text{ rad}$$

$$\theta_{C'} = \theta_C = 0.0127 \text{ rad}$$

$$\theta_{D'} = \theta_D = 0.0144 \text{ rad}$$

$$\theta_{E'} = 0.0161 \text{ rad}$$

$$\theta_{IO'} = \theta_{IO} = 0.0075 \text{ rad}$$

Commentary

Per ASCE 41-13 § 9.4.2.2.2, a strain-hardening slope equal to 3% of the elastic slope is modeled. Assuming a strength degradation slope (from Point C to D) equal to the initial elastic slope (from Point A to B) would result in a rotation at Point D larger than the rotation a Point E, so the rotation at Point D is assumed to be halfway between the rotations at Points C and E.

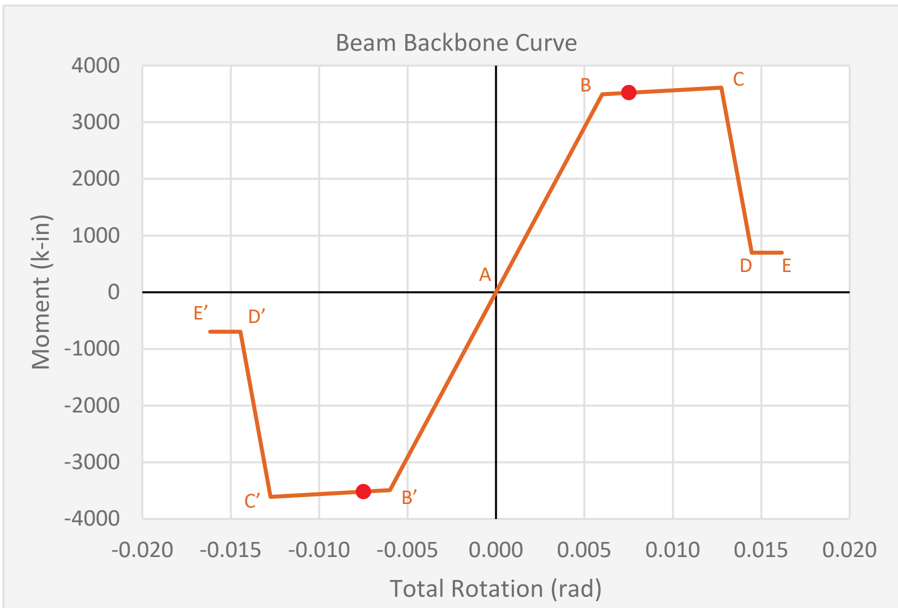


Figure 9-25 Beam backbone curve (with IO AC in red).

$$M_B = M_y = M_{CE} = 3{,}493 \text{ k in}$$

$$M_C = M_B + 0.03(M_B/\theta_B)(\theta_C - \theta_B)$$
$$= 3{,}493 \text{ k in} + 0.03((3{,}493 \text{ k in.})/(0.0060 \text{ rad}))(0.0127 \text{ rad} - 0.0060 \text{ rad})$$
$$= 3{,}610 \text{ k in}$$

$$M_D = c\, M_{CE} = 0.20\ (3{,}494 \text{ k in.}) = 699 \text{ k in}$$

$$M_E = M_D = 699 \text{ k in}$$

$$M_{B'} = M_B = 3{,}493 \text{ k in}$$

$M_{C'}$ $= M_C = 3{,}610$ k in

$M_{D'}$ $= M_D = 699$ k in

$M_{E'}$ $= M_E = 699$ k in

Beam Modeling Parameter and Acceptance Criteria Summary

Table 9-34 Beam Axial Load Summary

Story	P (kips)	P_{ye} (kips)	P_{CL} (kips)	P / P_{ye}	P / P_{CL}
3rd	106	566	364	0.19	0.29
2nd	106	566	364	0.19	0.29
1st	102	566	364	0.18	0.28

Table 9-35 Beam Modeling Parameter and Acceptance Criteria Summary

Story	M_y (k-in.)	θ_y (rad)	a / θ_y	b / θ_y	c	IO / θ_y
3rd	3,493	0.0060	1.12	1.69	0.20	0.25
2nd	3,493	0.0060	1.12	1.69	0.20	0.25
1st	3,536	0.0061	1.13	1.70	0.20	0.25

9.8.4 Column Modeling and Acceptance Criteria

9.8.4.1 Second Story Columns

Column Axial Loads

All nonlinear modeling parameters and acceptance criteria are a function of the axial load, P, at the target displacement. But since the target displacement depends on the pushover results, an initial estimate must be made and verified later. In this example, the axial load, P, will conservatively be taken as the limit state analysis axial loads calculated in the previous LSP.

P = Assumed force at target displacement

$= P_{UF} = P_{G,\max} + P_{LSA}$

$= 89$ kips $+ 145$ kips

$= 234$ kips

P_{ye} = Expected tensile strength

$= T_{ye} = F_{ye}A_g$

$= 53.1$ ksi$(14.4$ in.$^2)$

$= 765$ kips

P_{CL} = Lower-bound compressive strength

$= F_{cr}A_g$

$= (39.41 \text{ ksi})(14.4 \text{ in.}^2)$

$= 568$ kips

$P/P_{ye} = (234 \text{ kips})/(765 \text{ kips}) = 0.31$

$P/P_{CL} = (234 \text{ kips})/(568 \text{ kips}) = 0.41$

$0.2 \leq P/P_{CL} \leq 0.5$

Column Modeling Parameters (ASCE 41-13 Table 9-6)

F_{ye} = Expected column yield strength

= 53.1 ksi

Z = Column plastic section modulus

$= Z_x = 60.4 \text{ in}^3$ for W10×49

I = Column moment of inertia

$= I_x = 272 \text{ in}^4$ for W10×49

L = 12.5 ft, column length

$M_y = M_{CE} = \min\{1.18ZF_{ye}(1 - P/P_{ye}), ZF_{ye}\}$ (ASCE 41-13 Eq. 9-4)

$= \min\{1.18(60.4 \text{ in}^3)(53.1 \text{ ksi})(1 - 0.31), (60.4 \text{ in}^3)(53.1 \text{ ksi})\}$

$= \min\{2611 \text{ k-in.}, 3207 \text{ k-in.}\}$

$= 2611$ k-in.

$$\theta_y = \frac{ZF_{ye}L}{6EI}\left(1 - \frac{P}{P_{ye}}\right) \quad \text{(ASCE 41-13 Eq. 9-2)}$$

$$= \frac{(60.4 \text{ in}^3)(53.1 \text{ ksi})(12.5 \text{ ft})(12 \text{ in/ft})}{6(29{,}000 \text{ ksi})(272 \text{ in}^4)}\left(1 - 0.31\right)$$

$= 0.0070$ rad

- Flange Compactness

  $\lambda_f = b_f/2t_f = 8.93$

  $\lambda_{fa} = 52/\sqrt{F_{ye}} = 52/\sqrt{53.1} = 7.14$

  $\lambda_{fb} = 65/\sqrt{F_{ye}} = 65/\sqrt{53.1} = 8.92$

  $\lambda_f > \lambda_{fb}$

- Web Compactness

  $\lambda_w = h/t_w = 23.1$

  $\lambda_{wa} = 260/\sqrt{F_{ye}} = 260/\sqrt{53.1} = 35.7$

$\lambda_{wb} = 400/\sqrt{F_{ye}} = 400/\sqrt{53.1} = 54.9$

$\lambda_w < \lambda_{wa}$

- Modeling Parameters

$a_a/\theta_y = 11(1 - 5/3P/P_{CL}) = 11(1 - 5/3(0.41)) = 3.48$

$a_b/\theta_y = 1.00$

$a_f/\theta_y = a_b/\theta_y = 1.00$ (Since $\lambda_f > \lambda_{fb}$)

$a_w/\theta_y = a_a/\theta_y = 3.48$ (Since $\lambda_w < \lambda_{wa}$)

$a/\theta_y = \min\{a_f/\theta_y, a_w/\theta_y\} = 1.00$

$b_a/\theta_y = 17(1 - 5/3P/P_{CL}) = 17(1 - 5/3(0.41)) = 5.38$

$b_b/\theta_y = 1.50$

$b_f/\theta_y = b_b/\theta_y = 1.50$ (Since $\lambda_f > \lambda_{fb}$)

$b_w/\theta_y = b_a/\theta_y = 5.38$ (Since $\lambda_w < \lambda_{wa}$)

$b/\theta_y = \min\{b_f/\theta_y, b_w/\theta_y\} = 1.50$

$c_a/\theta_y = c_b/\theta_y = 0.20$

$c = 0.20$

Column Acceptance Criteria (ASCE 41-13 Table 9-6)

$\mathrm{IO}_a/\theta_y = IO_b/\theta_y = 0.25$

$\mathrm{IO}/\theta_y = 0.25$

Column Backbone Curve

As shown in Figure 9-26, the column hinge backbone curve is developed per ASCE 41-13 Figure 9-1 using the previously calculated modelling parameters. The previously calculated Immediate Occupancy acceptance criteria are shown with a red marker.

Column Modeling Parameter and Acceptance Criteria Summary

Table 9-36 Column Axial Load Summary

Story	P (kips)	P_{ye} (kips)	P_{CL} (kips)	P / P_{ye}	P / P_{CL}
3rd	96	765	568	0.13	0.17
2nd	234	765	568	0.31	0.41
1st	375	765	568	0.49	0.66*

* As required per ASCE 41-13 Table 9-6 footnote (b), since P/P_{CL} > 0.5 for the first story columns, they are modelled with Type 3 (force-controlled) load-deformation relationships per ASCE 41-13 Figure 7-4.

Commentary

Per ASCE 41-13 § 9.4.2.2.2, a strain-hardening slope equal to 3% of the elastic slope is modeled. Assuming a strength degradation slope (from Point C to D) equal to the initial elastic slope (from Point A to B) would result in a rotation at Point D larger than the rotation a Point E, so the rotation at Point D is assumed to be halfway between the rotations at Points C and E.

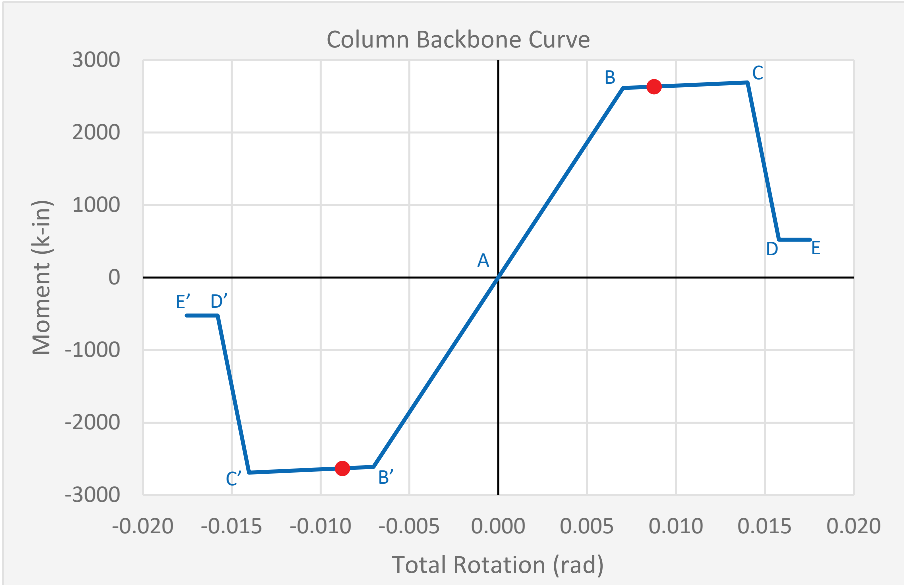


Figure 9-26 Column backbone curve.

The points on the column backbone curve are calculated as follows:

θ_B $= \theta_y = 0.0070$ rad

θ_C $= \theta_B + (a/\theta_y)\theta_y$

$= 0.0070$ rad $+ 1.00(0.0070$ rad$)$

$= 0.0140$ rad

θ_D $= (\theta_C + \theta_E)/2$

$= (0.0140$ rad $+ 0.0175$ rad$)$

$= 0.0158$ rad

θ_E $= \theta_B + (b/\theta_y)\theta_y$

$= 0.0070$ rad $+ 1.50(0.0070$ rad$)$

$= 0.0175$ rad

θ_{IO} $= \theta_B + (IO/\theta_y)\theta_y$

$= 0.0070$ rad $+ 0.25(0.0070$ rad$)$

$= 0.0088$ rad

$\theta_{B'}$ $= \theta_B = 0.0070$ rad

$\theta_{C'}$ $= \theta_C = 0.0140$ rad

$\theta_{D'}$ $= \theta_D = 0.0158$ rad

$\theta_{E'}$ $= 0.0175$ rad

$\theta_{IO'} = \theta_{IO} = 0.0088$ rad

$M_B = M_y = M_{CE} = 2{,}611$ k in.

$M_C = M_B + 0.03(M_B/\theta_B)(\theta_C - \theta_B)$

$= 2{,}611$ k in $+ 0.03((2{,}611$ k in.$)/(0.0070$ rad$))(0.0140$ rad $- 0.0070$ rad$)$

$= 2{,}689$ k in.

$M_D = c\, M_{CE} = 0.20\ (2{,}611$ k in.$) = 522$ k in.

$M_E = M_D = 522$ k in.

$M_{B'} = M_B = 2{,}611$ k in.

$M_{C'} = M_C = 2{,}689$ k in.

$M_{D'} = M_D = 522$ k in.

$M_{E'} = M_E = 522$ k in.

Table 9-37 Column Modeling Parameter and Acceptance Criteria Summary

Story	M_y (k-in.)	θ_y (rad)	a / θ_y	b / θ_y	c	IO / θ_y
3rd	3,207	0.0089	1.00	1.50	0.20	0.25
2nd	2,628	0.0070	1.00	1.50	0.20	0.25
1st	1,929	0.0052	0.05*	1.50*	0.10*	0.05*

* As required per ASCE 41-13 Table 9-6 footnote (b), since $P/P_{CL} > 0.5$ for the first story columns, they are modelled with Type 3 (force-controlled) load-deformation relationships per ASCE 41-13 Figure 7-4.

9.8.5 *Gravity Beam Connection Modeling and Acceptance Criteria*

The inelastic behavior of the gravity beam connections to the columns, (shown in Figure 9-27), is modeled per ASCE 41-13 § 9.4.3 and FEMA 355D, *State of the Art Report on Connection Performance* (FEMA, 2000e). As shown in FEMA 355D Figure 6-2 and Figure 6-5, the moment-rotation response of shear tab connections varies depending on the direction of the applied moment. For negative moment actions, the slab is assumed to be ineffective in tension (as shown in Figure 9-28), so a force couple develops between the top and bottom bolts in the shear tab. For positive moment actions, a force couple develops between the shear tab bolts and compression in the deck slab, as shown in Figure 9-29.

Commentary

See ASCE 41-13 § C9.4.3.1 for additional commentary regarding the significance of modeling gravity connections.

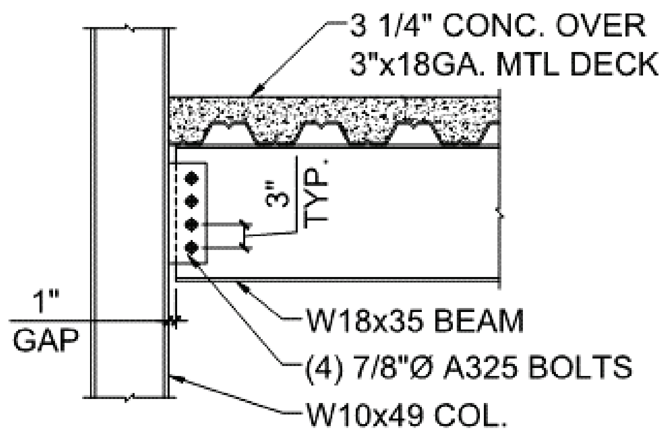


Figure 9-27 Gravity beam connection detail.

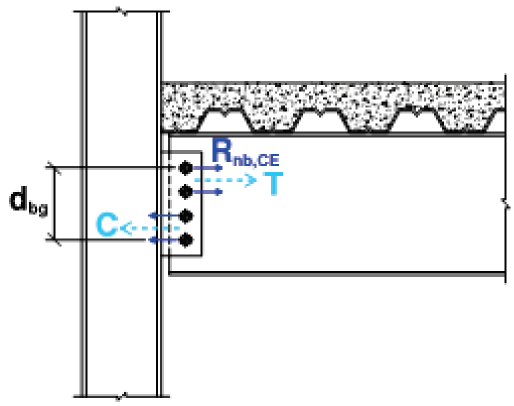


Figure 9-28 Negative moment at gravity beam connection.

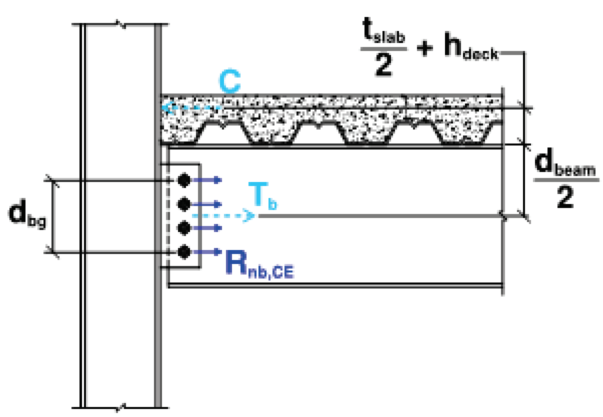


Figure 9-29 Positive moment at gravity beam connection.

Negative Moment (ASCE 41-13 Table 9-6 and FEMA 355D)

F_{ube} = Expected bolt tensile strength per testing
= 120.2 ksi

F_{nve} = 0.450 F_{ube} = per AISC 360-10 Table J3.2, A325-N bolts
= 0.450(120.2 ksi)
= 54.1 ksi

A_b = 0.601 in.^{2}

$R_{nb,CE}$ = $F_{nve}\,A_b$ = (54.1 ksi)(0.601 in.^{2}) = 32.5 kips/bolt

s_b = 3 in.

N_b = 4 bolts

d_{bg} = $(N_b - 1)s_b$ = (4-1)(3 in.) = 9 in.

T = $C = (N_b/2)R_{nb,CE}$
= (4/2)(32.5 kips)
= 65 kips

M_{CE} = $T(d_{bg} - s_b) = C(d_{bg} - s_b)$
= (65 kips)(9 in. – 3 in.)
= 390 k-in.

k_s = $28{,}000(d_{bg} - 5.6)$ (FEMA 355D Eq. 5-19)
= 28,000(9 – 5.6)
= 95,200 k-in./rad

θ_y = M_{CE} / k_s = (390 k-in.)/(95,200 k-in./rad)
= 0.0041 rad

θ_{binding} = $gap/(d_{\text{beam}}/2) - 0.02$ (FEMA 355D Eq. 5-19)
= (1 in.)/(17.7 in/2) – 0.02
= 0.093 rad

$$\theta_a = \min\begin{bmatrix}\theta_{\text{binding}} \\ 0.15 - 0.0036 d_{bg}\end{bmatrix} = \min\begin{bmatrix}0.093\text{ rad} \\ 0.118\text{ rad}\end{bmatrix} = 0.093\text{ rad}$$

$$\theta_b = \min\begin{bmatrix}\theta_{\text{binding}} \\ 0.15 - 0.0036 d_{bg}\end{bmatrix} = \min\begin{bmatrix}0.093\text{ rad} \\ 0.118\text{ rad}\end{bmatrix} = 0.093\text{ rad}$$

c = 0.4

$$\theta_{IO,p} = \min\begin{bmatrix}\theta_{\text{binding}} \\ 0.075 - 0.0018 d_{bg}\end{bmatrix} = \min\begin{bmatrix}0.093\text{ rad} \\ 0.059\text{ rad}\end{bmatrix} = 0.059\text{ rad}$$

Positive Moment (ASCE 41-13 Table 9-6, FEMA 355D)

f'_{ce} = Expected slab compressive strength per testing
= 5.94 ksi

b_{cf} = Column flange width
= 10 in for W10×49

t_{slab} = Slab thickness
= 3.25 in.

C_c = Slab compression force

$= 0.85 f'_{ce} b_{cf} t_{\text{slab}}$

= 0.85(5.94 ksi)(10 in.)(3.25 in.)

= 164 kips

T_b = Tension force resisted by bolt shear

$= N_b R_{nb,CE}$

= 4(32.5 kips)

= 130 kips

T $= C = \min\{T_b, C_c\}$

= min{130 kips, 164 kips }

= 130 kips

M_{CE} $= T(t_{\text{slab}}/2 + h_{\text{deck}} + d_{\text{beam}}/2)$

= (130 kips)(3.25 in./2 + 3 in. + 17.7 in./2)

= 1,752 k-in.

k_s $= 28{,}000(d_{bg} - 3.3)$ (FEMA 355D Eq. 6-4)

= 28,000(9 – 3.3)

= 159,600 k-in./rad

θ_y $= M_{CE}/k_s$ = (1,752 k-in.)/(159,600 k-in./rad)

= 0.0110 rad

θ_{binding} $= \text{gap}/(d_{\text{beam}}/2) - 0.02$ (FEMA 355D Eq. 6-3)

= (1 in.)/(17.7 in./2) – 0.02

= 0.093 rad

$$\theta_a = \min\begin{bmatrix} \theta_{\text{binding}} \\ 0.029 - 0.00020 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.093 \text{ rad} \\ 0.027 \text{ rad} \end{bmatrix} = 0.027 \text{ rad}$$

$$\theta_b = \min\begin{bmatrix} \theta_{\text{binding}} \\ 0.15 - 0.0036 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.093 \text{ rad} \\ 0.118 \text{ rad} \end{bmatrix} = 0.093 \text{ rad}$$

c = 0.4

$$\theta_{IO,p} = \min\begin{bmatrix} \theta_{\text{binding}} \\ 0.014 - 0.00010 d_{bg} \end{bmatrix} = \min\begin{bmatrix} 0.093 \text{ rad} \\ 0.013 \text{ rad} \end{bmatrix} = 0.013 \text{ rad}$$

Figure 9-30 shows the resulting gravity beam backbone curve.

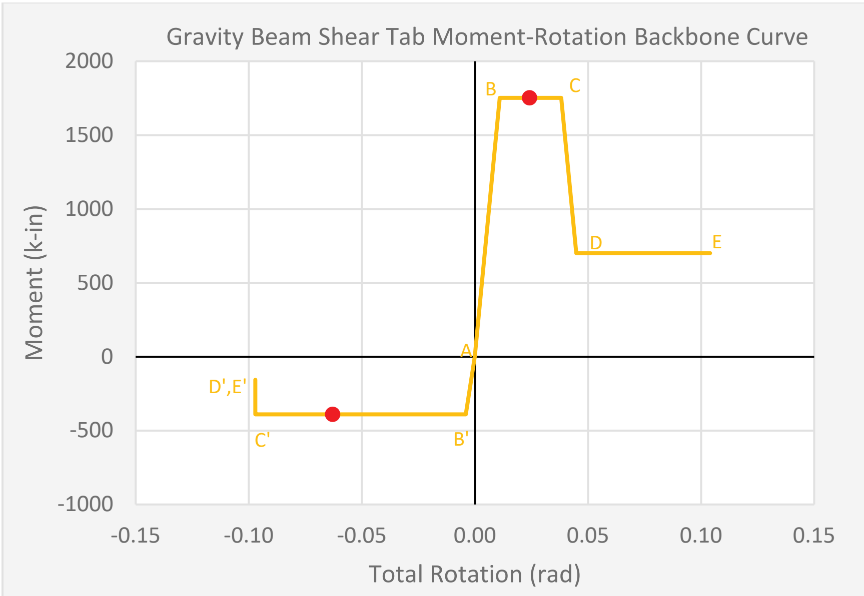


Figure 9-30 Gravity beam backbone curve (with IO acceptance criteria shown in red).

The points on the gravity beam shear tab backbone curve are calculated as follows:

- Positive Moment

M_B $= M_y = M_{CE} = 1{,}752$ k in.

M_C $= M_B = 1{,}752$ k in.

M_D $= c\, M_{CE} = 0.4\ (1{,}752$ k in.$) = 701$ k in.

M_E $= M_D = 701$ k in.

θ_B $= \theta_y = 0.011$ rad

θ_C $= \theta_B + \theta_a$

$= 0.011$ rad $+ 0.027$ rad

$= 0.038$ rad

θ_D $= \theta_C + (1 - c)\theta_y$

$= 0.038$ rad $+ (1 - 0.4)(0.011$ rad$)$

$= 0.045$ rad

θ_E $= \theta_B + \theta_b$

$= 0.011$ rad $+ 0.093$ rad

$= 0.104$ rad

θ_{IO} $= \theta_B + \theta_{IO,p}$

$= 0.011$ rad $+ 0.013$ rad

$= 0.024$ rad

- Negative Moment

$M_{B'} = M_y = M_{CE} = 390$ k in.

$M_{C'} = M_{B'} = 390$ k in.

$M_{D'} = c\, M_{CE} = 0.4$ (390 k in.) = 156 k in.

$M_{E'} = M_{D'} = 156$ k in.

$\theta_{B'} = \theta_y = 0.0041$ rad

$\theta_{C'} = \theta_{B'} + \theta_a$
= 0.0041 rad + 0.093 rad
= 0.0971 rad

$\theta_{D'} = \theta_{C'} = \theta_{E'} = 0.0971$ rad

$\theta_{E'} = \theta_B + \theta_b$
= 0.0041 rad + 0.093 rad
= 0.0971 rad

$\theta_{IO} = \theta_{B'} + \theta_{IO,p}$
= 0.0041 rad + 0.059 rad
= 0.0631 rad

9.8.6 Foundation Modeling and Acceptance Criteria (ASCE 41-13 Chapter 8)

The following foundation analysis parameters were determined from a site-specific geotechnical investigation. Chapter 5 of this *Guide* provides discussion on additional foundation modeling parameters. This section provides an example of Method 1 from ASCE 41-13 Chapter 8 with footings connected by tie beams idealized as a coupled I-shaped footing.

- Ultimate bearing pressure = 8,000 psf
- Subgrade modulus = 100 pci = 14,400 psf/in.

For the existing 6 foot square spread footings:

P_{ultimate} = (8,000 psf)(1 kip/1,000 lb)(6 ft)(6 ft)
= 288 kips

K_{ultimate} = (14,400 psf/in.)(1 kip/1,000 lb)(6 ft)(6 ft)
= 518.4 k/in.

Per ASCE 41-13 § 8.4.2, upper- and lower-bound studies (involving soil spring strength and stiffness values equal to twice and half the values calculated above, respectively) are required. However, for brevity, only the upper-bound values will be considered in this example:

P_y $= 2P_{\text{ultimate}}$
$= 2(288 \text{ kips})$
$= 576 \text{ kips}$

K $= 2K_{\text{ultimate}}$
$= 2(518.4 \text{ k/in.})$
$= 1{,}036.8 \text{ k/in.}$

Per ASCE 41-13 § 8.4.2.3.3, the soil springs are modeled with trilinear force-deformation relationships per ASCE 41-13 Figure 8-4b.

The uplift capacity of the spring is set equal to the total weight the footing, slab, and soil directly above the footing (tributary to each soil spring):

A_{footing} = Footing area tributary spring
$= 6 \text{ ft} (6 \text{ ft}) + (2 \text{ ft})(30 \text{ ft} - 6 \text{ ft})/2$
$= 60 \text{ ft}^2$

$P_{D,SOG}$ = Slab on grade dead load
$= 0.150 \text{ kcf} (4 \text{ in.})(1 \text{ ft}/12 \text{ in.})(60 \text{ ft}^2)$
$= 3 \text{ kips}$

$P_{D,\text{soil}}$ = Soil dead load
$= 0.120 \text{ kcf} (8 \text{ in.}) (1 \text{ ft}/12 \text{ in.})(60 \text{ ft}^2)$
$= 5 \text{ kips}$

$P_{D,\text{footing}}$ = Footing dead load
$= 0.150 \text{ kcf} (60 \text{ ft}^2)(3 \text{ ft})$
$= 27 \text{ kips}$

T_y = Total uplift load
$= P_{D,SOG} + P_{D,\text{soil}} + P_{D,\text{footing}}$
$= 35 \text{ kips}$

Modeling parameters and acceptance criteria are calculated per ASCE 41-13 Table 8-4 considering the two spread footings below the braced frame columns as a coupled I-shaped footing (case "c" per ASCE 41-13 Figure 8-3), as shown in Figure 9-31.

$B = 6 \text{ ft}$

$t_f = 6 \text{ ft}$

$t_w = 2 \text{ ft}$

$L_{\text{bay}} = 30 \text{ ft}$

$L = L_{\text{bay}} + t_f = 30 \text{ ft} + 6\text{ft} = 36 \text{ ft}$

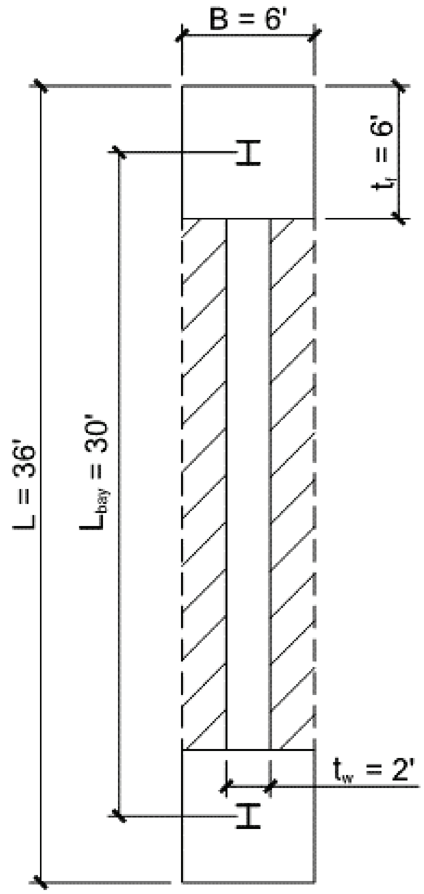


Figure 9-31 Foundation diagram.

$$A_f = 2Bt_f + t_w(L_{\text{bay}} - t_f) = 2(6\text{ ft})(6\text{ ft}) + (2\text{ ft})(30\text{ ft} - 6\text{ ft})$$
$$= 120\text{ ft}^2$$

$$A_{\text{rect}} = BL = (6\text{ ft})(36\text{ ft}) = 216\text{ ft}^2$$

$$q_c = 2q_{\text{ultimate}} = 2(8{,}000\text{ psf}) = 16{,}000\text{ psf}$$

$$P_D = P_{D,\text{col}} + P_{D,SOG} + P_{D,\text{soil}} + P_{D,\text{footing}}$$
$$= 123\text{ kips} + 3\text{ kips} + 5\text{ kips} + 27\text{ kips}$$
$$= 158\text{ kips/column}$$

$$P_L = P_{L,\text{col}}$$
$$= 54\text{ kips/column}$$

$$P = 2(P_D + 0.25\,P_L)$$
$$= (2\text{ columns})(158\text{ kips/column} + 0.25(54\text{ kips/column}))$$
$$= 343\text{ kips}$$

$$A_c = P/q_c$$
$$= (343\text{ kips})(1{,}000\text{ lb/kip})/(16{,}000\text{ psf})$$
$$= 21\text{ ft}^2$$

$$\frac{A_{\text{rect}} - A_f}{A_{\text{rect}}} = \frac{216\ \text{ft}^2 - 120\ \text{ft}^2}{216\ \text{ft}^2} = 0.44$$

$$\frac{A_c}{A_f} = \frac{21\ \text{ft}^2}{120\ \text{ft}^2} = 0.18$$

Using the above parameters and linear interpolation in ASCE 41-13 Table 8-4, the following modelling parameters and acceptance criteria are calculated:

$g = \theta_g = 0.0118$ rad

$d = \theta_d = 0.1000$ rad

$f = 0.5$

$\text{IO} = \theta_{\text{IO}} = 0.0063$ rad

Assuming the axis of rotation is halfway between the braced frame columns, the corresponding axial spring displacements may be estimated as follows.

$\Delta_g = \theta_g (L_{\text{bay}}/2) = 0.0118$ rad (30 ft)(12 in./ft)/2
$= 2.12$ in.

$\Delta_d = \theta_d (L_{\text{bay}}/2) = 0.1000$ rad (30 ft)(12 in./ft)/2
$= 18.0$ in.

$\Delta_{IO} = \theta_{IO} (L_{\text{bay}}/2) = 0.0063$ rad (30 ft)(12 in./ft)/2
$= 1.13$ in.

The foundation backbone curve is presented in Figure 9-32.

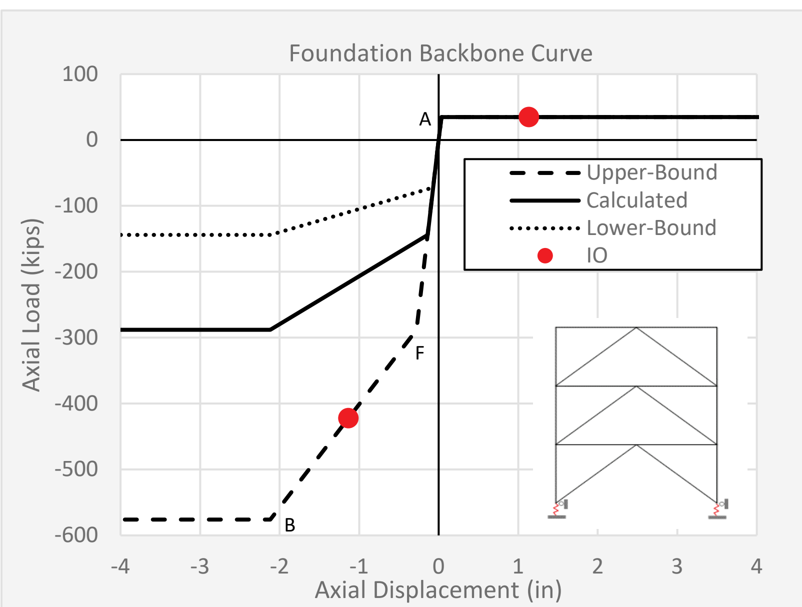


Figure 9-32 Foundation axial spring backbone curve.

Foundation rotations are monitored in the analysis model with rotation "gages" inserted between each braced frame column.

In this example, the structural capacity of the footing is assumed to exceed the bearing capacity of the soil. For a complete evaluation, the structural capacity of the footing should be calculated in accordance with ASCE 41-13 Chapter 10. If the structural capacity of the footing governs over the strength of the soil, the modelling parameters (i.e., backbone curve) and acceptance criteria should accordingly satisfy ASCE 41-13 Chapter 10.

9.8.7 Pushover Curve and Target Displacement (Flexible-Base Model)

The NSP target displacement is calculated per ASCE 41-13 § 7.4.3.2. As shown in the following figures, an idealized pushover curve is derived from the "actual" pushover curve determined from the nonlinear analysis using the graphical procedure outlined in ASCE 41-13 Figure 7-3. The first iteration of the pushover is shown in Figure 9-33.

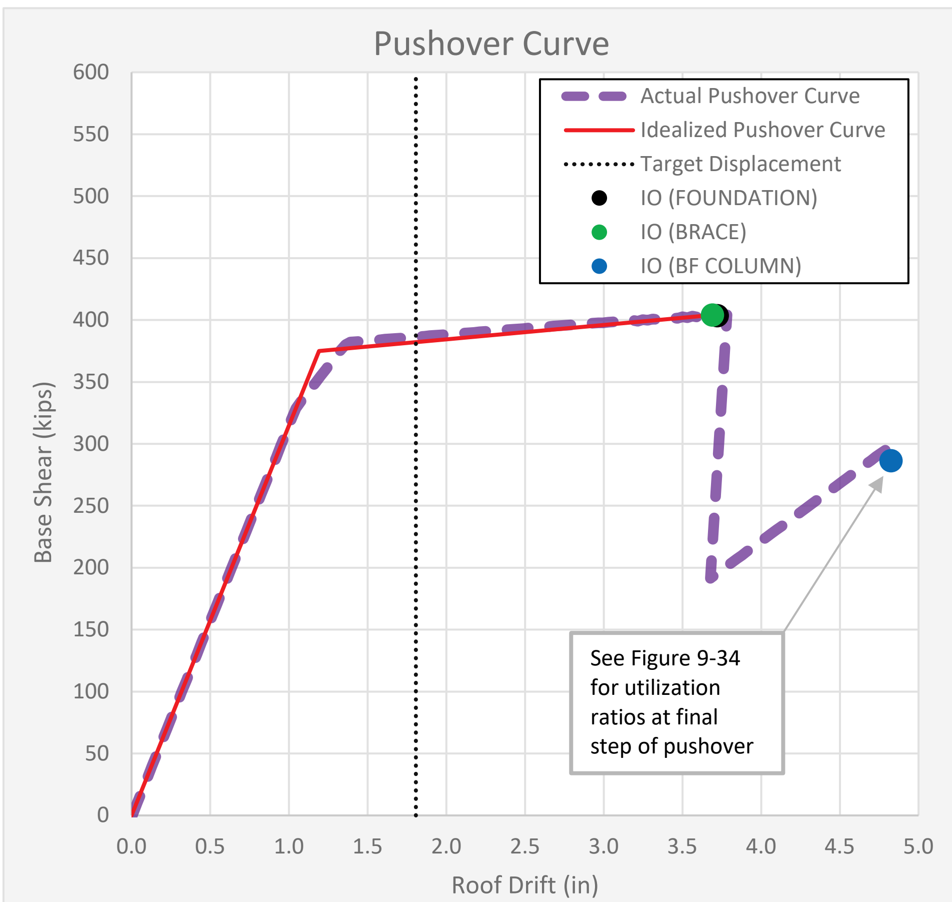


Figure 9-33 Pushover curve (flexible-base model).

V_y = 375 kips

K_i = 316 k/in.

K_e = 315 k/in.

T_i = 0.926 sec

T_e $= T_i\sqrt{K_i / K_e}$ (ASCE 41-13 Eq. 7-27)

$= (0.926 \text{ sec})\sqrt{(316 \text{ k/in.}) / (315 \text{ k/in.})}$

$= 0.927$ sec

S_a $= S_{X1}/T_e = 0.164/0.933 = 0.177\text{g}$

W = 3,839 kips

C_m = 0.8919 per analysis model

$\mu_{\text{strength}} = S_a C_m/(V_y/W)$

= (0.177)(0.8919)/(375/3839)

= 1.62

a = 60 for Site class D

C_0 = 1.2 (ASCE 41-13 Table 7-5)

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{aT_e^2}$$

$$= 1 + \frac{1.62 - 1}{60(0.927)^2}$$

= 1.012

$$C_2 = 1 + \frac{1}{800}\left(\frac{1 - \mu_{\text{strength}}}{T_e}\right)^2$$

$$= 1 + \frac{1}{800}\left(\frac{1 - 1.62}{0.927}\right)^2$$

= 1.001

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$= 1.2(1.012)(1.001)(0.177)\frac{(0.927)^2}{4\pi^2}(386.1)$$

= 1.81 in.

All components reach their IO acceptance criteria after the target displacement. However, several items that have been omitted from this evaluation for brevity should be reiterated. For a complete evaluation:

- Braced frame beam and column modelling parameters and acceptance criteria could be recalculated based on the axial compression loads, *P*, corresponding to the target displacement. As discussed in Sections 9.8.3.1 and 9.8.4.1 of this example, limit state analysis axial loads calculated in the previous LSP were used initially as a conservative estimate.
- The strength of the braced frame beam-to-column connections (with gusset plates) should be evaluated as force-controlled.
- The column-to-foundation anchorage strength should be evaluated, as discussed in Section 9.8.1 of this example.
- Lower-bound soil strength/stiffness properties should be considered, as discussed in Section 9.8.6 of this example.
- The structural capacity of the foundation components (i.e. square footing and grade beams) should be evaluated in accordance with ASCE 41-13 Chapter 10, as discussed in Section 9.8.6 of this example.
- An increased target displacement accounting for 3D building torsion effects should be considered, since a 2D model was used.
- Similarly, a representative secondary interior gravity column should be slaved at each diaphragm level and evaluated, or a 3D model with all gravity columns would need to be used for the analysis.

It should also be reiterated that the acceptance criteria were met in this example after a partial retrofit of the braced frame beams, as discussed in Section 9.8.3 of this example.

Figure 9-34 shows utilization ratios at the final step of the pushover. Utilization ratios equal the nonlinear displacement demands divided by the nonlinear displacement capacities (acceptance criteria).

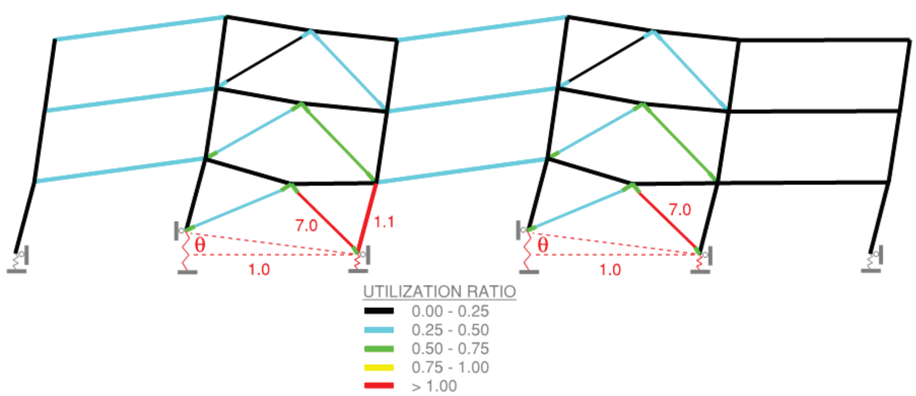


Figure 9-34 Pushover graphics (flexible-base model).

9.8.8 *Pushover Curve and Target Displacement (Fixed-Base Model)*

As shown in the previous section, the building behavior is dominated by foundation rocking, which occurs prior to significant yielding in the superstructure. The following fixed-base analysis results are therefore provided in this example to show the nonlinear behavior of older braced frames that are restrained from uplift.

V_y = 402 kips

K_i = 482 k/in.

K_e = 481 k/in.

T_i = 0.765 sec

$T_e = T_i\sqrt{K_i / K_e}$ (ASCE 41-13 Eq. 7-27)

$= (0.765\text{ sec})\sqrt{(482\text{ k/in.}) / (481\text{ k/in.})}$

$= 0.766$ sec

$S_a = S_{X1}/T_e = 0.164/0.766 = 0.214\text{g}$

W = 3,839 kips

C_m = 0.9070 per analysis model

$\mu_{\text{strength}} = S_a C_m/(V_y/W)$

$= (0.214)(0.9070)/(402/3839)$

$= 1.85$

a = 60 for Site class D

C_0 = 1.2 (ASCE 41-13 Table 7-5)

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{aT_e^2}$$

$$= 1 + \frac{1.85 - 1}{60(0.766)^2}$$

$= 1.024$

$$C_2 = 1 + \frac{1}{800}\left(\frac{1 - \mu_{\text{strength}}}{T_e}\right)^2$$

$$= 1 + \frac{1}{800}\left(\frac{1 - 1.85}{0.766}\right)^2$$

$= 1.002$

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$= 1.2(1.024)(1.002)(0.214)\frac{(0.766)^2}{4\pi^2}(386.1)$$

$= 1.51$ in.

As shown above, the fixed-base reduces the target displacement. The first-story braces quickly exceed their IO acceptance criteria and begin degrading in stiffness prior reaching the target displacement (refer to Figure 9-35 and Figure 9-36). For this example, the frame line meets the Immediate Occupancy performance level with foundation modeling but does not meet it by a large margin with base fixity. Per ASCE 41-13 § 8.4.2.3.2.1, the fixed-based assumption is not permitted for buildings being evaluated or retrofitted to Immediate Occupancy Performance Level that are sensitive to base rotations. While this example was not negatively sensitive to base rotations, the vast difference in building performance with and without foundation modeling clearly illustrates the importance of considering foundation stiffness in understanding the seismic response behavior.

The first-story braced frame beams eventually yield, and the first-story braced frame columns eventually reach their acceptance criteria after the target displacement. This analysis illustrates the secondary positive stiffness provided by the braced-frame beams after the braces buckle.

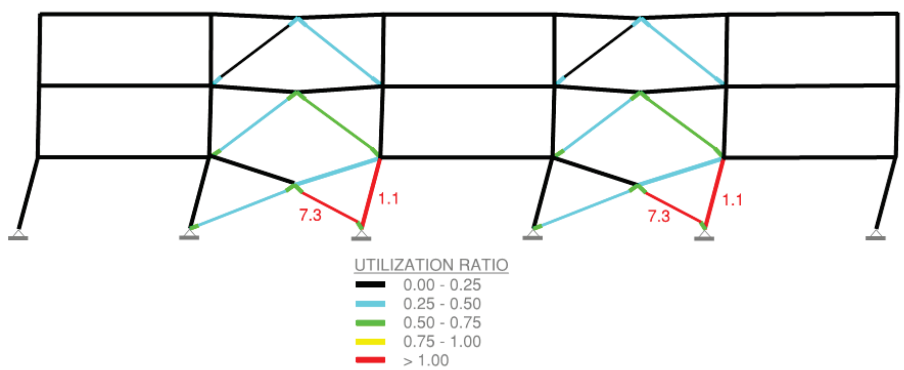


Figure 9-35 Pushover graphics (fixed-base model).

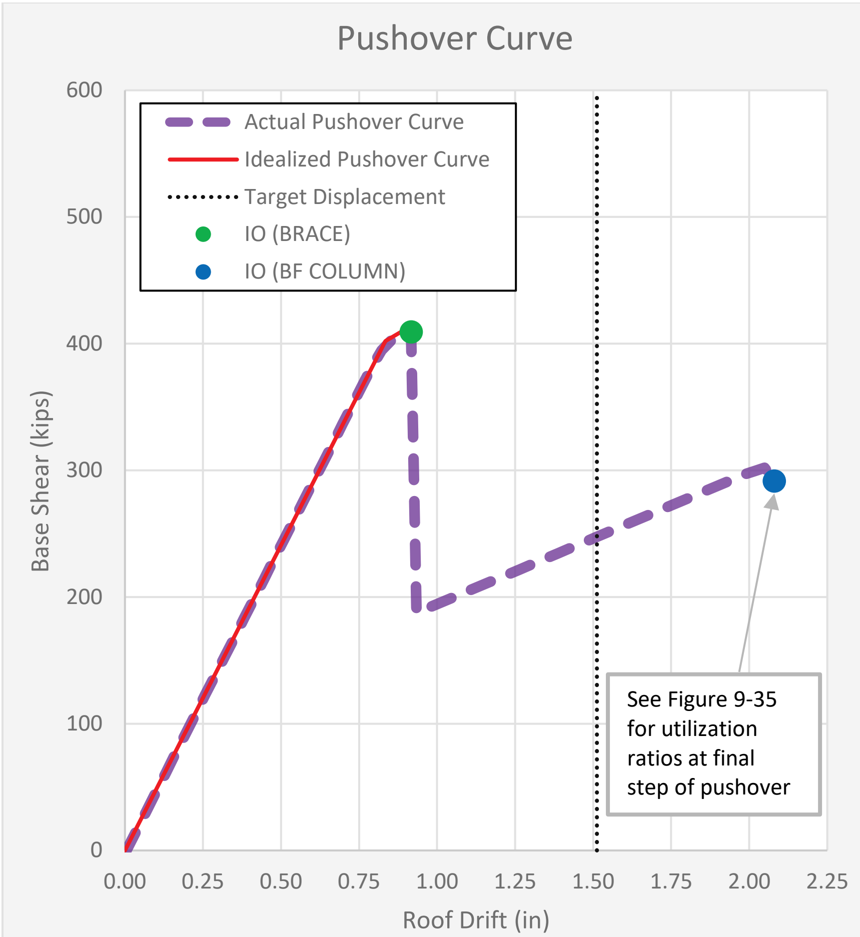


Figure 9-36 Pushover curve (fixed-base model).

Chapter 10

Concrete Shear Wall (C2) with Linear Static Procedure

10.1 Overview

This chapter provides discussion and example application of the Tier 3 systematic evaluation and retrofit procedures of ASCE 41-13 (ASCE, 2014) on a 1950s three-story concrete shear wall building in Seattle, Washington. The Tier 3 evaluation uses Basic Performance Objective for Existing Buildings (BPOE) and Risk Category II.

This example shows data collection requirements, evaluation of the lateral force-resisting system with added shear walls, design of fiber-reinforced polymer (FRP) reinforcing at the discontinuous columns and under-reinforced concrete walls, a diaphragm check and collector design where the discontinuous shear wall terminates at the first floor level, and evaluation of compatibility with the non-participating concrete beam/column frame.

This example illustrates the following:

- **Section 10.2:** Deficiencies identified from Tier 1 Screening and mitigation strategies.
- **Section 10.3**: Data collection requirements (ASCE 41-13 § 6.2 and § 10.2).
- **Section 10.4:** Analysis of the building using the linear static procedure (LSP) with the following calculations:
  - Preliminary pseudo seismic forces (ASCE 41-13 § 7.4.1.3)
  - Shear wall analysis and preliminary demand-capacity ratio (DCR) values for subject components (ASCE 41-13 § 7.4.1 and § 10.7.2.3)
  - Determination of C_1 and C_2 modification factors in each direction (ASCE 41-13 § 7.4.1.3)
  - Determination of final pseudo-seismic forces to be applied to the building (ASCE 41-13 § 7.4.1)
  - Determination of final DCR values for components using *m*-factors where they apply (ASCE 41-13 § 7.4.1 and § 10.7)

Example Summary

Building Type: C2

Performance Objective: BPOE

Risk Category: II

Location: Seattle, Washington

Level of Seismicity: High

Analysis Procedure: Linear Static (LSP)

Retrofit Procedure: Tier 3

Reference Document: ACI 318-11

- Determination of final element acceptance criteria (ASCE 41-13 § 7.5.2 and § 10.7)
- Confirmation of applicability of linear static procedure (ASCE 41-13 § 7.3.1.1)
- Column check at end of discontinuous center transverse concrete shear wall (ASCE 41-13 § 7.5.2.1 and § 10.7.1.2)

- **Section 10.5:** Evaluation of the lateral force-resisting system with added shear walls, including foundation check at selected location (ASCE 41-13 § 7.5.2.1, § 8.4.2.3.2.1, and § 10.7).
- **Section 10.6:** Rigid diaphragm check at first level (ASCE 41-13 § 10.7).
- **Section 10.7:** FRP reinforcing for existing concrete wall.
- **Section 10.8:** FRP reinforcing for existing concrete column supporting discontinuous shear wall.
- **Section 10.9:** Check of non-contributing concrete columns (ASCE 41-13 § 7.5.1.1).

This example is not a complete evaluation of the renovated structure, and it focuses only on selected structural elements. The building geometry and much of the analysis were taken from a Structural Engineers Association of Washington seminar example using ASCE 41-06 (SEAW, 2008). Chapter 11 of this *Guide* provides a nonlinear static procedure (NSP) evaluation of the same building shown here in Chapter 10.

10.2 Introduction

10.2.1 Building Description

The building is a three-story office building located in Seattle, Washington, illustrated in Figure 10-1, with a plan from the upper floors in Figure 10-2, the basement in Figure 10-3, and an elevation of the discontinuous interior shear wall in Figure 10-4. A seismic upgrade in accordance with ASCE 41-13 is being performed voluntarily at the request of the owner.

Building information:

- Concrete shear wall building (Type C2)
- Office use: Risk Category II
- Three stories above grade at Level 1 with a one-story basement below grade. The building is 42 feet tall above Level 1
- Floor live load: 125 psf (file storage throughout building)

Commentary

For the purposes of this design example, Level 1 is used as the base of the building with a focus on demonstrating how to apply ASCE 41-13. Modeling the building considering soil interaction at the basement level and foundations along with stiffness of structural elements is a common analysis method to capture the behavior of the structure. It greatly assists in determining displacement of structural elements as noted throughout this example. This analysis method is illustrated in the Chapter 11 design example in this *Guide*.

- Roof snow load: 25 psf
- Floor and roof dead load: 100 psf
- Seismic mass of building: 2,880 kips

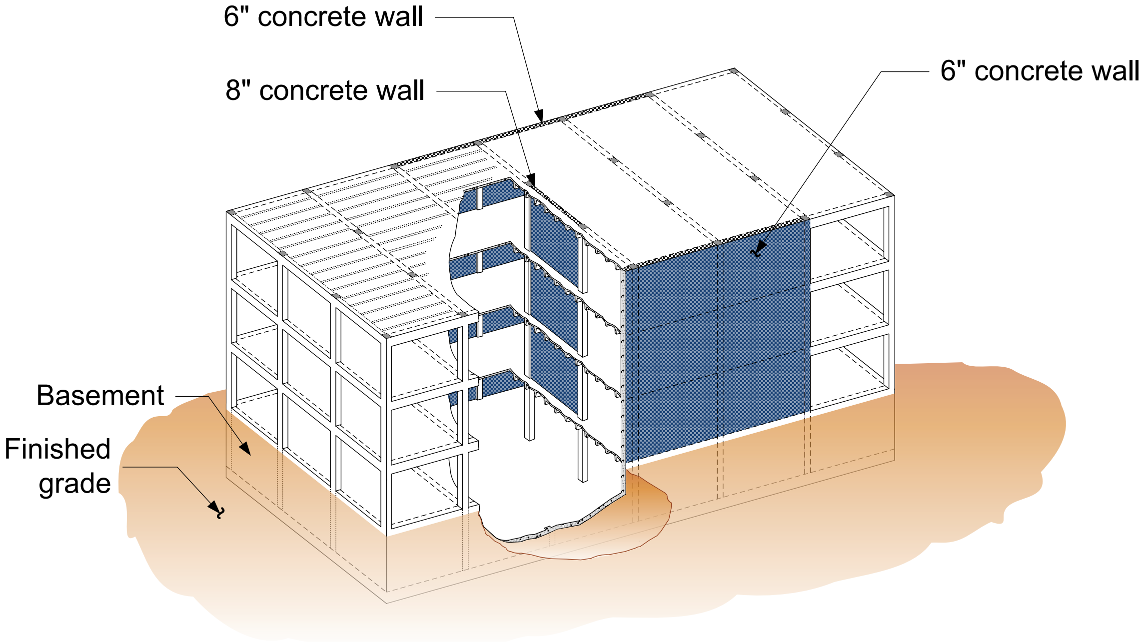


Figure 10-1 Three-dimensional view of the existing building.

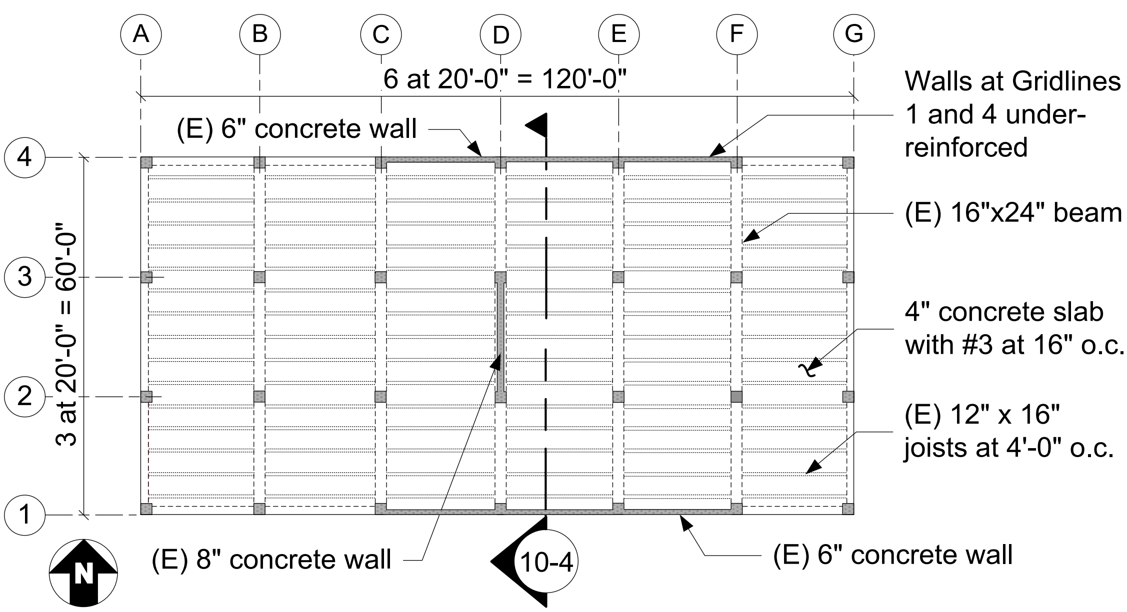


Figure 10-2 Floor plan for Levels 1, 2, and 3.

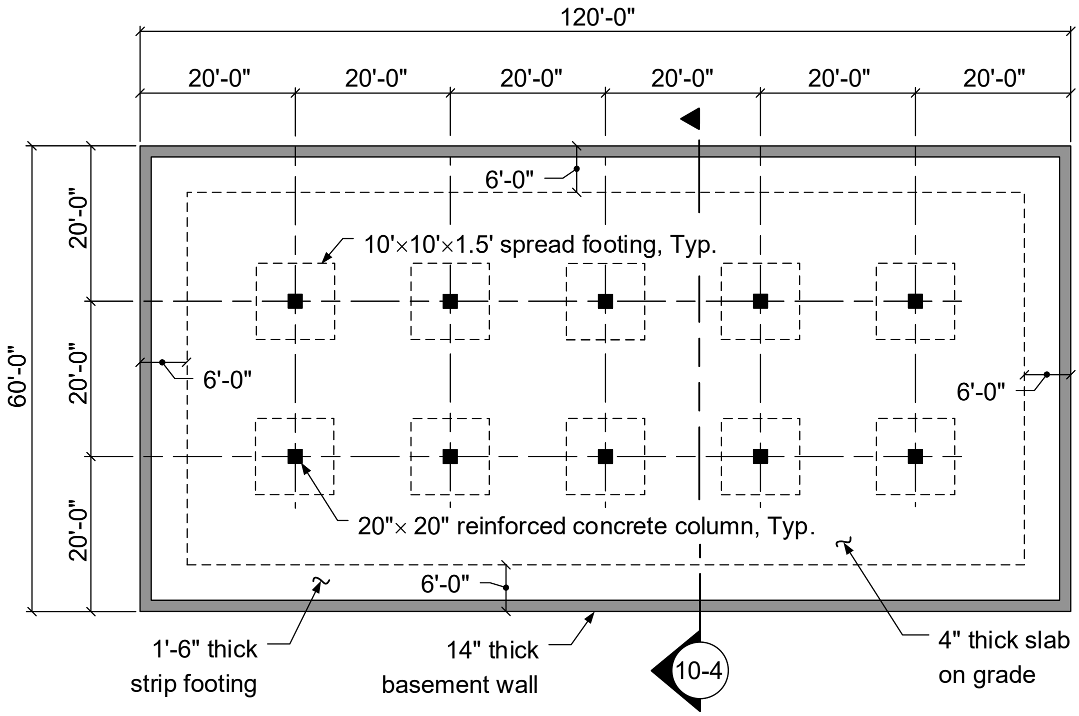


Figure 10-3 Basement plan.

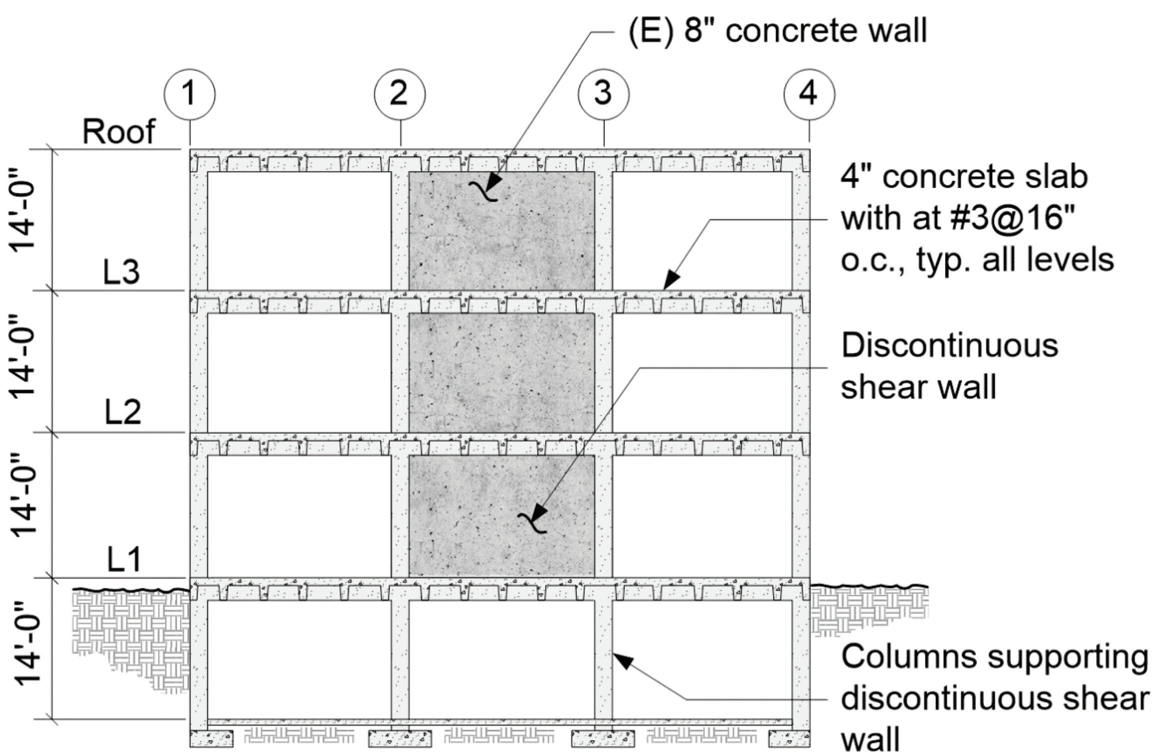


Figure 10-4 Building section/Wall D elevation.

10.2.2 Tier 1 Screening and Mitigation Strategy

An ASCE 41-13 Tier 1 evaluation was performed and the following structural deficiencies were identified, as shown in Figure 10-5:

- Vertical irregularity due to discontinuity at existing transverse shear wall along Gridline D
- Only one line of shear wall in the transverse direction
- Transverse wall at Gridline D did not pass the shear Quick Check
- Existing six-inch walls along Gridlines 1 and 4 do not meet minimum horizontal reinforcing requirements

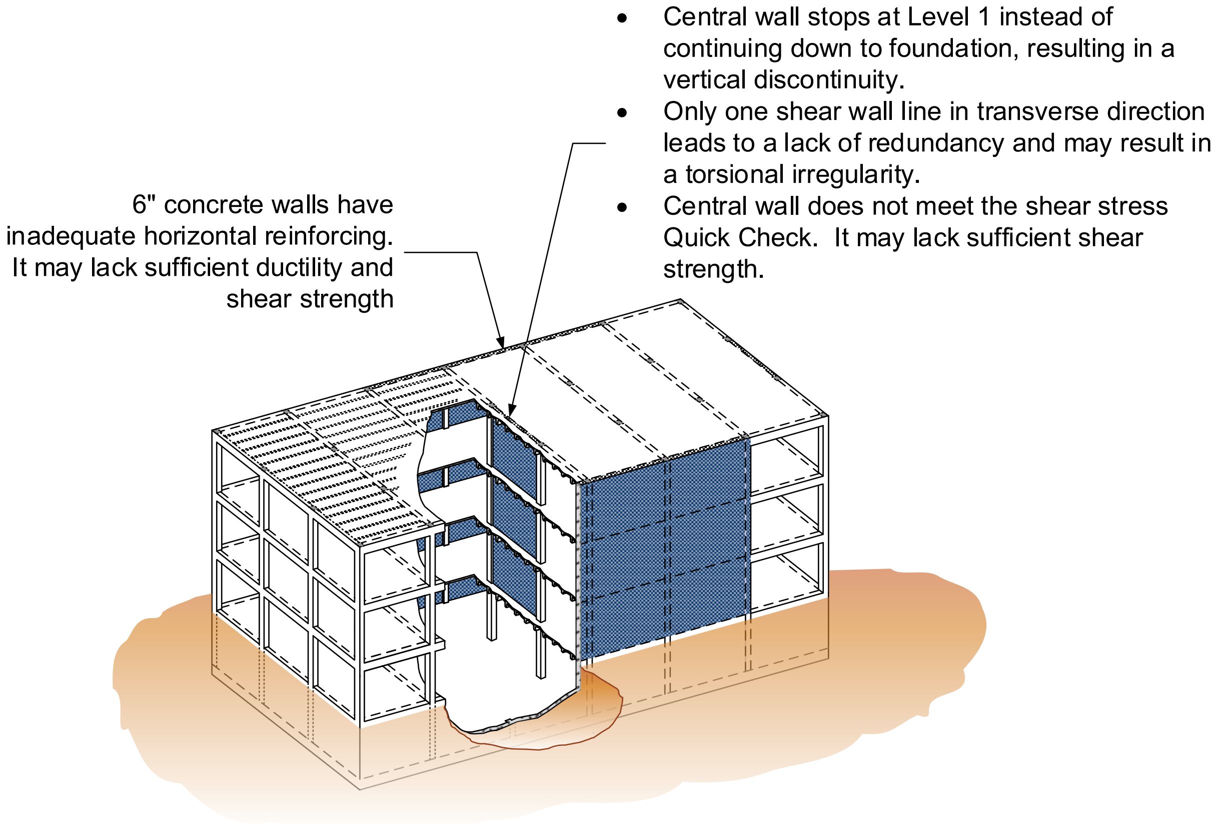


Figure 10-5 Tier 1 screening deficiencies in example building.

The seismic upgrade of the building is being done voluntarily. The owner and design team mitigation strategy for the building was selected to go directly to a Tier 3 systematic retrofit to address the significant seismic concerns identified in the ASCE 41-13 Tier 1 evaluation. To mitigate the seismic concerns, it was determined to add transverse concrete shear walls at each end of the building along Gridlines A and G to provide additional strength and stiffness in the transverse direction and to reduce seismic force and overturning demands on the existing discontinuous concrete shear wall at

Commentary

A Tier 2 deficiency-based analysis or retrofit could have been considered with a check of the BSE-1E Seismic Hazard Level for Life Safety Performance Level. This example goes directly to a Tier 3 systematic retrofit.

Gridline D. The preferred solution would have been for the concrete shear wall at Gridline D to continue to the foundation; however, the client does not want interior walls at the basement level as they would detrimentally impact the function of the space. The existing under-reinforced concrete shear walls along Gridlines 1 and 4 will be evaluated as force-controlled elements since they do not meet minimum horizontal reinforcing requirements. This example verifies the adequacy of the mitigation strategy and investigates if additional structural mitigation is required to meet the selected performance objective. Ultimately, additional strengthening of the concrete columns at the end of the discontinuous concrete shear wall at Gridline D, under-reinforced concrete shear walls at Gridlines 1 and 4, and diaphragm collector at the first level are determined to be required.

10.2.3 Seismic Design Parameters and Performance Objective (ASCE 41-13 § 2.2 and § 2.4.1)

The Basic Performance Objective for Existing Buildings (BPOE) was selected based on a voluntary seismic upgrade. ASCE 41-13 Table 2-1, "Basic Performance Objective for Existing Buildings (BPOE)," shows that for buildings of Risk Category II, the Tier 3 screening for the BSE-1E and BSE-2E Seismic Hazard Levels shall be based on Life Safety and Collapse Prevention Structural Performance, respectively. With BSE-1E being approximately 60% of BSE-2E Seismic Hazard Levels in the Seattle area, BSE-2E generally controls with the Collapse Prevention (CP) Performance Level. Accordingly, the design example illustrates the use of the BSE-2E Seismic Hazard Level with the CP Performance Level. At the end of the example, the final demand-capacity ratios (DCRs) for the acceptance criteria using the BSE-1E Seismic Hazard Level with the Life Safety (LS) Performance Level are also provided.

The site class, latitude, and longitude for the building are as follows:

- Site Class D
- Latitude: 47.6143°N
- Longitude: 122.3358°W

Spectral response acceleration parameters were obtained using the online tools described in Chapter 3 of this *Guide*, and are shown in Table 10-1 and Figure 10-6. For the short period and one-seconds acceleration, the BSE-1E and BSE-2E values are less than the associated BSE-1N and BSE-2N values, and so the uncapped 20% in 50-year and 5% in 50-year values are used for BSE-1E and BSE-2E. See Section 3.3.2 of this *Guide* for details and an example.

Table 10-1 Spectral Accelerations for Site in Seattle, Washington, Site Class D

Value	ASCE 41-13 Section 2.4.1 Spectral Ordinates				Uncapped BSE-2E and 1E	
	BSE-2N	BSE-1N	BSE-2E	BSE-1E	5% in 50 yr	20% in 50 yr
S_{XS} (g)	1.35	0.903	1.08	0.691	1.08	0.691
S_{X1} (g)	0.79	0.524	0.62	0.376	0.62	0.376
T_o (sec)	0.12	0.12	0.11	0.11	0.11	0.11
T_s(sec)	0.59	0.58	0.57	0.55	0.57	0.55

Figure 10-6 illustrates the general horizontal response spectra obtained using values from the online tools described in Chapter 3 of this *Guide*.

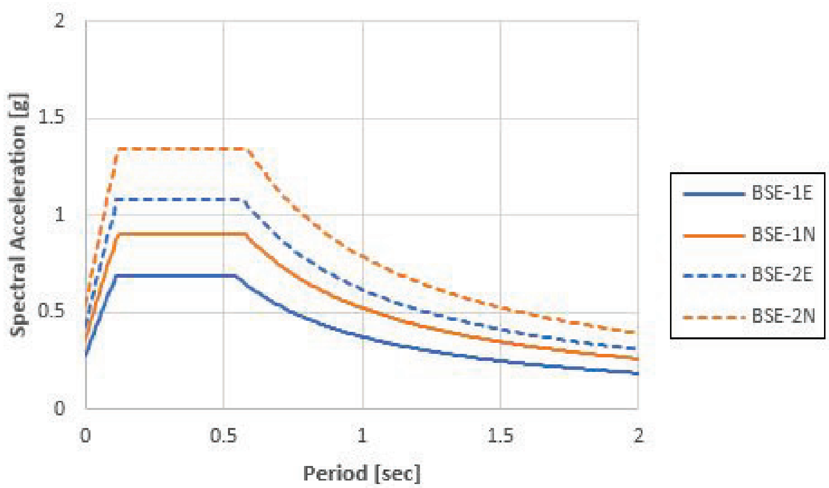


Figure 10-6 General response spectrum per ASCE 41-13 § 2.4.1.7.

10.2.4 Level of Seismicity (ASCE 41-13 § 2.5)

From the USGS website, S_{DS} and S_{D1} for BSE-1N are as follows (See Section 3.4 of this *Guide* for example of how to determine the Level of Seismicity):

$$S_{DS} = 0.903$$

$$S_{D1} = 0.524$$

Per ASCE 41-13 Table 2-5:

$$S_{DS} = 0.903 \geq 0.50$$

and:

$$S_{D1} = 0.524 \geq 0.20$$

Therefore, the site is in a High Level of Seismicity.

10.3 Data Collection Requirements (ASCE 41-13 § 6.2 and § 10.2)

Construction drawings were available for this building, identifying material properties, soil bearing capacity, and details for structural elements. According to ASCE 41-13 Table 6-1, for minimum level of knowledge with design drawings, knowledge factor, κ, is taken as 0.9. Per ASCE 41-13 § 6.2.4.2, a minimum level of knowledge is acceptable for linear procedures. A field assessment was completed per ASCE 41-13 § 10.2.3.2.1, Visual Condition Assessment, where condition and configuration of the structural elements were field verified and found to be consistent with original design drawings.

> **Commentary**
>
> All existing rebar development, hook, and lap lengths should be checked for adequacy. For this design example, all development, hook, and lap lengths are assumed to be adequate. When development, hook, and lap lengths are not adequate, refer to ASCE 41-13 § 10.3.5.

For the nonlinear portion of this example shown in Chapter 11, or to use a knowledge factor of 1.0, a "usual" level of knowledge is required. This level includes available original design drawings specifying the structural material properties and testing of the structural elements per ASCE 41-13 § 10.2.2.3, Test Methods to Quantify Material Properties, and ASCE 41-13 § 10.2.2.4, Minimum Number of Tests. For the concrete compressive strength per ASCE 41-13 § 10.2.2.4.1, Usual Data Collection, a minimum of one core per concrete strength used in the design or three minimum for the building are required. The specified concrete strength for all the structural elements in this example is the same, so a minimum of three cores is required. Per the same section in ASCE 41-13, to determine the yield and ultimate strength of the reinforcing steel, no testing is required since it is specified on the original design drawings.

> **Useful Tip**
>
> For force-controlled actions, lower bound material strengths are used. For deformation-controlled actions, expected strengths are used.

Material properties for existing elements are obtained from original design drawings. Per ASCE 41-13 § 10.2.2.1.2, the material properties specified on the construction documents are considered lower bound material properties. Corresponding expected material properties are calculated by multiplying lower bound properties by a factor taken from ASCE 41-13 Table 10-1. In this case, the factors are 1.5 for concrete compressive strength and 1.25 for reinforcing yield strength. Table 10-2 and Table 10-3 below summarize properties for existing walls and columns.

Concrete:

f'_{cl} = 2,500 psi (lower bound)

f'_{ce} = 1.5 × 2,500 psi = 3,750 psi (expected)

Reinforcing steel:

f_{yl} = 40,000 psi (lower bound)

f_{ye} = 1.25 × 40,000 psi = 50,000 psi (expected)

Table 10-2 Existing Wall Properties

Wall	Length (ft)	Thickness (in)	Reinforcing	End Column Size and Reinforcing At First Story	
1	60.0	6	#3 @ 18″ OC EW	22″ × 22″	(8) #9
4	60.0	6	#3 @ 18″ OC EW	22″ × 22″	(8) #9
D	20.0	8	#4 @ 16″ OC EW	22″ × 22″	(8) #9

Table 10-3 Existing Column Properties

Story	Size (in)	f'_c (psi)	Reinforcing	Ties
3rd	14 × 14	2,500	(4) #7	#4 @ 10″
2nd	16 × 16	2,500	(8) #6	#4 @ 10″
1st	18 × 18	2,500	(8) #7	#4 @ 10″
Basement	20 × 20	2,500	(8) #9	#4 @ 10″

10.4 Linear Static Procedure

In this section, the linear static procedure (LSP) is conducted for seismic analysis of the building. Pseudo seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements will be determined using linearly elastic, static analysis in accordance with ASCE 41-13 § 7.4.1.3 and compared against the acceptance criteria of ASCE 41-13 § 7.5.2.

10.4.1 Preliminary Pseudo Forces (ASCE 41-13 § 7.4.1.3.1)

The following method for determining the pseudo seismic force is an iterative process whereby a preliminary pseudo seismic force is used to determine preliminary maximum DCRs. Based on the DCR_{max} values in each direction, the final pseudo seismic forces are determined.

One shortcut to get the preliminary pseudo seismic force closer to the final value is to substitute the maximum *m*-factor in the direction under consideration for DCR_{max} in ASCE 41-13 Equation C7-3 to determine $\mu_{strength}$ and use this value to calculate C_1 and C_2.

ASCE 41-13 § 7.4.1.3 specifies that "Forces and deformations in elements and components shall be calculated for the pseudo seismic force of Section 7.4.1.3.1, using component stiffnesses calculated in accordance with Chapter 8 through Chapter 12." Accordingly, the preliminary pseudo seismic force, V_0, is determined using ASCE 41-13 Equation 7-21as follows

$$V_{Preliminary} = C_1 C_2 C_m S_a W \qquad \text{(ASCE 41-13 Eq. 7-21)}$$

Useful Tip

A less rigorous direct method is selecting C_1 and C_2 values from ASCE 41-13 Table 7-4 based on fundamental period and maximum *m*-factor or m_{max} used. This method is illustrated in Section 4.3 of this *Guide* and compared with the final C_1 and C_2 modification factors using the iterative method.

As defined for ASCE 41-13 Equation 7-22, "T = Fundamental period of the building in the direction under consideration, calculated in accordance with Section 7.4.1.2, including modification for SSI effects of Section 7.2.7, if applicable:" Therefore, the building period, T, is determined using ASCE 41-13 Equation 7-18 as follows:

$T = C_t h_n^{\beta}$, Method 2 (ASCE 41-13 Eq. 7-18)

where:

$C_t = 0.020$, $\beta = 0.75$

$h_n = 42$ ft

$T = 0.020(42\text{ ft})^{0.75} = 0.33$ seconds (same in both directions)

To determine DCR_{max} in ASCE 41-13 Equation C7-3, use $C_1 = C_2 = C_m = 1.0$ because ASCE 41-13 § C7.4.1.3.1 specifies "where DCR_{max} is the largest DCR computed for any primary component of a building in the direction of response under consideration, taking $C_1 = C_2 = C_m = 1.0$." The calculation of S_a in ASCE 41-13 Equation 7-21 is specified as "The value of S_a shall be obtained from the procedure specified in Section 2.4." As shown in ASCE 41-13 Equations. 2-5 to 2-10, T_0, T_S, S_{X1}, and S_{XS} are required to determine S_a. Therefore, these response acceleration parameters are determined per ASCE 41-13 § 2.4.1:

$T_0 = 0.2T_S$ (ASCE 41-13 Eq. 2-9)

$T_S = S_{X1}/S_{XS}$ (ASCE 41-13 Eq. 2-10)

One-second period spectral response parameter, S_{X1}:

$S_{X1} = 0.62$ (Given in Section 10.2.3 of this *Guide*)

Short-period spectral response parameter, S_{XS}:

$S_{XS} = 1.08$ (Given in Section 10.2.3 of this *Guide*)

$T_S = 0.62/1.08$ seconds $= 0.574$ seconds

$T_0 = 0.2(0.574)$ seconds $= 0.115$ seconds

$T_0 \leq T \leq T_S \longrightarrow S_a = S_{XS}/B_1$ (ASCE 41-13 Eq. 2-6)

Modifier B_1 is 1.0 for 5% damping: (ASCE 41-13 Eq. 2-11)

Useful Tip

Applying ASCE 41-13 Equation 2-11 with 5% damping results in modifier $B_1 = 1.0$.

$S_a = S_{SX}/B_1 = 1.08/1.0 = 1.08$

$$
\begin{aligned}
V_{\text{Preliminary}} &= C_1 C_2 C_m S_a W \\
&= 1.0(1.0)(1.0)(1.08)\ W \\
&= 1.08\ (2{,}880\text{ kips}) \\
&= 3{,}110\text{ kips}
\end{aligned}
$$

10.4.2 Preliminary Story Forces (ASCE 41-13 § 7.4.1.3.2)

Vertical distribution of seismic forces (story forces), F_x, are determined per ASCE 41-13 § 7.4.1.3.2, which is required to be applied to "all buildings except unreinforced masonry buildings with flexible diaphragms and seismically isolated structures."

$$F_x = C_{vx}V_0 \quad \text{(ASCE 41-13 Eq.7-24)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \quad \text{(ASCE 41-13 Eq. 7-25)}$$

where:

C_{vx} = the vertical distribution factor

w_x = the portion of the effective seismic weight, W, located on or assigned to level x

k = 1.0 since $T < 0.5$ seconds

Table 10-4 below summarizes the preliminary story forces.

Table 10-4 Preliminary Story Forces

Level	w_x (kips)	Story Height (feet)	h_x (ft)	$w_x h_x^k$ (kip-feet)	F_x (kips)	$\sum F_x$ (kips)
Roof	720	14	42	30,240	1,244	1,244
3	1,080	14	28	30,240	1,244	2,488
2	1,080	14	14	15,120	622	3,110
1	-	14	-	-	-	-
Total	2,880			75,600	3,110	

10.4.3 Preliminary Wall Demand (ASCE 41-13 § 7.4.1.3 and ASCE 41-13 § 10.7)

As indicated in Section 10.2.2 of this *Guide*, the proposed mitigation strategy is to add transverse concrete shear walls at each end of the building along Gridlines A and G to provide additional strength and stiffness in the transverse direction and to reduce seismic force and overturning demands on the existing discontinuous concrete shear wall at Gridline D, as shown in Figure 10-7.

First, the material properties for the new elements are assumed:

Concrete:

$$f'_{cl} = 5{,}000 \text{ psi}$$

$$f'_{ce} = 1.3 f'_c = 1.3(5{,}000 \text{ psi}) = 6{,}500 \text{ psi}$$

Commentary

A factor to translate lower-bound or design concrete strength to expected compressive strength for concrete is not specifically addressed in ASCE 41-13 for new concrete. A factor of 1.3 is commonly used. ASCE 41-13 Table 10-1 applies to existing materials.

Reinforcing steel:

f_{yl} = 60,000 psi

f_{ye} = $1.25f_y$ = 1.25(60,000 psi) = 75,000 psi

Table 10-5 summarizes the material properties for the new shear walls.

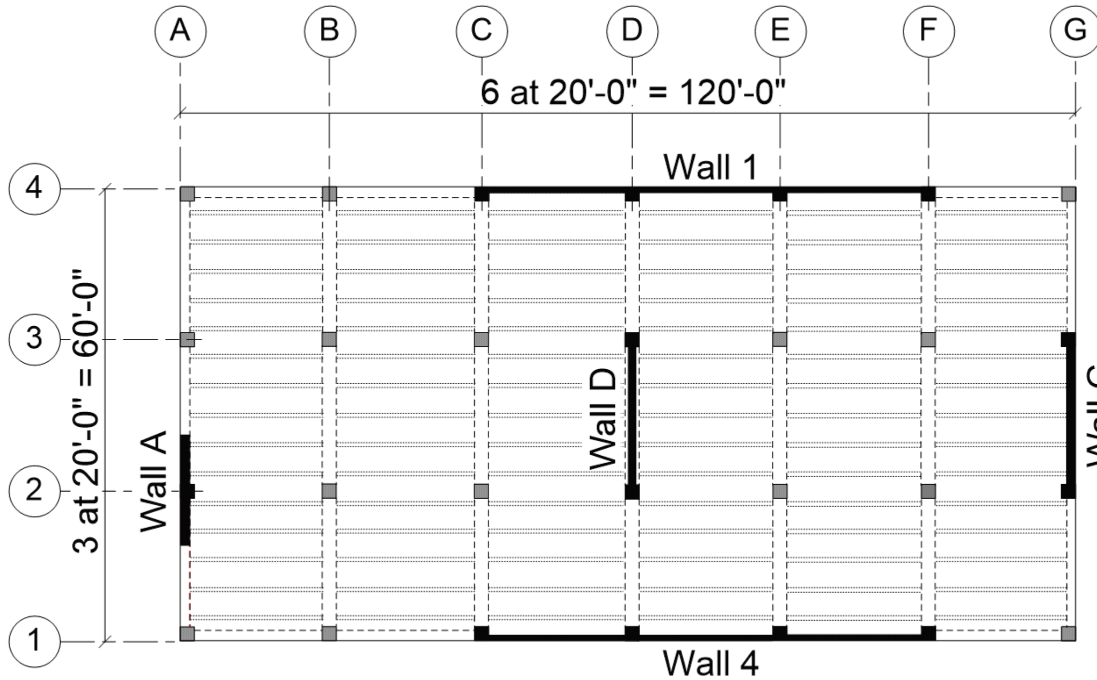


Figure 10-7 Plan of Levels 1, 2, and 3 of proposed mitigation strategy showing new shear walls.

Table 10-5 New Shear Wall Properties

Wall	Length (ft)	Thickness (inches)	Typical Reinforcing	Wall Boundary Reinforcing
A	14.0	24	#6 @ 9″ OC EF	(6) #6
G	20.0	10	#4 @ 12″ OC EF	(6) #8

Useful Tip

Developing a computer model of the building will enable the determination of distribution of forces and displacements, and would be beneficial. Although not required, the model could also be used to perform a linear dynamic procedure (LDP) more representative of the structure's behavior or to reduce design overturning moments.

Wall sizing and reinforcing are preliminary and will be confirmed based on deformation-controlled acceptance criteria at the end of this example.

Story forces are distributed to existing and preliminary added concrete shear walls using a spreadsheet based on a rigid concrete diaphragm analysis, relative rigidities of concrete shear walls, and 5% accidental torsion. Per ASCE 41-13 § 7.2.9.1, the concrete floor diaphragms are considered rigid with the displacements of the concrete diaphragm less than half the average story drift of the concrete shear walls of the story below the diaphragms.

Table 10-6 summarizes the wall demands where "W1FR" denotes the wall on Gridline 1 at the story below the roof.

Table 10-6 Element Demands per ASCE 41-13 § 7.3.1 with $V_{Preliminary} = S_aW$

Wall	Floor	ID[1]	% of story shear (kips)	Story Shear (kips)	Story Moment (kip-feet)	Shear V_{UD} (kips)	Moment M_{UD} (kip-feet)
1	R	W1FR	0.55	1,244	17,418	684	9,580
1	3	W1F3	0.55	2,488	52,255	1,369	28,740
1	2	W1F2	0.55	3,110	95,800	1,711	52,690
4	R	W4FR	0.55	1,244	17,418	684	9,580
4	3	W4F3	0.55	2,488	52,255	1,369	28,740
4	2	W4F2	0.55	3,110	95,800	1,711	52,690
A	R	WAFR	0.40	1,244	17,418	498	6,967
A	3	WAF3	0.40	2,488	52,255	995	20,902
A	2	WAF2	0.40	3,110	95,800	1,244	38,320
D	R	WDFR	0.23	1,244	17,418	286	4,006
D	3	WDF3	0.23	2,488	52,255	572	12,019
D	2	WDF2	0.23	3,110	95,800	715	22,034
G	R	WGFR	0.39	1,244	17,418	485	6,793
G	3	WGF3	0.39	2,488	52,255	970	20,379
G	2	WGF2	0.39	3,110	95,800	1,213	37,362

10.4.4 Preliminary Element DCRs (ASCE 41-13 § 7.3.1 and ASCE 41-13 § 10.7)

Element DCRs are calculated per ASCE 41-13 § 7.3.1.1:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \qquad \text{(ASCE 41-13 Eq.7-16)}$$

> **Commentary**
>
> ASCE 41-13 Equation 7-16 defines DCR as Q_{UD}/Q_{CE}. The capacity does not include the *m*-factor or κ. The DCR is thus a measure of required component ductility. In this *Guide*, the term "acceptance ratio" is used to define $Q_{UD}/(m\kappa Q_{CE})$.

For this initial DCR check, the deformation-controlled demands and expected capacities for all elements, regardless of behavior, are required to be used. Since the linear static procedure is used, per ASCE 41-13 §7.5.2.1.1 "Deformation-Controlled Actions for LSP or LDP," Q_{UD} is calculated as:

$$Q_{UD} = Q_G \pm Q_E \qquad \text{(ASCE 41-13 Eq.7-34)}$$

Q_{CE}, "Expected strength of the component or element, calculated as specified in Chapter 8 through Chapter 13," is determined for each element in accordance with ASCE 41-13 Chapter 10.

10.4.4.1 Wall 1 at Level 2, W1F2

The demands and calculations are similar for Wall 1 at Level 2 and Wall 4 at Level 2 (W4F2), as shown in Figure 10-8, with V_{UD} = 1,711 kips, M_{UD} = 52,690 kip-ft (see Table 10-6).

> **Commentary**
>
> Wall 1 is under-reinforced and will ultimately be evaluated as a force-controlled element. For the purposes of determining preliminary wall demands, the wall will be considered deformation-controlled, and expected material strengths will be used.

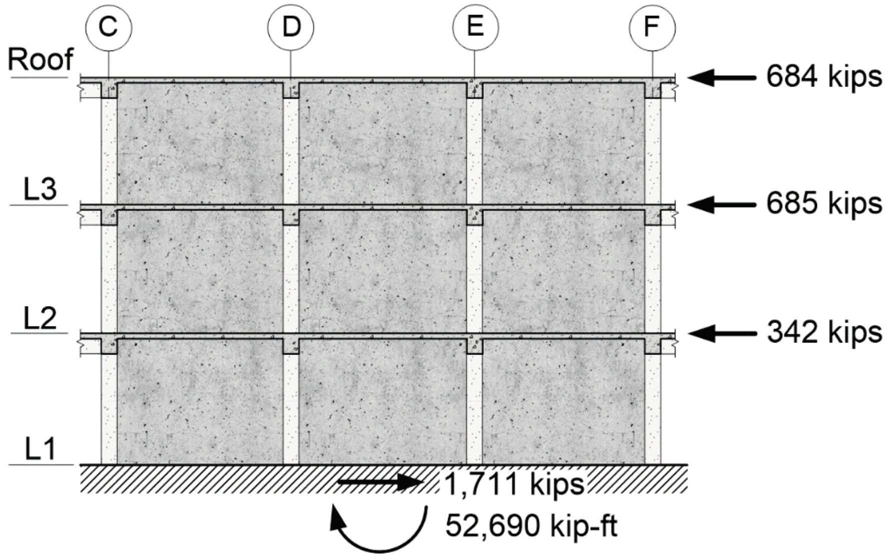


Figure 10-8 Shear wall 1 and 4 at Gridline 1 and Gridline 4.

Component strengths are determined in accordance with ASCE 41-13 § 10.7.2.3. ASCE 41-13 § 10.7.2.3 states that "Component strengths shall be computed according to the general requirements of Section 10.3.2, with the additional requirements of this section. Strength shall be determined considering the potential for failure in flexure, shear, or development under combined gravity and lateral load." So in the following calculations, both the flexure and shear strength under Q_G and Q_E is determined for each shear wall.

Use ACI 318-11 (ACI, 2011), with ϕ = 1.0:

Wall properties:	Material properties (expected strength):
h_w = height = 42 ft	f'_{ce} = 3,750 psi
l_w = length = 60 ft	f_{ye} = 50,000 psi
t = thickness = 6 in	

Determine DCR$_V$ = V_{UD}/V_{CE} for Shear Strength

The shear capacity of Wall 1 at Level 2 is determined per ACI 318-11 Equation 21-7:

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$$

where:

A_{cv} = net wall area

= l_w × thickness

= 6(60 ft)(12 in./ft)

= 4,320 in.^{2}

α_c is determined by h_w/l_w ratio:

h_w/l_w = 42 ft/60 ft = 0.7 < 1.5, therefore

α_c = 3.0

λ = 1.0 for normal weight concrete

ρ_t = ratio of transverse shear reinforcing
= #3 bars at 18" o.c.
= 0.11 in.^{2}/18 in./6 in.
= 0.0010

For expected strength determination, use f'_{ce} = 3,750 psi and f_{ye} = 50,000 psi

$$V_n = \left(4{,}320 \text{ in.}^2\right)[3.0(1.0)\sqrt{3{,}750 \text{ psi}} + 0.0010\left(50{,}000 \text{ psi}\right)] \text{ (ACI 318-11§ 11.9.3)}$$

$$= 1{,}010 \text{ kips} \le (10)A_{cp}\sqrt{f'_c}$$

$$= \frac{10\left(4{,}320 \text{ in.}^2\right)\sqrt{3{,}750} \text{ psi}}{1{,}000 \text{ lbs/kip}}$$

$$= 2{,}645 \text{ kips}$$

V_{CE} = V_n = 1,010 kips

DCR$_V$ = V_{UD}/V_{CE} = 1,711 kips/1,010 kips = 1.69

Determine DCR$_M$ = M_{UD}/M_{CE} for Flexure Strength

The moment capacity, M_n, of Wall 1 at Level 2 is determined in accordance with ACI 318-11 Chapter 10:

M_n is dependent upon P, axial load.

P_G is determined in accordance with ASCE 41-13 § 7.2:

P_G = 1.1(Q_D+Q_L+Q_S) (ASCE 41-13 Eq. 7-1)

P_G = 0.9Q_D (ASCE 41-13 Eq. 7-2)

Tributary area = 4(20 ft)(1/2)(20 ft) = 800 ft^2

Q_D = 100 psf (floor or roof) + weight of wall above level 2
= [3(800 ft^2)(100 psf) + 2(14 ft)(6/12 ft)(60 ft)(150 pcf)]/(1000 lbs/kip)
= 366 kips

Q_L = 25% of unreduced live load = 0.25(125 psf) = 31.3 psf (floor)
= 2(800 ft^2)(31.3 psf)/(1000 lbs/kip) = 50 kips

where snow loads are less than 30 psf, Q_s is permitted to be zero.

Q_S = 0

$P_{G\min}$ = 1.1(366 kips + 50 kips) = 458 kips

$P_{G\max}$ = 0.9(366 kips) = 329 kips

To determine M_n, Q_E is assumed to be 0, since there is no change in axial load to the wall due to earthquake effects. Therefore, $P_{UD} = P_G$.

Use both to determine minimum M_n (P_G from ASCE 41-13 Equation 7-1 is used in this example, as the difference between the two M_n values was not significant):

Run *spColumn*® software (StructurePoint, 2016) with f'_c = 3,750 psi, f_y = 50,000 psi, and ϕ = 1.0. Based on moment-axial interaction diagram (see Figure 10-9):

$\phi M_n = M_{CE}$ = 64,040 kip-ft

$DCR_M = M_{UD}/M_{CE}$ = 52,690 kip-ft/64,040 kip-ft = 0.83

A comparison of the DCR values shows that the walls are shear-controlled:

$DCR_V = 1.69 > DCR_M = 0.83$

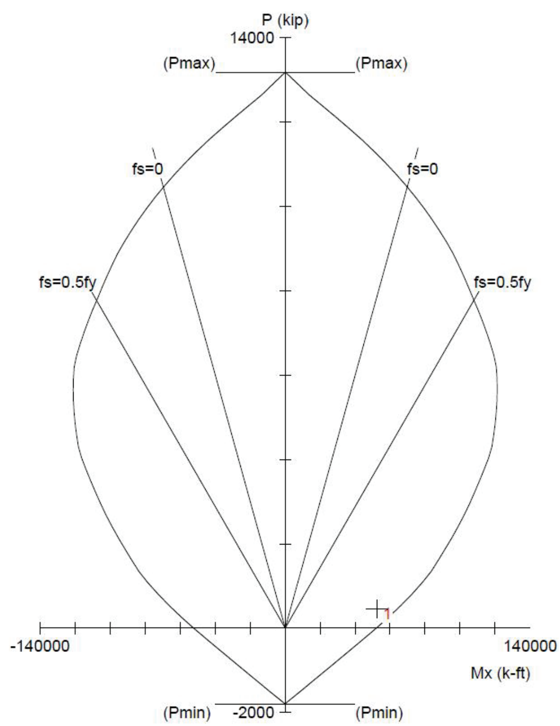


Figure 10-9 Moment-axial interaction diagram from *spColumn*®.

10.4.4.2 Wall A at Level 2 (WAF2)

DCR values are calculated for Wall A at Level 2, shown in Figure 10-10 with V_{UD} = 1,244 kips, M_{UD} = 38,320 kip-ft (from Table 10-6).

The calculations are similar to Wall 1 with new material properties:

Wall properties: Material properties:

h_w = height = 42 ft f'_{ce} = 6,500 psi

l_w = length = 14 ft f_{ye} = 75,000 psi

t = thickness = 24 in.

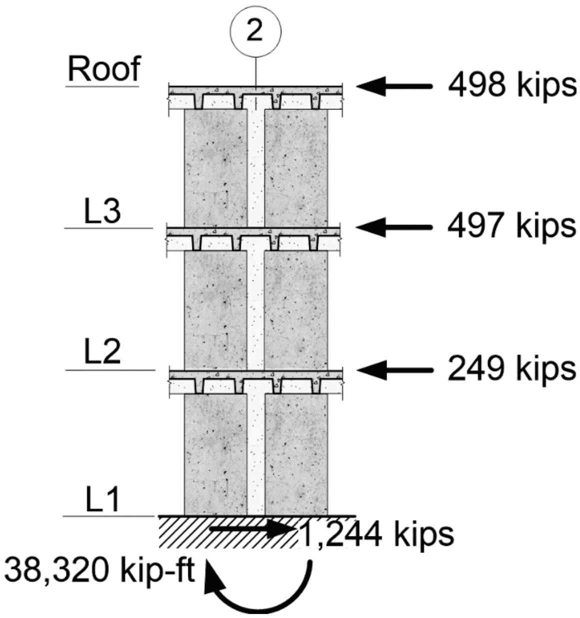


Figure 10-10 Wall A.

Determine DCR_V and DCR_M

$DCR_V = V_{UD}/V_{CE}$

where:

$V_{CE} = V_n = 1{,}890$ kips

DCR_V = 1,244 kips / 1,890 kips = 0.66

$DCR_M = M_{UD}/M_{CE}$

where:

$\phi M_n = M_{CE} = 9{,}600$ kip-ft

DCR_M = 38,320 kip-ft / 9,600 kip-ft = 3.99

A comparison of the DCR values shows that the walls are flexure-controlled:

$DCR_V = 0.66 < DCR_M = 3.99$

10.4.4.3 Wall D at Level 2 (WDF2)

DCR values are calculated for Wall D at Level 2, shown in Figure 10-11 with V_{UD} = 715 kips, M_{UD} = 22,034 kip-ft (from Table 10-6).

The calculations are similar to Wall 1 with existing material properties:

Wall properties:

h_w = height = 42 ft

l_w = length = 20 ft

t = thickness = 8 in

Material properties:

f'_{ce} = 3,750 psi

f_{ye} = 50,000 psi

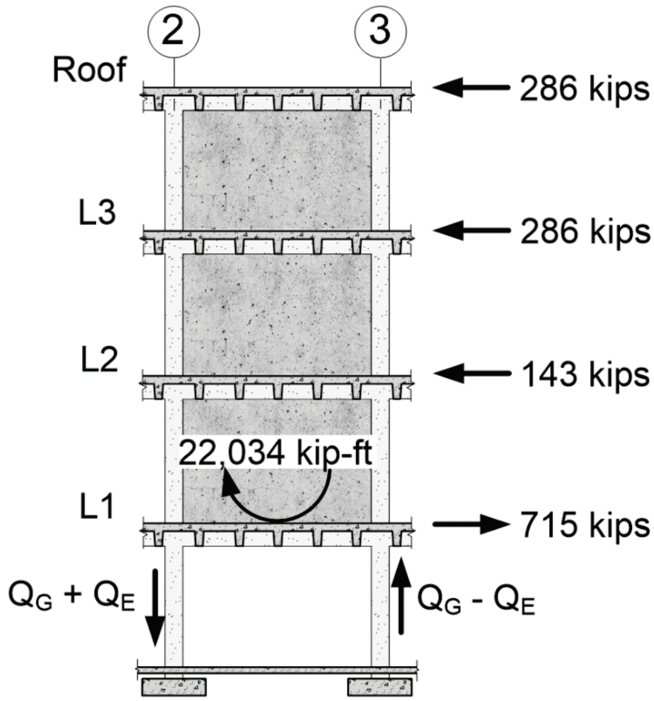


Figure 10-11 Wall D.

Determine DCR$_V$ and DCR$_M$

DCR$_V$ = V_{UD}/V_{CE}

$V_{CE} = V_n$ = 388 kips

DCR$_V$ = 715 kips / 388 kips = 1.84

DCR$_M$ = M_{UD}/M_{CE}

where:

$\phi M_n = M_{CE}$ = 11,930 kip-ft

DCR$_M$ = 22,034 kip-ft / 11,930 kip-ft = 1.85

A comparison of the DCR values shows that the walls are flexure-controlled:

$\text{DCR}_V = 1.84 < \text{DCR}_M = 1.85$

10.4.4.4 Wall G at Level 2 (WGF2)

DCR values are calculated for Wall G at Level 2, shown in Figure 10-12 with V_{UD} = 1,213 kips, M_{UD} = 37,362 kip-ft (from Table 10-6).

The calculations are similar to Wall A with new material properties:

V_{UD} = 1,213 kips, M_{UD} = 37,362 kip-ft

Wall properties:

h_w = height = 42 ft

l_w = length = 20 ft

t = thickness = 10 in

Material properties:

f'_{ce} = 6,500 psi

f_{ye} = 75,000 psi

> **Commentary**
>
> When new concrete shear walls, like Wall G, intersect existing concrete beams the reinforcing in the beams may not be adequate to resist the shear and moment demands. For this example, the reinforcing was adequate and no additional structural mitigation was required.

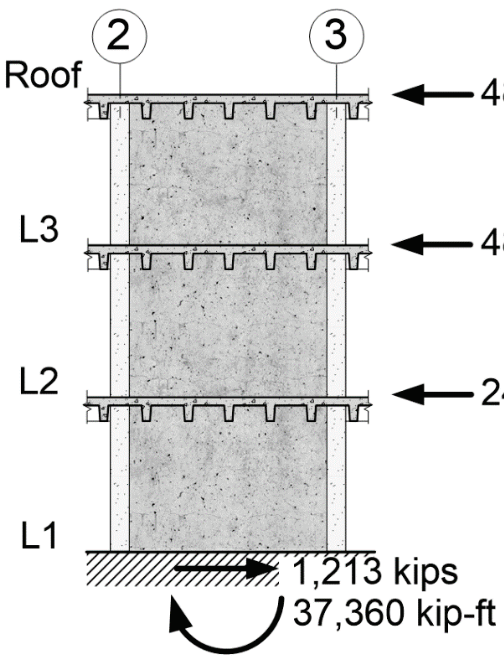


Figure 10-12 Wall G.

> **Commentary**
>
> The new shear wall on Line G is flexurally-controlled. The ends of the wall are the existing columns at Lines 2 and 3. These existing columns do not have confinement reinforcing that would meet current boundary zone reinforcing. It is assumed for this example that confinement reinforcing will be provided as part of the retrofit that meets current code detailing requirements and enables the wall to develop its flexural strength. This is discussed in more detail in Chapter 11 of this *Guide.*

Determine DCR_V and DCR_M

$\text{DCR}_V = V_{UD}/V_{CE}$

$V_{CE} = V_n$ = 980 kips

DCR_V = 1,213 kips / 980 kips = 1.24

$\text{DCR}_M = M_{UD}/M_{CE}$

where:

$\phi M_n = M_{CE}$ = 12,230 kip-ft

DCR_M = 37,362 kip-ft / 12,230 kip-ft = 3.05

A comparison of the DCR values shows that the walls are flexure-controlled:

$DCR_V = 1.20 < DCR_M = 3.05$

10.4.4.5 Gridline D Columns at Basement Supporting Discontinuous Shear Wall

For this initial DCR check, Q_{UD} and Q_{CE} are used (even though action is considered force-controlled), in order to get a true comparison of relative element capacities and building behavior and calculate C_1 and C_2 more accurately.

Determine Q_{UD}

$Q_{UD} = Q_G \pm Q_E$ (ASCE 41-13 Eq. 7-34)

where:

Q_E = the vertical force in the columns based on the overturning moment at the base of the wall
= 22,030 k-ft/20 ft
= 1,102 kips

$Q_G = 1.1(Q_D + Q_L + Q_S)$ (ASCE 41-13 Eq. 7-1)

where:

Q_D = supported dead load, including: (1) floor dead load and roof dead load; (2) weight of the walls above; and (3) weight of the column
= 4(20 ft)(20 ft)(100 psf) + 3(14 ft)(20/2 ft)(10/12 ft)(150 pcf) + (14 ft)($20^2 + 18^2 + 16^2 + 14^2$ in.^{2})/(12 in./ft)^2(150 pcf)

Q_D = 160 kips + 53 kips + 17 kips = 230 kips

Q_L = 25% of unreduced live load (0.25)(125 psf) = 31.3 psf)
= 3(20 ft)(20 ft)(31.3 psf) / (1000 lbs/kip)
= 37.6 kips

where snow loads are less than 30 psf, Q_s is permitted to be zero.

$Q_S = 0$

Q_G = 1.1(230 kips + 37.6 kips + 0) = 294 kips
= $0.9Q_D$ (ASCE 41-13 Eq. 7-2)

where:

Q_D = 230 kips

Q_G = 0.9(230 kips) = 207 kips

$Q_{UD} = Q_G + Q_E$ = 294 kips + 1,102 kips = 1,396 kips (compression)
= $Q_G - Q_E$ = |207 kips – 1,102 kips| = 895 kips (tension)

Use ACI 318-11, let $\phi = 1.0$:

Column properties:

$A_g = (20 \text{ in.})(20 \text{ in.}) = 400 \text{ in.}^2$

$A_{ST} = 8(1.00 \text{ in.}^2) = 8.0 \text{ in}^2$

Material properties:

$f'_{ce} = 3{,}750$ psi

$f_{ye} = 50{,}000$ psi

Determine Q_{CE}

Maximum axial compressive capacity is determined using *spColumn*® software with the moment demand, M_{UD}, equal to 195 kip-ft based on the moment induced in the column from the computed displacement of the concrete floor diaphragm at the first level. The shear demand for the column, V_{UD}, is computed to be 29.3 kips.

Since DCR is based on deformation-controlled actions (ASCE 41-13 Equation 7-16), use $f'_{ce} = 3{,}750$ psi and $f_{ye} = 50{,}000$ psi:

Based on moment-axial interaction diagram from *spColumn*®:

$\phi P_N = P_{CE} = 1{,}320 \text{ kips} = Q_{CE}$

Determine DCR

ASCE 41-13 § 7.3.1.1 specifies that "The magnitude and distribution of inelastic demands for existing and added primary elements and components shall be defined by demand-capacity ratios (DCRs) and computed in accordance with Equation 7-16." Therefore, per ASCE 41-13 Equation 7-16:

DCR $= Q_{UD}/Q_{CE} = 1{,}391 \text{ kips}/1{,}320 \text{ kips} = 1.05$ compression

Maximum axial tensile capacity per ACI 318-11 Section 10.2.5 (Calculations assume the moment demand, M_{UD}, is equal to 0):

$P = f_y A_{ST}$
$= (50 \text{ ksi})(8.0 \text{ in.}^2)$
$= 400$ kips

DCR $= Q_{UD}/Q_{CE} = 899 \text{ kips}/400 \text{ kips} = 2.25$ tension

Commentary

There is not 400 kips of dead load to resist tension in concrete columns at D2 and D3. This will be discussed later when reviewing acceptance criteria.

Check shear in columns

$\phi V_N = V_{CE}$

Determine V_{CE}:

Per ASCE 41-13 § 10.4.2.3, Equation 10-3:

$k = 1.0$ (Ductility demand is less than 2)

$M = 195$ kip-ft

V = 29.3 kips

d = 0.8(20 ft)/(12 in./ft)

M/Vd = 195 kip-ft/(29.3 kips)(0.8)(20 in.)/(12 in./ft) = 5.0 (need not be taken greater than 4)

N_u = 203 kips in compression

λ = 1.0 (Normal weight concrete)

A_v = 0.40 in.^{2}

$$V_n = kV_0 = k\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f_c'}}{M/Vd}\sqrt{1+\frac{N_U}{6\sqrt{f_c'}A_g}}\right)0.8A_g\right]$$

$$V_{CE} = \phi V_n$$

$$= 1.0\left[\frac{(0.40)(50{,}000\text{ psi})(20\text{ in.})(0.8)}{10\text{ in.}} + (1)\left(\frac{6\sqrt{3{,}750}\text{ psi}}{4}\sqrt{1+\frac{203{,}000\text{ lbs}}{6\sqrt{3{,}750}\text{ psi}(20\text{ in.})^2}}\right)(0.8)(20\text{ in.})^2\right]$$

= 77.4 kips

Determine DCR

DCR = Q_{UD}/Q_{CE} = 29.3 kips/77.4 kips = 0.38 column in shear

A comparison of the maximum Wall D and column DCR values shows that the columns are stronger than the wall they support in compression but less strong in tension. Therefore, the tension of the columns will control the wall performance.

$DCR_{wall} = 1.85 < DCR_{col} = 2.25$

10.4.4.6 Summary of Preliminary Element DCRs

Calculations in the previous sections are repeated for other walls. The results are summarized in Figure 10-13 and Table 10-7.

10.4.5 Confirm Applicability of Linear Procedure (ASCE 41-13 § 7.3.1)

ASCE 41-13 § 7.3.1.1 states: *"If a component DCR exceeds the lesser of 3.0 and the m-factor for the component action and any irregularity described in Section 7.3.1.1.3 or Section 7.3.1.1.4 is present, then linear procedures are not applicable and shall not be used."*

For this building, the *m*-factor value for most components governs over the DCR maximum of 3.0, and there are some DCR values greater than the specified limit (the lower of 3.0 or the *m*-factor). However, the building does not have either irregularity (weak story or torsional strength). Therefore, linear procedures are acceptable.

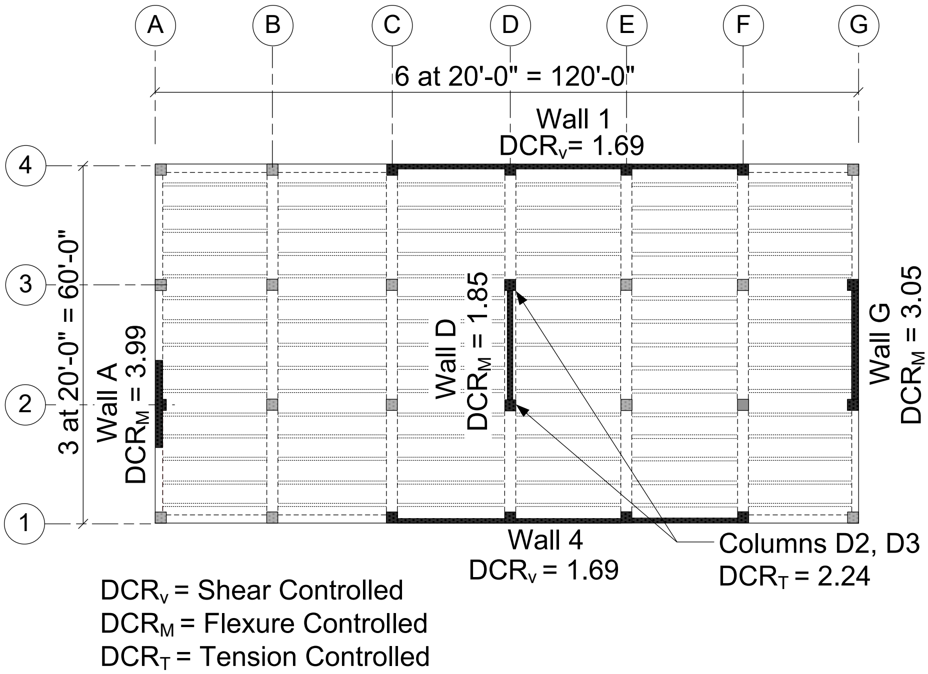


Figure 10-13 Preliminary element controlling DCR.

Table 10-7 Summary of Preliminary Element DCRs

Element	Location	Level	ID	Action	Q_{UD} (kips or k-ft)	Q_{CE} (kips or k-ft)	DCR
Wall	Gridline 1	Level 1	W1F2	Shear* Flexure	1,711 52,690	1,031 64,040	1.69 0.83
Wall	Gridline 4	Level 1	W4F2	Shear* Flexure	1,711 52,690	1,031 64,040	1.69 0.83
Wall	Gridline A	Level 1	W4F2	Shear Flexure*	1,244 38,320	1,890 9,600	0.66 3.99
Wall	Gridline D	Level 1	W4F2	Shear Flexure*	715 22,034	388 11,930	1.84 1.85
Wall	Gridline G	Level 1	W4F2	Shear Flexure*	1,213 37,362	980 12,230	1.24 3.05
Column	Gridline D	Base-ment	D2/D3	Axial Comp. Axial Tension* Shear	1,391 899 29.3	1,320 400 77.4	1.06 2.24 0.38

* Controlling failure mode

$(DCR_{max})_{TRANS} = 3.99$

$(DCR_{max})_{LONG} = 1.69$

10.4.6 Pseudo Seismic Force Calculations (ASCE 41-13 § 7.4.1 and ASCE 41-13 § 10.7)

> **Commentary**
>
> See Section 4.3 of this *Guide* for a discussion of C_1 and C_2 modification factors.

Determine final pseudo seismic force, V, per ASCE 41-13 § 7.4.1:

$$V = C_1 C_2 C_m S_a W \qquad \text{(ASCE 41-13 Eq.7-21)}$$

10.4.6.1 Transverse Direction

First, final C_1 and C_2 are determined per ASCE 41-13 § 7.4.1.3.1:

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{aT^2} \qquad \text{(ASCE 41-13 Eq. 7-22)}$$

and $T \leq 0.7$ s, therefore,

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{\text{strength}} - 1}{T}\right)^2 \qquad \text{(ASCE 41-13 Eq. 7-23)}$$

a = 60 (Site Class D)

T = 0.33 seconds (see previous calculation in Section 10.4.1 of this *Guide*)

Determine ratio of elastic strength demand to yield strength coefficient, μ_{strength}, per ASCE 41-13 § C7.4.1.3.1:

$$\mu_{\text{strength}} = \frac{DCR_{\max}}{1.5}(C_m) > 1.0 \qquad \text{(ASCE 41-13 Eq. C7-3)}$$

where:

DCR_{max} = 3.99 (Wall A, see previous calculations in Section 10.4.4 of this *Guide*)

C_m = 0.8 (effective mass factor per ASCE 41-13 Table 7-4)

$\mu_{\text{strength}} = 3.99(0.8)/1.5 = 2.13$

$$C_1 = 1 + \frac{(2.13-1)}{60(0.33)^2} = 1.17$$

$$C_2 = 1 + \frac{\left(\frac{2.13-1}{0.33}\right)^2}{800} = 1.01$$

$V_{TRANS} = 1.17(1.01)(0.8)(1.08)W = 1.02W$

> **Commentary**
>
> C_1 and C_2 are final. The iteration to determine DCR_{max} has to be conducted only once in each direction.

This compares to $1.04W$ calculating the pseudo seismic force using the alternate method with values for modification factors C_1, C_2 found in ASCE 41-13 § Table 7-3, which is illustrated in Section 4.3 of this *Guide.*

10.4.6.2 Longitudinal Direction

Determine final C_1 and C_2 per ASCE 41-13 § 7.4.1.3.1:

$$C_1 = 1 + \frac{\mu_{\text{strength}} - 1}{aT^2} \qquad \text{(ASCE 41-13 Eq. 7-22)}$$

and $T \leq 0.7$s, therefore,

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{\text{strength}} - 1}{T}\right)^2 \qquad \text{(ASCE 41-13 Eq. 7-23)}$$

a = 60 (Site Class D)

T = 0.33 seconds (see previous calculation in Section 10.4.1 of this *Guide*)

Determine ratio of elastic strength demand to yield strength coefficient, μ_{strength}, per ASCE 41-13 § C7.4.1.3.1:

$$\mu_{\text{strength}} = \frac{\text{DCR}_{\max} C_m}{1.5} > 1.0 \qquad \text{(ASCE 41-13 Eq. C7-3)}$$

$\text{DCR}_{\max}$ = 1.69 (Wall 1, see previous calculations in Section 10.4.4 of this *Guide*)

C_m = 0.8 (per ASCE 41-13 Table 7-4)

$$\mu_{\text{strength}} = \frac{1.69(0.8)}{1.5} = 0.90 < 1.0 \longrightarrow \mu_{\text{strength}} = 1.0$$

$$C_1 = 1 + \frac{1.0 - 1}{60(0.33)^2} = 1.0$$

$$C_2 = 1 + \frac{\left(\frac{1.0-1}{0.33}\right)^2}{800} = 1.0$$

$$V_{LONG} = 1.0(1.0)(0.8)(1.08)W = 0.86\text{W}$$

This compares to $0.86W$ calculating the pseudo seismic force using the alternate values for modification factors C_1 and C_2 found in ASCE 41-13 Table 7-3, which is illustrated in Section 4.3 of this *Guide*.

10.4.7 Final Pseudo Seismic Forces (ASCE 41-13 § 7.4.1.3)

Determine final pseudo seismic force, V, per ASCE 41-13 § 7.4.1:

10.4.7.1 Transverse Direction

V_{TRANS}= $1.02W$ (see previous calculations in Section 10.4.6.1 of this *Guide*)

V_{TRANS} demands are scaled by the factor $V/V_{\text{Preliminary}} = 1.02W/1.08W = 0.94$. A summary of the final story forces in the transverse direction are listed in Table 10-8.

Table 10-8 Final Story Forces – Transverse Direction

Level	w_x (kips)	Story Ht (feet)	h (ft)	$w_x h_x^k$ (kip-feet)	F_x (kips)	$\sum F_x$ (kips)	$\sum M_x$ (kip-feet)
Roof	720	14	42	30,240	1,169	1,169	16,370
3	1,080	14	28	30,240	1,169	2,338	49,100
2	1,080	14	14	15,120	585	2,923	90,020
1	-	14	-	-	-	-	-
Total	2,880			75,600	2,923		

10.4.7.2 Longitudinal Direction

$V_{\text{LONG}} = 0.86W$ (see previous calculations)

V_{LONG} demands are scaled by the factor $V/V_0 = 0.86/1.08 = 0.80$. A summary of the final story forces in the transverse direction are listed in Table 10-9.

Table 10-9 Final Story Forces – Longitudinal Direction

Level	w_x (kips)	Story Ht (feet)	h (ft)	$w_x h_x^k$ (kip-feet)	F_x (kips)	$\sum F_x$ (kips)	$\sum M_x$ (kip-feet)
Roof	720	14	42	30,240	991	991	13,870
3	1,080	14	28	30,240	991	1,981	41,610
2	1,080	14	14	15,120	495	2,477	76,285
-	-	14	-	-	-	-	-
Total	2,880			75,600	2,477		

10.4.8 Final Wall Demands

Table 10-10 and Table 10-11 and Figure 10-14 summarize the final wall demands.

Table 10-10 Final Element Demands for Determination of Element Acceptance per ASCE 41-13 § 7.5.2.2

Wall	Floor	ID	% of story shear (kips)	Story Shear (kips)	Story Moment (kip-feet)	Shear V_{UD} (kips)	Moment M_{UD} (kip-feet)
1	R	W1FR	0.55	991	13,870	545	7,629
1	3	W1F3	0.55	1,981	41,610	1,090	22,890
1	2	W1F2	0.55	2,477	76,285	1,362	41,960
4	R	W4FR	0.55	991	13,870	545	7,629
4	3	W4F3	0.55	1,981	41,610	1,090	22,890
4	2	W4F2	0.55	2,477	76,285	1,362	41,960
A	R	WAFR	0.40	1,169	16,370	468	6,548
A	3	WAF3	0.40	2,338	49,100	935	19,640
A	2	WAF2	0.40	2,923	90,020	1,169	36,010
D	R	WDFR	0.23	1,169	16,370	269	3,765
D	3	WDF3	0.23	2,338	49,100	538	11,290
D	2	WDF2	0.23	2,923	90,020	672	20,700
G	R	WGFR	0.39	1,169	16,370	456	6,384
G	3	WGF3	0.39	2,338	49,100	912	19,150
G	2	WGF2	0.39	2,923	90,020	1,140	35,110

Commentary

The DCRs in Table 10-11 are to compare the DCRs from the preliminary pseudo seismic force values and are not the same used for the acceptance criteria.

Table 10-11 Final Element DCRs (Using Expected Strength Properties)

Element	Location	Level	ID	Action	Q_{UD} (kips or k-ft)	Q_{CE} (kips or k-ft)	DCR
Wall	Gridline 1	Level 1	W1F2	Shear* Flexure	1,362 41,960	1,010 64,040	1.35 0.66
Wall	Gridline 4	Level 1	W4F2	Shear* Flexure	1,362 41,960	1,010 64,040	1.35 0.66
Wall	Gridline A	Level 1	WAF2	Shear Flexure*	1,169 36,010	1,890 9,600	0.62 3.75
Wall	Gridline D	Level 1	WDF2	Shear Flexure*	672 20,700	388 11,930	1.73 1.74
Wall	Gridline G	Level 1	WGF2	Shear Flexure*	1,140 35,110	980 12,230	1.16 2.87
Column	Gridline D	Basement	D2 D3	Axial Comp Axial Tension* Shear	1,324 832 27.5	1,320 400 77.4	1.00 2.08 0.36

Note: * indicates the action with the largest DCR for each wall

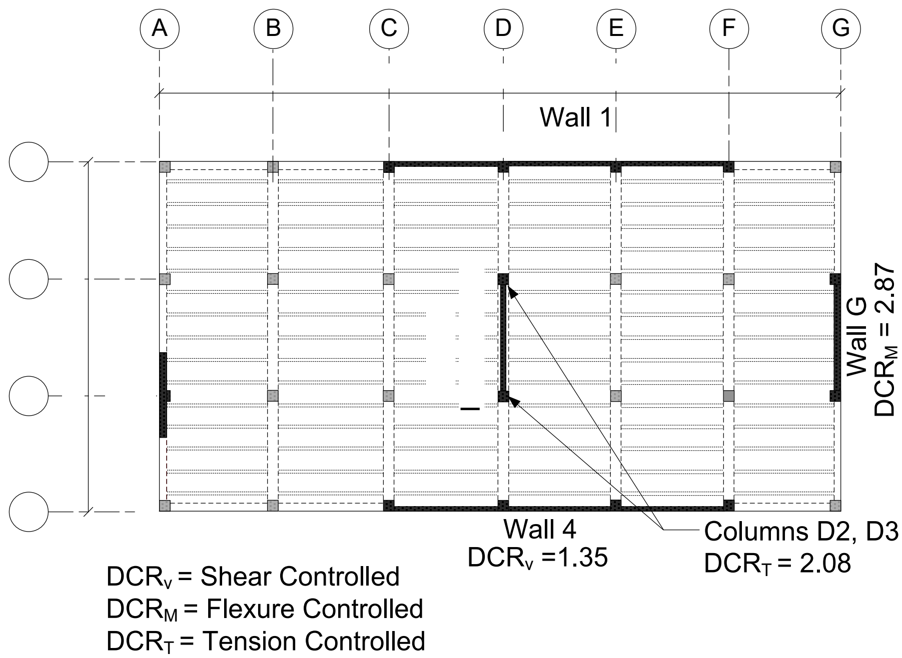


Figure 10-14 Final element DCRs.

10.5 Evaluation and Retrofit of Shear Walls (ASCE 41-13 § 7.5.1 and ASCE 41-13 § 10.7)

10.5.1 Element m-Factors and Final Element Acceptance Ratios for Shear Walls (ASCE 41-13 § 7.5.1 and § 10.7)

Elements classified:

The target Performance Level has been selected as Collapse Prevention (CP), as defined in ASCE 41-13 § 2.3.

Commentary

For discussion of primary and secondary members see Section 4.4 of this *Example Application Guide*. For discussion of force-controlled and deformation-controlled elements see Section 4.5 of this *Example Application Guide*.

According to ASCE 41-13 § 7.5.1, each component shall first be classified as primary or secondary in accordance with ASCE 41-13 § 7.5.1.1 and each action shall be classified as deformation controlled or force controlled as follows:

Deformation controlled:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

Force controlled:

$$\kappa Q_{CL} > Q_{UF} \qquad \text{(ASCE 41-13 Eq. 7-37)}$$

The knowledge factor, κ, was established as 0.9 in Section 10.3.

m-factors for concrete shear walls are determined per ASCE 41-13 § 10.7.

10.5.1.1 Wall 1 at Level 2, W1F2

The calculations are similar for Wall 1 at Level 1 and Wall 4 at Level 1 (W4F2), with V_{UD} = 1,362 kips and M_{UD} = 41,960 kip-ft from Table 10-10.

Wall 1 has the lowest transverse steel ratio ρ_t at 0.0010; other walls are at 0.0015 or higher. ASCE 41-13 § 10.7.2.3 specifies that "Where an existing shear wall or wall segment has a transverse reinforcement percentage, ρ_n, less than 0.0015, the wall shall be considered force-controlled." As a force-controlled element, lower bound material properties are to be used and *m*-factors do not apply, per ASCE 41-13 § 7.5.2.2.2. There is no inelastic mechanism in the system for Wall 1, other than possibly the floor slab which could limit the demands. This is uncommon and not the case for this example. Therefore, using a force-delivery reduction factor, *J*, does not apply.

Commentary

In ASCE 41-06 § 6.7.2.3 the user is directed to use a ρ_n of 0.0015 for the contribution of wall reinforcing for the shear strength of the wall when reinforcing is less than 0.0015. This was deleted in ASCE 41-13 and further clarified that the wall should be considered force controlled.

Using the same calculations for determining the expected strength for Wall 1 using lower bound material properties of f_{cl} = 2,500 psi and f_{yl} of 40,000 psi V_{CL} and M_{CL} are as follows:

V_{CL} = 838 kips

M_{CL}= 52,440 kip-ft

For force-controlled elements the acceptance criteria is given as

$$\kappa Q_{CL} > Q_{UF} \quad \text{(ASCE 41-13 Eq. 7-37)}$$

Calculating for shear:

0.9(838 kips) = 754 kips < V_{UD} = 1,362 kips ⟶ Wall is not acceptable in shear

For flexure:

0.9(52,040 kip-ft) = 46,840 kip-ft > M_{UD}
= 41,960 kip-ft ⟶ Wall acceptable in flexure

Accordingly, FRP reinforcing should be added to the wall to improve the shear performance as a force-controlled element to mitigate the structural deficiency.

10.5.1.2 Wall A at Level 1, WAF2

M_{UD} = 36,010 kip-ft

M_{CE} = 9,600 kip-ft (see Table 10-10, wall is flexure-controlled)

Useful Tip

Per ACI 440 (2017), when FRP is being designed to seismically strengthen walls for shear or flexure, it should be designed as a force-controlled element.

Use ASCE 41-13 Table 10-21 (wall is flexure-controlled) to determine *m*-factor. The following factors are required for determining the *m*-factor:

> **Commentary**
>
> When calculating *m*-factors, expected strength (f'_{ce} and f_{ye}) should be used for concrete and rebar yield strength instead of lower bound material properties.

> **Commentary**
>
> Per FEMA 274 Section C6.8.2.4, the term "$(A_s - A_s')f_y$" is intended to penalize flanged (T- and L-shaped) wall sections with unbalanced amounts of reinforcing at each end of the wall; these walls generate large compression strains in the stem of the wall when large amounts of reinforcing in the flange are in tension and accordingly have significantly less ductility. Therefore, since Wall A is a planar wall with symmetrical reinforcing layout, the flanged wall term $(A_s\text{-}A_s')$ may be taken as 0.

> **Commentary**
>
> Per ASCE 41-13 § 10.7.2.4, for cantilevered concrete shear walls the lateral force is assumed to be uniformly distributed over the height of the wall. To determine the maximum shear force, *V*, in the shear wall.

> **Commentary**
>
> κ is equal to 1.0 for new construction with material testing values confirmed during construction.

- Confined boundary: Yes (new shear wall)
- Component type: Primary
- Performance level: CP
- Use expected strength properties f'_{ce} = 6,500 psi, f_{ye} = 75,000 psi

Axial stress:

Determine *P* in accordance with ASCE 41-13 § 7.2:

$Q_G = 1.1(Q_D + Q_L + Q_S)$ (ASCE 41-13 Eq. 7-1)

$= 0.9Q_D$ (ASCE 41-13 Eq. 7-2)

where:

Tributary area = (1/4)(20 ft)(40 ft) = 200 ft^2

Q_D = 3(200 ft^2)(100 psf) + 2(14 ft)(24/12 ft)(14 ft)(150 pcf) = 178 kips

Q_L = 2(200 ft^2)(31.3 psf) = 12.5 kips

Q_S = 0

Q_G = 1.1(178 kips + 12.5 kips +0)

$= P_G$ = 210 kips

$(A_S - A_s')f_y$ = 0 (see commentary box)

$$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} = \frac{0 + 210 \text{ kips}}{(24 \text{ in.})(14 \text{ ft})(12 \text{ in./ft})(6.5 \text{ ksi})}$$

$$= \frac{210 \text{ kips}}{26{,}210 \text{ kips}} = 0.01 \leq 0.1$$

Shear stress (shear at the flexure yield of the wall per ASCE 41-13 § 10.7.2.4):

$V = M_{CE}/(h_{\text{wall}}/2)$ = 9,600 kip-ft/(42 ft/2) = 457.1 kips

$$\frac{V}{t_w l_w \sqrt{f_c'}} = \frac{457{,}100 \text{ lbs}}{(24 \text{ in.})(14 \text{ ft})(12 \text{ in./ft})\sqrt{6{,}500} \text{ psi}} = 1.41 \leq 4$$

According to ASCE 41-13 Table 10-21, m = 6.0

For a deformation-controlled wall:

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq 7-36)

6.0(1.0)(9,600 kip-ft)= 57,600 kip-ft > M_{UD} = 36,010 kip-ft⟶ Wall is acceptable

10.5.1.3 Wall D at Level 2, WDF2

M_{UD} = 20,700 kip-ft

M_{CE} = 11,930 kip-ft (see Table 10-10 of this *Guide*; wall is flexure-controlled)

Use ASCE 41-13 Table 10-21 (wall is flexure-controlled) to determine *m*-factor. The following factors are required to determining the *m*-factor:

- Confined boundary: No (existing columns do not have confined boundaries with ductile ties)
- Component type: Primary
- Performance level: CP
- Use expected strength properties f'_{ce} = 3,750 psi, f_{ye} = 50,000 psi

Axial stress:

Q_G = P_G = 219 kips (based on tributary area of wall)

$$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} = \frac{0 + 219 \text{ kips}}{(8 \text{ in.})(20 \text{ ft})(12 \text{ in./ft})(3.75 \text{ ksi})} = \frac{219 \text{ kips}}{7,200 \text{ kips}} = 0.03 \leq 0.1$$

Shear stress:

V = $M_{CE}/(h_{\text{wall}}/2)$ = 11,930 kip-ft/(42 ft/2) = 568.1 kips

$$\frac{V}{t_w l_w \sqrt{f_c'}} = \frac{568,100 \text{ lbs}}{(8 \text{ in.})(20 \text{ ft})(12 \text{ in./ft})\sqrt{3,750} \text{ psi}} = 4.83 > 4 \text{ but} < 6$$

Interpolating ASCE 41-13 Table 10-21, *m* = 3.37

For a deformation controlled wall:

$m\kappa Q_{CE} > Q_{UD}$

3.37(0.9)(11,930 k-ft) = 40,200 k-ft > M_{UD} = 20,700 k-ft

Therefore, the wall is acceptable.

10.5.1.4 Wall G at Level 2, WGF2

M_{UD} = 35,110 kip-ft (see Table 10-10 of this *Guide*; wall is flexure-controlled)

M_{CE} = 12,230 kip-ft

Use ASCE 41-13 Table 10-21 (wall is flexure-controlled) to determine *m*-factor:

- Confined boundary: Yes (new shear wall)

- Component type: Primary
- Performance level: CP
- Use expected strength properties f'_{ce} = 6,500 psi, f_{ye} = 75,000 psi

Axial stress:

$$P_G = 154 \text{ kips (see previous calculations)}$$

$$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c} = \frac{0 + 154 \text{ kips}}{(10 \text{ in.})(20 \text{ ft})(12 \text{ in./ft})(6.5 \text{ ksi})} = \frac{154 \text{ kips}}{18{,}000 \text{ kips}}$$

$$= 0.01 \le 0.1$$

Shear stress:

$$V = M_{CE}/(h_{\text{wall}}/2) = 12{,}230 \text{ kip-ft}/(42 \text{ ft}/2) = 596.6 \text{ kips}$$

$$\frac{V}{t_w l_w \sqrt{f'_c}} = \frac{596{,}600 \text{ lbs}}{(10 \text{ in.})(20 \text{ ft})(12 \text{ in./ft})\sqrt{6{,}500 \text{ psi}}} = 3.08 \le 4$$

According to ASCE 41-13 Table 10-21, m = 6.0

For a deformation controlled wall:

$$m\kappa Q_{CE} > Q_{UD}$$

6.0(1.0)(12,230 kip-ft) = 73,380 kip-ft > M_{UD} = 35,110 kip-ft

Therefore, the wall is acceptable.

10.5.1.5 Summary Element *m*-factors

m-factors calculated for each of the walls are summarized in Table 10-12.

Table 10-12 Summary of *m*-Factors (Deformation-Controlled Components)

Element	Location	Lvl	t_w (in)	l_w (ft)	h_w (ft)	f'_c (psi)	f_y (psi)	Axial Cond.	Shear Cond.	Confined Boundary?	Comp. Type	Contr. Action	*m*	Action
W1F2	Grid 1	1	6	60	42	2,500	40,000	-	-	No	Primary CP	Force	NA	Shear*
W4F2	Grid 4	1	6	60	42	2,500	40,000	-	-	No	Primary CP	Force	NA	Shear*
WAF2	Grid A	1	24	14	42	6,500	75,000	0.01	1.41	Yes	Primary CP	Def.	6	Flexure*
WDF2	Grid D	1	8	20	42	3,750	50,000	0.01	4.83	No	Primary CP	Def.	3.37	Flexure*
WGF2	Grid G	1	10	20	42	6,500	75,000	0.01	3.08	Yes	Primary CP	Def.	6	Flexure*

* Controlling failure mode

NA: Not applicable as *m*-factors do not apply to force-controlled actions

10.5.2 Acceptance of Columns and Foundation Supporting Discontinuous Shear Wall Gridline D (ASCE 41-13 § 7.5.2.1 and § 10.7.1.2)

10.5.2.1 Gridline D Columns at Basement Supporting Discontinuous Shear Wall

For the final DCR check, use Q_{UF} and Q_{CL} for force-controlled action per ASCE 41-13 § 10.7.2.4.

Two methods to determine Q_{UF} are demonstrated in this example. The first method uses the pseudo seismic overturning force modified by C_1 and C_2 and J factors, which is defined by ASCE 41-13 Equation 7-35. The second method uses the expected moment capacity of the wall, which is the first method defined in ASCE 41-13 § 7.5.2.1. It should be noted that it is not required to take the greater of the two. Engineering judgment should be used to determine which result is more applicable to the structure.

Method Using the Pseudo Seismic Overturning Forces

Determine Q_{UF}:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

From previous calculations in Section 10.4.4.5 of this *Guide.*

Q_G = 289 kips (for compression cases)
= 203 kips (for tension cases)

Q_E = M_{UD}/l_w for Wall D at first floor (WDF2)
= 20,700 kip-ft / 20 ft
= 1,035 kips

C_1 = 1.17

C_2 = 1.01 (see previous calculations in Section 10.4.6.1 of this *Guide*)

J may be determined based on DCR values or estimated using Level of Seismicity per ASCE 41-13 § 2-5 and Table 2-5. In ASCE 41-13 Equation 7-35, J is defined as "Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest demand capacity ratio (DCR) of the components in the load path delivering force to the component in question". ASCE 41-13 § 7.5.2.1.2 specifies an alternative method for determining J, which is "values of J equal to 2.0 for a high level of seismicity, 1.5 for a moderate level of seismicity, and 1.0 for a low level of seismicity shall be permitted where not based on calculated DCRs. J shall be taken as 1.0 for the Immediate Occupancy Structural Performance Level." The Level of

Commentary

Q_G for compression and tension forces is based on the component gravity load combinations per ASCE 41-13 § 7.2 giving the highest compression and tension loads.

Commentary

Per ASCE 41-13 § 7.5.2.1.2, "where the forces contributing to Q_{UF} are delivered by components of the seismic-force-resisting system that remain elastic J shall be taken as 1.0." Therefore, J = 2.0 is acceptable in this case since Wall D is yielding.

Seismicity is determined using ASCE 41-13 Table 2-5. For S_{DS} = 0.903g > 0.5g and S_{D1} = 0.524g > 0.2g, the corresponding Level of Seismicity is High.

High Level of Seismicity requires J = 2.0 per ASCE 41-13 § 7.5.2.1.2:

$$Q_{UF} = 289 \text{ kips} + \frac{1{,}035 \text{ kips}}{(1.17)(1.01)(2.0)} = 727 \text{ kips (compression)}$$

$$= 203 \text{ kips} - \frac{1{,}035 \text{ kips}}{(1.17)(1.01)(2.0)} = -235 \text{ kips (tension)}$$

Using Expected Strength of Wall (M_{CE})

$Q_E = M_{CE}/l_w$
= 11,930 kip-ft/20 ft = 596 kips

Q_{UF} = 289 kips + 596 kips = 885 kips (compression)
= |203 kips – 596 kips| = 393 kips (tension)

Using engineering judgment/best practices, the moment from the expected strength of Wall D results is selected as the action to be used in checking for the columns that support the discontinuous shear wall. The J or force delivery reduction factor is an approximate value. As noted above, per ASCE 41-13 § 7.5.2.1.2, either method would be acceptable.

Final Member Acceptance Determination

Maximum axial compressive capacity is determined using *spColumn®* software with the moment demand, M_{UD}, equal to 165 kip-ft based on the moment induced in the column from the computed displacement of the concrete floor diaphragm at the first level. The shear demand for the column, V_{UD}, is computed to be 27.5 kips.

Since member acceptance is based on force-controlled action, use f'_{cL} = 2,500 psi and f_{yl} = 40,000 psi.

Based on the moment-axial interaction diagram from *spColumn®*:

$Q_{CL} = \phi P_N$ = 900 kips

$\kappa Q_{CL} > Q_{UF}$

0.9(900 kips) = 810 kips < 885 kips (compression)

Therefore, the columns are not acceptable in compression.

Determine Q_{CL} (tension) per ACI 318-11 Section 10.2.5, with ϕ = 1.0:

$P = f_y A_{ST}$ = 40 ksi (8.0 in.^{2}) = 320 kips

$Q_{CL} = P$ = 320 kips

$\kappa Q_{CL} > Q_{UF}$

0.9(320 kips) = 288 kips < 393 kips (tension)

Although the tension capacity of the column is exceeded, the columns should be acceptable in tension because there is not enough dead load in the column and footing below to develop the 288 kip tensile capacity of the column. See Section 10.5.2.2 of this *Guide* for evaluation of uplift at the foundation. Therefore, the columns are acceptable in tension.

ASCE 41-17 Revision

In ASCE 41-17, axial tension in columns and wall piers is now permitted to be treated as a deformation-controlled action with the commensurate *m*-factor or acceptance criteria as the flexure action.

Check Shear in Columns

$\phi V_N = V_{CL}$

Determine V_{CL}:

Per ASCE 41-13 § 10.4.2.3, Equation 10-3

k = 1.0 (Ductility demand is less than 2)

M = 165 kip-ft

V = 27.5 kips

d = 0.8(20 in.)/12 in./ft

M/Vd = 165 kip-ft/[(27.5 kips)(0.8)(20 in.)/(12 in./ft)] = 4.5 (need not be taken greater than 4)

N_U = axial compression force, 203 kips in compression (see previous calculations)

λ = 1.0 (normal weight concrete)

A_v = 0.40 in^2

$$V_n = kV_0 = k\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_n}{6\sqrt{f'_c}A_g}}\right)0.8A_g\right]$$

$$V_{CE} = \phi V_n$$

$$= 1.0\left[\frac{(0.40\text{ in.}^2)(40{,}000\text{ psi})(20\text{ in.})(0.8)}{10\text{ in.}} + (1)\left(\frac{6\sqrt{2{,}500}\text{ psi}}{4}\sqrt{1+\frac{203{,}000\text{ lbs}}{6\sqrt{2{,}500}\text{ psi}(20\text{ in.})^2}}\right)(0.8)(20\text{ in.})^2\right]$$

= 65.0 kips

Determine acceptance ratio:

$Q_{UD}/\kappa Q_{CL}$ = 27.5 kips/[0.9(65.0 kips)] = 0.47 in shear

To mitigate the seismic concern, FRP reinforcing at the concrete columns is selected for use. This will provide confinement reinforcing for the columns plus the small increase in compression capacity needed to meet the compression demands on the columns. Another option would be to perform a more comprehensive material testing of the existing concrete and reinforcing to change κ from 0.9 to 1.0.

10.5.2.2 Foundations at Gridline D Columns

In accordance with ASCE 41-13 § 7.5.1, foundations shall satisfy the criteria specified in ASCE 41-13 Chapter 8. Acceptance criteria for linear procedures with foundation modeled as fixed bases are specified in ASCE 41-13 § 8.4.2.3.2.1.

Foundation properties:

- Footing dimensions: 10' × 10' × 18"
- Allowable soil bearing capacity: 4.0 ksf

Prescriptive expected bearing capacity determined per ASCE 41-13 § 8.4.1.1:

Let $Q_{CE} = q_c A_{ftg} = 3q_{\text{allow}} A_{ftg} = 3(4.0\text{ ksf})(10\text{ ft})(10\text{ ft}) = 1{,}200\text{ kips}$

Foundation Compression Check at Gridline D columns

Figure 10-15 provides a summary of forces.

$$Q_{UD} = Q_G \pm Q_E \qquad \text{(ASCE 41-13 Eq. 7-34)}$$

Q_G = 289 kips for columns in compression (see calculations in Section 10.4.4.4)

$Q_E = M_{UD}/l_w$ for Wall D at first floor (WDF2)
= 20,700 k-ft/20 ft = 1,035 kips (compression or tension)

Q_{UD} = 289 kips + 1,035 kips = 1,324 kips compression

ASCE 41-13 § 8.4.2.3.2.1 "Foundation Modeled as a Fixed Base" specifies that "If the base of the structure is assumed to be completely rigid, the foundation soil shall be classified as deformation controlled. Component actions shall be determined by Equation 7-34. Acceptance criteria shall be based on Equation 7-36, *m*-factors for foundation soil shall be 1.5 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention, and the use of upper-bound component capacities shall be permitted."

ASCE 41-13 § 8.4.2 states that "Where foundation components are not modeled explicitly, the analysis shall be bounded by the upper- and lower-bound foundation capacity as defined in this section. In lieu of explicit evaluation of uncertainties in foundation characteristics, it shall be permitted to take the upper-bound stiffness and bearing capacity values and shear-sliding and axial load-settlement relationships as two times the expected values and the lower-bound stiffness and capacity values as one-half of the expected values."

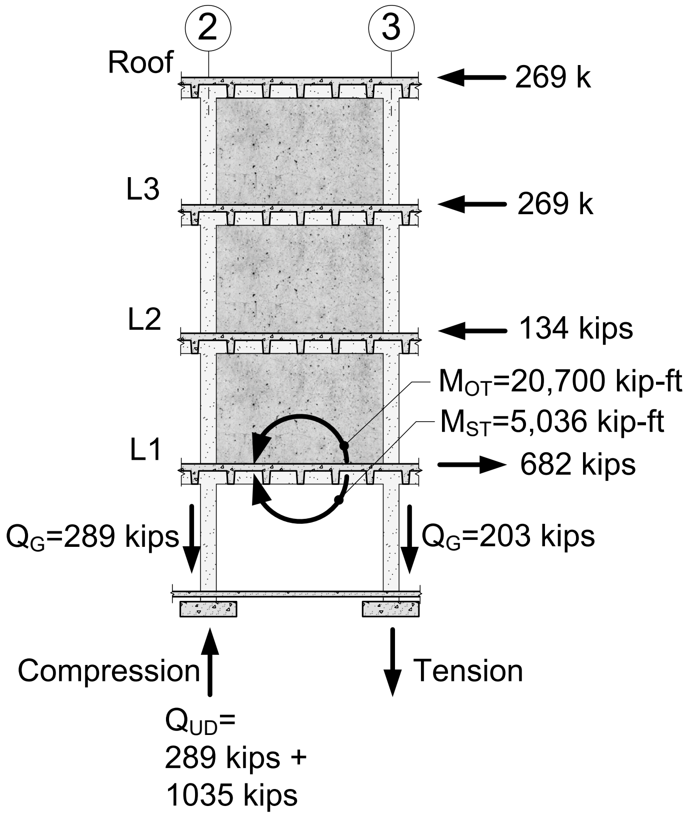


Figure 10-15 Foundation check at Gridline D.

The upper-bound soil capacity is $2Q_{CE}$.

$m\kappa(2Q_{CE}) > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

$m\kappa(2Q_{CE})$= (4.0)(0.9)(2)(1,200 kips) = 8,640 kips

$\geq Q_{UD}$ = 1,324 kips

The lower-bound soil capacity is 1/2 Q_{CE}.

$m\kappa(1/2Q_{CE}) > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

$m\kappa(1/2Q_{CE})$ = (4.0)(0.9)(1/2)(1,200 kips) = 2,160 kips

$\geq Q_{UD}$= 1,324 kips

Therefore, the foundation is acceptable in compression.

Commentary

Because the soils information is dependent on the original construction drawings, a κ = 0.9 value is used.

10.5.2.3 Overturning/Tension Check at Foundation Gridline D

Determine the Stabilizing Moment Produced by the Dead Load on Wall D:

Stabilizing moment, M_{ST}, is calculated:

Dead Loads:

Floor DL + Roof DL = 160 kips

Weight of walls above = 3(14 ft)(20/2 ft)(10/12 ft)(150 pcf) = 52.5kips

Weight of column full height = (14 ft)(20^2+18^2+16^2+14^2 in.^{2}) /(12 in./ft)^2 (150 pcf) = 17.2 kips

Weight of tributary slab-on-grade at basement = (20 ft)^2 (4/12 ft)(150 pcf) = 20.0 kips

Weight of soil over footing = (10 ft)^2 (6 in./12)(120 pcf) = 6.0 kips

Weight of column footing = (10 ft)^2 (18 in./12)(150 pcf) = 22.5 kips

M_{ST} = 20 ft(160 kips+52.5/2 kips+17.2 kips+20.0 kips+6 kips+22.5 kips)

= 5,039 kip-ft

Q_{CE} = 0.9M_{ST}
= overturning moment due to seismic forces reduced per ASCE 41-13 § 7.2 load combinations.

Q_{UD} = 20,700 k-ft (see previous calculations)

ASCE 41-13 § 8.4.2.3.2.1 states that "Where overturning results in an axial uplift force demand from linear analysis, this uplift shall be considered deformation controlled, and an *m*-factor of 1.5 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention applied to the expected restoring dead load shall be used."

Commentary

Since the dead load resisting overturning does not rely on material strength, a κ = 1.0 value is used.

$m\kappa Q_{CE} > Q_{UD}$ (For overturning use κ equal to 1.0)

(4)(1)(0.9)(5,039 k-ft) = 18,140 k-ft < Q_{UD} = 20,700 k-ft.

Accordingly, the foundation is not acceptable with overturning due to tension at foundation. Additional dead load is required at foundation to resist overturning.

$M_{OT} = m(0.9)M_{ST} = m\kappa Q_{CE}$

The additional dead load centered on the column necessary is calculated as:

$(M_{OT} - m\kappa Q_{CE})/[L\kappa m(0.9)]$ = (20,700 k-ft – 18,140 k-ft)/[(20 ft)(1)(4)(0.9)]

= 35.6 kips

Potential solutions are to remove existing concrete slab on grade and provide reinforced mat over existing footing to match required dead load, or provide micro piles/rock anchors.

Tension that can be developed at end columns of Wall D below the first level are checked:

Dead load of column, tributary slab on grade, soil over footing, footing, and additional dead load required to resist overturning.

4.7 kips + 20 kips + 6 kips + 22.5 kips + 35.6 kips = 88.8 kips < 288 kip column capacity.

Therefore, column at end of discontinuous Shear Wall D is acceptable in tension as previously noted when checking the column due to expected moment capacity of Shear Wall D.

10.5.3 Summary for BSE-2E CP Performance (ASCE 41-13 § 7.5.2 and § 10.7)

Table 10-13 and Figure 10-16 provide a summary of the acceptability of the walls and foundation.

Table 10-13 Final Acceptance Ratios for BSE-2E CP Performance

Element	Location	Level	Action	Q_{UD} or Q_{UF} (k or k-ft)	Q_{CE} or Q_{CL}	m	mkQ_{CE} or kQ_{CL} (k or k-ft)	Q_{UC}/mkQ_{CE} or Q_{UF}/kQ_{CL}	Acceptable
W1F2	Gridline 1	1	Shear*	1,362	838	NA	754	1.80	NO
W4F2	Gridline 4	1	Shear*	1,362	838	NA	754	1.80	NO
WAF2	Gridline A	1	Flexure*	36,010	9,600	6	57,600	0.63	YES
WDF2	Gridline D	1	Flexure*	20,700	11,930	3.37	40,200	0.51	YES
WGF2	Gridline G	1	Flexure*	35,110	12,230	6	73,380	0.48	YES
Column	Gridline D	BSMNT	Compression*	885	900	NA	810	1.09	NO
			Tension	393	320	NA	288	1.37	YES
			Shear	27.5	65	NA	58.5	0.47	YES
Foundation	Gridline D	FDN	Bearing	1,324	2,400	4	8,640	0.15	YES
Overturning	Gridline D	FDN	Overturning	20,700	4,535	4	18,140	1.14	NO

* Controlling failure mode

NA Not applicable as *m*-factors do not apply to force-controlled actions

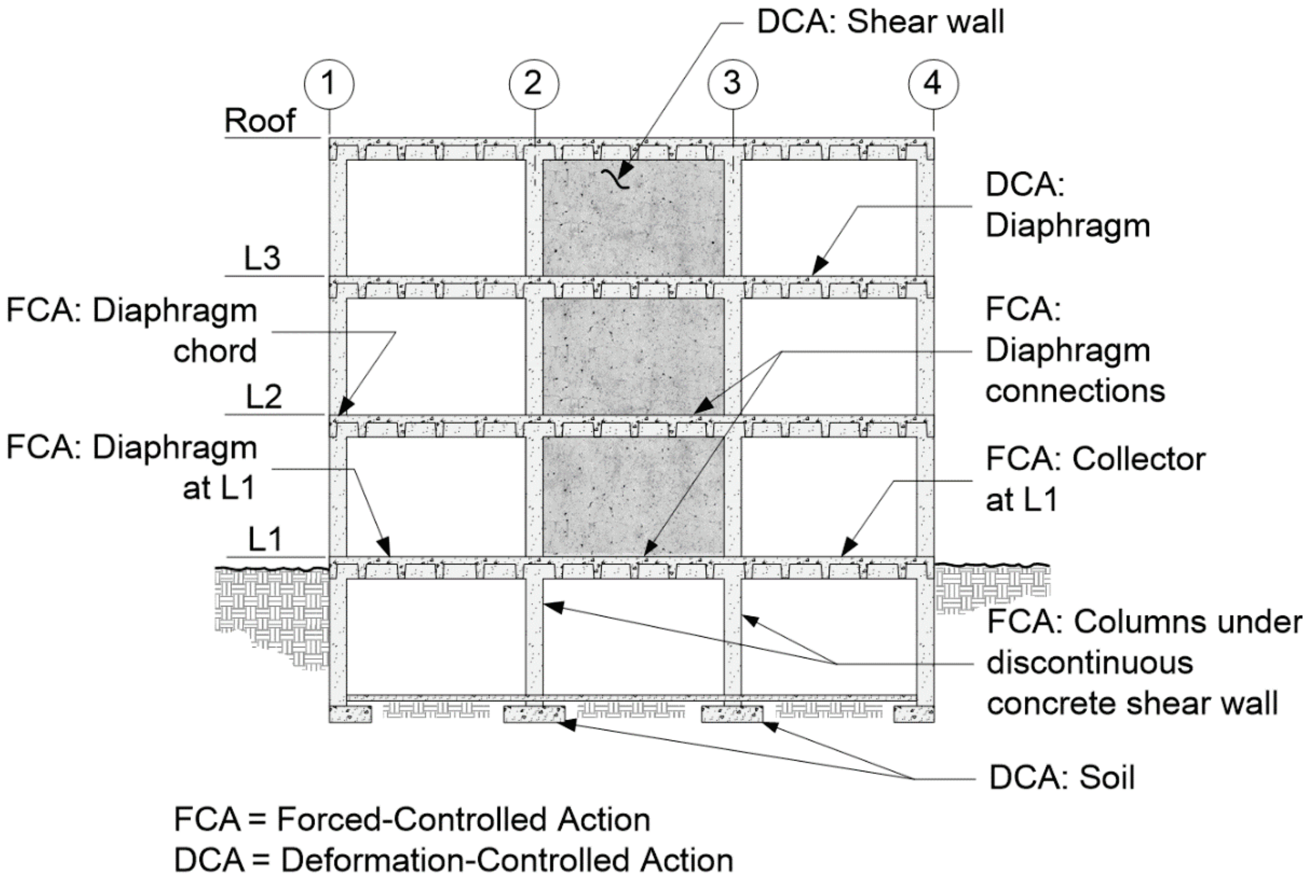


Figure 10-16 Deformation-controlled and force-controlled actions at Gridline D.

10.5.4 Summary for BSE-1E LS Performance (ASCE 41-13 § 7.5.2 and ASCE 41-13 § 10.7)

The BSE-1E Seismic Hazard Level and LS Performance Level are checked in the same manner as the BSE-2E Seismic Hazard Level and CP Performance Level shown above and the following information from the online tools described in Chapter 3 of this *Guide*:

$S_{X1} = 0.376$

$S_{XS} = 0.691$

$V_{TRANS} = 0.588$

$V_{LONG} = 0.553$

Table 10-14 summarizes the results. BSE-1E results do not govern the design.

Table 10-14 Final Acceptance Ratios for BSE-1E LS Performance

Element	Location	Level	Action	Q_{UD} or Q_{UF}	Q_{CE} or Q_{CL}	m	$m\kappa Q_{CE}$ or κQ_{CL}	Q_{UC}/mkQ_{CE} or Q_{UF}/kQ_{C}	Acceptable
W1F2	Gridline 1	1	Shear*	876	838	NA	754	1.16	NO
W4F2	Gridline 4	1	Shear*	876	838	NA	754	1.16	NO
WAF2	Gridline A	1	Flexure*	20,760	9,600	4	38,400	0.54	YES
WDF2	Gridline D	1	Flexure*	11,940	11,930	2.29	27,320	0.44	YES
WGF2	Gridline G	1	Flexure*	20,240	12,230	4	48,920	0.41	YES
Column	Gridline D	BSMNT	Compression*	570+	922	NA	830	0.69	YES+
			Tension	78+	320	NA	288	0.27	YES
			Shear	15.9	65	NA	58.5	0.27	YES
Foundation	Gridline D	FDN	Bearing	886	2,400	3	6,480	0.14	YES
Overturning	Gridline D	FDN	Overturning	11,940	4,574	3	13,600	0.88	YES

* Controlling failure mode

NA Not applicable as *m*-factors do not apply to force controlled actions

\+ Force delivery reduction factor applied to pseudo force at end of discontinuous shear wall. If M_{CE} of wall D is applied the columns at the end fail in compression as noted in the BSE-2E/CP analysis.

10.6 Rigid Diaphragm Check (ASCE 41-13 § 7.2.9)

The rigid concrete diaphragm, shown in Figure 10-17, is evaluated at the first floor level for transfer and distribution of seismic forces from the discontinuous shear wall at Gridline D and to the perimeter concrete shear walls along Gridlines A and G. First, the ability to transfer lateral forces from the discontinuous shear Wall D to the adjacent concrete diaphragm is checked, and then the ability for the first level concrete diaphragm to distribute the load to the perimeter walls is checked.

Commentary

The shear transfer/collector demands should be evaluated for all walls at all levels. In this example, the diaphragm is only evaluated at the Level 1.

Per ASCE 41-13 § 10.10.2.4, the connection between the concrete Wall D and diaphragm shall be considered force-controlled. This is also true for the struts, collectors, and chords.

10.6.1 Slab Connection to Concrete Wall D at Level 1

With the symmetry of the building and shear walls at the perimeter of the basement level, the lateral forces from Wall D will be equally distributed on each side of the shear wall into the Level 1 slab. The connection at Level 1 consists of #3 at 16 inches on center, 4-inch slab reinforcing being continuous through the wall.

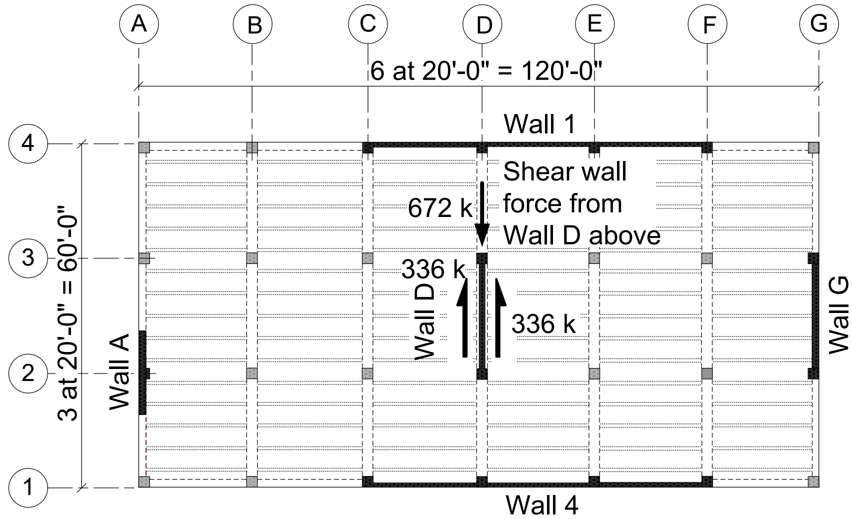


Figure 10-17 Level 1 diaphragm.

Diaphragm connection demand:

$Q_{E\text{DIAPH}} = V_{\text{WallD}}/2$

$Q_{E\text{DIAPH}}$ = 672 kips/2 = 336 kips

For force-controlled action of the diaphragm and diaphragm chord, using LSP, determine Q_{UF} per ASCE 41-13 § 7.5.2.1.2:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

C_1 = 1.17, C_2 = 1.01 (see previous calculations in Section 10.4.6.1 of this *Guide*)

J = 1.0 with concrete diaphragm assumed to remain elastic since it is being evaluated as a force-controlled element

Q_G = 0 to consider diaphragm forces only

Q_{UF} = 284 kips

Determine shear capacity across the critical plane per ACI 318-11 Sections 11.1.1, 11.2.1.1, and 11.4.7.2:

$Q_{CL\text{DIAPH}} = V_{N\text{Slab}}$

$$V_{N\text{Slab}} = 2\sqrt{f'_{clb}}A_c + \frac{A_v f_{ylb} d}{s}$$

where:

f'_{clb} = 2,500 psi

= lower bound concrete strength

d = 20 ft

= length over which the local shear will act, equal to the length of the shear wall

A_c = (4 in.)(d)

= area of concrete diaphragm at interface of slab and shear Wall D

A_v = 0.11 in.^{2} = area of steel across the shear plane

f_{ylb} = 40,000 psi = lower bound steel strength

s = 16 in. = spacing of reinforcing

Q_{CL} = V_N

$$= \left[(2\sqrt{2500}\text{ psi})(4\text{ in.}) + \frac{0.11\text{ in.}^2(40{,}000\text{ psi})}{16\text{ in.}} \right]$$

$$(20\text{ ft})(12\text{ in./ft}) / (1000\text{ lbs/ft})$$

= 162 kips

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

(0.9)(162 kips) = 146 kips < Q_{UF} = 284 kips → Diaphragm connection not acceptable

$Q_{UF}/\kappa Q_{CL}$ = 284 kips/146 kips = 1.95

The capacity of the connection between the diaphragm and concrete shear wall is the same as the diaphragm capacity; therefore, a diaphragm collector is required, or the diaphragm adjacent to the discontinuous shear wall will need to be strengthened. A diaphragm collector will be provided.

10.6.2 Diaphragm Collector Design

A collector for the full width of the diaphragm will be provided to act in compression on one side of the wall and in tension on the other. Both conditions will be checked. See Figure 10-18 below.

10.6.2.1 Demand on Collectors

Determine Q_{UF} and Q_{CL}:

Q_{UF} = 284 kips

Length of diaphragm collector = 60 ft

Ratio capacity based on initial diaphragm connection check:

κQ_{CL} = 60/20(0.9)(162kips) = 438 kips

$\kappa Q_{CL} > Q_{UF}$

(0.9)(438 kips) = 394 kips > 284 kips → Collector length is acceptable

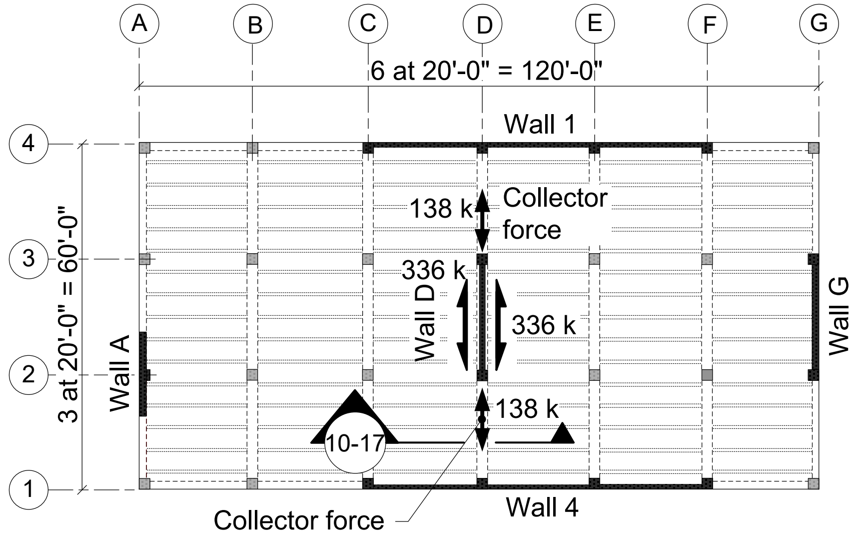


Figure 10-18 Level 1 diaphragm collectors at discontinuous shear wall.

10.6.2.2 Seismic Force on Collector

The seismic force on collector, shown on Figure 10-18, will be total seismic force transferred to the diaphragm on each side, less the amount transferred directly by the adjacent concrete slab:

$$Q_{UF} - \kappa Q_{CL\ \text{direct slab}} = Q_{UF\ \text{Collector}}$$

568 kips – (2)(0.9)(162 kips) = 276 kips or 138 kips each side of wall

Determine tension requirements:

Neglect the tension reinforcing in the concrete beam.

Provide (4) #7 each side of the beam for a total of (8) #7

A_S = 4.8 in.^{2}

f_{yl} = 60 ksi

$Q_{CL\ \text{Tension}} = A_S f_{yl}$

Q_{CL} = (4.8)(60 kips) = 288 kips

$\kappa Q_{CL} > Q_{UF}$

(1.0)(288 kips) = 288 kips > 138 kips → Collector reinforcing acceptable

Commentary

κ is equal to 1.0 for new construction with material testing values confirmed during construction.

Check compression on collector:

Gravity demands on beam

$$Q_G = 1.1(Q_D + Q_L + Q_S) \quad \text{(ASCE 41-13 Eq. 7-1)}$$

Tributary width = 20 ft

Q_{DL} = (20 ft)(100 psf) = 2,000 plf

Q_{LL} = (0.25)(20 ft)(125 psf) = 625 plf

M_{UD} = 144 kip-ft (Gravity moment)

P_{UD} = ±140 kips (Collector force)

Beam length = 20 feet

Beam reinforcing

(3) #9 top and bottom

#4 ties at inches on center

Commentary

Conservatively, only the existing concrete beam is considered in compression with the direct load transfer from concrete shear wall above to adjacent existing concrete beam.

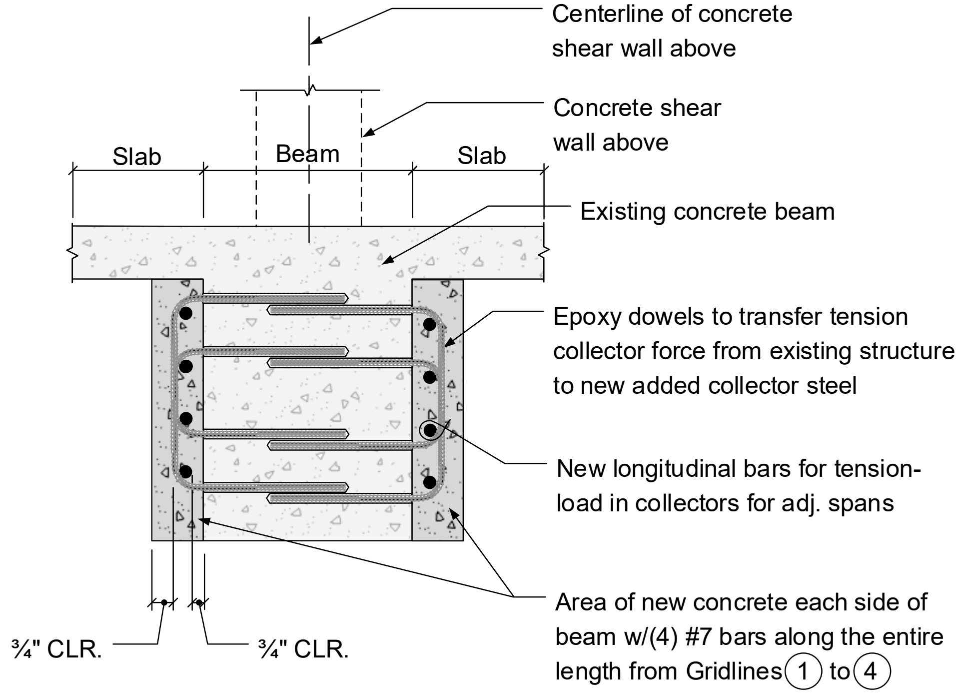


Figure 10-19 Collector detail.

10.6.2.3 Check Axial Load with *spColumn*®:

As shown in Figure 10-20, per *spColumn*® beam is acceptable in combined flexure and compression.

10.6.3 Check Overall Level 1 Diaphragm

Per ASCE 41-13 § 7.5.1.2 Table C7-1 Footnote "c," the diaphragm flexure and shear are considered to be force controlled because they distribute the lateral forces from the discontinuous concrete shear Wall D from the level above.

10.6.3.1 Flexural Demand on the Level 1 Diaphragm

With the Level 1 being the base of the structure and soil each side of the floor, the flexural demand on the diaphragm is from the distribution of the seismic forces from the discontinuous interior concrete shear Wall D.

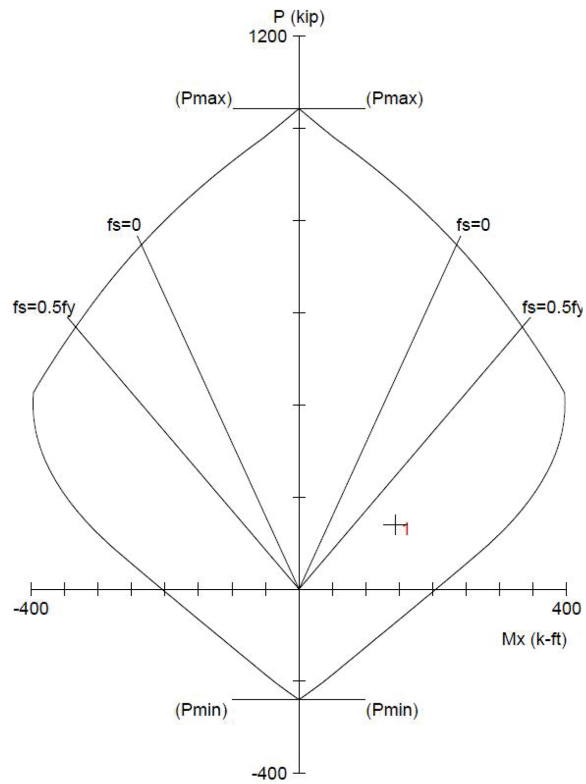


Figure 10-20 P-M curve output from *spColumn*®.

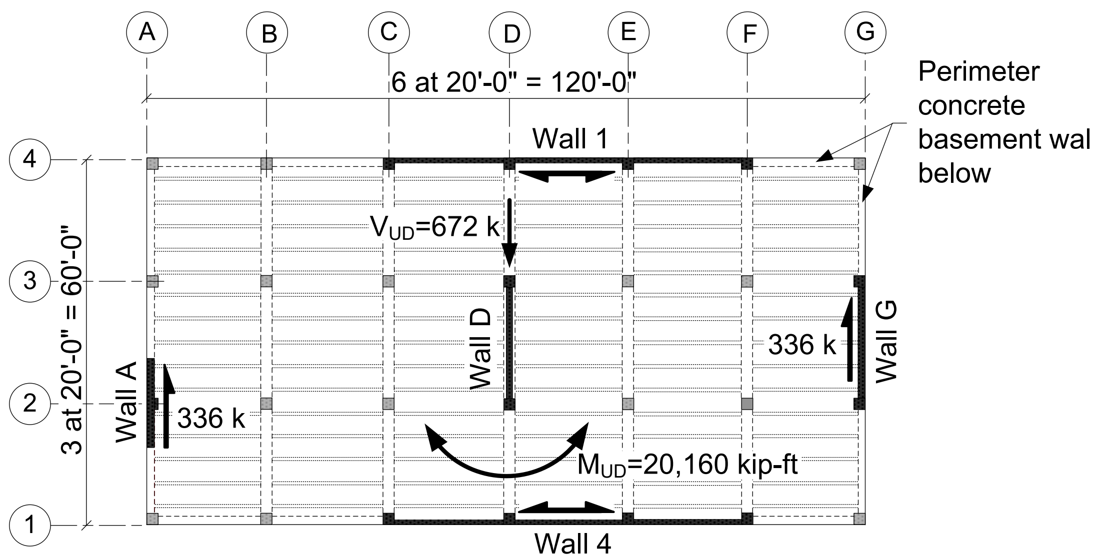


Figure 10-21 Level 1 diaphragm forces from discontinuous concrete shear wall.

M_{UD} $= PL/4$

P = 672 kips

L = 120 ft

M_{UD} = (672 kips)(120 ft)/4 = 20,160 kip-ft

Shear demand on diaphragm

V_{UD}= 336 kips

Determine flexural capacity of floor diaphragm, M_N, by using *spColumn*®:

For the diaphragm capacity, the perimeter wall is used like a flange, the reinforcing in the wall and the continuous reinforcing in the floor joist across the diaphragm.

Reinforcing at perimeter wall: #4 horizontal at 16 inches on center

Depth of wall considered: 14/2 or 7 feet (Based on engineering judgment)

Continuous reinforcing for joist: (2) #4 (joist every four feet)

Length of diaphragm joist considered effective: 60/4 or 15 feet

Number of joists considered: 3

Diaphragm depth = 60 ft

Slab thickness = 4 in.

Material Properties

f'_{clb} = 2,500 psi

f_{ylb} = 40,000 psi

Results from *spColumn*®:

M_N = 10,540 kip-ft

$\kappa Q_{CL} > Q_{UF}$

(0.9)(10,540 kip-ft) = 9,486 kip-ft < 20,160 kip-ft

DCR = 20,160 kip-ft/9,486 kip-ft = 2.15

V_{Clb} = 486 kips (from previous calculations)

$\kappa Q_{CL} > Q_{UF}$

(0.9)(486 kips) = 437 kips > 336 kips

DCR = 336 kips/437 kips = 0.77 < 2.15

Level 1 diaphragm is flexurallycontrolled.

10.6.3.2 Seismic Mitigation

Provide (7) #7 chord reinforcing at perimeter wall and re-run *spColumn*®

M_N = 24,464 kip-ft

$\kappa Q_{CL} > Q_{UF}$

(0.9)(24,464 kip-ft) = 22,076 kip-ft > 20,160 kip-ft

Diaphragm chord is acceptable.

10.7 FRP Design of Existing Concrete Shear Walls at Gridlines 1 and 4

From the analysis for Wall 1 in Section 10.5.1.1 of this *Guide* (Wall 4 similar), the wall was found to not meet the acceptance criteria per ASCE 41-13 Equation 7-37:

$\kappa Q_{CL} > Q_{UF}$ for shear

From Table 10-13 of this *Guide*: Q_{CL} = 838 kips and Q_{UF} = 1362 kips

κ = 0.9

0.9(838 kips) = 754 kips < 1362 kips → Wall is not acceptable in shear

Fiber reinforced polymer (FRP) will be designed to provide additional shear capacity to the wall. The FRP is designed following the equations of ACI 440.2R-17 (ACI, 2017).

$V_n = V^*_n + \psi_f V_f$ (ACI 440.2R-17 Eq.13.7.2.2.b)

where:

V^*_n = the nominal shear strength of the existing wall

ψ_f = 0.85 (FRP reduction factor)

V_f = shear provided by FRP

Commentary

FRP is a desirable alternate to shotcrete because it adds minimal mass to the building and does not increase the width of the wall which can be important if the wall is part of an egress route.

The FRP will be designed to provide $\psi_f V_f$ such that $V_n = Q_{CL} + \psi_f V_f > Q_{UF}/\kappa$

Therefore, $\psi_f V_f \geq$ 1362 kips/0.9 – 838 kips = 675 kips

The following carbon FRP (CFRP) system will be used in design:

t_f = thickness per ply
= 0.023 in.

f^*_{fu} = ultimate tensile strength
= 115 ksi

ε^*_{fu} = rupture strain
= 0.012 in./in.

E_f = modulus
= 9,600 ksi

The wall is located in an enclosed and conditioned space and a carbon FRP material will be used. Therefore, per ACI 440.2R-17 Table 9.4, an environmental-reduction factor, C_E, of 0.95 is suggested.

$f_{fu} = C_E f_{fu}^*$

$\varepsilon_{fu} = C_E \varepsilon_{fu}^*$

f_{fu} = (0.95)(115 ksi) = 109 ksi

ε_{fu} = (0.95)(0.012) = 0.011 in./in.

As per ACI 440.2R-17 Section 13.7.3, FRP should be provided on two faces of the wall if the ratio of the existing transverse steel reinforcement to gross concrete area, ρ_t, is less than 0.0015. ρ_t, is 0.0010, so FRP will be provided on two faces of the wall.

ACI 440.2R-17 Equation 13.7.2.2c gives the capacity of a two-sided retrofit as:

$V_f = 2t_f\varepsilon_{fe}E_f d_{fv}$

where:

$\varepsilon_{fe} = \kappa_v \varepsilon_{fu}$ (ACI 440.2R-17 Eq. 11.4.1.2a)
= (0.28)(0.011) = 0.003 ≤ 0.004

$\kappa_v = k_1 k_2 L_e / 468\varepsilon_{fu}$ (ACI 440.2R-17 Eq. 11.4.1.2b)
= (0.73)(0.99)(1.99)/[(468)(0.011)] = 0.28

$k_1 = (f_{clb}/4000)^{2/3}$ (ACI 440.2R-17 Eq. 11.4.1.2d)
= $(2500 \text{ psi}/4000 \text{ psi})^{2/3}$ = 0.73

$k_2 = (d_{fv} - 2L_e)/d_{fv}$ (ACI 440.2R-17 Eq. 11.4.1.2e)
= (576 – 2(1.99))/576 = 0.99

$d_{fv} = 0.8L_w$ (ACI 318 Chapter 21)
= (0.8)(60 ft)(12 in./ft) = 576 in.

$L_e = 2500/(n_f t_f E_f)^{0.58}$ (ACI 440.2R-17 Eq. 11.4.1.2c)
= $2500/[(0.023)(9600)(10^3)]^{0.58}$ = 1.99 in. (one ply)

V_f = (2)(0.023)(0.003)(9600)(576)
= 763 kips

Using a single ply on each side of wall: $\psi_f V_f$ = (0.85)(763) = 649 kips

From above, $\psi_f V_f \geq$ 1362 kips /0.9 – 838 kips = 676 kips. Design is not sufficient.

> **Commentary**
>
> The FRP design should be considered a force-controlled action unless a deformation-controlled classification is justified based on experimental data. As such, boundary zone confinement is not required.

Repeat calculations using two plies (n_f = 2) on each face of the wall.

L_e = 1.33 in.

k_2 = 0.99

κ_v = 0.19

ε_{fe} = 0.002

V_f = 1,017 kips

Using two plies on each side of wall: $\psi_f V_f$ = 0.85 × 1,017 kips = 864 kips > 676 kips

κV_n = 0.9(838 kips + 864 kips) = 1,532 kips > Q_{UF} OK

Install two layers of FRP on each face of the wall for the full wall length, as shown in Figure 10-22.

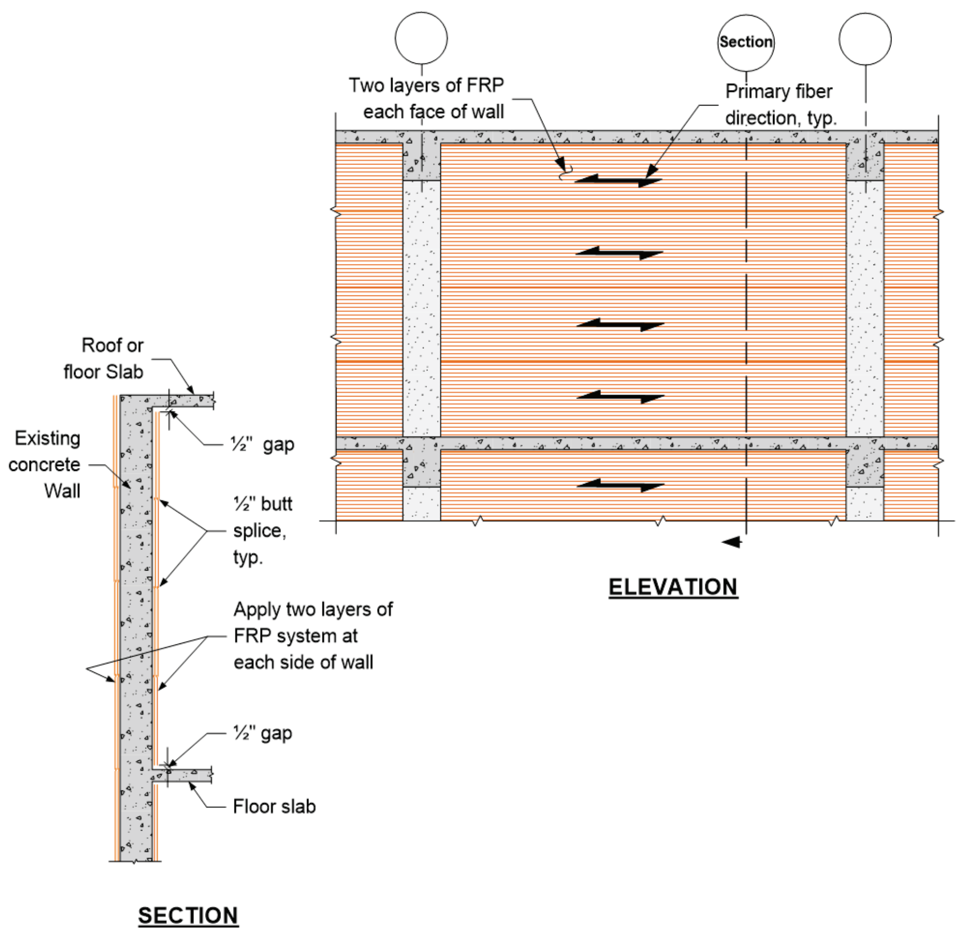


Figure 10-22 FRP shear wall reinforced detail.

10.8 FRP Design of Columns Supporting Discontinuous Shear Wall Gridline D

The analysis in Section 10.5.2.1 of this *Guide* shows that Gridline D columns at the end of the discontinuous shear wall are not acceptable in compression.

$\kappa Q_{CL} > Q_{UF}$ (ASCE 41-13 Eq. 7-37)

From Table 10-13 of this *Guide*, Q_{UF} = 885 kips and Q_{CL} = 900 kips

κ = 0.9

0.9(900 kips) = 810 kips < 885 kips (compression)

Therefore, columns are not acceptable in compression

Fiber reinforced polymer will be designed to provide additional confinement to the columns to increase the compressive strength. The FRP is designed following the equations of ACI 440.2R-17.

$\phi P_n = 0.8f[0.85f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}]$ (ACI440.2R-17 Eq. 12.1b)

where f'_{cc} is compressive strength of the confined concrete.

The objective is to strengthen the column such that $\kappa Q_{CL} > Q_{UF}$ = 889 kips.

The following FRP will be used:

t_f = thickness per ply
= 0.013 in.

f^*_{fu} = ultimate tensile strength
= 550 ksi

ε^*_{fu} = rupture strain
= 0.017

E_f = modulus
= 0.017

The column is located in an enclosed and conditioned space and a carbon FRP (CFRP) material will be used. Therefore, per ACI 440.2R-17 Table 9.4, an environmental-reduction factor of 0.95 is suggested.

$f_{fu} = C_E f^*_{fu}$

$\varepsilon_{fu} = C_E \varepsilon^*_{fu}$

f_{fu} = (0.95)(550 ksi) = 523 ksi

ε_{fu} = (0.95)(0.017) = 0.016 in./in.

The required axial compression strength of column is obtained by reordering ACI 440.2R-17 Equation 12.1a:

f'_{cc} = $[0.85(A_g - A_{st})]^{-1}[(Q_{UF}/\kappa 0.8\phi) - f_y A_{st}]$

where:

ϕ = 1

κ = 0.9

$P_n = Q_{CL}$

f'_{cc} = $[0.85((20 \text{ in.})^2 - (8 \times 1)\text{in.}^2)]^{-1}[889 \text{ kips}/(0.9 \times 0.8 \times 1.0) - 40 \text{ ksi} \times 8 \text{ in.}^2 \times 1]$
= 2.74 ksi

Therefore, the minimum required f'_{cc} is 2.74 ksi.

Required confining stress is obtained by reordering ACI 440.2-17 Equation 12-1g:

$f_l = (f'_{cc} - f'_c)/3.3\kappa_a$ (ACI 440.2R-17 Eq. 12.1h)
= (2.74 ksi – 2.5 ksi)/(3.3 × 0.45) = 0.16 ksi

$\kappa_a = (A_e/A_c)(b/h)^2 = 0.45$ (ACI 440.2R-17 Eq. 12.1.2b)

$A_e/A_c = (1 - \rho_g)^{-1}\{1 - [(b/h)(h - 2r_c)^2 + (h/b)(b - 2r_c)^2]/3A_g - \rho_g\}$
(ACI 440.2R-17 Eq. 12.1.2d)
$= (1 - 0.02)^{-1}\{1 - [(1)(18 \text{ in.})^2 + (1)(18 \text{ in.})^2]/(3 \times 400 \text{ in.}^2) - 0.02\}$
= 0.45

$\rho_g = 8 \text{ in.}^2/400 \text{ in.}^2 = 0.02$

$b = h$ = 20 in. and assume radius of corner, r_c = 1 in.

Minimum confinement ratio f_l/f'_c should not be less than 0.08 as per ACI 440.2R-17 Section 12.1:

f_l/f'_c = 0.16 ksi/2.5 ksi = 0.064 which is less than 0.08.

Therefore, use a minimum f_l = 0.20 ksi so f_l/f'_c = 0.20 ksi /2.5 ksi = 0.08.

Required number of plies obtained by reordering ACI 440.2-17 Equation 12-1h:

$$f_l = \frac{2E_f n t_f \varepsilon_{fe}}{D}$$ (ACI 440.2R-17 Eq. 12.1h)

And solve for number of plies (n):

$n = [f_l(b^2 + h^2)^{1/2}]/(2E_f t_f \varepsilon_{fe})$

Commentary

Once the required minimum confined concrete strength is determined, the number of layers of FRP required to achieve this confined concrete strength is calculated.

$$\varepsilon_{fe} = \kappa_e \varepsilon_{fu} = (0.55)(0.016) \leq 0.004 \qquad \text{(ACI 440.2R-17 Eq. 12.2)}$$

$$\begin{aligned} n &= (0.20\text{ ksi})(400\text{ in.}^2 + 400\text{ in.}^2)^{1/2}/[(2)(33000\text{ ksi})(0.013\text{ in.})(0.004)] \\ &= 1.6 \\ &= 2.0\text{ pl} \end{aligned}$$

The maximum strain in the FRP confined concrete is limited to 0.01 to prevent excessive cracking.

$$\varepsilon_{ccu} = \varepsilon_c'[1.5 + 12\kappa_b(f_l/f_c')(\varepsilon_{fe}/\varepsilon_c')^{0.45}] < 0.01 \qquad \text{(ACI 440.2R-17 Eq. 12.1j)}$$

$$\begin{aligned} \varepsilon_{ccu} &= 0.002[1.5 + 12(0.43)(0.20\text{ ksi}/2.5\text{ ksi})(0.004/0.002)^{0.45}] \\ &= 0.003 < 0.010\ldots\text{OK} \end{aligned}$$

Install two layers of FRP on the column with primary fibers horizontal, as shown in Figure 10-23.

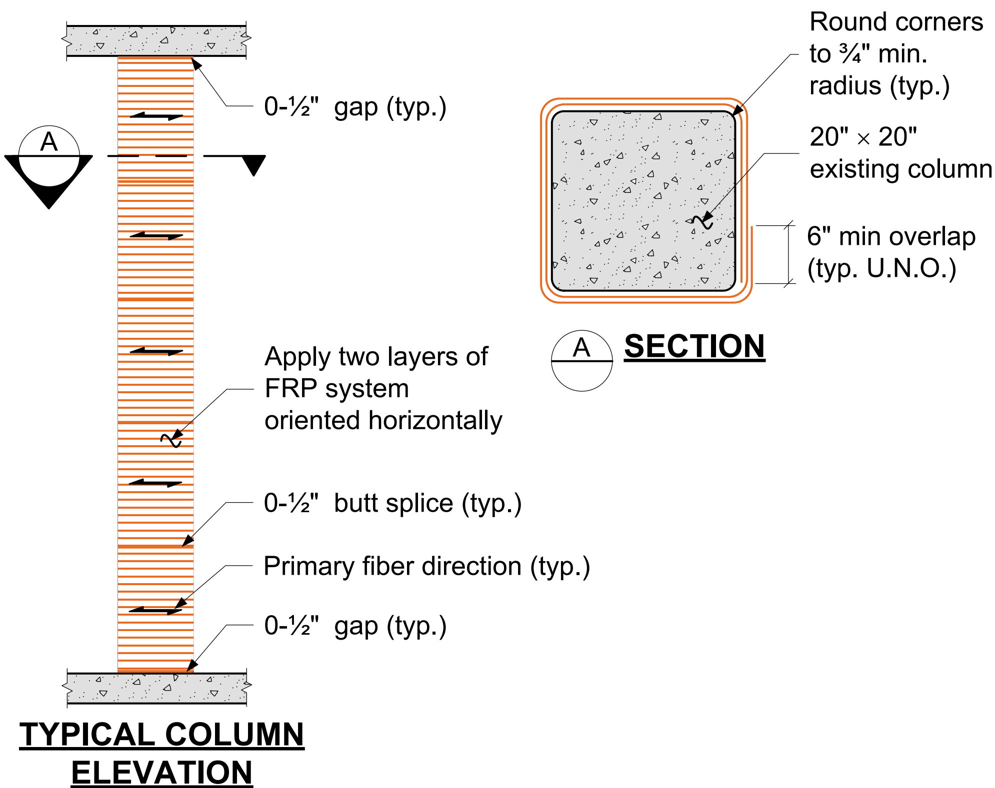


Figure 10-23 FRP at concrete column.

It is necessary to verify the flexural and shear capacities of this FRP-retrofit column. The moment and shear resisted by this column are provided in Section 10.5.2.1 of this *Guide*:

$$M_u = 165\text{ k-ft}$$

$$V_u = 27.5\text{ kips}$$

The axial load is given previously as:

$$P_u = 885 \text{ kips}$$

The shear capacity of the column is adequate without retrofit. The concrete component (alone) of shear resistance neglecting the beneficial effect of axial load is:

$$V_c = 2\sqrt{f_c'}\, b_w d = 2\sqrt{2500} \text{ psi} \times 20 \text{ in.} \times 17.5 \text{ in.} = 35 \text{ kips}$$

> **Commentary**
>
> The FRP is installed with the primary fibers horizontal, so no additional tensile capacity is provided in the vertical direction. The FRP retrofit cannot be installed as 1.6 plies, so 2 plies are installed on the columns. This increases the confined concrete strength above the required 2.74 ksi to 2.9 ksi. This is accounted for in the P-M analysis.

Thus, only axial load-moment (P-M) interaction in considered. Program RESPONSE 2000 (Bentz, 2000) is used to calculate all P-M interaction curves presented in Figure 10-24.

As seen in Figure 10-24, the column P-M demand (885 kips – 165 k-ft) exceeds the capacity calculated using prescribed ASCE 41-13 reduction factors, ϕ = 1.0 and κ = 0.90. The behavior is clearly compression-dominated; nonetheless the retrofit provided must be assessed in terms of its P-M response.

The retrofit provides confinement to the column but no additional longitudinal reinforcement. To model this, the compressive strength of concrete is increased from f_c' to confined compressive strength resulting from the retrofit, f_{cc}'.

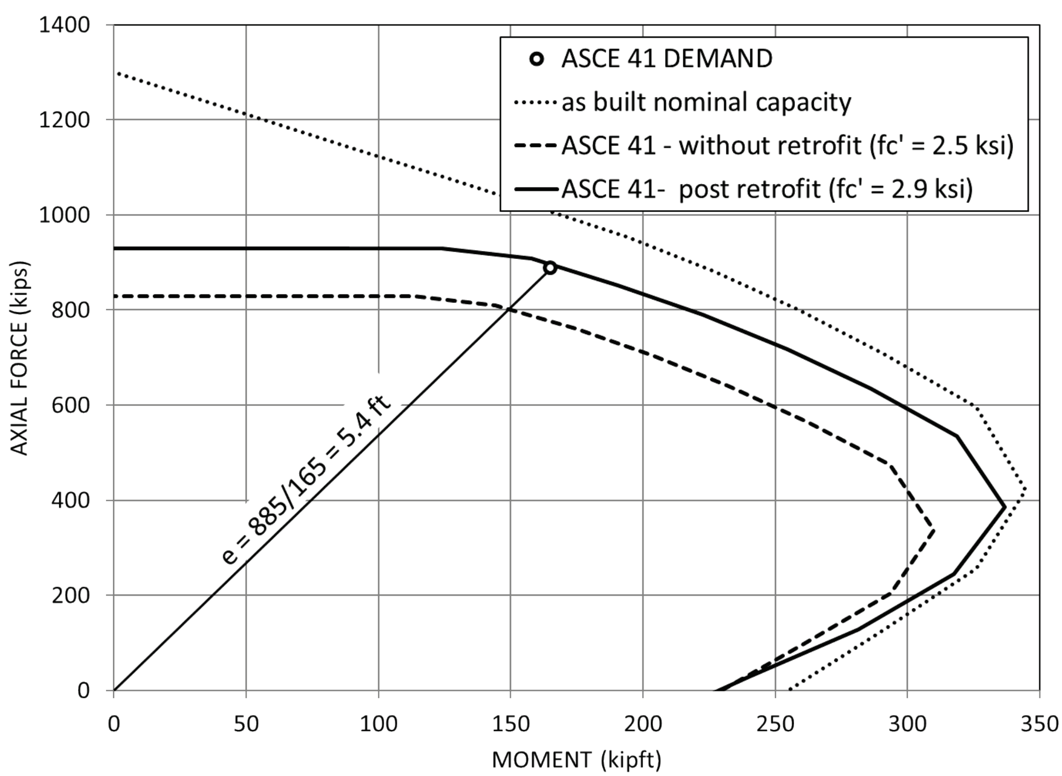


Figure 10-24 P-M interaction diagram for column from RESPONSE 2000 program.

The two ply retrofit provides 125% of the calculated required confinement (1.6 plies). Therefore the value of f_l provided is 1.25 × 0.20 ksi = 0.25 ksi [from minimum required f_l / f'_c = 0.20 ksi/2.5 ksi = 0.08].

From ACI 440.2R-17 Equation 12.1g, f^*_{cc} may be calculated:

$$f'_{cc} = f_l \,\psi_f (3.3\kappa_a) + f'_c = (0.25 \text{ ksi})(0.95)(3.3)(0.45) + 2.5 \text{ ksi} = 2.9 \text{ ksi}$$

The P-M interaction diagram calculated using f'_{cc} = 2.9 ksi is shown to be adequate for the column demand (Figure 10-24).

A single P-M-V interaction analysis was conducted using RESPONSE 2000. With P-M-V = 885 kips – 165 k-ft – 27.5 kips and κ = 0.9, the results indicate that the column is inadequate in compression with f'_c = 2.5 ksi but adequate with f'_{cc} = 2.9 ksi.

10.9 Check Non-Contributory Concrete Frames (ASCE 41-13 § 7.5.1.1 and § 10.4)

A finite element model would provide a thorough analysis of the building's behavior. Figure 10-25 below shows the primary and secondary elements of the building. If the secondary frames were included as part of an overall analysis model, then one would need only take the demands placed upon them by the pseudo seismic force and compare that to the expected strength of the secondary element multiplied by the appropriate *m*-factor. If the secondary frames were not included in the overall analysis model, then a separate model of the secondary frames should be built or another method employed to determine the demands on the beams, columns, slabs, and joints due to the displacement of the primary lateral force-resisting system with the pseudo-seismic forces. For this example, we will determine building drifts based on the relative stiffness used in the initial lateral analysis used to determine the distribution of pseudo seismic forces to individual concrete shear walls. This analysis assumes a rigid floor and roof diaphragm.

Note

For more of a discussion on primary and secondary elements, see Section 4.4 of this *Guide.*

Per ASCE 41-13 § 10.3.1.2.1, the design displacement shall consider cracked sections accounting for expected inelastic response. ASCE 41-13 § 10.3.1.2.1 refers to Table 10-5 to determine appropriate cracked section modifiers.

10.9.1 Story Drifts

The following method was used to determine story drift with the first term being deflection due to flexural deformation and the second term being deflection due to shear deformation. Cracked sections were used per ASCE 41-13 Table 10-5.

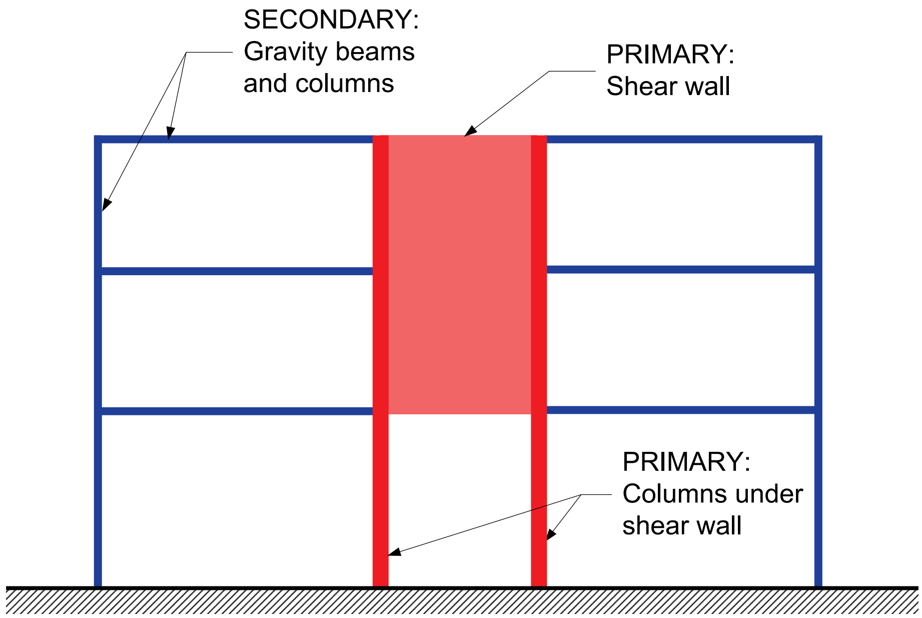


Figure 10-25 Primary and secondary elements.

$$\delta_x = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG}$$

Transverse direction:

$\delta_{R,TRAN}$ = 0.094 in.

$\delta_{3,TRAN}$ = 0.189 in.

$\delta_{2,TRAN}$ = 0.237 in.

Longitudinal direction:

$\delta_{R,LONG}$ = 0.055 in.

$\delta_{3,LONG}$ = 0.109 in.

$\delta_{2,LONG}$ = 0.137 in.

10.9.2 Column Loads

First, column loads are determined as they apply to applicable *EI* in determining applied moments and shear to columns. The column at gridline C2 is taken as a typical secondary column.

Load combination per ASCE 41-13 § 7.2.

$P_G = 1.1(P_D + P_L + P_S)$ (ASCE 41-13 Eq. 7-1)

$= 0.9P_D$ (ASCE 41-13 Eq. 7-2)

P_D = supported dead load, including: (1) floor dead load and roof dead load; and (2) weight of column

= 3(20 ft)(20 ft)(100 psf) + [(14 ft)(14/12 ft)^2]+[(14 ft)(16/12 ft)^2](150 pcf)

= 120+6.6 kips

= 127 kips

P_L = supporting live load, including floor live load

= 2(20 ft)(20 ft)(125 psf)

= 100 kips

P_S = snow load: 25 psf. Snow load less than 30 psf therefore can be considered zero.

= 0

P_G = 1.1(127 kips + 100 kips + 0)

= 250 kips or

= 0.9(127 kips)

= 114 kips

10.9.3 Moment and Shear Demands on Secondary Columns

Determine moment and shear demands on secondary columns based on building displacements. From above, the maximum displacement is between the first and second level, with the stiffest columns at the first level. For column properties, see Table 10-2 of this *Guide*.

Determine M_{UD}:

Moment for columns at first floor due to displacement (at 2nd Floor):

$$M_u = \frac{6EI\delta}{L^2}$$ Fixed top and bottom per ACI 318-11 Section 8.10.3

E_{CE} = 3,490,000 psi

I = 18 in.(18 in.)^3/12

= 8,748 in.^{4}

Column loads are 228 and 114 kips (see previous calculations)

$A_G f'_{ce}$ = (18 in.)(18 in.)(3.75 ksi) = 1,215 kips

P_G = 250 kips (see previous calculations)

$P_G / A_G f'_{ce}$ = 250 kips/1,215 kips = 0.21

Therefore, design gravity load is $0.21 A_G f'_c > 0.1 A_G f'_c$.

Effective flexural rigidity per ASCE 41-13 Table 10-5 with gravity load less than $0.5 A_G f'_c$ and greater than $0.1 A_G f'_c$.

$EI = 0.39\ E_c I_g = 0.39(3{,}490{,}000\text{ psi})(8{,}748\text{ in.}^4)$

$L = 12\text{ ft}$

Transverse Direction:

$$M_{u,x} = \frac{(6)(0.39)(3{,}490{,}000\text{ psi})(8{,}748\text{ in.}^4)(0.237\text{ in.})}{\left[(12\text{ ft})(12\text{ in./ft})\right]^2(1000\text{ lbs/kip})(12\text{ in./ft})} = 68.0\text{ k-ft}$$

Longitudinal Direction:

$$M_{u,y} = \frac{(6)(0.39)(3{,}490{,}000\text{ psi})(8{,}748\text{ in.}^4)(0.137\text{ in.})}{\left[(12\text{ ft})(12\text{ in./ft})\right]^2(1000\text{ lbs/kip})(12\text{ in./ft})} = 39.3\text{ k-ft}$$

Determine V_{UD}

Shear for columns at first story columns due to displacement (at second floor):

$$V_u = \frac{12EI\delta}{L^3}$$ Fixed top and bottom per ACI 318-11 Section 8.10.3

$E_{CE} = 3{,}490{,}000\text{ psi}$

$I = 18\text{ in.}(18\text{ in.})^3/12 = 8{,}748\text{ in.}^4$

Effective shear rigidity per ASCE 41-13 Table 10-5 with gravity load less than $0.5A_g f'_c$ and greater than $0.1A_g f'_c$

$EI = 0.4E_c I_g = (0.4)(3{,}490{,}000\text{ psi})(8{,}748\text{ in.})$

$L = 12\text{ ft}$

Transverse direction:

$$V_u = \frac{(12)(0.4)(3{,}490{,}000\text{ psi})(8{,}748\text{ in.}^4)(0.237\text{ in.})}{\left[(12\text{ ft})(12\text{ in./ft})\right]^3(1000\text{ lbs/kip})} = 11.8\text{ kips}$$

Longitudinal direction:

$$V_{u,y} = \frac{(12)(0.4)(3{,}490{,}000\text{ psi})(8{,}748\text{ in.}^4)(0.137\text{ in.})}{\left[(12\text{ ft})(12\text{ in./ft})\right]^3(1000\text{ lbs/kip})} = 6.7\text{ kips}$$

10.9.4 Shear- or Flexure-Controlled Existing Columns

Next, whether the gravity/secondary columns are shear-controlled or flexurally-controlled is determined. The secondary members will initially be considered deformation-controlled, so expected strengths are used. Columns above the first level are checked first. This level has the maximum drift and the largest columns. For column properties, see Table 10-2 of this *Guide*.

Determine M_{CE}:

P_G from previous calculations;

$P_{G\max}$ = 228 kips

$P_{G\min}$ = 114 kips

Run *spColumn*® with f_c' = 3,750 psi, f_y = 50,000 psi, and ϕ = 1.0. Based on moment-axial interaction diagram from *spColumn*®:

M_{CE} = ϕM_n = 251 kip-ft = Q_{CE} (for $P_{G\max}$ = 228 kips)

M_{CE} = ϕM_n = 218 kip-ft = Q_{CE} (for $P_{G\min}$ = 114 kips) (governs)

Determine V_{CE}:

Per ASCE 41-13 § 10.4.2.3, Equation 10-3

k = 1.0 (Ductility demand is less than 2)

M = 68 kip-ft (see previous calculations)

V = 11.8 kips (see previous calculations)

d = 0.8(18 in.)

M/Vd = 68 kip-ft/(11.8)(0.8)(18 in.)(12) = 4.8 (need not be taken greater than 4)

N_U = 114 kips (see previous calculations)

λ = 1.0 (Normal weight concrete)

$$V_n = kV_0 = k\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_U}{6\sqrt{f'_c}A_g}}\right)0.8A_g\right]$$

$$V_{CE} = V_n = 1.0\left[\frac{(0.40\text{ in.}^2)(50{,}000\text{ psi})(18\text{ in.})(0.8)}{10\text{ in.}} + 1\left(\frac{6\sqrt{3750}\text{ psi}}{4}\sqrt{1+\frac{114{,}000\text{ lbs}}{6\sqrt{3750}\text{ psi}(18\text{ in.})^2}}\right)(0.8)(18\text{ in.})^2\right]$$

= 62.1 kips = Q_{CE}

V_{CE} > $M_{CE}(2)/L_C$

62.1 kips > [218 kip-ft (2)] /12 ft = 36.3 kips for minimum gravity load

69.4 kips > [251 kip-ft (2)] /12 ft= 41.8 kips for maximum gravity load

Therefore, secondary columns at first floor are flexure-controlled.

10.9.5 Column Deformation-Compatibility Moment

Since flexure controls, only flexural compatibility needs to be checked.

Find *m*-factor for moment compatibility:

Per ASCE 41-13 § 10.4, use Table 10-9 (reinforced concrete columns):

Component Type: Secondary

Building Performance: Collapse Prevention

> **Commentary**
>
> Per ASCE 41-13 § 10.4.2.2.2 there are three potential failure mechanisms. Condition 2 or *ii* is flexure failure where $s/d \geq 0.5$.

Condition (*ii*), flexure-shear failure, where yielding in flexure is expected before shear failure and s/d is greater than 0.5:

P = 114 kips (see previous calculations for minimum gravity load)

$P / A_g f_c$ = (114 kips)/[(18 in.)(18 in.)(3.75 ksi)]
= 0.9 < 0.1 (For minimum gravity load)
= (228 kips) / (18 in.)(18 in.)(3.75 ksi)
= 0.19 > 0.1 but less than 0.6

ρ = $A_v/b_w s$
= 0.40 in.^{2} / (18 in.)(10 in.)
= 0.0022 ≤ 0.006

$$\frac{V}{b_w d\sqrt{f'_c}} = \frac{11{,}800 \text{ lbs}}{(18 \text{ in.})(18 \text{ in.})(0.8)\sqrt{3{,}750} \text{ psi}} = 0.74 \leq 3$$

Therefore,

m = 5.00 (for minimum gravity load)

m = 4.46 (interpolated, for maximum gravity load)

$m\kappa Q_{CE} > Q_{UD}$

(5.00)(0.9)(218 kip-ft) = 981 kip-ft > 68 kip-ft (for minimum gravity load)

(4.46)(0.9)(251 kip-ft) = 875 kip-ft > 68 kip-ft (for maximum gravity load)

DCR = 0.08 Columns are acceptable in flexure

Chapter 11

Concrete Shear Wall (C2) with Nonlinear Static Procedure

11.1 Overview

This chapter provides discussion and example application of the Tier 3 systematic evaluation and retrofit procedures of ASCE 41-13 (ASCE, 2014) on the same 1950s three-story concrete shear wall building studied in Chapter 10 of this *Example Application Guide* using the linear static procedure (LSP). The example in this chapter applies the nonlinear static procedure (NSP) to the building, as NSP is another analytical approach of the ASCE 41-13 Tier 3 systematic evaluation procedure. The following information regarding the building is provided in Chapter 10 and will not be repeated in this chapter: building description (Section 10.2.1), Tier 1 screening and mitigation strategy (Section 10.2.2), seismic design parameters and Performance Objective (Section 10.2.3), and data collection requirements (Section 10.3).

Example Summary

Building Type: C2

Performance Objective: BPOE

Risk Category: II

Location: Seattle, Washington

Level of Seismicity: High

Analysis Procedure: Nonlinear Static (NSP)

Evaluation Procedure: Tier 3

Reference Documents:

ACI 318-14

FEMA 440

This example demonstrates three-dimensional nonlinear modeling of reinforced concrete shear wall structures, determination of the target displacement for the NSP, performance evaluation of reinforced concrete shear walls and columns with the NSP, and three-dimensional explicit modeling of foundation components, including kinematic interaction and radiation damping soil-structure interaction effects. Additionally, the seismic performance of shear wall elements is compared between the LSP and NSP, as well as NSP results obtained from fixed-base and flexible-base models. The sections of this chapter are organized as follows:

- **Section 11.2:** This section highlights the following aspects of a three-dimensional analytical model for the reinforced concrete shear wall building:
  - Fiber section for reinforced concrete shear walls
  - Nonlinear constitutive stress-strain relations for concrete and reinforcing steel materials
  - In-plane shear stress-strain relations for reinforced concrete shear walls
  - Nonlinear plastic hinge model for reinforced concrete columns

    - Elastic models for beams and diaphragms with effective stiffness values

- **Section 11.3:** This section introduces the following critical steps for using the Nonlinear Static (Pushover) Analysis in ASCE 41-13:
    - Perform preliminary analysis for idealized force-displacement curve (ASCE 41-13 § 7.4.3.2.4)
    - Confirm applicability of NSP (ASCE 41-13 § 7.3.2.1)
    - Determine preliminary target displacement (ASCE 41-13 § 7.4.3.3)
    - Evaluate actual and accidental torsional effects (ASCE 41-13 § 7.2.3.2)
    - Determination of final target displacement (ASCE 41-13 §7.4.3.3.2)
- **Section 11.4:** This section discusses performance evaluation of reinforced concrete shear walls:
    - Deformation-controlled and force-controlled actions for reinforced concrete shear walls (ASCE 41-13 § 10.7.2.3 and § 7.5.1.2)
    - Acceptance criteria for shear and flexural responses (ASCE 41-13 § 10.7.2.4.2)
- **Section 11.5:** This section discusses performance evaluation of reinforced concrete columns (ASCE 41-13 § 10.4.2)
    - Axial response
    - Shear response
    - Flexural response
- **Section 11.6:** This section discusses three-dimensional explicit modeling of foundation components (ASCE 41-13 § 8.4.2)
    - Modeling foundation flexibility
    - Modeling foundation capacity
- **Section 11.7:** This section discusses kinematic interaction and radiation damping soil-structure interaction effects (ASCE 41-13 § 8.5)
    - Base slab averaging
    - Embedment
    - Target displacements considering kinematic interaction effects
    - Foundation damping soil-structure interaction effects

- Shear wall performance evaluated using flexible-base building model considering soil-structure interaction effects

Retrofitting was found to be needed in Chapter 10, and the retrofit scheme of adding walls at Gridlines A and G identified in Chapter 10 is assumed. Figure 11-1 through Figure 11-4 illustrate the configuration and dimensions of the building and include the new walls on Gridlines A and G.

11.2 Three-Dimensional Nonlinear Modeling Approach

A three-dimensional analytical building model is created for performing the NSP. The model incorporates both material and geometric nonlinearities in terms of constitutive behavior of structural components and *P-Δ* effects. Modeling of nonlinear foundation components is briefly introduced in Section 11.6 of this *Guide*. A critical step of the NSP is to confirm the applicability of the NSP in accordance with ASCE 41-13 § 7.3.2.1 and § 7.4.3.3. This step requires creating a full model of the structure, performing comparative preliminary nonlinear static analyses by pushing the structural model beyond its peak capacity, as well as comparative response spectrum analyses. Comparisons need to be made between nonlinear static analyses considering and not considering *P-Δ* effects. In addition, the comparative response spectrum analyses need to use both multiple modes to produce 90% mass participation and only the fundamental mode.

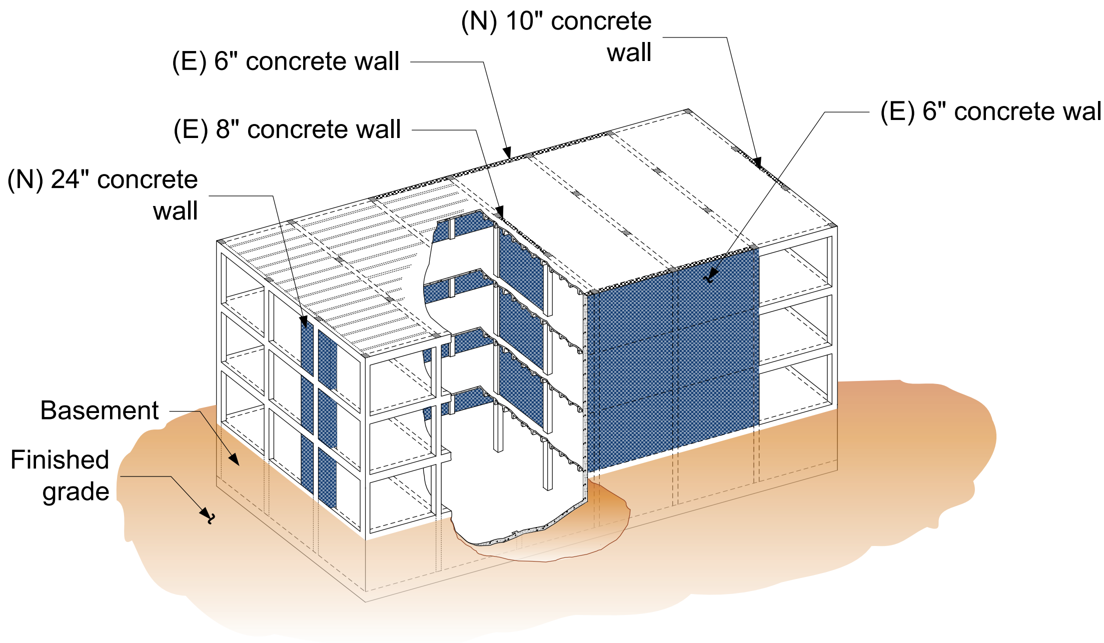


Figure 11-1 Three-dimensional image of the building including the new walls added in the proposed retrofit.

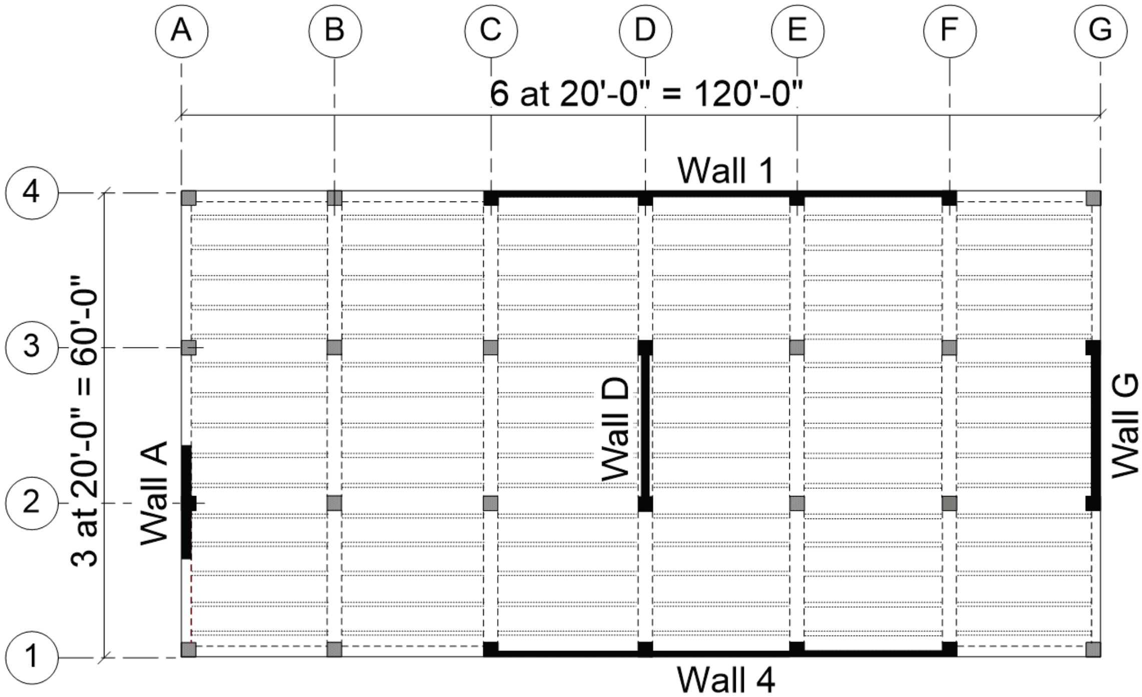


Figure 11-2 Floor plan, including walls on Gridline A and Gridline G for the retrofit.

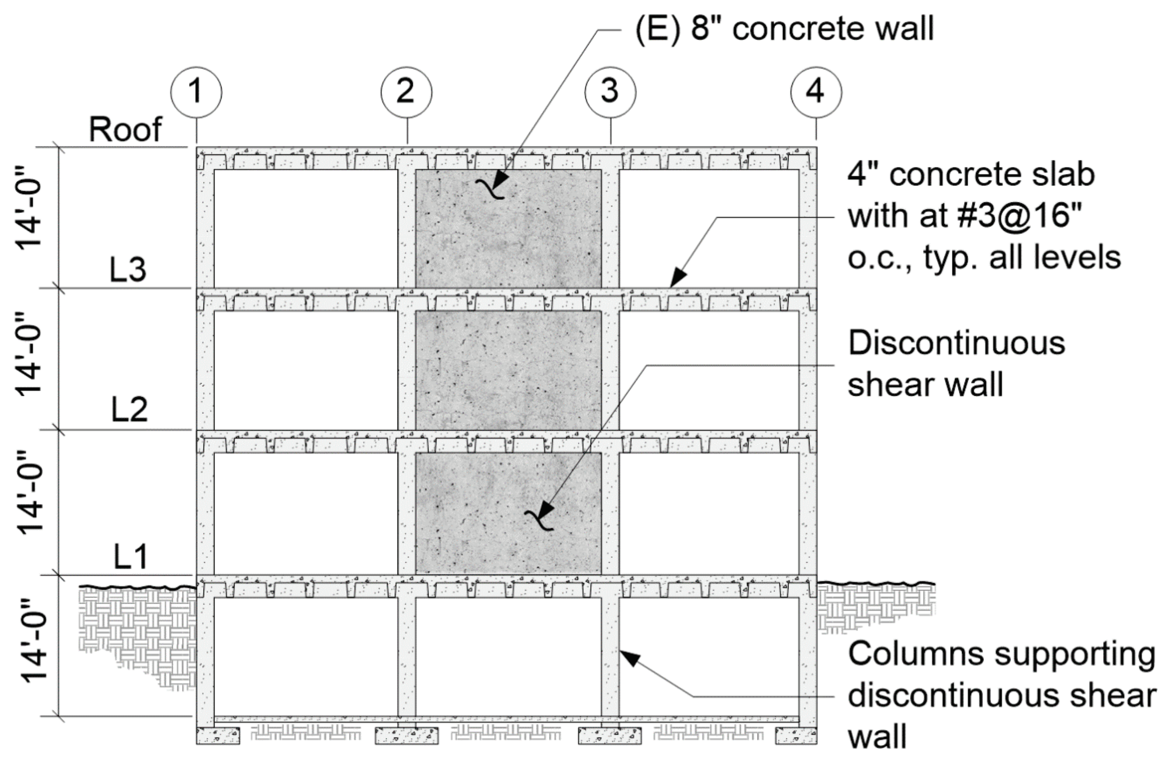


Figure 11-3 Building section/Wall D elevation.

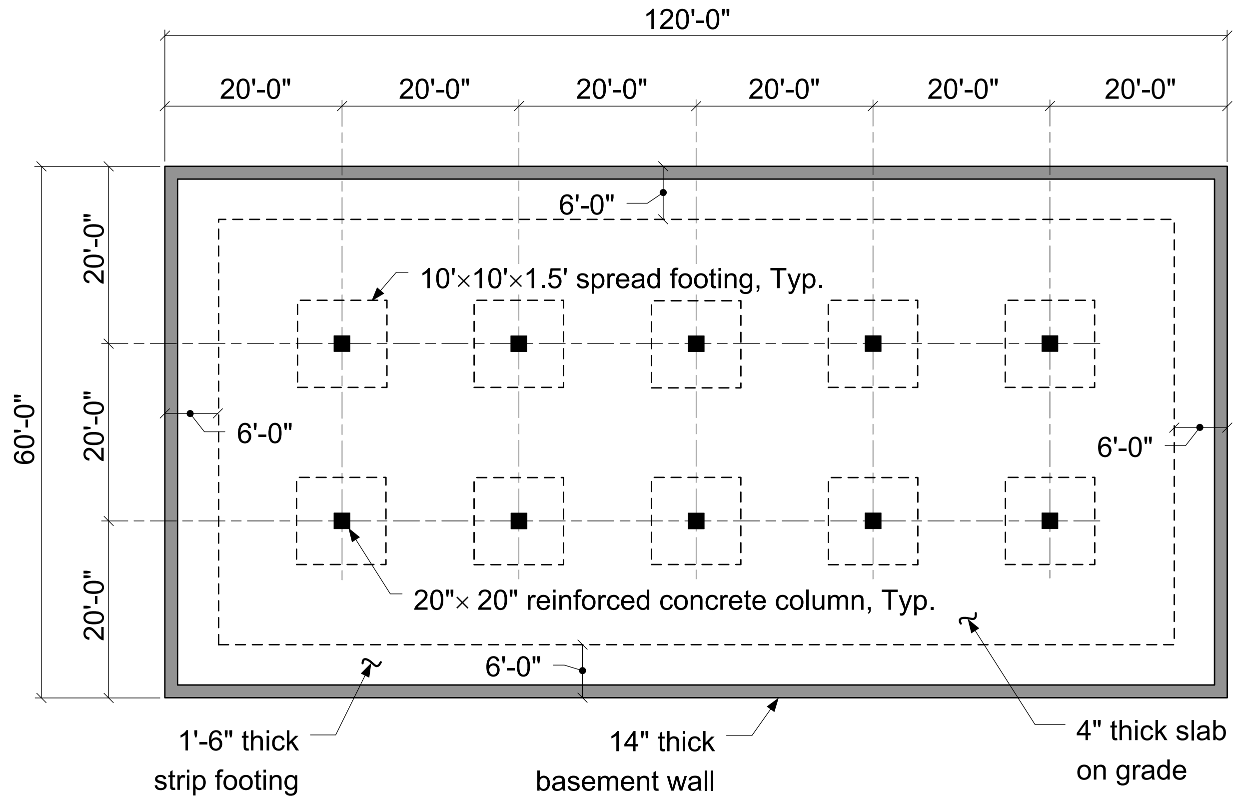


Figure 11-4 Basement plan.

11.2.1 Modeling Approach for Structural Components

- **Modeling and Analysis Software:** The nonlinear modeling program PERFORM-3D® V5.0.1 (Computers and Structures Inc., 2013) was used for this example.
- **Scope of Model:** ASCE 41-13 § 7.5.1.1 distinguishes between primary and secondary components. For this example, the primary components are the reinforced concrete shear walls and the secondary components are the columns and beams. Both primary and secondary components are included in the model. Both fixed base and flexible base with foundation soil spring boundary conditions are considered for the model in separate cases. Figure 11-5 and Figure 11-6 show the analytical building model with a fixed base created using the modeling software. The nodes at the wall and column bottoms are pinned (the three translational degrees of freedom are restrained while the three rotational degrees of freedom are released) as it is assumed that the spread and strip footings supporting the columns and walls do not possess significant moment-resisting capacity.

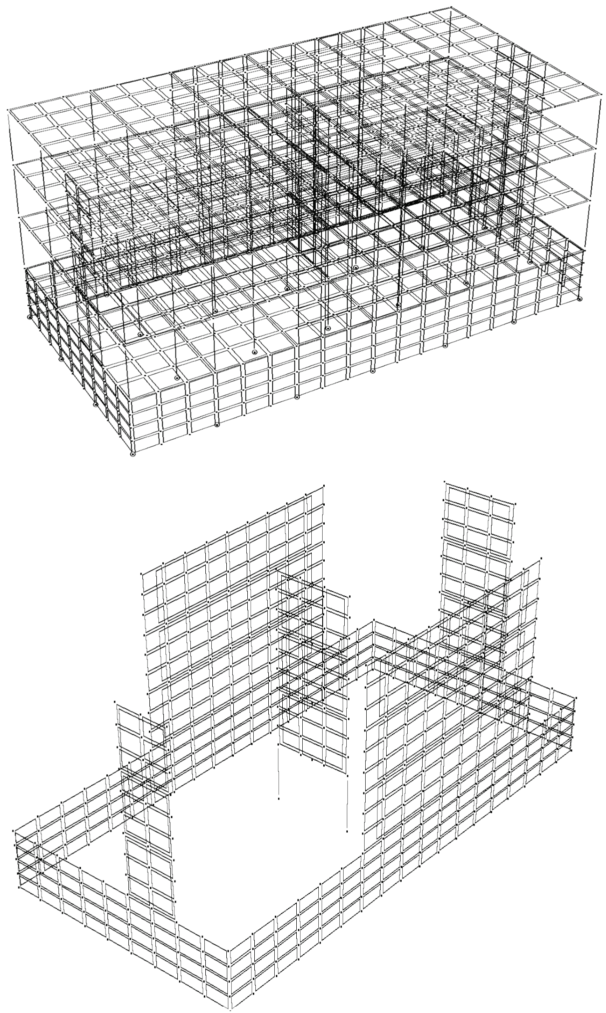
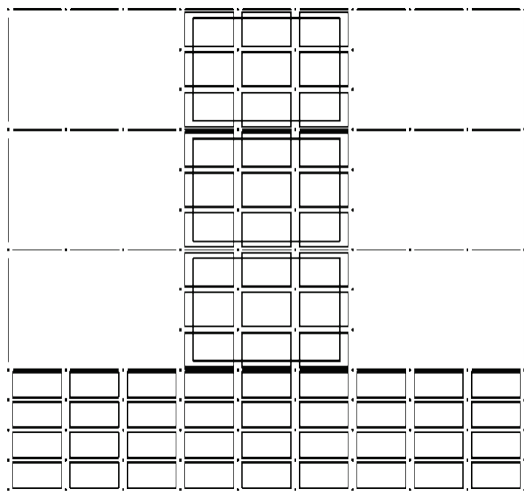
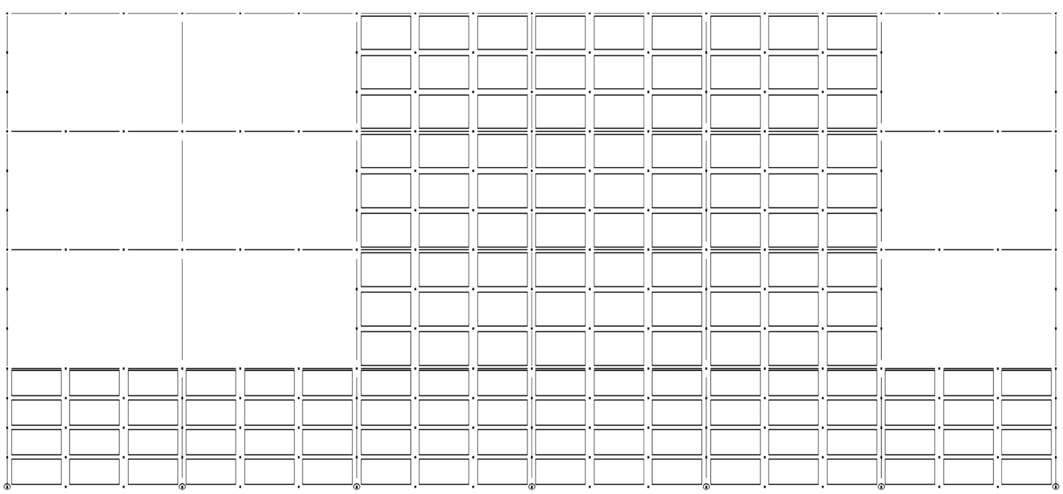


Figure 11-5 Three-dimensional views of analytical building model created using PERFORM-3D® (entire building and structural walls).

- **Material Properties:** Expected material properties are used for all the components made of concrete and reinforcing steel in the analytical model. Confined concrete properties derived from expected unconfined concrete properties and reinforcement layouts are used in modeling the boundary elements of the new walls on Gridlines A and G (identified as Walls A and G). For existing concrete, the expected strength, f_c' , is 1.5 times of the nominal strength, f_c' . For new concrete, the factor of 1.3 is a little lower than the factor of 1.5 specified in ASCE 41-13 Table 10-1 based on engineering judgment and common industry practice that an increase of 1.5 times is unlikely. For confined concrete in the boundary elements of the new shear walls, the peak confined concrete strength is derived from f_{ce}', and it is higher than f_{ce}' due to the confinement effects. Section 11.2.2.1 of this *Guide* further describes the nonlinear constitutive stress-strain relations of concrete used in the model. When evaluating component actions per ASCE 41-13 § 7.5.1.2 and § 7.5.1.3, lower-bound material strengths are used to determine capacities of force-controlled actions, and expected material strengths are used for deformation-controlled actions. Per ASCE 41-13 § 6.2.4.3, nonlinear analyses require either usual or comprehensive levels of knowledge per ASCE 41-13 Table 6-1, so it is assumed that those requirements are met. Therefore, the knowledge factor, κ, is taken as 1.0.



(a) East-west view (Gridline G)



(b) North-south view (Gridline 1)

Figure 11-6 Elevation views of analytical building model.

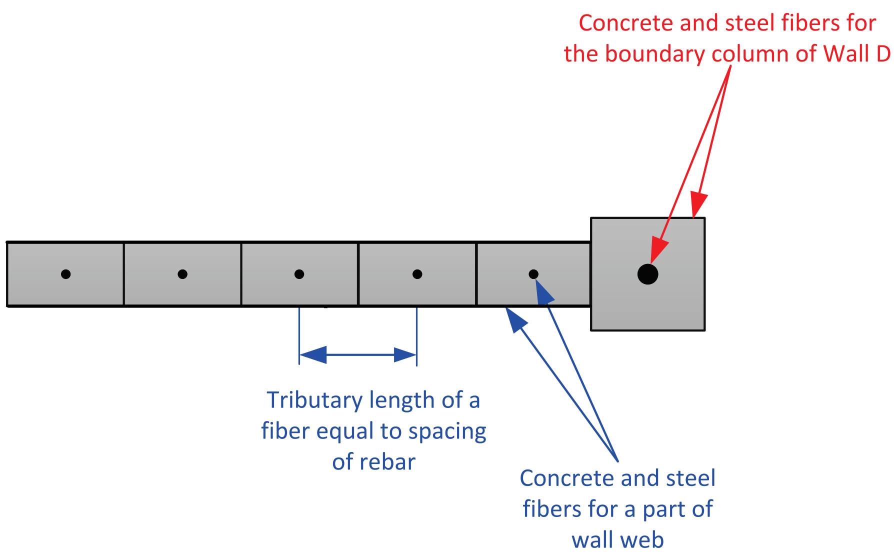
- **Modeling of Reinforced Concrete Shear Walls:**
  - Fiber-discretized wall sections are employed to model nonlinear in-plane flexural and axial response of shear walls. Nonlinear constitutive stress-strain relations of unconfined and confined concrete as well as reinforcing steel materials are assigned to the

Commentary

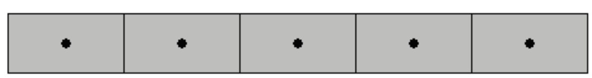
Although a fiber wall model is more complicated in the modeling process than a lumped plastic hinge model, the fiber wall model has a number of advantages over the lumped plastic hinge model for flexural response of shear walls:

- The fiber model can capture the instantaneous P-M interaction at each analysis step, while the flexure-only plastic hinge model cannot.
- The layout of materials on the wall section is directly modeled using fibers with different material constitutive relations, while the plastic hinge model uses one moment-rotation relation to approximate the global flexural behavior of the plastic hinge region.
- The fiber model can capture the compressive or tensile response at each fiber location on the wall section, while the plastic hinge model only captures the global wall rotation.

corresponding material fibers on the section. As an example, Figure 11-7 illustrates the section model for elements of Wall D. Depending on the location of the wall elements (at the center or at the edge), different sections are assigned to the elements. The other shear walls and basement retaining walls are modeled using the same approach. As the flexural behavior of the shear walls is modeled using the fiber model, the modeling parameters for the lumped plastic hinge model, *a*, *b* and *c* in ASCE 41-13 Table 10-19, are not used in the modeling process. When the analysis results are assessed, the plastic rotation of the walls is compared with the acceptable plastic hinge rotation listed in ASCE 41-13 Table 10-19 to determine the acceptance of wall flexural response.



a. Edge element of Wall D



b. Central element of Wall D

Figure 11-7 Fiber-discretized sections for wall elements of Wall D.

- In-plane shear behavior of shear walls is modeled using nonlinear shear stress-strain curves determined in accordance with ASCE 41-13 Table 10-20.
- Out-of-plane flexural and shear behavior of shear walls are modeled elastically. The out-of-plane effective moment of inertia is set to be 50% of the gross sectional moment of inertia. This effective moment of inertia is within the range of 25% to 50% of gross moment of inertia specified by ACI 381-14 Table 6.6.3.1.1(b) (ACI, 2014) for

flat slabs. The flexural rigidity specified in ASCE 41-13 Table 10-5 is only for in-plane effective stiffness.

- The basement retaining walls are assumed to have a thickness of 14 inches and two curtains of #9 vertical reinforcing steel bars with a 12-inch spacing on center. Although not needed for the LSP evaluation in Chapter 10, the concrete and reinforcement of the retaining wall are assumed to have nominal material properties of f'_c = 2,500 psi and f_y = 40 ksi, in order to be consistent with the material properties of other existing walls. The expected material properties derived from these nominal properties are used in the retaining wall model. The retaining walls are modeled using the same approach as the shear walls.

- **Modeling of Reinforced Concrete Columns:** Following the modeling approach of ASCE 41-13 § 10.4.2, nonlinear rotational hinges are used to model the flexural behavior at column ends. The backbone moment-rotation curves for such hinges are determined in accordance with ASCE 41-13 Table 10-8 and implemented using the inelastic "FEMA Column, Concrete Type" model in PERFORM-3D®. P-M_2-M_3 interaction is considered using a yield surface whose shape is defined using shape parameters suggested by the software. As the focus of this example is the shear walls, the secondary frames consisting of beams, columns, and beam-column joints are modeled with less sophistication than the shear walls. The shear walls were modeled with fiber sections and distributed plasticity along the height and width (each wall element can experience inelastic response at any fiber location). The columns, by contrast, are modeled with lumped plasticity at the ends and the beams are modeled elastically. The beam-column joints are not modeled using a nonlinear joint model, but are assumed to be rigid connections.

- **Modeling of Reinforced Concrete Beams:** The reinforced concrete beams are modeled using elastic beam elements. The effective stiffness values of the elements are determined in accordance with ASCE 41-13 Table 10-5. The effective flexural and shear stiffness values used in the model are $0.3E_cI_g$ and $0.4E_cA_w$, respectively. Because the floor slab is explicitly modeled using elastic shell elements, the beams are not regarded as T-beams and only the web is used to calculate the effective stiffness of the beam elements, in order not to double count the contribution of the slab. As will be described later in this section, the effective flexural stiffness of the slab is $1/3E_cI_g$, where I_g is the gross moment of inertia of the floor slab. As shown in Figure 11-8, the flange slab elements on the one or two sides of the beam elements share the same nodes with the beam elements. Therefore, these elements are coupled in the model, which is close to the flanged construction in reality. The flange action of the beams is considered by the floor model consisting of both the beam and flange slab elements, and both types of elements are assigned with effective flexural stiffness to capture reduced stiffness caused by cracked concrete.

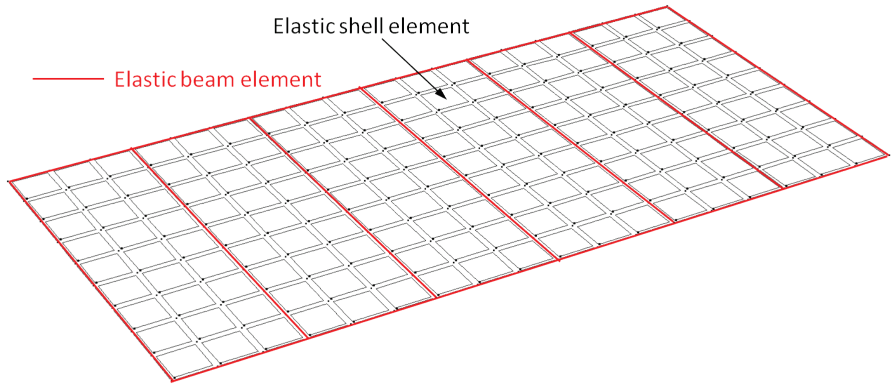


Figure 11-8 An explicitly modeled floor diaphragm with elastic shell and beam elements (each bay is divided into 9 shell elements).

- **Modeling of Diaphragms:** Roof and floor diaphragms are explicitly modeled using elastic shell elements. The effective stiffness values of the shell elements are determined in accordance with ASCE 41-13 Table 10-5. In accordance with ASCE 41-13 § 10.4.4.2, the effective flexural stiffness of the slab used in the model is $1/3E_cI_g$. Although the overall procedure is NSP, the analytical structural model is a combination of linear and nonlinear materials and elements. Thus, to account for the cracked property and other nonlinear response of the linear diaphragm model, a reduced moment of inertia of $1/3E_cI_g$ is used. The axial stiffness of the slab used in the model is E_cA_g. Neither ASCE 41-13 Table 10-5 nor §10.4.4.2 specify an effective axial stiffness of reinforced-concrete diaphragm. In this situation, the axial rigidity of “Walls-cracked” listed in ASCE 41-13 Table 10-5 may be used as an approximation. For “Walls-cracked” under compression, the axial rigidity is E_cA_g while the axial rigidity is E_sA_s for “Walls-cracked” under tension. As the diaphragms are modeled using elastic shell elements that have the same axial rigidity for both compression and tension, the axial rigidity, E_cA_g, is used. Ideally, nonlinear shell elements with different membrane stiffness for compression and tension should be used to capture the axial rigidity of cracked diaphragms. However, due to the unavailability of such nonlinear shell elements in the analysis software, elastic shell elements are used. Figure 11-8 shows the model of a diaphragm with beams.

- **Modeling of Foundation Components:**
    - Fixed-base option: All nodes at the bottom of retaining walls and columns are pinned (the three translational degrees of freedom are fixed while the three rotational degrees of freedoms are released).
    - Flexible-base option: Method 3 outlined in ASCE 41-13 § 8.4.2.5 is employed to explicitly model the spread and strip footings as well as soil springs beneath them. See Chapter 5 of this *Guide* for more details on the different foundation analysis methods in ASCE 41-13. More details regarding the flexible foundation model for this example can be found in Section 11.6 in this *Guide*.
- **Damping:** Per ASCE 41-13 § 7.2.3.6, 5% damped response spectra are used for the NSP with the fixed-base model, which is also used for the LSP. For the flexible-base model, the effective damping ratio of the structure-foundation system is used to modify the acceleration response spectrum in accordance with ASCE 41-13 § 8.5.2. More details regarding the effective damping ratio of the structure-foundation system can be found in Section 11.6 in the *Guide*.
- **Concurrent Seismic Effects:** ASCE 41-13 § 7.2.5.1.2 indicates that when the NSP procedure is used, "it shall be permitted to determine the forces and deformations associated with 100% of the displacements in any single direction that generates the maximum deformation and component action demands. Further concurrent seismic effects need not be considered in the critical direction(s) …." There are no shared columns or intersecting shear walls in this building example. Accordingly, uni-directional NSP analysis is used to evaluate the shear walls without considering concurrent seismic effects.

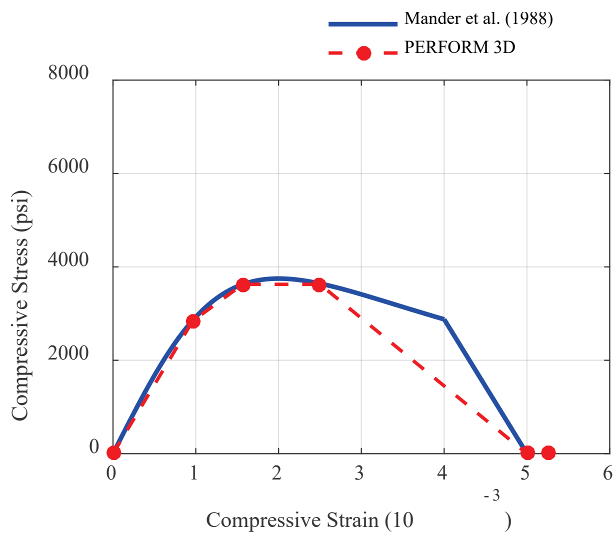
11.2.2 Additional Information Required for NSP

Detailed information on structural properties was provided in Chapter 10 of this *Guide*. Additional information is needed for some aspects of NSP modeling and is introduced in this section.

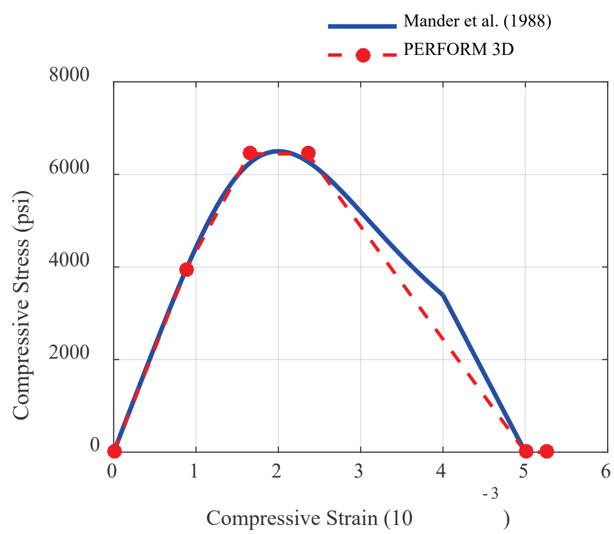
11.2.2.1 Nonlinear Constitutive Stress-Strain Relations of Concrete and Reinforcing Steel

One-dimensional constitutive stress-strain relations for confined and unconfined concrete as well as reinforcing steel are needed for creating fiber-based shear wall elements. Figure 11-9 illustrates the stress-strain relations of the unconfined existing and new concrete materials. The theoretical stress-strain curves are computed based on the widely used formulas for confined and unconfined concrete materials proposed by Mander et al.

(1988) and then are approximated by the 1-D concrete material available in the software. Unconfined existing concrete is used for the fiber section of Walls 1 and 4 and basement retaining walls, while unconfined new concrete is used for the fibers at the web of Walls A and G outside the boundary elements.



(a) Unconfined existing concrete material (f'_{ce} = 3,750 psi)



(b) Unconfined new concrete material (f'_{ce} = 6,500 psi)

Figure 11-9 Constitutive stress-strain model for unconfined existing and new concrete materials.

Figure 11-10 and Figure 11-11 illustrate the boundary elements of Walls A and G and their corresponding fiber models. In these two figures, the areas of confined and unconfined concrete fibers are illustrated by the deep and light grey rectangles, respectively. The area of the confined concrete fiber is calculated based on the region enclosed by the confining transverse reinforcement, while the area of the unconfined concrete fibers is calculated based on the out-to-out width of the wall.

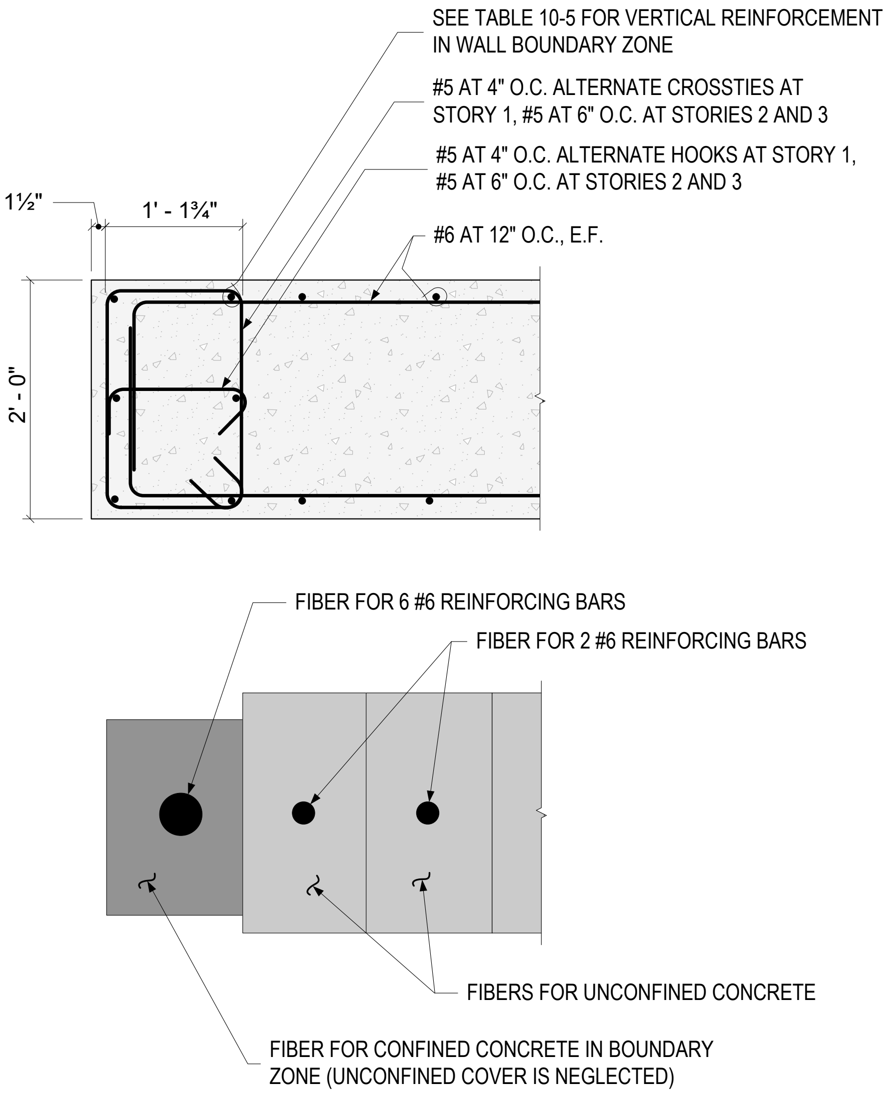


Figure 11-10 Boundary element of Wall A and its corresponding fiber model.

> **Commentary**
>
> Detailing of the new boundary elements of Walls A and G is conducted in accordance with ACI 318-14 Section 18.10.6. Although the seismic analysis is performed in accordance with ASCE 41-13, the retrofitting should comply with current building codes.

Figure 11-12 shows the stress-strain curves of the confined existing and new concrete materials in the boundary elements of Walls A and G. For the confined concrete in the wall boundary, the ultimate compressive strain capacity at the end of the blue curve corresponds to the first hoop fracture and is determined based on an energy balance approach (Mander et al., 1988). The residual strength of the confined concrete after the first hoop fracture is neglected, so the stress of the approximated curve drops to zero after the ultimate strain capacity is reached. The formulas proposed by Mander et al. (1988) are implemented in SAP2000 v18.2 (Computers and Structures, Inc., 2016) and are used to determine the blue curves shown in

Figure 11-9 and Figure 11-12. Subsequently, the blue curve is fitted by the red curve that is implemented in PERFORM-3D® for 1-D concrete material. The round red dots shown in Figure 11-9 and Figure 11-12 represent the parameters that need to be determined by the engineer for defining the red curve in PERFORM-3D®.

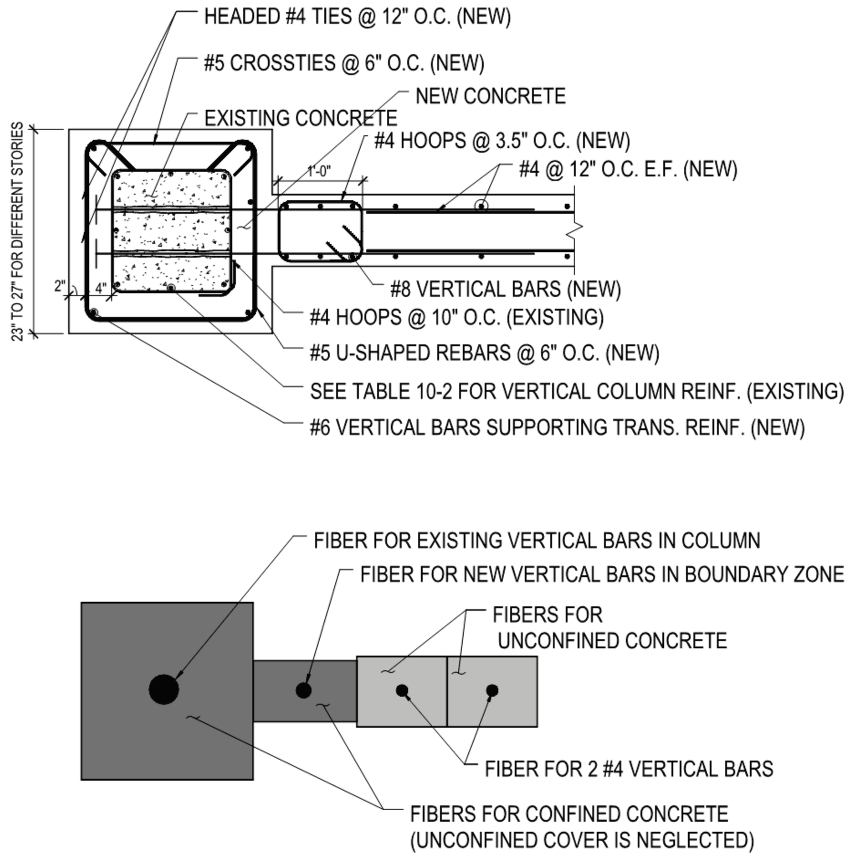
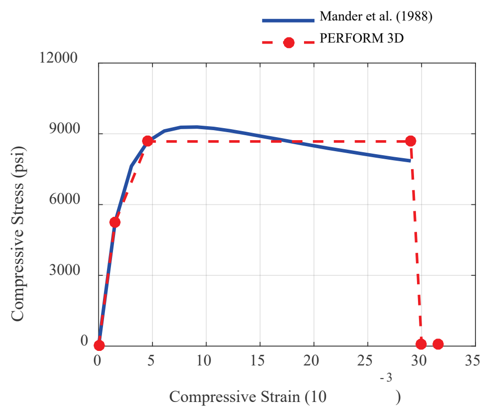


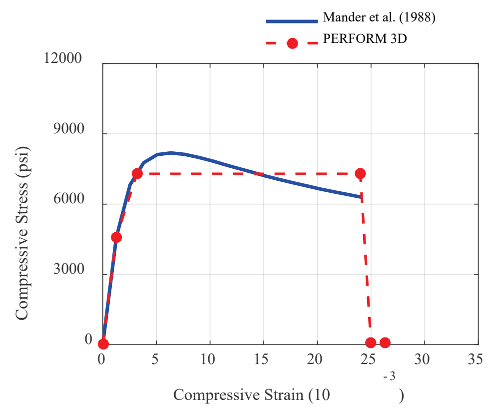
Figure 11-11 Boundary element of Wall G and its corresponding fiber model.

Figure 11-13 illustrates the stress-strain relations for the existing and new reinforcing steel materials. The red dashed curves are approximations of the theoretical curves in blue. Due to the transverse confinement effects in the boundary zones of Walls A and G, the strength and ductility of the confined concrete material are significantly improved as compared to the unconfined concrete.

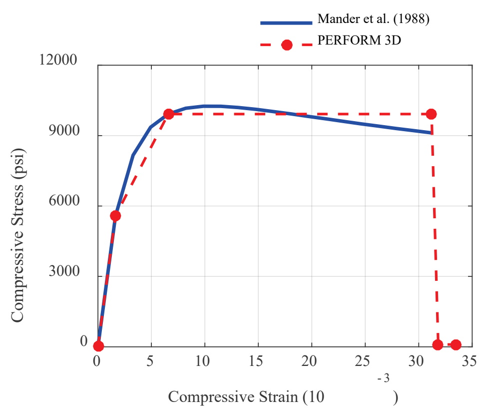
ASCE 41-13 § 10.3.3.1 specifies usable strain limits for concrete and steel reinforcement. Accordingly, the compressive strain limit for unconfined concrete used in the NSP is 0.005. The strain limits for unconfined and confined concrete are shown in Figure 11-9 and Figure 11-12, respectively. The tensile and compressive strain limits for reinforcing steel are 0.05 and 0.02, respectively. As can be seen in Figure 11-13, the material stress drops to almost zero after the strain exceeds the usable strain limit.



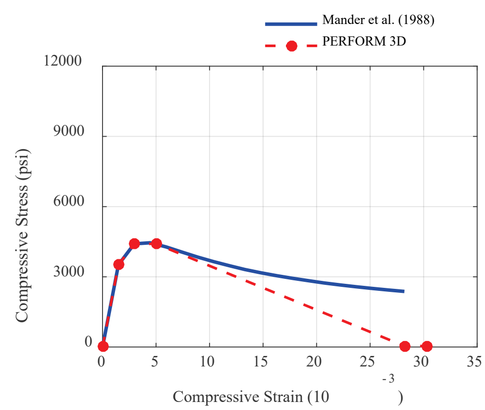
(a) Confined concrete at boundary element of new Wall A at Story 1



(b) Confined concrete at boundary element of new Wall A at Stories 2 and 3

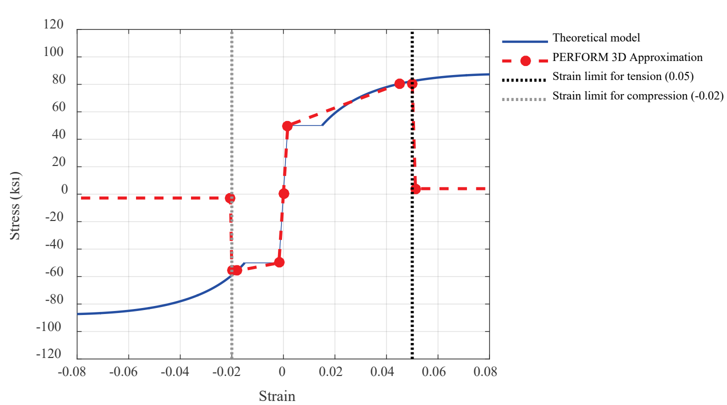


(c) Confined concrete at boundary element of new Wall G at Story 1

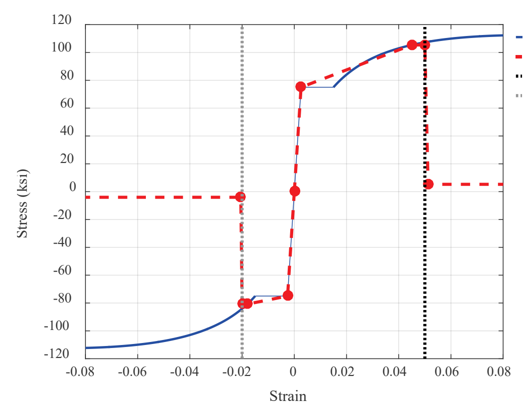


(d) Confined concrete at retrofitted existing boundary columns of Wall G at Story 1

Figure 11-12 Constitutive stress-strain model for confined existing and new concrete materials.



(a) Existing reinforcing steel material (f_{ye} =50 ksi)



(b) New reinforcing steel material (f_{ye} =75 ksi)

Figure 11-13 Constitutive stress-strain models for existing and new reinforcing steel materials.

11.2.2.2 In-Plane Shear Stress-Strain Relations for Reinforced Concrete Shear Walls

In-plane shear stress-strain relations of the shear walls are constructed for modeling the in-plane shear response per ASCE 41-13 § 10.7.2.2 and § 10.7.2.3. The critical modeling parameters listed in ASCE 41-13 Table 10-20 are used in determining the shear stress-strain relations and utilize the tri-linear force-displacement relations from ASCE 41-13 Figure 10-1c. Table 11-1 lists the required parameters for constructing the curves.

Table 11-1 Parameters for In-Plane Shear Stress-Strain Curves of Shear Walls

Wall loc.	Lvl	t_w (in)	l_w (ft)	A_w (in.^{2})	A_s (in.^{2})	A'_s (in.^{2})	f'_{ce} (psi)	f'_{ye} (ksi)	$P_{G,max}$ (kips)	Axial Con.	d (%)	e (%)	g (%)	c (%)	f (%)	V_{CE} (1) (kips)	v_{CE} (2) (ksi)
Grid 1	1	6	60	4,320	2.2	2.2	3,750	50	453	0.03	1.0	2.0	0.4	0.2	0.6	1,010	0.23
Grid 4	1	6	60	4,320	2.2	2.2	3,750	50	453	0.03	1.0	2.0	0.4	0.2	0.6	1,010	0.23
Grid A	1	24	14	4,032	6.2	6.2	6,500	75	205	0.01	1.0	2.0	0.4	0.2	0.6	1,574	0.39
Grid D	1	8	20	1,920	1.5	1.5	3,750	50	219	0.03	1.0	2.0	0.4	0.2	0.6	388	0.20
Grid G	1	10	20	2,400	4.0	4.0	6,500	75	154	0.01	1.0	2.0	0.4	0.2	0.6	987	0.41

(1) Shear capacity determined per ACI 318 using expected material strength properties.

(2) Ultimate shear stress $v_{CE} = V_{CE}/A_w$.

Axial stress ratios are such that all walls use the first row of ASCE 41-13 Table 10-20. The in-plane shear stress-strain relations for the retaining walls are modeled using the same approach. The shear stress-strain curves are shown in Figure 11-14.

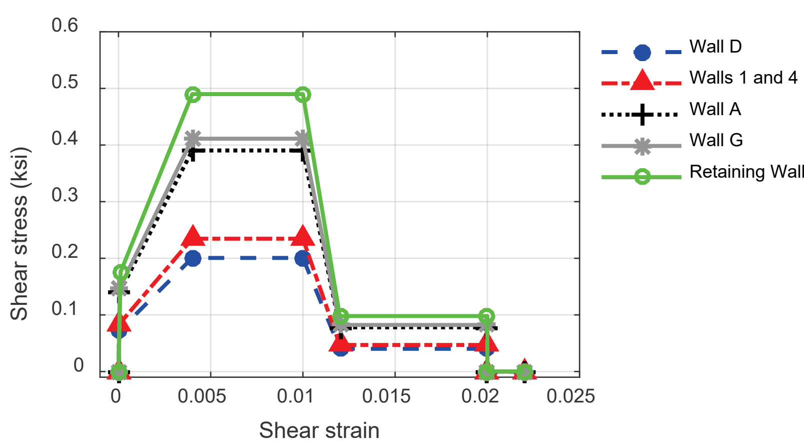


Figure 11-14 In-plane stress-strain curves for shear walls.

Using Wall 1 as an example, the axial compression ratio is calculated as:

$$\frac{\left(A_s - A_s'\right) f_y + P}{t_w l_w f_c'} \qquad \text{(ASCE 41-13 Table10-20)}$$

$$= \frac{\left(4.40\text{ in.}^2 - 4.40\text{ in.}^2\right)\left(50\text{ ksi}\right) + 453\text{ kips}}{\left(6\text{ in.}\right)\left(720\text{ in.}\right)\left(3.750\text{ ksi}\right)}$$

$$= 0.028 < 0.05$$

Therefore, according to ASCE 41-13 Table 10-20, d = 1.0%, e = 2.0%, g = 0.4%, c = 0.2%, and f = 0.6%.

The procedure for calculating V_{CE} is shown in Section 10.4.4.1 and v_{CE} = V_{CE}/A_w = 1,010 kips / 4,320 in.^{2} = 0.23 ksi.

11.2.2.3 Classification of Diaphragms

Rigidity of the diaphragms is classified per ASCE 41-13 § 7.2.9.1. Lateral force patterns corresponding to the pseudo seismic forces shown in Table 10-8 and Table 10-9 of this *Guide* are applied to the diaphragm nodes of the analytical model in the east-west and north-south directions, and nonlinear static pushover analyses are performed to obtain deformation of the diaphragms and shear walls. As the analysis is performed stepwise, the ratio between the maximum lateral deformation of the diaphragm and the average drift of the shear walls in the story immediately below the diaphragm is determined for each step, and the largest ratio of all the steps is compared to 0.5. At each step, the maximum lateral deformation of the diaphragm is taken as the maximum diaphragm drift subtracting the average drift of the shear walls spanning along the analysis direction in the story immediately below the diaphragm. Table 11-2 lists the largest deformation ratios. All of the ratios in the table are smaller than 0.5, so the diaphragms are classified as rigid per the criterion in ASCE 41-13 § 7.2.9.1.

Table 11-2 Ratio of Deformation between Diaphragms and Shear Walls

Level	Δ_d / Δ_w under E-W pseudo seismic forces[1]	Δ_d / Δ_w under N-S pseudo seismic forces[2]
Roof	0.14	0.30
Level 3	0.21	0.22
Level 2	0.10	0.22

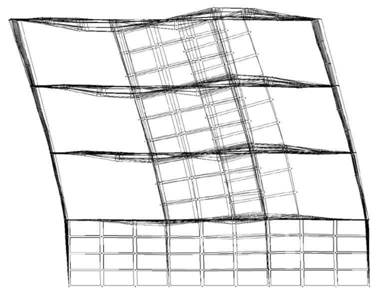
Note: Δ_d is the maximum lateral deformation of the diaphragm. Δ_w is the average story drift of shear walls in the direction of pseudo seismic force.

11.2.2.4 Modal Properties

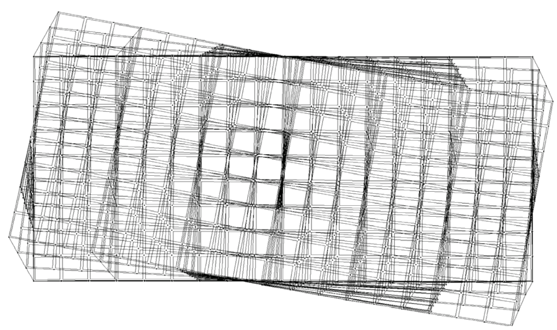
Figure 11-15 shows the first three mode shapes of the building and the corresponding period. Mode 1 is the fundamental mode in the north-south direction, and Mode 3 is the fundamental mode in the east-west direction. Mode 2 is a torsional mode.

The mode shapes and periods shown in Figure 11-15 are initial modal properties for the building in its initial elastic state. The mode shapes and periods are determined using the initial elastic stiffness properties of the nonlinear and linear materials and elements of the model.

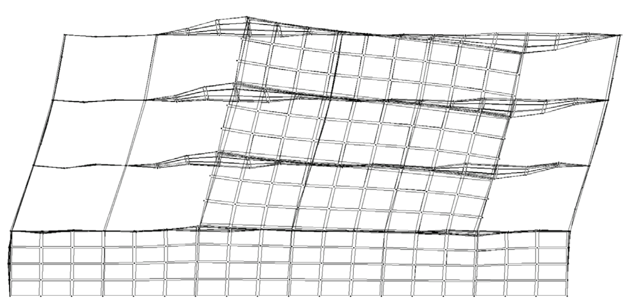
These modal properties are obtained from the fixed-base building model. Section 11.6.2 presents modal properties of a flexible-base building model.



(a) Mode 1 (T_1 =0.245 s, translational vibration in north-south direction)



(b) Mode 2 (T_2 =0.115 s, torsional vibration)



(c) Mode 3 (T_3 =0.113 s, translational vibration in east-west direction)

Figure 11-15 First three mode shapes and periods.

11.3 Nonlinear Static (Pushover) Analysis

The nonlinear static (pushover) analysis is performed on the analytical building model. Before running the pushover analysis, a linear static gravity analysis is performed by applying gravity loads to the analytical model in accordance with ASCE 41-13 Equation 7-3. The floor and roof dead load of 100 psf and 25% of the floor live load of 125 psf listed in Section 10.2.1 of this *Guide* are applied to the nodes of the diaphragms according to the tributary area of each node. The following load combination is used to define the gravity load applied to the structure:

$$Q_G = Q_D + Q_L + Q_S \qquad \text{(ASCE 41-13 Eq. 7-3)}$$

where:

Q_D = Action caused by dead loads

Q_L = Action caused by live load, equal to 25% of the unreduced live load obtained in accordance with ASCE 7 but not less than the actual live load

Q_S = Action caused by effective snow load. In this example, the flat roof snow load is 25 psf, which is less than 30 psf, so the effective snow load is neglected per ASCE 41-13 § 7.2.2

After the gravity analysis is finished, the pushover analysis starts at the deformed structural state with full gravity loads applied. The gravity loads will be kept constant during the pushover analysis. The lateral force pattern for the pushover analysis can be either the fundamental mode shape in the direction under consideration, or a load pattern defined by the user. The former is used for the analyses in the following sections, while a user-defined load pattern consistent with the pseudo seismic forces calculated in Table 10-8 and Table 10-9 of this *Guide* is used only for classification of diaphragms per ASCE 41-13 § 7.2.9.1, which is discussed in Section 11.2.2.3 of this *Guide*.

The node at the center of the roof is set to be the control node. The pushover analysis is displacement-controlled in a monotonic manner, i.e., the displacement of the control node increases monotonically at each analysis step, while the magnitude of the lateral forces applied to each node is adjusted at each step to achieve equilibrium between the external forces and the internal resistance of the deformed structure at this step. When such equilibrium is achieved, the displacement of the control node increases by an increment, and the analysis proceeds to the next step.

The pattern of the lateral forces (ratio between forces applied to different nodes) is an invariant that is not altered during the analysis, while only the magnitude of these forces is scaled up and down by the same factor at each step, in order to accommodate the deformation increment and the softening or hardening of the structure. When the analysis meets a termination condition that is defined in advance (such as target displacement of the control node, maximum deformation of components, or maximum number of iterations within one step), the analysis is terminated. Then, deformation and force of the structural components can be obtained for each step of the analysis and compared to the corresponding acceptance criteria. Figure 11-16 summarizes the critical steps for NSP.

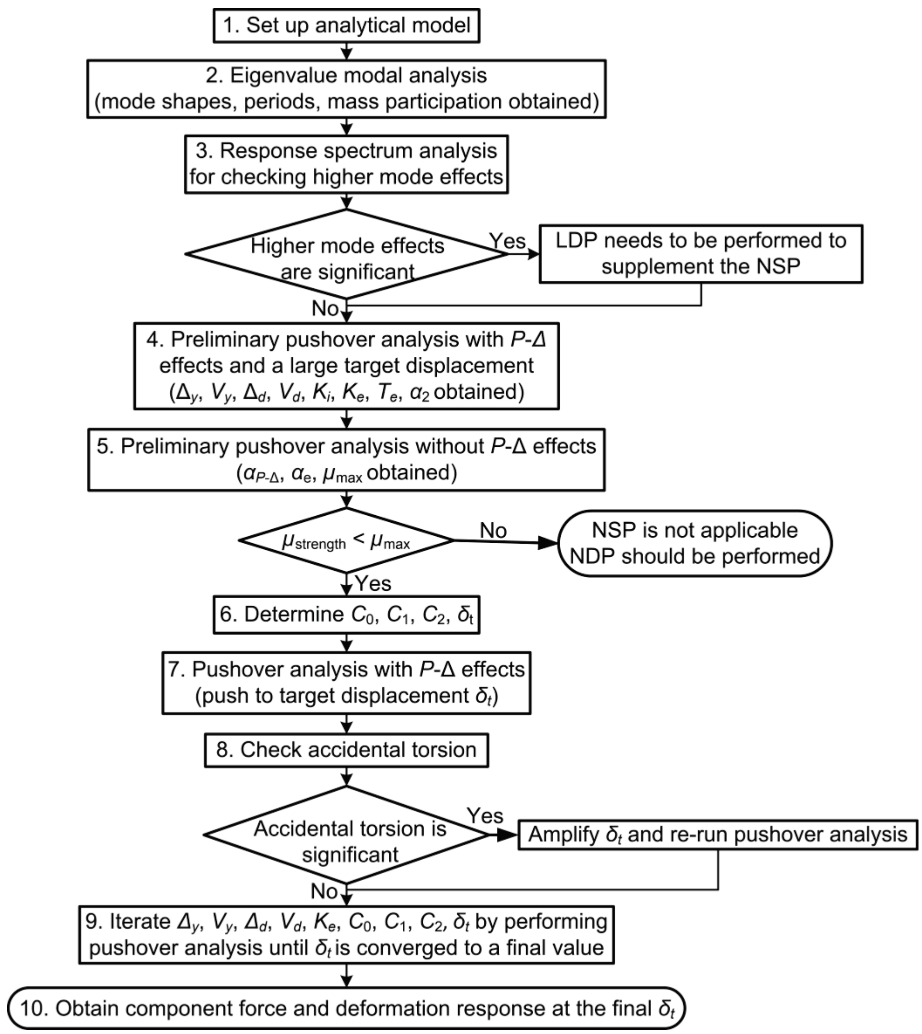


Figure 11-16 Flowchart for critical steps of NSP.

11.3.1 *Preliminary Analysis for Idealized Force-Displacement Curve (ASCE 41-13 § 7.4.3.2.4)*

In order to determine the target displacement for the NSP, a preliminary pushover analysis with a sufficiently large target displacement at the control node is performed in each of the two principal building directions. The base shear versus control-node displacement curve obtained from the analysis needs to incorporate an ascending branch up to the peak base shear force and also a post-peak descending branch, so that the applicability of NSP can be determined according to ASCE 41-13 § 7.3.2.1 and § 7.4.3.3. Figure 11-17 and Figure 11-18 show the base shear versus roof displacement curves obtained from the east-west and north-south preliminary analyses.

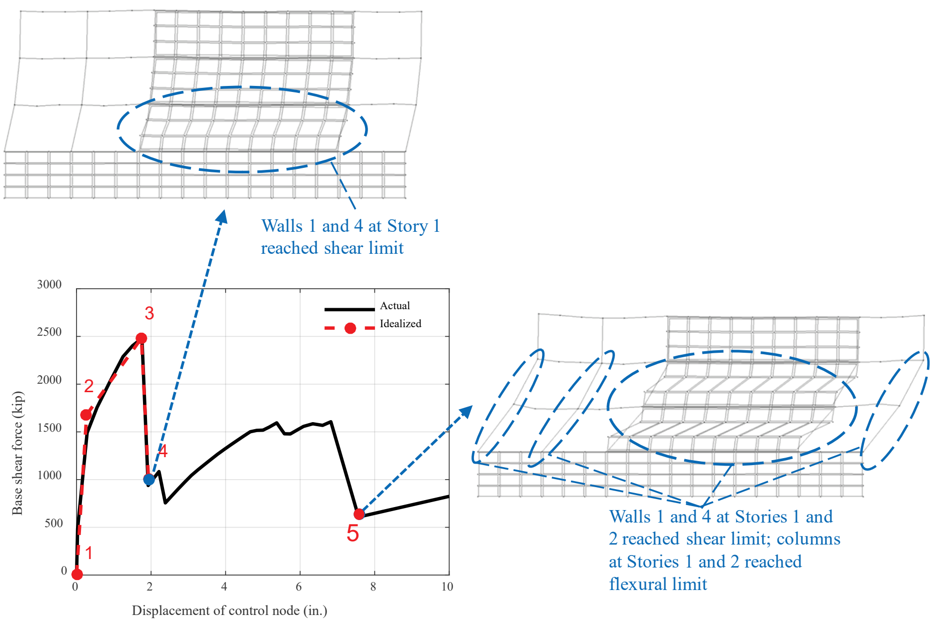


Figure 11-17 Preliminary force-displacement curve for east-west analysis.

In the east-west analysis, Walls 1 and 4 at Story 1 reached peak shear capacity at Point 3 shown in Figure 11-17, and the lateral capacity of the whole structure dropped to about 1,000 kips at Point 4 after the shear failure. After shear failure of Walls 1 and 4 at Story 1, the lateral force was resisted by the out-of-plane bending of Walls A, D, and G and the gravity columns, which resulted in an increasing of the total base shear between Points 4 and 5. Walls 1 and 4 at Stories 2 and 3 did not lose lateral force-resisting capacity at Point 4, and as the roof displacement increases, the two walls continued to resist lateral force until Story 2 of Walls 1 and 4 lost shear capacity at a displacement around 7.5 in. Different from the east-west analysis, the structure in the north-south analysis exhibited a nearly monotonically descending base shear versus control node displacement curve up to a 20-inch control node displacement, as shown in Figure 11-18.

Starting from Point 3, failures of the three walls and the columns occurred one after another with small intervals of control-node displacement, so although there is some small local increase of force after Point 3, the global trend of the static pushover response is strength degradation.

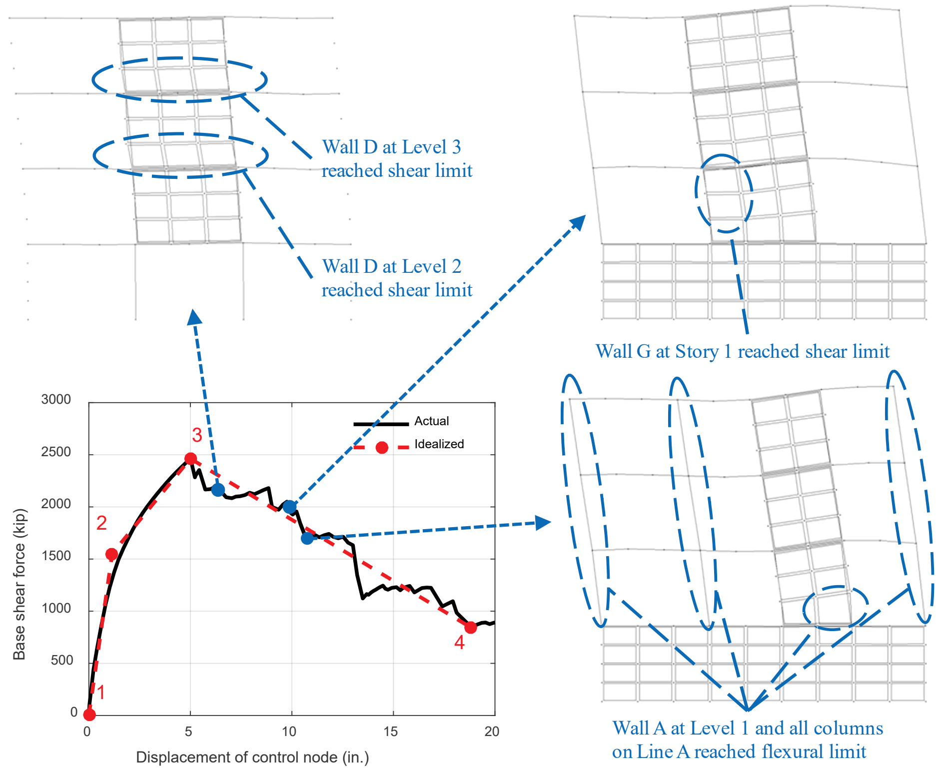


Figure 11-18 Preliminary force-displacement curve for north-south analysis.

The idealized force-displacement curves shown in the figure are constructed per ASCE 41-13 § 7.4.3.2.4. Although not clearly stated in ASCE 41-13 § 7.4.3.2.4, FEMA 440 Section 4.3 (FEMA, 2005) provides more details for determining the bi-linear idealized force-displacement curve. As FEMA 440 Section 4.3 states, "The intersection of the two idealized segments defines effective lateral stiffness (K_e), the effective yield strength (V_y), and effective positive post-yield stiffness ($\alpha_1 K_e$). The intersection point is determined by satisfying two constraints. First, the effective stiffness, K_e, must be such that the first segment passes through the calculated curve at a point where the base shear is 60% of the effective yield strength. Second, the areas above and below the calculated curve should be approximately equal."

From the idealized force-displacement curve, a number of key parameters for determining the target displacement can be obtained, including Δ_y, V_y, Δ_d, V_d, Δ_4, V_4, K_i and K_e. (Δ_y, V_y) and (Δ_d, V_d) correspond to the yield point (Point 2 in Figure 11-18) and the peak point (Point 3 in Figure 11-17 and 11-18), respectively. Point 4 (Δ_4, V_4) is located at the descending branch. The force of Point 4, denoted as V_4, is specified to be $0.6V_y$ by ASCE 41-13 § 7.4.3.2.4. For the north-south preliminary analysis, the smallest post-peak force is $0.74V_y$ ($> 0.6\ V_y$), so for this analysis, V_4 is equal to the smallest force on the post-peak branch. For the east-west analysis, V_4 is equal to $0.6\ V_y$. K_e is calculated as V_y/Δ_y and K_i is the tangent of the force-displacement curve at the first step of the analysis. The values of these parameters are listed in Table 11-3, as well as T_i and T_e. T_i is the initial elastic fundamental period in the direction under consideration obtained from an eigenvalue analysis, while T_e is the effective fundamental period calculated as follows:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \quad \text{(ASCE 41-13 Eq. 7-27)}$$

Table 11-3 Key Parameters of Preliminary Force-Displacement Curves

Analysis Direction	Point 2*		Point 3*		Point 4*					
	Δ_y (in.)	V_y (kips)	Δ_d (in.)	V_d (kips)	Δ_4 (in.)	V_4 (kips)	K_i (kips/in.)	K_e (kips/in.)	T_i (s)	T_e (s)
E-W	0.27	1,679	1.74	2,482	1.93	1,007	17,268	6,261	0.11	0.19
N-S	1.14	1,547	5.00	2,463	18.83	843	3,451	1,362	0.25	0.39

* Points 2, 3, and 4 are shown in Figure 11-16 and Figure 11-17.

11.3.2 Coefficients for Calculating Target Displacement

As indicated in ASCE 41-13 § 2.2.1 and its commentary C2.2.1, while Tier 1 and Tier 2 procedures only need to have a check at the BSE-1E Seismic Hazard Level, the Tier 3 procedure requires a check at both the BSE-1E and BSE-2E Seismic Hazard Levels to conduct a full-building assessment to ensure sufficient robustness and margin of safety beyond the design-level earthquake. Therefore, for both the BSE-2E and BSE-1E Seismic Hazard Levels, the target displacement of the control node on the roof is calculated as follows:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \quad \text{(ASCE 41-13 Eq. 7-28)}$$

Calculation of the coefficients in ASCE 41-13 Equation 7-28 is discussed as follows.

11.3.2.1 Calculation of C_0

C_0 is calculated using PERFORM-3D® with the option of using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by ordinate of the shape vector at the control node. The obtained C_0 values for the east-west and north-south analyses are 1.164 and 1.339, respectively. Alternatively, C_0 can be determined according to ASCE 41-13 Table 7-5. For a 3-story shear building subjected to a triangular load pattern, the tabulated C_0 value is 1.2, which is close to the C_0 values calculated by the software program.

11.3.2.2 Calculation of $\mu_{strength}$

$\mu_{strength}$ is the ratio of elastic strength demand to yield strength coefficient calculated as follows:

$$\mu_{\text{strength}} = \frac{S_a}{V_y / W} C_m \qquad \text{(ASCE 41-13 Eq. 7-31)}$$

The required parameters and results of $\mu_{strength}$ are listed in Table 11-4 and Table 11-5. Among the required parameters, W is the effective seismic mass specified in ASCE 41-13 § 7.4.1.3.1. To be consistent with the LSP for the same building, the effective seismic weight used herein is 720 kips (roof) + 1,080 kips (Level 3) + 1,080 kips (Level 2) + 1,080 kips (Level 1) = 3,960 kips (total weight). C_m is the effective mass factor, which is taken as the effective modal mass participation factor calculated for the fundamental mode using an eigenvalue analysis; the C_m values calculated using PERFORM-3D® are 0.694 and 0.645 for the east-west and north-south analyses, respectively. Alternatively, C_m can be determined in accordance with ASCE 41-13 Table 7-4; for a 3-story concrete shear wall building, the tabulated value for C_m is 0.8; the values of V_y are obtained from Table 11-4. For example, $\mu_{strength}$ in the east-west direction for the BSE-2E Seismic Hazard Level is calculated as

$$\mu_{\text{strength}} = \frac{S_a}{V_y / W} C_m = \frac{1.08}{1{,}679 / 3{,}960}\left(0.694\right) = 1.768$$

Table 11-4 Calculation of $\mu_{strength}$ for the BSE-2E Seismic Hazard Level

Analysis Direction	T_e (s)	S_a (g)	V_y (kips)	W (kips)	C_m	$\mu_{strength}$
E-W	0.19	1.08	1,679	3,960	0.694	1.768
N-S	0.39	1.08	1,547	3,960	0.645	1.783

Table 11-5 Calculation of $\mu_{strength}$ for the BSE-1E Seismic Hazard Level

Analysis direction	T_e (s)	S_a (g)	V_y (kips)	W (kips)	C_m	$\mu_{strength}$
E-W	0.19	0.69	1,679	3,960	0.694	1.129
N-S	0.39	0.69	1,547	3,960	0.645	1.139

Useful Tip

As indicated by ASCE 41-13 § 7.3.2.1, if $\mu_{strength}$ is greater than the maximum strength ratio, μ_{max}, the NSP is not applicable and a nonlinear dynamic procedure (NDP) is recommended to capture strength degradation and dynamic P-Δ effects to confirm dynamic stability of the building.

In ASCE 41-13 Equation 7-32, if Δ_d/Δ_y is greater than $\mu_{strength}$, the second term does not need to be calculated since it is always positive. The second term is calculated herein for

11.3.2.3 Confirm Applicability of Nonlinear Static Procedure (ASCE 41-13 § 7.3.2.1 and § 7.4.3.3.2)

To confirm the applicability of NSP, $\mu_{strength}$ needs to be compared against μ_{max}. μ_{max} can be calculated as follows:

$$\mu_{max} = \frac{\Delta_d}{\Delta_y} + \frac{|\alpha_e|^{-h}}{4} \qquad \text{(ASCE 41-13 Eq. 7-32)}$$

where $h = 1 + 0.15 \ln(T_e)$; and the effective negative post-yield slope ratio, α_e, shall be calculated as follows:

$$\alpha_e = \alpha_{P-\Delta} + \lambda\left(\alpha_2 - \alpha_{P-\Delta}\right) \qquad \text{(ASCE 41-13 Eq. 7-33)}$$

where:

α_2 = Negative post-yield slope ratio as defined in ASCE 41-13 Figure 7-3

$\alpha_{P-\Delta}$ = Negative slope ratio caused by P-Δ effects

λ = Near-field effect factor
= 0.8 if $S_{X1} \geq 0.6$ for BSE-2N
= 0.2 if $S_{X1} < 0.6$ for BSE-2N

As indicated in ASCE 41-13 § C7.4.3.3.2, "the negative slope caused by P-Δ effects, $\alpha_{P-\Delta}$, is based on the restoring force needed to balance the overturning moment caused by the weight of the structure displaced by an amount Δ, acting at the effective height of the first mode. It can be determined using structural analysis software by comparing the stiffness results of an analysis run with P-Δ effects to one run without P-Δ effects considered." Figure 11-19 shows the force-displacement curves with and without P-Δ effects.

The calculation of α_e and μ_{max} is shown in Table 11-6. Using the east-west analysis under BSE-2E level hazard as an example, the calculations corresponding to Table 11-6 are as follows:

$$K_{negative} \text{ (without } P\text{-}\Delta) = \frac{(2{,}668 \text{ k-}1{,}809 \text{ k} \times 0.6)}{(2.3 \text{ in.-}3.073 \text{ in.})}$$

$$= \text{-}2{,}047 \text{ kip/in.}$$

$$K_{negative} \text{ (with } P\text{-}\Delta) = \frac{(2{,}482 \text{ k-}1{,}679 \text{ k} \times 0.6)}{(1.743 \text{ in.-}1.925 \text{ in.})}$$

$$= \text{-}8{,}102 \text{ kip/in.}$$

$$\alpha_{P-\Delta} = \frac{K_{negative}(\text{with } P-\Delta) - K_{negative}(\text{without } P-\Delta)}{K_e}$$

$$= \frac{-8{,}102 \text{ kip/in.} - \left(-2{,}047 \text{ kip/in.}\right)}{6{,}261 \text{ kip/in.}}$$

$$= -0.97$$

$$\alpha_2 = \frac{K_{\text{negative}}(\text{with } P-\Delta)}{K_e}$$

$$= \frac{-8{,}102 \text{ kip/in.}}{6{,}261 \text{ kip/in.}}$$

$$= 1.29$$

$$\alpha_e = \alpha_{P-\Delta} + \lambda\left(\alpha_2 - \alpha_{P-\Delta}\right)$$

$$= -0.97+0.8[-1.29-(-0.97)]$$

$$= -1.23$$

$$h = 1+0.15\ln T_e$$

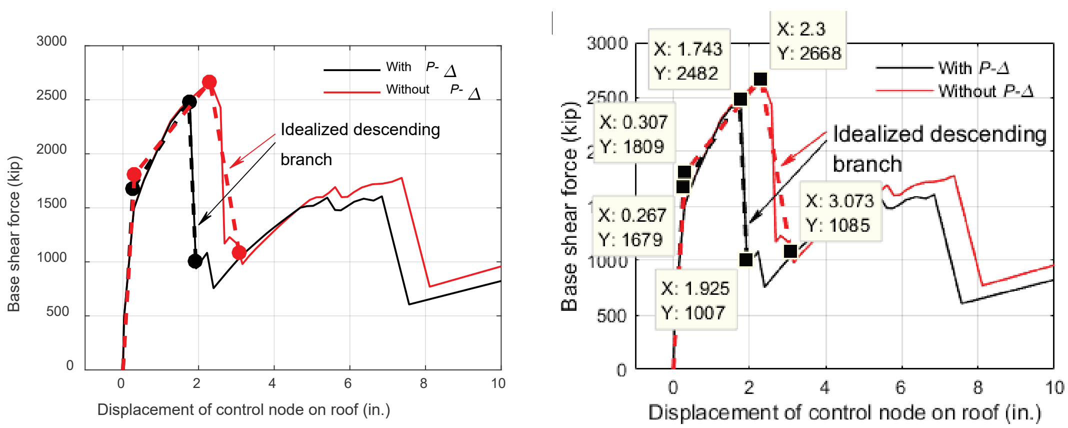
$$= 1+0.15\ln(0.19)$$

$$= 0.75$$

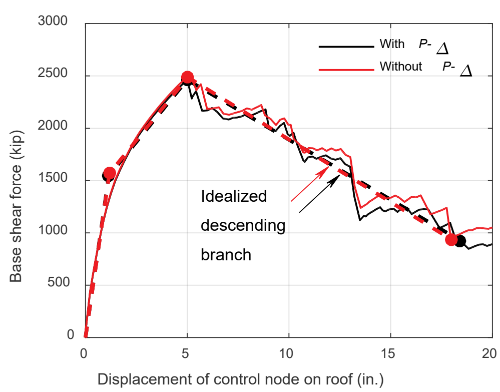
$$\mu_{\max} = \frac{\Delta_d}{\Delta_y} + \frac{\left|\alpha_e\right|^{-h}}{4}$$

$$= \frac{1.74}{0.27} + \frac{\left|-1.23\right|^{-0.75}}{4}$$

$$= 6.66$$



(a) east-west analysis



(b) north-south analysis

Figure 11-19 Comparison of post-yield descending slopes with and without P-Δ effects.

Table 11-6 Calculation of α_e and μ_{max}

Analysis Direction	K_e (kip / in.)	$K_{negative}$ (with P-Δ) (kip / in.)	$K_{negative}$ (without P-Δ) (kip / in.)	$\alpha_{P\text{-}\Delta}$	α_2	λ	α_e	h	μ_{max}
E-W	6,261	-8,102	-2,047	-0.97	-1.29	0.8	-1.23	0.75	6.66
N-S	1,362	-114	-119	0.0037	-0.0837	0.8	-0.066	0.86	6.97

Note: Tabulated values are same for both BSE-1E and BSE-2E Seismic Hazard Levels.

The $\mu_{strength}$ values in Table 11-4 and Table 11-5 are much smaller than the corresponding μ_{max} values in Table 11-6. Thus, the ASCE 41-13 § 7.3.2.1.1 requirement is satisfied, strength degradation effects are not considered significant, and an NDP analysis is not triggered.

In addition to comparing $\mu_{strength}$ and μ_{max}, higher mode effects need to be examined to confirm the applicability of NSP, per ASCE 41-13 § 7.3.2.1.2. For each of the two building principal directions, two response spectrum analyses are performed. The first analysis uses the first 50 modes with a cumulative mass participation of 97%. The second analysis incorporates only one fundamental mode in the direction under consideration. As indicated by ASCE 41-13 § 7.3.2.1.2, "Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the correponding story shear considering only the first mode response." As shown in Table 11-7, for each story, the ratio of shear force between the two analyses is less than 1.30, which indicates that the higher mode effects are insignificant for this structure.

Table 11-7 Response Spectrum Analysis Results for Confirming Applicability of NSP

Analysis Direction	Story	50 modes: Cumulative Mass Participation (%)	50 modes: Story Shear (kips)	3 modes: Cumulative Mass Participation (%)	3 modes: Story Shear (kips)	Ratio of Story Shear Between 50 and 3 Modes
E-W	3	97	1,032	69	1,011	1.02
	2		2,190		2,185	1.00
	1		2,842		2,834	1.00
	Basement		2,960		2,917	1.01
N-S	3	97	1,136	64	1,087	1.05
	2		2,158		2,150	1.00
	1		2,698		2,664	1.01
	Basement		2,914		2,753	1.06

Note that the 3 modes in Table 11-7 capture the fundamental mode in each direction (Modes 1 and 3) and the torsional mode between them (Mode 2), as illustrated in Figure 11-15.

By comparing μ_{strength} and $\mu_{\max}$ and examining the higher mode effects, the applicability of NSP in both principal directions is confirmed.

11.3.2.4 Calculation of C_1

C_1 is calculated as follows:

$$C_1 = 1+\frac{\mu_{\text{strength}}-1}{aT_e^2} \qquad \text{(ASCE 41-13 Eq. 7-29)}$$

where a is a site class factor and a = 60 for Site Class D. For the east-west analysis with BSE-2E Seismic Hazard Level:

$$C_1 = 1+\frac{1.768-1}{60(0.19)^2} = 1.354$$

For the north-south analysis with BSE-2E Seismic Hazard Level, C_1 = 1.086. For the east-west and north-south analyses with BSE-1E Seismic Hazard Level, the C_1 values are 1.060 and 1.015, respectively.

11.3.2.5 Calculation of C_2

C_2 is calculated as follows:

$$C_2 = 1+\frac{1}{800}\left(\frac{\mu_{\text{strength}}-1}{T_e}\right)^2 \qquad \text{(ASCE 41-13 Eq. 7-30)}$$

For the east-west analysis with the BSE-2E Seismic Hazard Level,

$$C_2 = 1+\frac{1}{800}\left(\frac{1.768-1}{0.19}\right)^2 = 1.020$$

For the north-south analysis with the BSE-2E Seismic Hazard Level, C_2 = 1.005. For the east-west and north-south analyses with the BSE-1E Seismic Hazard Level, the C_2 values are 1.001 and 1.000, respectively.

11.3.3 Preliminary Target Displacement (ASCE 41-13 § 7.4.3.3)

With the coefficients calculated in the preceding sections, the preliminary target displacement of the control node can be calculated as follows:

For the east-west analysis with the BSE-2E Seismic Hazard Level,

$$\delta_t = C_0C_1C_2S_a\frac{T_e^2}{4\pi^2}g \qquad \text{(ASCE 41-13 Eq. 7-28)}$$

$$= 1.164(1.354)(1.020)(1.08)\left(\frac{0.19\text{ s}}{2\pi}\right)^2\left(386.4\text{ in./s}^2\right)$$

$$= 0.613\text{ in.}$$

For the north-south analysis with the BSE-2E Seismic Hazard Level,

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{(ASCE 41-13 Eq. 7-28)}$$

$$= 1.339(1.086)(1.005)(1.08)\left(\frac{0.39\text{ s}}{2\pi}\right)^2 \left(386.4\text{ in./s}^2\right)$$

$$= 2.349\text{ in.}$$

For the east-west and north-south analyses with the BSE-1E Seismic Hazard Level, the preliminary target displacements are 0.301 in. and 1.397 in. respectively.

11.3.4 Actual and Accidental Torsional Effects (ASCE 41-13 § 7.2.3.2)

As indicated in ASCE 41-13 § C7.2.3.2.1, "The total torsional moment at a story shall be equal to the sum of the actual torsional moment and the accidental torsional moment …" In this example, since a three-dimensional building model with stiff (semi-rigid) diaphragms is employed in the analysis, the actual torsion is captured. The following steps are performed to examine the accidental torsional effect in this example:

1. For each of the two building principal directions, a pushover analysis is conducted with the preliminary target displacement. After the analysis is finished, the shear force of each story is extracted at the end of the analysis. The displacement multiplier caused by actual torsion, η_{actual}, is also obtained by comparing the maximum diaphragm displacement and the average diaphragm displacement.

2. The accidental torsional moment is calculated using the story shear force obtained in the last sub-step and the torsional moment is applied to each story. As indicated in ASCE 41-13 § 7.2.3.2.1, "the accidental torsional moment at a story shall be calculated as the seismic story shear force multiplied by a distance equal to 5% of the horizontal dimension at a given floor level measured perpendicular to the direction of the applied load."

3. After applying the accidental torsional moment to each level above the grade, the pushover analysis that is performed in the first sub-step is rerun. After the analysis is finished, the displacement multiplier that considers both actual and accidental torsional effects is calculated, denoted as $\eta_{\text{actual+accidental}}$.

4. Per ASCE 41-13 § 7.2.3.2.2, "Increased forces and displacements caused by accidental torsion need not be considered if either of the following conditions apply: (a) the accidental torsional moment is less than 25% of

the actual torsional moment, or (b) the ratio of the displacement multiplier caused by the actual torsion is less than 1.1 at every floor." In this example, $\eta_{actual+accidental}$ is compared with η_{actual} to confirm the necessity of considering accidental torsion. If accidental torsion needs to be considered per ASCE 41-13 § 7.4.3.3.3, "the target displacement shall be modified to consider the effects of torsion in accordance with Section 7.2.3.2." If not, the preliminary target displacement does not need any modification for torsion. In this example, comparisons between $\eta_{actual+accidental}$ and η_{actual} are shown in Table 11-8. It can be seen that for both directions, the ratio of $\eta_{actual+accidental}$ to η_{actual} for all the floors is smaller than 1.1 and, thus, the preliminary target displacements do not need to be modified for torsion.

Table 11-8 Displacement Multipliers of Actual and Accidental Torsion

Analysis Direction	Diaphragm	η_{actual}	$\eta_{actual+accidental}$	$\eta_{actual+accidental}$ / η_{actual}
E-W	Roof	1.004	1.021	1.017
	Level 3	1.005	1.016	1.011
	Level 2	1.002	1.011	1.010
N-S	Roof	1.053	1.088	1.033
	Level 3	1.031	1.068	1.036
	Level 2	1.020	1.030	1.010

11.3.5 Final Target Displacement (ASCE 41-13 § 7.4.3.3.2)

As indicated in ASCE 41-13 § 7.4.3.2.4, "(V_d, Δ_d) shall be a point on the actual force-displacement curve at the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is least." As shown in Section 11.3.3, the preliminary target displacements are much smaller than the Δ_d shown in Table 11-3. Therefore, iterations are necessary to obtain the final target displacement.

Using the preliminary target displacements, nonlinear static analyses are performed and the parameters V_y, Δ_y, C_0, T_e, C_m, C_1, and C_2 are recalculated to determine an updated target displacement, δ_t, for each iteration. When the relative difference between the updated δ_t and δ_t of the last calculation is smaller than 5%, the iteration can be stopped and the updated δ_t can be regarded as the target displacement. Note that there is no provision for the amount of relative difference (or absolute difference in displacement) specified in ASCE 41-13 for terminating the iteration. The termination condition for the iteration is subject to engineering judgment. The general rule is that when the difference of target displacement between two iterations is small enough, the iteration can be terminated. Table 11-9 and Table 11-11 show the updated δ_t through iterations and the final δ_t. It can be seen that in this example, only 1 to 2 iterations are needed to obtain the final target displacement. Table 11-10 shows the details of the iterations for east-west analysis under the BSE-2E Seismic Hazard Level. Figure 11-20 shows the force-displacement curve obtained from analyses with the final target displacements.

Table 11-9 Iterations for Determining Final Target Displacement (BSE-2E)

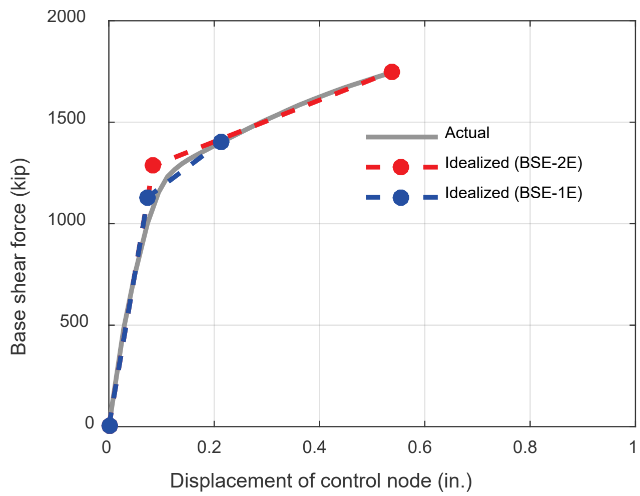
Analysis Direction	1st calculation	2nd calculation		3rd calculation		Final Δ_t (in.)
	δ_t (in.)	δ_t (in.)	Rel. Diff. (%)	δ_t (in.)	Rel. Diff. (%)	
E-W	0.613	0.521	-15	0.537	3.07	0.537
N-S	2.349	2.264	-3.62	N/A	N/A	2.264

Table 11-10 Details for Iteration on Target Displacement in East-West Direction (BSE-2E)

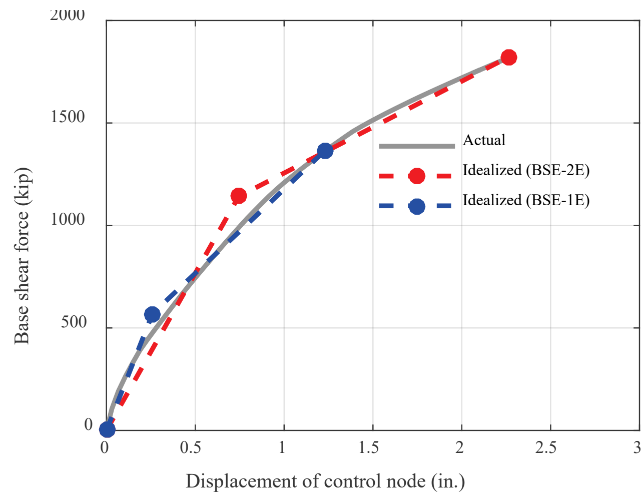
Calculation	S_a	W (kips)	V_y (kips)	C_m	$\mu_{strength}$	T_e (s)	C_0	C_1	C_2	δ_t (in.)	Rel. Diff. (%)
1st	1.08	3,960	1,679	0.694	1.768	0.190	1.164	1.354	1.020	0.613	
2nd			1,276		2.326	0.121	1.170	2.522	1.151	0.521	-15%
3rd			1,250		2.374	0.120	1.171	2.593	1.164	0.537	3.07%

Table 11-11 Iterations for Determining Final Target Displacement (BSE-1E)

Analysis Direction	1st calculation	2nd calculation		3rd calculation		4st calculation		Final Δ_t (in.)
	Δ_t (in.)	δ_t (in.)	Rel. Diff. (%)	δ_t (in.)	Rel. Diff. (%)	δ_t (in.)	Rel. Diff. (%)	
E-W	0.301	0.202	-32.9	0.219	8.42	0.215	-1.86	0.215
N-S	1.397	1.190	-14.8	1.231	3.45	N/A	N/A	1.231



(a) east-west analysis



(b) north-south analysis

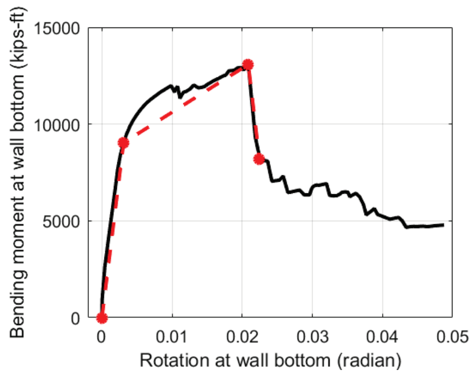
Figure 11-20 Force-displacement curves for final target displacements.

11.4 Performance Evaluation of Reinforced Concrete Shear Walls

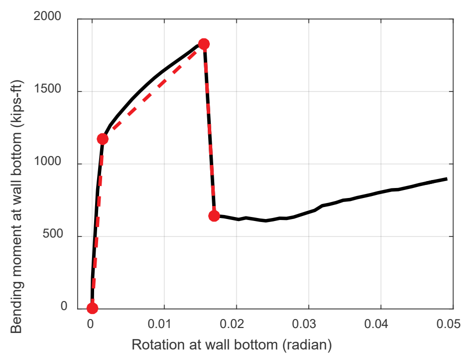
11.4.1 Deformation-Controlled and Force-Controlled Actions for Reinforced Concrete Shear Walls (ASCE 41-13 § 10.7.2.3)

As introduced in Section 10.5.1.1 of this *Guide*, per ASCE 41-13 § 10.7.2.3, when the transverse steel ratio is less than 0.0015, the wall shall be considered force-controlled. Walls 1 and 4 have a transverse steel ratio of 0.0010, and are thus considered force-controlled components.

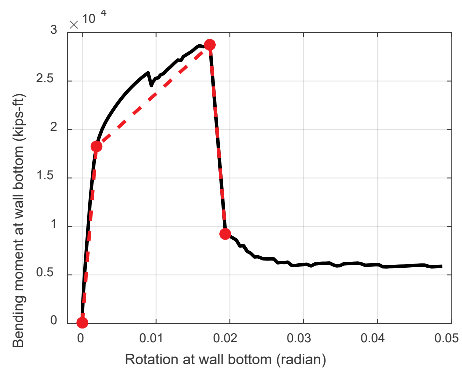
The transverse steel ratio of Walls A, D, and G is greater than 0.0015, and their shear and flexural actions are evaluated in accordance with ASCE 41-13 § 7.5.1.2. The in-plane shear action of these walls can be approximated by the Type 1 curve of ASCE 41-13 Figure 7-4. As shown in Figure 11-14 and Table 11-1, $d = 1.0\% > 2g = 2(0.4\%) = 0.8\%$. Per ASCE 41-13 § 7.5.1.2, “Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the plastic range is such that $d \geq 2g$; otherwise, they should be classified as force-controlled.” Therefore, the in-plane shear responses of Walls A, D, and G are classified as deformation-controlled actions. Figure 11-21 shows the moment-rotation behavior of Walls A, D, and G at the bottom section at Level 1.



(a) Wall A



(b) Wall D



(c) Wall G

Figure 11-21 Moment-rotation behavior of Walls A, D, and G.

The corresponding yield deformation and peak deformation, *g* and *d*, are listed in Table 11-12. It can be seen that the *d*/*g* ratio of all these walls is much greater than 2, so the flexural behavior of these walls is classified as deformation-controlled.

In summary, Walls 1 and 4 are considered force-controlled while Walls A, D, and G are considered deformation-controlled.

Table 11-12 Flexural Behavior of Walls A, D, and G

Wall location	Location	*d* (radian)	*g* (radian)	*d* / *g*	Action
Grid A	Level 1	0.02087	0.003094	6.7	Deformation-controlled
Grid D	Level 1	0.01556	0.001497	10.4	Deformation-controlled
Grid G	Level 1	0.01731	0.001892	9.1	Deformation-controlled

11.4.2 Acceptance Criteria for Shear and Flexural Responses (ASCE 41-13 § 10.7.2.4.2)

As indicated in ASCE 41-13 § 7.5.1.3, a lower-bound estimate of the component strength shall be used in evaluating the behavior of force-controlled actions. Accordingly, the shear and flexural response of Walls 1 and 4 are evaluated using their lower-bound capacities. Table 11-13 and Table 11-14 compare the demand and capacity of the shear and flexural responses of Walls 1 and 4 along with the counterparts obtained from the LSP. Using Wall 1 as an example, the maximum shear at Level 1 in the east-west analysis is 778 kips. The shear capacity of the wall calculated using lower bound material strength is 838 kips, which is shown in Section 10.5.1.1 of this *Guide*. Thus, the lower-bound wall capacity and the maximum shear demand obtained from NSP are compared as follows:

$$\frac{\text{Max. shear demand from NSP}}{Q_{CL}} = \frac{778 \text{ kips}}{838 \text{ kips}} = 0.93$$

The shear demand of the same wall obtained from LSP, Q_{UF}, is compared with κQ_{CL} as follows:

$$\frac{Q_{UF}}{\kappa Q_{CL}} = \frac{1{,}362 \text{ kips}}{0.9\left(838 \text{ kips}\right)} = 1.81$$

The values of Q_{UF} and Q_{CL} are shown in Section 10.5.1.1. The ratio of $Q_{UF}/\kappa Q_{CL}$ is termed an "acceptance ratio" here to provide a simple means of comparison.

Table 11-13 Demand and Capacity Acceptance Ratios of Walls 1 and 4 Subjected to BSE-2E Seismic Loads (Force-Controlled Component with Lower-Bound Strength Properties)

Wall location	Level	Action	Max. force/moment from NSP (k or k-ft)	Q_{UF} from LSP(1) (k or k-ft)	Q_{CL}(3) (k or k-ft)	NSP force demand/Q_{CL}	$Q_{UF}/\kappa Q_{CL}$(1), (2)
Grid 1	1	Shear*	778	1,362	838	0.93	1.81
		Flexure	24,024	41,960	52,440	0.46	0.89
Grid 4	1	Shear*	778	1,362	838	0.93	1.81
		Flexure	24,009	41,960	52,440	0.46	0.89

* Controlling failure mode

(1) Q_{UF} for shear and flexure is obtained from Table 10-13 and Table 10-11 of this *Guide*, respectively

(2) Knowledge factor κ = 0.9 (previous calculations in Section 10.5.1.1 of this *Guide*)

(3) As introduced at the beginning of Section 11.4.1 of this *Guide*, Walls 1 and 4 are considered to be force-controlled components because of the insufficient transverse reinforcement. Thus, Q_{CL} for flexure is calculated using sectional axial-flexural interaction analysis software with lower-bound material properties.

Table 11-14 Demand and Capacity Acceptance Ratios of Walls 1 and 4 Subjected to BSE-1E Seismic Loads (Force-Controlled Component with Lower-Bound Strength Properties)

Wall location	Level	Action	Max. force/moment from NSP (k or k-ft)	Q_{UF} from LSP(1) (k or k-ft)	Q_{CL}(3) (k or k-ft)	NSP force demand/Q_{CL}	$Q_{UF}/\kappa Q_{CL}$(2) (LSP)
Grid 1	1	Shear*	657	876	838	0.78	1.16
		Flexure	19,808	N/A	52,440	0.38	N/A
Grid 4	1	Shear*	657	876	838	0.78	1.16
		Flexure	19,775	N/A	52,440	0.38	N/A

* Controlling failure mode

(1) Q_{UF} obtained from Table 10-14 of this *Guide*

(2) Knowledge factor $\kappa = 0.9$ (previous calculations in Section 10.5.1.1 of this *Guide*)

(3) As introduced at the beginning of Section 11.4.1 of this *Guide*, Walls 1 and 4 are considered to be force-controlled components because of the insufficient transverse reinforcement. Thus, Q_{CL} for flexure is calculated using sectional axial-flexural interaction analysis software with lower-bound material properties.

Table 11-15 and Table 11-16 compare the maximum shear strain and rotation of Walls A, D, and G with the corresponding acceptance criteria listed in ASCE 41-13 Table 10-19 and Table 10-20. It can be seen that for the BSE-2E Seismic Hazard Level, all the shear and flexural deformations are acceptable for the Collapse Prevention (CP) level. While Walls A and G are dominated by flexure, Wall D is dominated by shear. For the BSE-1E Seismic Hazard Level, all the deformations are acceptable for the Life Safety (LS) level.

Table 11-15 Demand and Capacity Acceptance Ratios of Walls A, D, and G Subjected to BSE-2E Seismic Loads (Deformation-Controlled Components with Expected Strength Properties)

Wall location	Level with max. def.	Action	Total shear strain or rotation (unitless/rad)	Elastic rotation limit(3) (rad)	Plastic rotation (rad)	Acceptable shear strain or rotation (unitless / rad)			NSP Def. accp. ratio at CP(1)	LSP force accp. ratio(2)
						IO	LS	CP		
Grid A	1	Shear	8.59E-04	N/A	N/A	0.004	0.015	0.020	0.04	--
	1	Flexure*	4.22E-03	3.09E-3	1.13E-03	0.005	0.015	0.020	0.06	0.63
Grid D	2	Shear*	2.91E-03	N/A	N/A	0.004	0.015	0.020	0.15	--
	1	Flexure	1.20E-03	1.50E-3	N/A	0.002	0.008	0.015	Elastic	0.51
Grid G	1	Shear	1.67E-03	N/A	N/A	0.004	0.015	0.020	0.08	--
	1	Flexure*	3.00E-03	1.89E-3	1.11E-03	0.005	0.015	0.020	0.06	0.48

* Controlling action

(1) NSP deformation acceptance ratio at CP = Maximum shear strain or plastic rotation / acceptance criterion at CP

(2) LSP force acceptance ratio = $Q_{UD}/m\kappa Q_{CE}$ (per previous calculations in Table 10-13) (DCR_{LSP} compares forces while NSP deformation acceptance ratio compares deformation)

(3) The elastic rotation limit corresponds to the deformation at the second red dot shown in Figure 11-21

Table 11-16 Demand and Capacity Acceptance Ratios of Walls A, D, and G Subjected to BSE-1E Seismic Loads (Deformation-Controlled Components with Expected Strength Properties)

Wall location	Level with max. def.	Action	Total shear strain or rotation (unitless/rad)	Elastic rotation limit(3) (rad)	Plastic rotation (rad)	Acceptable shear strain or rotation (unitless / rad)			NSP Def. accp. ratio at LS(1)	LSP force accp. ratio(2)
						IO	LS	CP		
Grid A	1	Shear	2.96E-04	N/A	N/A	0.004	0.015	0.020	0.02	--
	1	Flexure*	2.22E-03	3.09E-3	N/A	0.005	0.015	0.020	Elastic	0.54
Grid D	2	Shear*	1.20E-03	N/A	N/A	0.004	0.015	0.020	0.08	--
	1	Flexure	9.06E-04	1.50E-3	N/A	0.002	0.008	0.015	Elastic	0.44
Grid G	1	Shear	4.70E-04	N/A	N/A	0.004	0.015	0.020	0.03	--
	1	Flexure*	1.54E-03	1.89E-3	N/A	0.005	0.015	0.020	Elastic	0.41

* Controlling action

(1) NSP deformation acceptance ratio at LS = Maximum shear strain or plastic rotation / acceptance criterion at LS

(2) LSP force acceptance ratio = $Q_{UD}/m\kappa Q_{CE}$ (per previous calculations in Table 10-14) (DCR for LSP compares forces while DCR for NSP compares deformation)

(3) The elastic rotation limit corresponds to the deformation at the second red dot shown in Figure 11-21

11.4.3 LSP and NSP for Evaluating Shear Walls

Table 11-13 and Table 11-14 highlight a significant finding for this particular design example: Force-controlled walls analyzed using the LSP did not meet the specified Performance Objective, but they did meet the Performance Objective when using the NSP. This will not always be the case, but it shows the potential benefit of using the more involved nonlinear procedure. This observation is also consistent with the ASCE 41-13 C7.5.1 statement: "Buildings that do not comply with the linear analysis acceptance criteria may comply with nonlinear acceptance criteria. Therefore, performing a nonlinear analysis is recommended to minimize or eliminate unnecessary seismic retrofit. Design professionals are encouraged to consider the limitations of linear procedures and to pursue nonlinear analyses where linear acceptance criteria are not met."

Some discussion is provided to help with this comparison. The LSP directly determines pseudo seismic forces applied to the structure, although the forces are typically higher than the actual capacity of the structure. When the lateral force-resisting components are evaluated, the *m*-factor is used to amplify the structural capacity of deformation-controlled actions to account for the ability of ductile components to dissipate seismic energy through inelastic deformation. As the force-controlled actions are typically brittle and lack of energy dissipation ability, their capacities are not amplified when compared with the pseudo seismic demands.

In contrast, the NSP estimates the seismic target displacement of a structure rather than directly estimating seismic force. Based on the estimated target displacement, the seismic force demands are obtained using a nonlinear structural model that explicitly considers yielding and softening of structural components. Thus, for force-controlled actions, the force demand obtained from NSP can be directly compared with the capacity. For deformation-controlled actions, the deformation response is compared with the acceptance criteria on deformation, rather than evaluating the force demand.

As shown in Table 11-13 and Table 11-14, the force and moment demands on the force-controlled Walls 1 and 4 obtained from LSP are much larger than those obtained from the NSP. For the force-controlled Walls 1 and 4, the *m*-factor is not involved in determination of the capacity. The $Q_{UF}/\kappa Q_{CL}$ acceptance ratios obtained from the LSP are much higher than the maximum force demand to Q_{CL} ratio obtained from the NSP for Walls 1 and 4. Walls A, D, and G are deformation-controlled, and their deformation response is compared with the corresponding acceptance criteria. The maximum shear strain or plastic rotation acceptance criterion for the NSP is substantially less than the $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio for the LSP, implying the LSP acceptance criteria are comparatively more stringent.

11.5 Performance Evaluation of Reinforced Concrete Columns (ASCE 41-13 § 10.4.2)

As an example, the Column D-3 supporting the discontinuous shear wall at Gridline D in the basement is evaluated with the NSP. The north-south analysis result caused by the BSE-2E Seismic Hazard Level is evaluated herein.

11.5.1 Axial Response

Gravity loads are added to the analytical model in a stepwise approach by gradually increasing the gravity load applied to the model before the lateral loads are applied. At the end of the gravity load application, the axial compression of Column D-3 in the basement is 243 kips. During the static pushover analysis for the BSE-2E Seismic Hazard Level in the north-south direction, the maximum and minimum axial compressive forces of Column D-3 in the basement are 868 kips and 243 kips, respectively. Column D-3 was not subjected to tension in the nonlinear static analysis.

Per ASCE 41-13 Table C7-1, axial compression of columns is typically classified as a force-controlled action. For the force-controlled action of axial compression, per ASCE 41-13 § 7.5.3.2.2, primary and secondary components shall have lower-bound strengths not less than the maximum analysis forces. The lower-bound axial capacity of Column D-3 in the basement is calculated using ACI 318-14 Equation 22.4.2.2 as follows:

$$P_0 = 0.8\phi\left[0.85f_c'\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$= 0.8(1.0)[0.85(2.5 \text{ ksi})(400 \text{ in.}^2 - 8.0 \text{ in.}^2) + (40 \text{ ksi})(8.0 \text{ in.}^2)]$

$= 922$ kips

where:

A_g = gross sectional area of the column

A_{st} = total area of nonprestressed longitudinal reinforcement

f_c' = the lower-bound (nominal) compressive strength of the concrete

f_y = lower-bound (nominal) yield strength of rebar

ϕ = strength reduction factor taken as 1.0 in the ASCE 41-13 context

Comparing the lower-bound strength with the analysis force at the target displacement, $Q_{CL} = P_0 = 922$ kips $> N_{u,\max} = 868$ kips. Thus, the column is adequate for axial compression.

11.5.2 Shear Response

Per ASCE 41-13 Table C7-1, shear of columns in reinforced concrete frames is typically classified as a force-controlled action. Shear strength of the column is determined per ASCE 41-13 § 10.4.2.3.1. In ASCE 41-13 Equation 10-3, M/Vd is the largest ratio of moment to shear times effective depth under design loadings for the column but shall not be taken greater than 4 or less than 2. The M/Vd at each step of the static pushover analysis is evaluated and the maximum value in the analysis is 1,613 >> 4, and, thus, M/Vd is taken as 4. The maximum N_u in ASCE 41-13 Equation 10-3 is 868 kips and the minimum is 243 kips.

The k factor in ASCE 41-13 Equation 10-3 is determined per ASCE 41-13 § C10.4.2.3.1: "For a column with experiencing flexural yielding before shear failure ($V_p < V_o$), displacement ductility demand is defined as the ratio of maximum displacement demand to yield displacement. The yield displacement is the lateral displacement of the column, determined using the effective rigidities from ASCE 41-13 Table 10-5, at a shear demand resulting in flexural yielding of the plastic hinges, V_p. The maximum displacement demand for the column can be estimated as the maximum interstory displacement demand. Alternatively, the interstory displacement demand can be refined by accounting for the interstory displacements caused by rigid body rotations at the column's base and top."

For $N_{u,\max}$ = 868 kips, the flexural capacity at the column's top, M_{CE}, is determined by running a P-M interaction analysis program with f'_{ce} = 3,750 psi, = f_{ye} = 50,000 psi, and ϕ = 1.0. Thus, M_{CE} is 384 kip-ft.

As the Column D-3 (shown in Figure 11-22) is pinned at the bottom and rigidly connected to the diaphragm at the top, the column is studied using an idealized column model with the bottom pinned and the top fixed. The relation between drift Δ and end moment M on the column top is $\Delta = ML^2/3EI_{\text{eff}}$ (Hibbeler, 2014), where L is the clear height of the column and EI_{eff} is the effective flexural stiffness determined in accordance with ASCE 41-13 Table 10-5. Although the story height between Level 1 and the basement slab is 14 ft, the clear height of the column, L, is assumed to be 12 ft, which considers the beam at the column top and is consistent with the clear height used in the LSP. Figure 11-22 shows the relationship between end moment and lateral drift on the column top.

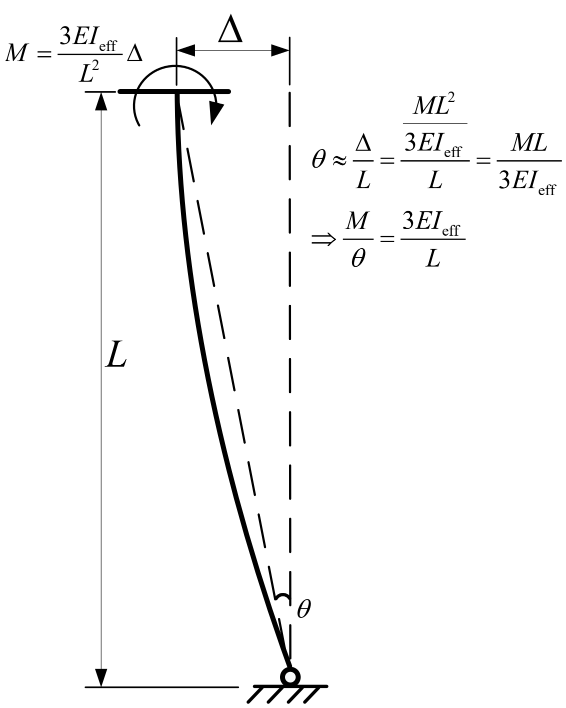


Figure 11-22 Drift of a basement column with a pinned support at base and a fixed support at top.

For Column D-3 in the basement,

$$0.1 < \frac{P_G}{A_g f'_{ce}} = \frac{243 \text{ kips}}{\left(400 \text{ in.}^2\right)\left(3.75 \text{ ksi}\right)} = 0.162 < 0.5$$

where P_G is the column compression caused by design gravity loads. Although the definition of f_c' in ASCE 41-13 Table 10-5 is vague, ASCE 41-17 Table 5 clearly shows that f_{ce}' is used to categorize the axial compression ratio of columns.

The effective flexural stiffness determined in accordance with ASCE 41-13 Table 10-5 and interpolating between the $0.3E_cI_g$ value for $P_G = 0.1A_g f_c'$ and the $0.7E_cI_g$ value for $P_G = 0.5A_g f_c'$ is $0.362E_cI_g$.

$$\begin{aligned}
EI_{\text{eff}} &= 0.362E_cI_g \\
&= 0.362\left(57{,}000\sqrt{3{,}750\text{ psi}}\right)\left[\frac{(20\text{ in.})^4}{12}\right]/144/1{,}000 \\
&= 116{,}997\text{ kip-ft}^2
\end{aligned}$$

The displacement at flexural yielding of the column top is calculated as

$$\begin{aligned}
\Delta_y &= \frac{M_{CE}L^2}{3EI_{\text{eff}}} \\
&= \frac{(384\text{ kip-ft})(144\text{ ft}^2)}{3(116{,}997\text{ kip-ft}^2)} \\
&= 0.158\text{ ft} \\
&= 1.90\text{ in.}
\end{aligned}$$

For north-south analysis under the BSE-2E Seismic Hazard Level, the drift of Column D-3 is 0.073 in, which is much smaller than $\Delta_y = 1.90$ in. Per ASCE 41-13 § 10.4.2.3.1, the displacement ductility demand k is taken as 1.0 in ASCE 41-13 Equation 10-3. Using lower-bound material strength, the shear capacity V_n is calculated as follows:

$$V_n(N_{u,\max}) = kV_o \qquad \text{(ASCE 41-13 Eq. 10-3)}$$

$$\begin{aligned}
&= 1.0\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f_c'}}{M/Vd}\sqrt{1+\frac{N_{u,\max}}{6\sqrt{f_c'}A_g}}\right)0.8A_g\right] \\
&= 1.0\left[\frac{2\left(0.2\text{ in.}^2\right)(40{,}000\text{ psi})(20\text{ in.})(0.8)}{10\text{ in.}}\right. \\
&\qquad \left. +1.0\left(\frac{6\sqrt{2{,}500\text{ psi}}}{4}\sqrt{1+\frac{868{,}000\text{ lbs}}{6\sqrt{2{,}500\text{ psi}}\left(400\text{ in.}^2\right)}}\right)(0.8)\left(400\text{ in.}^2\right)\right]/1{,}000 \\
&= 94.5\text{ kips}
\end{aligned}$$

where:

V_o = shear strength of column without modification for flexural ductility

A_v = effective area of all bar legs or wires within spacing s

f_y = lower-bound (nominal) yield strength of the rebar

d = effective depth that shall be permitted to be assumed equal to 0.8

λ = 0.75 for lightweight aggregate concrete and 1.0 for normal-weight aggregate concrete

f'_c = lower-bound (nominal) compressive strength of the concrete

M/Vd = the largest ratio of moment to shear times effective depth under design loadings for the column but shall not be taken greater than 4 or less than 2

N_u = axial compression force (set to zero for tension force)

A_g = gross cross-sectional area of the column

For $N_{u\,\min}$ = 243 kips, shear capacity of the column is determined as

$$V_n(N_{u,\min}) = kV_o$$

$$= 1.0\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_{u,\min}}{6\sqrt{f'_c}A_g}}\right)0.8A_g\right]$$

$$= 1.0\left[\frac{2\left(0.2\text{ in.}^2\right)\left(40{,}000\text{ psi}\right)\left(20\text{ in.}\right)\left(0.8\right)}{10\text{ in.}}\right.$$

$$\left.+1.0\left(\frac{6\sqrt{2{,}500\text{ psi}}}{4}\sqrt{1+\frac{243{,}000\text{ lbs}}{6\sqrt{2{,}500\text{ psi}}\left(400\text{ in.}^2\right)}}\right)\left(0.8\right)\left(400\text{ in.}^2\right)\right]/1{,}000$$

$$= 67.3\text{ kips}$$

Under the BSE-2E Seismic Hazard Level in the north-south direction, the maximum shear force of Column D-3 in the analysis is only 0.65 kips. Thus, the shear strength of Column D-3 in the basement is adequate for the BSE-2E Seismic Hazard Level in the north-south direction.

11.5.3 Flexural Response

Because Column D-3 is pinned at the bottom in the building model, only the plastic rotation at the column top is assessed. Per ASCE 41-13 § 10.4.2.2.2, "Columns not controlled by inadequate splices, condition i, ii, or iii in Table 10-8, shall be classified based on V_o per Eq. 10-3, using expected material properties, the plastic shear demand on the column, V_p, defined as the shear demand at flexural yielding of plastic hinges, and the transverse reinforcement detailing, as shown in Table 10-11."

To determine V_p in ASCE 41-13 Table 10-11, run a commercially available P-M interaction analysis program with f'_c = 3,750 psi, f_{ye} = 50,000 psi, and ϕ = 1.0.

For $N_{u,\max}$= 868 kips, M_{CE} = 384 kip-ft,

$$V_p = \frac{M_{CE}}{L} = \frac{384 \text{ kip-ft}}{12 \text{ ft}} = 32 \text{ kips.}$$

For $N_{u,\min}$= 243 kips, M_{CE} = 392 kip-ft,

$$V_p = \frac{M_{CE}}{L} = \frac{392 \text{ kip-ft}}{12 \text{ ft}} = 32.7 \text{ kips}$$

Note that $V_n = M_{CE}/L$ is only applicable to the basement columns in the analytical model with a pinned support at the bottom and rigidly connected to the diaphragm at the top. For columns in the other stories with both ends connected to the diaphragm, $V_n = 2M_{CE}/L$ should be used. Different from the calculation of V_n (V_o) using lower-bound material strength, V_o is calculated using ASCE 41-13 Equation 10-3 but with expected material strength f'_{ce} = 3,750 psi and f_{ye} = 50,000 psi per ASCE 41-13 § 10.4.2.2.2. The V_o values for the maximum and minimum axial compression are 109.2 kips and 79.8 kips, respectively. The ratios of V_p / V_o are calculated as follows:

For $N_{u,\max}$= 868 kips,

$$\frac{V_p}{V_o} = \frac{32 \text{ kips}}{109.2 \text{ kips}} = 0.293 < 0.6$$

For $N_{u,\min}$= 243 kips,

$$\frac{V_p}{V_o} = \frac{32.7 \text{ kips}}{79.8 \text{ kips}} = 0.410 < 0.6$$

Assume that the transverse reinforcements of the column are closed hoops with 90º hooks. According to ASCE 41-13 Table 10-11, the condition to be used in ASCE 41-13 Table 10-8 is Condition (ii) (flexure-shear failure, where yielding in flexure is expected before shear failure), because of the combination of $V_p / V_o \leq 0.6$ and the assumed closed hoops with 90-degree hooks.

For Condition (ii) in ASCE 41-13 Table 10-8,

$$\frac{N_{u,\max}}{A_g f'_{ce}} = \frac{868\text{ kips}}{\left(400\text{ in.}^2\right)\left(3.75\text{ ksi}\right)} = 0.579 < 0.6$$

As indicated in ASCE 41-13 Table 10-8 Footnote ©, the axial load P should be based on the maximum expected axial loads caused by gravity and earthquake loads. Therefore, $N_{u,\max}$ = 868 kips is used for the calculation in ASCE 41-13 Table 10-8.

$$0.1 < \frac{N_{u,\max}}{A_g f'_{ce}} = \frac{868\text{ kips}}{\left(400\text{ in.}^2\right)\left(3.75\text{ ksi}\right)} = 0.579 < 0.6$$

$$0.0005 < \rho = \frac{A_v}{b_w s} = \frac{2\left(0.2\text{ in.}^2\right)}{\left(20\text{ in.}\right)\left(10\text{ in.}\right)} = 0.002 < 0.006$$

The maximum shear force on the column in the north-south analysis under BSE-2E Seismic Hazard Level is 0.65 kips.

$$\frac{V}{b_w d\sqrt{f'_{ce}}} = \frac{650\text{ lbs}}{\left(20\text{ in.}\right)\left(16\text{ in.}\right)\sqrt{3,750\text{ psi}}} = 0.033 << 3$$

Although the definition of f'_c in ASCE 41-13 Table 10-8 for reinforced concrete columns is vague, ASCE 41-17 Table 10-7 and Table 10-19 for reinforced concrete beams and walls clearly show that the expected material strength f'_{ce} should be used for categorizing the shear ratio.
Conducting linear interpolation for the cases in ASCE 41-13 Table 10-8, the plastic rotation angles for the three performance levels IO, LS, CP are determined as follows:

$$\begin{cases} \text{IO: } \theta_{\text{plastic}} = 0.00239 \\ \text{LS: } \theta_{\text{plastic}} = 0.00526 \\ \text{CP: } \theta_{\text{plastic}} = 0.00645 \end{cases}$$

The effective flexural stiffness of the column is determined per ASCE 41-13 Table 10-5. As calculated in the last subsection:

$$EI_{\text{eff}} = 116{,}997\text{ kip-ft}^2$$

For the columns in the basement, as shown in Figure 11-23, the rotational stiffness is calculated as:

$$\begin{aligned} \frac{M}{\theta} &= \frac{3EI_{\text{eff}}}{L} \\ &= \frac{3\left(0.362E_c I_g\right)}{L} \end{aligned}$$

$$= \frac{3\left(116{,}997 \text{ kip-ft}^2\right)}{12 \text{ ft}}$$
$$= 29{,}249 \text{ kip-ft/rad}$$

For $N_{u,\,\max}$ = 868 kips and $M_{CE}\,(N_{u,\,\max})$ = 384 kip-ft, the yield rotation, θ_y, is calculated as follows:

$$\theta_y,(N_{u,\max}) = \frac{M_{CE}\left(N_{u,\max}\right)L}{3EI_{\text{eff}}}$$
$$= \frac{\left(384 \text{ kip-ft}\right)\left(12 \text{ ft}\right)}{3\left(116{,}997 \text{ kip-ft}^2\right)}$$
$$= 0.0131 \text{ rad}$$

For $N_{u,\,\min}$ = 243 kips and $M_{CE}\,(N_{u,\,\min})$ = 392 kip-ft, the yield rotation θ_y associated with $N_{u,\,\min}$ and $M_{CE}\,(N_{u,\,\min})$ is calculated as follows:

$$\theta_y,(N_{u,\min}) = \frac{M_{CE}\left(N_{u,\min}\right)L}{3EI_{\text{eff}}}$$
$$= \frac{\left(392 \text{ kip-ft}\right)\left(12 \text{ ft}\right)}{3\left(116{,}997 \text{ kip-ft}^2\right)}$$
$$= 0.0134 \text{ rad}$$

The maximum rotation at the column top subjected to BSE-2E Seismic Hazard Level in the north-south direction is 0.000314 rad << θ_y = 0.0134 rad. Thus, the column responds elastically without having any plastic end rotation.

As illustrated in Figure 11-23, the columns in the stories above the basement are studied using an idealized column model with both ends fixed. The relation between the top drift Δ and end moment M is $\Delta = (ML^2/6EI_{\text{eff}})$ (Hibbeler, 2014). Substituting Δ by $\Delta \approx \theta L$ into $\Delta = (ML^2/6EI_{\text{eff}})$, the relation between yield rotation θ_y and moment capacity M_{CE} can be derived as $\theta_y = (ML/6EI_{\text{eff}})$.

For both the north-south and east-west analyses, all columns are assessed using the same procedure outlined above in Section 11.5 of this *Guide*. For all columns, the seismic performance for axial, shear, and flexural response meets the acceptance criteria.

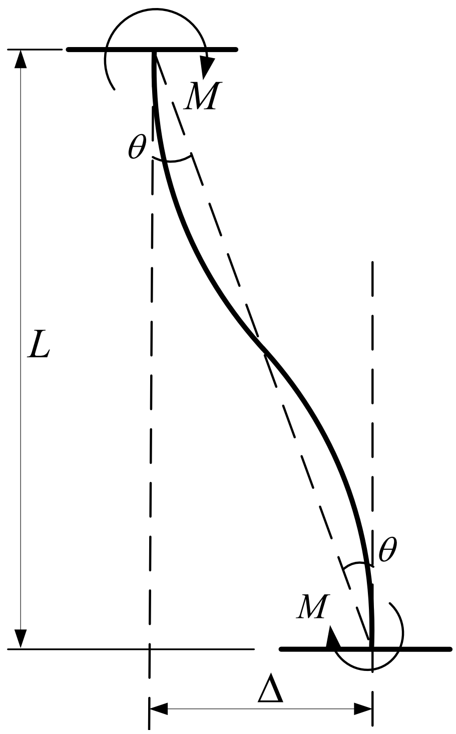


Figure 11-23 Drift of columns in stories above basement.

11.6 Three-Dimensional Explicit Modeling of Foundation Components (ASCE 41-13 § 8.4.2)

To more accurately estimate seismic performance of the building, the building structure is modeled with a flexible base instead of a fixed base with all nodes at the bottom of walls and columns pinned. The foundation flexibility will have an impact on the target displacement for NSP and may also affect the distribution of seismic forces among the lateral force-resisting components. For this building with shallow foundation components, ASCE 41-13 § 8.4.2 outlines three different methods for modeling the foundation system, depending on the rigidity of foundation components relative to the foundation soil and if explicit coupling of axial and overturning foundation response is desired or not. As shown in the subsequent sections, the spread and strip footings supporting columns and basement retaining walls of this building are classified as flexible relative to the foundation soil. Moreover, explicit axial and overturning foundation response is desired to be modeled. Due to these two reasons, Method 3 in ASCE 41-13 § 8.4.2.5 is used to model the foundation components of the building. In this method, the foundation components are explicitly modeled, and bilinear compression-only springs are used to model the soil reaction beneath the foundation components. More details regarding the different methods for modeling foundation systems can be found in Chapter 5 of this *Guide*.

11.6.1 Modeling Foundation Flexibility

Assume that the unit weight of foundation soil, γ, is 120 pcf. Per ASCE 41-13 § 2.4.1.6.1, for Site Class D, the average shear wave velocity is in the range of 600 ft/s to 1,200 ft/s. Considering that the Site Class is D in this example, the shear wave velocity v_{s0} is assumed to be 1,000 ft/s. The initial shear modulus of the foundation soil, G_0, is calculated in accordance with ASCE 41-13 Equation 8-4 as follows:

$$
\begin{aligned}
G_0 &= \frac{\gamma v_{s0}^2}{g} && \text{(ASCE 41-13 Eq. 8-4)}\\
&= \frac{(120 \text{ pcf})(1{,}000 \text{ ft/s})^2}{32.2 \text{ ft/s}^2}\\
&= 3{,}726{,}708 \text{ psf}
\end{aligned}
$$

The effective shear modulus ratio G/G_0 is determined using ASCE 41-13 Table 8-2. For Site Class D and $S_{XS} = 1.08$, G/G_0 equals to 0.468. Thus, the effective shear modulus of the foundation soil is calculated as:

$$
G = G_0(G/G_0) = (3{,}726{,}708 \text{ psf})(0.468) = 1{,}744{,}099\text{psf}
$$

Per ASCE 41-13 § 8.4.2.2, Poisson's ratio of the foundation soil is taken as 0.25. The unit subgrade spring coefficient, k_{sv}, is determined in accordance with ASCE 41-13 Equation 8-11 as follows:

$$
\begin{aligned}
k_{sv} &= \frac{1.3G}{B_f(1-\nu)} && \text{(ASCE 41-13 Eq. 8-11)}\\
&= \frac{1.3(1{,}744{,}099 \text{ psf})}{(10 \text{ ft})(1-0.25)}\\
&= 302{,}310 \text{ psf/ft}
\end{aligned}
$$

Interior Spread Footing

The flexibility of the spread footing supporting gravity columns is determined using ASCE 41-13 Equations C8-2 and C8-3. Dimensions of the spread footing are 10 ft (L) × 10 ft (B) × 1.5 ft (t), which are consistent with the footing dimensions used in ASCE 41-13 § 10.5.2.2. As indicated by ASCE 41-13 § C8.4.2.1, "for rectangular plates (with plan dimensions L and B, thickness t, and mechanical properties E and ν) on elastic supports (for instance, mat foundations or isolated footings) subjected to a point load in the center, the foundation may be considered rigid where"

$$4k_{sv}\sum_{m=1}^{5}\sum_{n=1}^{5}\frac{\sin^2\left(\frac{m\cdot\pi}{2}\right)\sin^2\left(\frac{n\cdot\pi}{2}\right)}{\left[\pi^4 D\left(\frac{m^2}{L^2}+\frac{n^2}{B^2}\right)^2\right]+k_{sv}}<0.03 \qquad \text{(ASCE 41-13 Eq. C8-2)}$$

where:

$$D = \frac{E_c t^3}{12\left(1-\nu_c\right)^2} \qquad \text{(ASCE 41-13 Eq. C8-3)}$$

where E_c and ν_c are the elastic modulus and Poisson's ratio of the concrete material of the footing and t is the thickness of the footing. To consider cracked stiffness of the concrete spread footing, a reduced elastic modulus of concrete, $0.25E_c$, is used in ASCE 41-13 Equation C8-3, in lieu of the nominal elastic modulus E_c. The reduction factor of 0.25 is obtained from ACI 318-14 Table 6.6.3.1.1(a), as there are no foundation entries in ASCE 41-13 Table 10-5. The elastic modulus of concrete is calculated using expected material strength in accordance with ACI 318-14 as follows:

$$E_c = 57{,}000\sqrt{3{,}750 \text{ psi}}\left(144\right) = 5.026\times10^8 \text{ psf}$$

The Poisson's ratio of concrete, ν_c, is taken as 0.2 (Wright, 2015).

Thus, coefficient D is calculated as follows:

$$D = \frac{0.25E_c t^3}{12\left(1-\nu_c\right)^2} = \frac{0.25\left(5.026\times10^8 \text{ psf}\right)\left(1.5 \text{ ft}\right)^3}{12\left(1-0.2\right)^2} = 5.52\times10^7 \text{ lb-ft}$$

D, L, B, and k_{sv} are substituted by 5.52×10^7 lb-ft, 10 ft, 10 ft, and 302,310 psf/ft into ASCE 41-13 Equation C8-2, and the left-hand side of ASCE 41-13 Eq. C8-2 yields 0.56, which is much greater than 0.03. Therefore, the spread footing is considered to be flexible relative to the foundation soil. Note that the calculation of ASCE 41-13 Equation C8-2 is quite involved and needs a detailed spreadsheet or small program to complete. In the interest of space, it is not shown here.

Perimeter Strip Footing

The width and thickness of the strip footing supporting the perimeter retaining walls are assumed to be 6 ft and 1.5 ft, respectively. The flexibility of the strip footing relative to the foundation soil is determined in accordance with ASCE 41-13 Equation C8-1. To consider cracked stiffness of the concrete strip footing, a reduced elastic modulus of concrete, $0.25E_c$ is used, similar to the spread footing. Figure 11-24 shows the composite section consisting of strip footing and retaining wall for evaluating rigidity of strip footing relative to the foundation soil. L in ASCE 41-13 Equation C8-1 is taken as the wall length between two column lines, which is 20 ft. I is taken as the moment of inertia of the composite section with both the strip footing and the 14-inch-thick retaining wall on top of the footing.

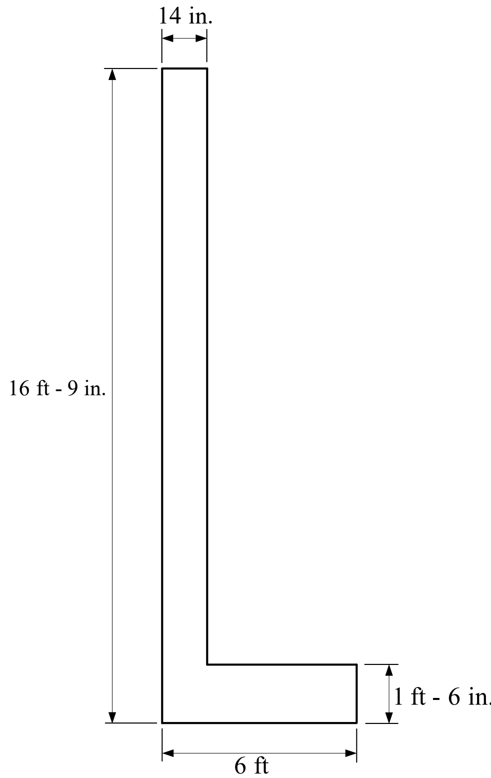


Figure 11-24 Section of retaining wall and strip footing used to evaluate rigidity of strip footing relative to foundation soil.

In accordance with ASCE 41-13 Equation C8-1:

$$\frac{0.25E_cI}{L^4} = \frac{0.25\left(5.0264\times10^8\text{ psf}\right)\left(718.6\text{ ft}^4\right)}{\left(20\text{ ft}\right)^4} = 564{,}337\text{ psf}$$

$$\frac{2}{3}k_{sv}B = \frac{2}{3}\frac{1.3G}{B\left(1-\nu\right)}B$$

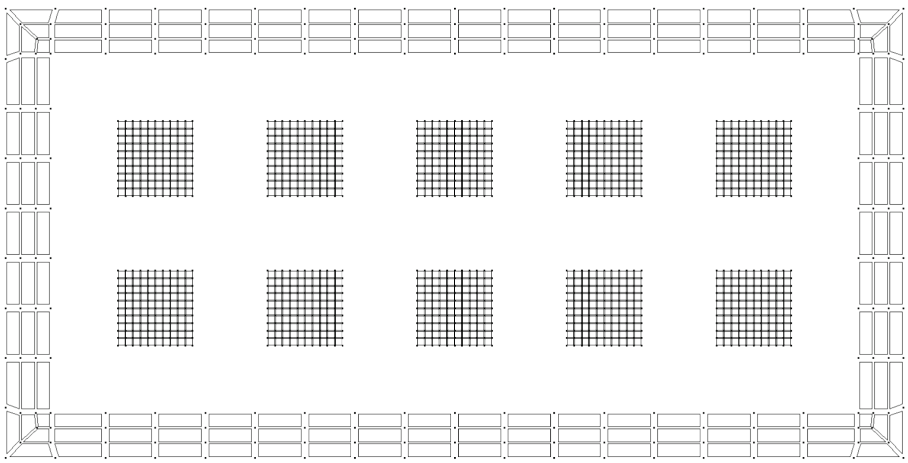
$$= \frac{2}{3}\frac{1.3(1{,}744{,}099\text{ psf})}{\left(1-0.25\right)}$$

$$= 2{,}015{,}403$$

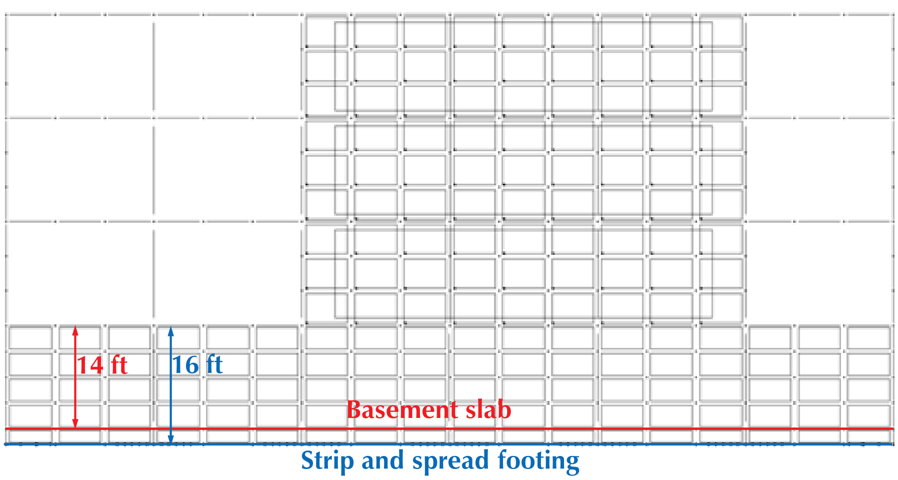
$$\frac{0.25E_cI}{L^4} = 564{,}337\text{ psf} < \frac{2}{3}k_{sv}B = 2{,}015{,}403\text{ psf}$$

Thus, the strip footing is also considered to be flexible relative to the foundation soil.

Because both the spread and strip footings are flexible relative to the foundation soil, Method 3 "Shallow Foundation Not Rigid Relative to the Soil" outlined in ASCE 41-13 § 8.4.2.5 is employed to model the foundation system. The strip and spread footings are modeled using elastic shell elements, as shown in Figure 11-25a. Additionally, a 4-inch-thick basement slab is added to the building model as a diaphragm at a depth of 14 ft from the finished grade, as shown in Figure 11-25b and Figure 11-26. The strip and spread footings are laterally connected by the basement slab.



(a) Spread and strip footings modeled by elastic shell elements



(b) Depth of basement slab and shallow foundation

Figure 11-25 Explicitly modeled spread and strip footings.

An elasto-plastic compression-only soil spring is connected to each footing node shown in Figure 11-25b. The elastic stiffness of the spring is the product of the unit subgrade spring coefficient, k_{sv}, and the tributary area of the node, A_{tr}. For example, the 10 ft by 10 ft spread footing is meshed into 100 square shell elements. The nodes that are not along the perimeter of the footing have an tributary area of 1 ft^2. Thus, the spring stiffness for such as node is calculated as

$$k_{\text{spring}} = A_{tr}\, k_{sv} = (1\ \text{ft}^2)(302{,}310\ \text{psf/ft}) = 302{,}310\ \text{lb/ft}$$

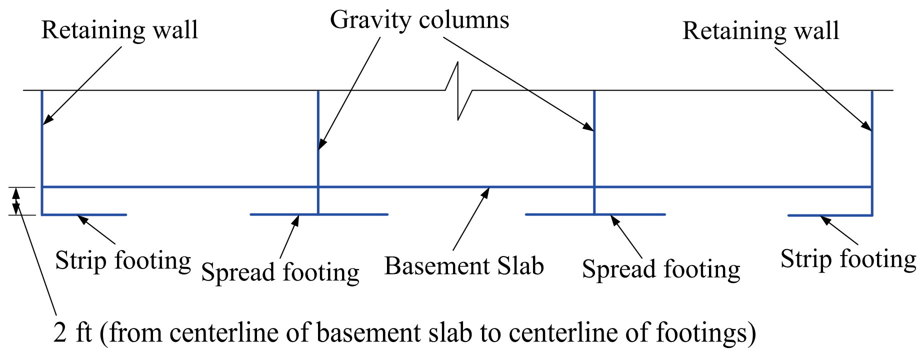


Figure 11-26 Schematic diagram for basement slab and footings of building model.

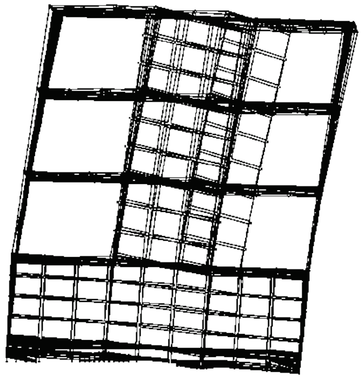
11.6.2 Modeling Foundation Capacity

The lower-bound prescriptive expected bearing capacity is determined in accordance with ASCE 41-13 § 8.4.1.1:

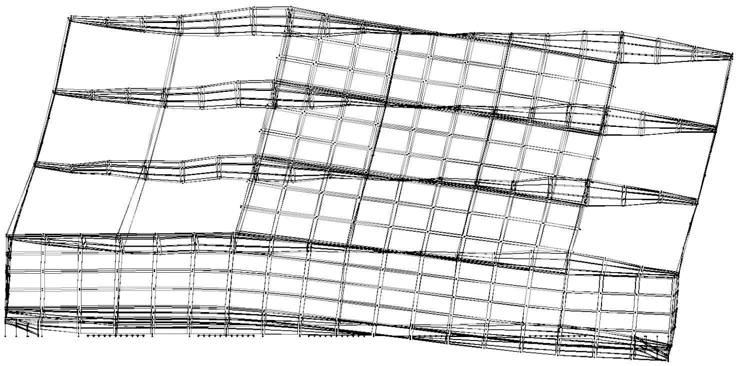
$$0.5q_c = 0.5(3q_{\text{allow}}) = 0.5[3(4{,}000\ \text{psf})] = 6{,}000\ \text{psf}$$

The value of 4,000 psf of q_{allow} described in ASCE 41-13 Section 10.5.2.2 is used here. The capacity of each soil spring is taken as $0.5q_cA_{tr}$. The factor is taken as 0.5 in accordance with ASCE 41-13 § 8.4.2 and Figure 8-1(a) for lower-bound foundation bearing capacity.

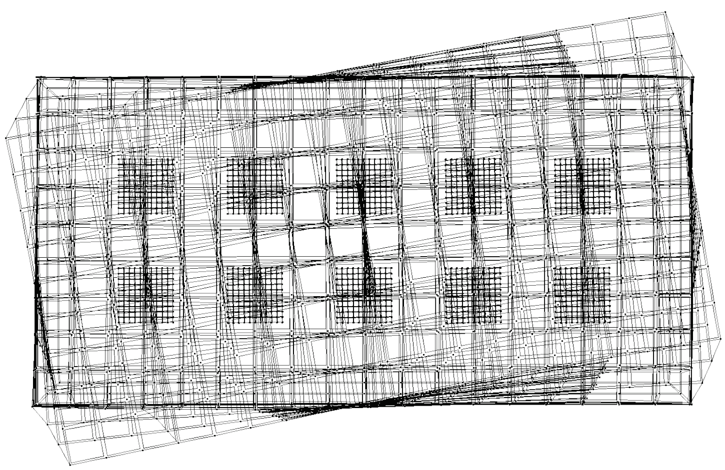
The first three mode shapes and periods of the flexible-base building model are shown in Figure 11-27. By comparing Figure 11-15 and Figure 11-27, it can be seen that the periods are elongated due to the flexible base. The period of Mode 1 (north-south vibration) increases from 0.245 s to 0.347 s; Mode 2 (torsion) of the fixed-base model becomes Mode 3 of the flexible-base model, and the period increases from 0.115 s to 0.135 s; and Mode 3 (east-west vibration) of the fixed-base model becomes Mode 2, and the period increases from 0.113 s to 0.160 s.



(a) Mode 1 (T_1 =0.347 s, translational vibration in north-south direction)



(b) Mode 2 (T_2 =0.160 s, translational vibration in east-west direction)



(c) Mode 3 (T_3 =0.135 s, torsional vibration)

Figure 11-27 First three mode shapes of building model with a flexible base.

11.7 Kinematic Interaction and Radiation Damping Soil-Structure Interaction Effects (ASCE 41-13 § 8.5)

ASCE 41-13 § 7.2.7 requires that the effects of soil-structure interaction (SSI) be evaluated for those buildings "in which an increase in fundamental period caused by SSI effects results in an increase in spectral acceleration." This will be evaluated in this section.

As indicated by ASCE 41-13 § 8.5.1, "kinematic interaction effects shall be represented by ratio of response spectra (RRS) factors RRS_{bsa} for base slab averaging, and RRS_e for embedment, which are multiplied by the spectral acceleration ordinates on the response spectrum calculated in accordance with Section 2.4. Reduction of the response spectrum for kinematic interaction effects shall be permitted subject to the limitations in Sections 8.5.1.1 and 8.5.1.2."

11.7.1 Base Slab Averaging

The area of the foundation footprint is calculated as

$$A_{\text{base}} = 60(120)\text{ ft}^2 = 7{,}200\text{ ft}^2$$

The effective foundation size in feet, b_e, is calculated as follows:

$$b_e = \sqrt{A_{\text{base}}} = \sqrt{7{,}200\text{ ft}^2} = 84.9\text{ ft} < 260\text{ ft} \qquad \text{(ASCE 41-13 Eq. 8-18)}$$

Obtained from eigenvalue analysis, the fundamental period in the north-south direction, $T_{\text{N-S}}$, is 0.347 s and in the east-west direction, $T_{\text{E-W}}$ is 0.160 s. As indicated by ASCE 41-13 § 8.5.1.1, T shall not be taken as less than 0.20 s when using ASCE 41-13 Equations 8-17 and 8-19, so $T_{\text{E-W}}$ is taken as 0.20 s.

b_0 is calculated using ASCE 41-13 Equation 8-17 as follows:

$$\begin{cases} b_{0,\text{ E-W}} = 0.0001\left(\dfrac{2\pi b_e}{T_{\text{E-W}}}\right) = 0.0001\left[\dfrac{2\pi(84.9\text{ ft})}{0.20}\right] = 0.267 < 1 \\ b_{0,\text{ N-S}} = 0.0001\left(\dfrac{2\pi b_e}{T_{\text{N-S}}}\right) = 0.0001\left[\dfrac{2\pi(84.9\text{ ft})}{0.347}\right] = 0.154 < 1 \end{cases}$$

As both $b_{0,\text{N-S}}$ and $b_{0,\text{E-W}}$ are smaller than 1, B_{bsa} is determined using ASCE 41-13 Equation 8-16 as follows:

$$\begin{cases} B_{bsa,\text{ E-W}} = 1 + b_{0,\text{ E-W}}^2 + b_{0,\text{ E-W}}^4 + \dfrac{b_{0,\text{ E-W}}^6}{2} + \dfrac{b_{0,\text{ E-W}}^8}{4} + \dfrac{b_{0,\text{ E-W}}^{10}}{12} \\ \qquad = 1 + 0.267^2 + 0.267^4 + \dfrac{0.267^6}{2} + \dfrac{0.267^8}{4} + \dfrac{0.267^{10}}{12} \\ \qquad = 1.077 \\ B_{bsa,\text{ N-S}} = 1 + b_{0,\text{ N-S}}^2 + b_{0,\text{ N-S}}^4 + \dfrac{b_{0,\text{ N-S}}^6}{2} + \dfrac{b_{0,\text{ N-S}}^8}{4} + \dfrac{b_{0,\text{ N-S}}^{10}}{12} = 1.024 \end{cases}$$

RRS_{bsa} is determined using ASCE 41-13 Equation 8-15:

$$\begin{cases} RRS_{bsa,\text{ E-W}} = 0.25 + 0.75\left\{\dfrac{1}{b_{0,\text{ E-W}}^2}\left[1 - \exp\left(-2b_{0,\text{ E-W}}^2\right)B_{bsa,\text{ E-W}}\right]\right\}^{1/2} \\ \qquad = 0.25 + 0.75\left\{\dfrac{1}{0.267^2}\left[1 - \exp\left(-2 \times 0.267^2\right)\left(1.077\right)\right]\right\}^{1/2} \\ \qquad = 0.972 \\ RRS_{bsa,\text{ N-S}} = 0.25 + 0.75\left\{\dfrac{1}{b_{0,\text{ N-S}}^2}\left[1 - \exp\left(-2b_{0,\text{ N-S}}^2\right)B_{bsa,\text{ N-S}}\right]\right\}^{1/2} \\ \qquad = 0.25 + 0.75\left\{\dfrac{1}{0.154^2}\left[1 - \exp\left(-2 \times 0.154^2\right)\left(1.024\right)\right]\right\}^{1/2} \\ \qquad = 0.996 \end{cases}$$

11.7.2 Embedment

The ratio of response spectra (*RRS*) factor for embedment, RRS_{e}, is determined per ASCE 41-13 § 8.5.1.2. Since the Site Class is D and the foundation components are assumed to be laterally connected by the basement slab, which is assumed to be connected to the top of each footing, reduction for embedment is permitted for this building. The effective shear wave velocity, v_s, is calculated as

$$v_s = nv_{s0} = \sqrt{G / G_0}\,v_{s0} = \sqrt{0.468}\left(1{,}000\text{ ft/s}\right) = 684\text{ ft/s}$$

The foundation embedment depth is taken as the distance from grade to the bottom of the footings (per Figure 11-25 and Figure 11-26, 14'-0" for the foundation story plus 2'-0" to the center of the footing plus (1'-6")/ 2 for half of the footing), totaling 16'-9". Thus, this yields:

$$e = 16\text{ ft} + 9\text{ in.} = 16.75\text{ ft}$$

Useful Tip

The foundation embedment depth is taken as the distance between ground surface and bottom surface of footings.

RRS_e is determined using ASCE 41-13 Equation 8-19 as follows:

$$\begin{cases} RRS_{e,\text{ E-W}} = 0.25 + 0.75\cos\left(\dfrac{2\pi e}{T_{\text{E-W}} v_s}\right) \\ \qquad = 0.25 + 0.75\cos\left[\dfrac{2\pi(16.75\text{ ft})}{(0.20\text{ s})(684\text{ ft/s})}\right] \\ \qquad = 0.789 > 0.5 \\ RRS_{e,\text{ N-S}} = 0.25 + 0.75\cos\left(\dfrac{2\pi e}{T_{\text{N-S}} v_s}\right) \\ \qquad = 0.25 + 0.75\cos\left[\dfrac{2\pi(16.75\text{ ft})}{(0.347\text{ s})(684\text{ ft/s})}\right] \\ \qquad = 0.928 > 0.5 \end{cases}$$

The RRS for kinematic interaction is the product of RRS_{bsa} and RRS_e and is calculated as follows. ASCE 41-13 § 8.5.1 limits RRS to a maximum of 0.5.

$$\begin{cases} RRS_{\text{E-W}} = RRS_{bsa,\text{ E-W}} \times RRS_{e,\text{ E-W}} = 0.972 \times 0.789 = 0.767 > 0.5 \\ RRS_{\text{N-S}} = RRS_{bsa,\text{ N-S}} \times RRS_{e,\text{ N-S}} = 0.996 \times 0.928 = 0.924 > 0.5 \end{cases}$$

11.7.3 Target Displacement Considering Kinematic Interaction Effects

To determine the target displacement taking into account the kinematic soil-structure interaction effects, the spectrum acceleration, S_a, for the building is multiplied by RRS to reduce the seismic demand.

$$\begin{cases} S_{a,\ RRS,\text{ E-W}} = RRS_{\text{E-W}} S_{a,\text{ E-W}} = 0.767(1.08) = 0.828 \\ S_{a,\ RRS,\text{ N-S}} = RRS_{\text{N-S}} S_{a,\text{ N-S}} = 0.924(1.08) = 0.998 \end{cases}$$

Starting from the target displacements in Table 11-9 and Table 11-11 for the fixed-base model, parameters T_e, μ_{strength}, C_0, C_1, C_2, are iterated with S_a replaced by $S_{a,RRS}$, the target displacement δ_t calculated by ASCE 41-13 Equation 7-28 converges to a final value when the difference between two iterations is less than 5%. For this example, δ_t converged after one or two iterations. The iteration process is similar to that for the fixed-base model shown in Section 11.3.5. For the BSE-2E Seismic Hazard Level:

$$\begin{cases} \delta_{t,\ RRS,\text{ E-W}} = 0.537\text{ in.} \rightarrow 0.433\text{ in.} \rightarrow 0.448\text{ in.} \\ \delta_{t,\ RRS,\text{ N-S}} = 2.264\text{ in.} \rightarrow 3.422\text{ in.} \rightarrow 3.411\text{ in.} \end{cases}$$

For the BSE-1E Seismic Hazard Level, iterations for the target displacement are:

$$\begin{cases} \delta_{t,\,RRS,\,\text{E-W}} = 0.215 \text{ in.} \rightarrow 0.241 \text{ in.} \rightarrow\ 0.226 \text{ in.} \rightarrow 0.233 \text{ in.} \\ \delta_{t,\,RRS,\,\text{N-S}} = 1.231 \text{ in.} \rightarrow 2.249 \text{ in.} \rightarrow 2.017 \text{ in.} \rightarrow 2.011 \text{ in.} \end{cases}$$

11.7.4 Foundation Damping Soil-Structure Interaction Effects

Starting from the target displacement considering kinematic interaction effects under BSE-2E Seismic Hazard Level, $\delta_{t,\,RRS,\text{E-W}}$ = 0.448 in. and $\delta_{t,RRS,\text{N-S}}$ = 3.411 in., the influence of foundation damping on the target displacement is calculated per ASCE 41-13 § 8.5.2.

The effective structure height, h, is taken as 70% of the total structure height for multistory structures per ASCE 41-13 § 8.5.2.

$$h = 0.70\ (16.75 \text{ ft} + 14 \text{ ft} + 14 \text{ ft} + 14 \text{ ft}) = 41.1 \text{ ft}$$

The equivalent foundation radius, r_x, for translation is determined using ASCE 41-13 Equation 8-28 as follows:

$$r_x = \sqrt{A_{\text{base}} / \pi} = \sqrt{7{,}200 / \pi} = 47.9 \text{ ft}$$

To determine the foundation damping caused by radiation damping, β_f, using ASCE 41-13 Equation 8-21, a number of parameters need to be calculated.

$$c_e = 1.5\ (e / r_x) + 1 = 1.5(16.75/47.9) + 1 = 1.525$$

M^*_{fixed} is the effective mass for the fundamental mode determined using the fixed-base model. For the fixed-base building model, the effective mass factors for the fundamental mode in the east-west and north-south directions calculated by PERFORM-3D® are 0.6936 and 0.6449, respectively. The total seismic weight of the structure is 3,960 kips. So the effective mass of the fundamental mode in the east-west and north-south directions is determined as follows:

$$\begin{cases} M^*_{\text{fixed, E-W}} = 0.6936\left(\dfrac{3{,}960 \text{ kips}}{32.2 \text{ ft/s}^2}\right) = 85.3 \text{ k-s}^2/\text{ft} \\ M^*_{\text{fixed, N-S}} = 0.6449\left(\dfrac{3{,}960 \text{ kips}}{32.2 \text{ ft/s}^2}\right) = 79.3 \text{ k-s}^2/\text{ft} \end{cases}$$

Alternatively, ASCE 41-13 Equation 8-26 can be used to determine the effective modal mass.

The effective translation stiffness K^*_{fixed} of the fixed-base model is calculated using ASCE 41-13 Equation 8-25 as follows:

$$\begin{cases} K^*_{\text{fixed, E-W}} = M^*_{\text{fixed, E-W}} \left(\dfrac{2\pi}{T_{\text{fixed, E-W}}}\right)^2 \\ \qquad = \left(85.3 \text{ kip-s}^2/\text{ft}\right)\left(\dfrac{2\pi}{0.11 \text{ s}}\right)^2 \\ \qquad = 278{,}307 \text{ kip/ft} \\ K^*_{\text{fixed, N-S}} = M^*_{\text{fixed, N-S}} \left(\dfrac{2\pi}{T_{\text{fixed, N-S}}}\right)^2 \\ \qquad = \left(79.3 \text{ kip-s}^2/\text{ft}\right)\left(\dfrac{2\pi}{0.25 \text{ s}}\right)^2 \\ \qquad = 50{,}090 \text{ kip/ft} \end{cases}$$

ASCE 41-13 Equation 8-29 requires the lengthening ratio for the initial elastic fundamental period. The lengthening ratio is calculated as follows:

$$\begin{cases} \left(\dfrac{\tilde{T}}{T}\right)_{\text{E-W}} = \dfrac{0.16}{0.11} = 1.455 \\ \left(\dfrac{\tilde{T}}{T}\right)_{\text{N-S}} = \dfrac{0.35}{0.25} = 1.400 \end{cases}$$

where $\tilde{T}$ is the initial elastic fundamental period of the flexible-base model in seconds and T is the initial elastic fundamental period of the fixed-base model in seconds. In the background research for determining foundation damping reported by FEMA 440, a single-degree-of-freedom model with a pair of translational spring and dashpot and another pair of rotational spring and dashpot at the base are used to determine the foundation damping soil-structure interaction effect. For the translational spring, the effective translational stiffness of the foundation, K_x, is calculated using ASCE 41-13 Equation 8-27 as follows:

$$K_x = \frac{8}{2-\nu} G r_x = \frac{8}{2-0.25}\left(1744.1 \text{ ksf}\right)\left(47.9 \text{ ft}\right) = 381{,}908 \text{ kip/ft}$$

where:

v = Poisson's ratio of the foundation soil (the value of 0.25 is consistent with the value used in the evaluation of foundation spring stiffness)

G = effective shear modulus

r_x = equivalent foundation radius for translation

For the rotational spring of the single-degree-of-freedom model used to determine the foundation damping soil-structure interaction effect, the effective rotational stiffness K_θ is calculated using ASCE 41-13 Equation 8-24:

$$\begin{cases} K_{\theta,\text{ E-W}} = \dfrac{K^*_{\text{fixed, E-W}}\left(h\right)^2}{\left(\dfrac{\tilde{T}}{T}\right)^2_{\text{E-W}} - 1 - \dfrac{K^*_{\text{fixed, E-W}}}{K_x}} \\ \quad = \dfrac{\left(278{,}307 \text{ kip/ft}\right)\left(41.1 \text{ ft}\right)^2}{1.455^2 - 1 - \dfrac{278{,}307 \text{ kip/ft}}{381{,}908 \text{ kip/ft}}} \\ \quad = 1.211\times10^9 \text{ kip-ft/rad} \\ K_{\theta,\text{ N-S}} = \dfrac{K^*_{\text{fixed, N-S}}\left(h\right)^2}{\left(\dfrac{\tilde{T}}{T}\right)^2_{\text{N-S}} - 1 - \dfrac{K^*_{\text{fixed, N-S}}}{K_x}} \\ \quad = \dfrac{\left(50{,}090 \text{ kip/ft}\right)\left(41.1 \text{ ft}\right)^2}{1.400^2 - 1 - \dfrac{50{,}090 \text{ kip/ft}}{381{,}908 \text{ kip/ft}}} \\ \quad = 1.021\times10^8 \text{ kip-ft/rad} \end{cases}$$

The equivalent foundation radius for rotation, r_θ, is calculated using ASCE 41-13 Equation 8-23:

$$\begin{cases} r_{\theta,\text{ E-W}} = \left[\dfrac{3\left(1-\nu\right)K_{\theta,\text{ E-W}}}{8G}\right]^{1/3} \\ \quad = \left[\dfrac{3\left(1-0.25\right)\left(1.211\times10^9 \text{ kip-ft/rad}\right)}{8(1744.1 \text{ ksf})}\right]^{1/3} \\ \quad = 58.02 \text{ ft} \\ r_{\theta,\text{ N-S}} = \left[\dfrac{3\left(1-\nu\right)K_{\theta,\text{ N-S}}}{8G}\right]^{1/3} \\ \quad = \left[\dfrac{3\left(1-0.25\right)\left(1.021\times10^8 \text{ kip-ft/rad}\right)}{8(1744.1 \text{ ksf})}\right]^{1/3} \\ \quad = 25.44 \text{ ft} \end{cases}$$

The a_1 and a_2 factors in ASCE 41-13 Equation 8-21 are determined as follows:

$$\begin{cases} a_{1,\text{ E-W}} = c_e \exp\left(4.7 - 1.6\dfrac{h}{r_{\theta,\text{ E-W}}}\right) \\ \qquad = 1.525 \exp\left(4.7 - 1.6\dfrac{41.1\text{ ft}}{58.02\text{ ft}}\right) \\ \qquad = 53.98 \\ a_{1,\text{ N-S}} = c_e \exp\left(4.7 - 1.6\dfrac{h}{r_{\theta,\text{ N-S}}}\right) \\ \qquad = 1.525 \exp\left(4.7 - 1.6\dfrac{41.1\text{ ft}}{25.44\text{ ft}}\right) \\ \qquad = 12.64 \end{cases}$$

$$\begin{cases} a_{2,\text{ E-W}} = c_e \left[25\ln\left(\dfrac{h}{r_{\theta,\text{ E-W}}}\right) - 16\right] \\ \qquad = 1.525\left[25\ln\left(\dfrac{41.1}{58.02}\right) - 16\right] \\ \qquad = -37.54 \\ a_{2,\text{ N-S}} = c_e \left[25\ln\left(\dfrac{h}{r_{\theta,\text{ N-S}}}\right) - 16\right] \\ \qquad = 1.525\left[25\ln\left(\dfrac{41.1}{25.44}\right) - 16\right] \\ \qquad = -6.11 \end{cases}$$

The effective period lengthening ratio is determined as follows:

$$\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}} = \left\{1 + \frac{1}{\mu}\left[\left(\frac{\tilde{T}}{T}\right)^2 - 1\right]\right\}^{0.5} \qquad \text{(ASCE 41-13 Eq. 8-29)}$$

where μ = expected ductility demand. For nonlinear procedures, μ is the maximum displacement divided by the yield displacement (δ_t / Δ_y for NSP). For the analysis under the BSE-2E Seismic Hazard Level, at the target displacement considering flexible base and kinematic soil-structure interaction ($\delta_{t,RRS,\text{E-W}}$ = 0.448 in. and $\delta_{t,RRS,\text{N-S}}$ = 3.411 in.):

$$\begin{cases} \mu_{RRS,\text{E-W}} = \dfrac{\delta_{t,RRS,\text{E-W}}}{\Delta_{y,RRS,\text{E-W}}} = \dfrac{0.448\text{ in.}}{0.153\text{ in.}} = 2.928 \\ \mu_{RRS,\text{N-S}} = \dfrac{\delta_{t,RRS,\text{N-S}}}{\Delta_{y,RRS,\text{N-S}}} = \dfrac{3.411\text{ in.}}{1.253\text{ in.}} = 2.722 \end{cases}$$

$\Delta_{t,RRS,\text{E-W}}$ and $\Delta_{t,RRS,\text{N-S}}$ are determined using the same procedure described in Section 11.3.1 but based on the base shear vs. top drift curve obtained from the structural model with flexible foundations. The $\Delta_{y,\text{ E-W}}$ for the BSE-2E level east-west analysis of the fixed-base model is 0.084 in. As expected, the $\Delta_{y,\text{ E-W}}$ of 0.084 in. is smaller than the $\Delta_{\text{y}, RRS, \text{ E-W}}$ of 0.153 in for the flexible-base model. Please note than the Δ_y of 0.27 in. shown in Table 11-3 is for the preliminary east-west analysis result rather than the final analysis result.

Substituting μ into ASCE 41-13 Equation 8-29, the effective period lengthening ratio is calculated as follows:

$$\begin{aligned}
\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{E-W}} &= \left\{1+\frac{1}{\mu_{RRS,\text{E-W}}}\left[\left(\frac{\tilde{T}}{T}\right)^2_{\text{E-W}}-1\right]\right\}^{0.5} \\
&= \left\{1+\frac{1}{2.928}\left[1.455^2-1\right]\right\}^{0.5} \\
&= 1.175
\end{aligned}$$

$$\begin{aligned}
\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{N-S}} &= \left\{1+\frac{1}{\mu_{RRS,\text{N-S}}}\left[\left(\frac{\tilde{T}}{T}\right)^2_{\text{N-S}}-1\right]\right\}^{0.5} \\
&= \left\{1+\frac{1}{2.722}\left[1.400^2-1\right]\right\}^{0.5} \\
&= 1.163
\end{aligned}$$

The foundation damping caused by radiation damping, β_f, in percent is calculated in accordance with ASCE 41-13 Equation 8-21 as follows:

$$\begin{cases}
\beta_{f,\text{E-W}} = a_{1,\text{E-W}}\left[\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{E-W}}-1\right]+a_{2,\text{E-W}}\left[\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{E-W}}-1\right]^2 \\
\quad = 53.98(1.175-1)-37.54(1.175-1)^2 \\
\quad = 8.297(\%) \\
\beta_{f,\text{N-S}} = a_{1,\text{N-S}}\left[\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{N-S}}-1\right]+a_{2,\text{N-S}}\left[\left(\frac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)_{\text{N-S}}-1\right]^2 \\
\quad = 12.64(1.163-1)-6.11(1.163-1)^2 \\
\quad = 1.898(\%)
\end{cases}$$

The effective damping of the structure-foundation system in percent is calculated in accordance with ASCE 41-13 Equation 8-20:

$$\begin{cases} \beta_{0,\text{E-W}} = \beta_{f,\text{E-W}} + \dfrac{\beta}{\left(\dfrac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)^3_{\text{E-W}}} \\ \quad = 8.297 + \dfrac{5}{(1.175)^3} \\ \quad = 11.379(\%) > \beta = 5\% \\ \beta_{0,\text{N-S}} = \beta_{f,\text{N-S}} + \dfrac{\beta}{\left(\dfrac{\tilde{T}_{\text{eff}}}{T_{\text{eff}}}\right)^3_{\text{N-S}}} \\ \quad = 1.898 + \dfrac{5}{(1.163)^3} \\ \quad = 5.077(\%) > \beta = 5\% \end{cases}$$

Per ASCE 41-1 3§ 2.4.1.7, the response spectrum ordinate for 5% viscous damping is adjusted by the B_1 factor. The B_1 factor is calculated using ASCE 41-13 Equation 2-11 as follows:

$$\begin{cases} B_{1,\text{E-W}} = \dfrac{4}{5.6 - \ln \beta_{0,\text{E-W}}} \\ \quad = \dfrac{4}{5.6 - \ln(100 \times 0.11379)} \\ \quad = 1.263 \\ B_{1,\text{N-S}} = \dfrac{4}{5.6 - \ln \beta_{0,\text{N-S}}} \\ \quad = \dfrac{4}{5.6 - \ln(100 \times 0.05077)} \\ \quad = 1.006 \end{cases}$$

For the BSE-2E Seismic Hazard Level, S_{XS} =1.080 and S_{X1} =0.624. In accordance with ASCE 41-13 Equation 2-9:

$$T_S = \frac{S_{X1}}{S_{XS}} = \frac{0.624}{1.080}\text{ s} = 0.578\text{s}$$

At the target displacement considering flexible base and kinematic soil-structure interaction ($\delta_{t,RRS,\text{E-W}}$ = 0.448 in. and $\delta_{t,RRS,\text{N-S}}$ = 3.411 in.):

$$\begin{cases} T_{\text{eff}}\Big|_{\delta_{t,RRS,\text{E-W}}} = 0.166\text{ s} < T_S = 0.578\text{ s} \\ T_{\text{eff}}\Big|_{\delta_{t,RRS,\text{N-S}}} = 0.479\text{ s} < T_S = 0.578\text{ s} \end{cases}$$

Therefore, per ASCE 41-13 Equation 2-6:

$$\begin{cases} S_{a,\text{ E-W}} = S_{XS} / B_{1,\text{ E-W}} = 1.080 / 1.263 = 0.855 \\ S_{a,\text{ N-S}} = S_{XS} / B_{1,\text{ N-S}} = 1.080 / 1.006 = 1.074 \end{cases}$$

Considering both the kinematic and foundation soil-structure interaction effects, the spectrum acceleration is determined as:

$$\begin{cases} S_{a,\,SSI,\text{ E-W}} = RRS_{\text{E-W}} S_{a,\text{ E-W}} = 0.767\left(0.855\right) = 0.656 \\ S_{a,\,SSI,\text{ N-S}} = RRS_{\text{N-S}} S_{a,\text{ N-S}} = 0.924\left(1.074\right) = 0.992 \end{cases}$$

Starting from the target displacements in Section 11.7.3, iterate on parameters including μ_{strength}, T_e, C_0, C_1, C_2, $\widetilde{T}_{\text{eff}} / T_{\text{eff}}$, B_1 and S_a, the target displacement δ_t calculated by ASCE 41-13 Equation 7-28 converges to a final value when the difference between two iterations is less than 5%. For this example, δ_t converged after two iterations:

$$\begin{cases} \delta_{t,\,SSI,\text{ E-W}} = 0.448 \text{ in.} \rightarrow \ 0.289 \text{ in.} \rightarrow 0.278 \text{ in.} \\ \delta_{t,\,SSI,\text{ N-S}} = 3.411 \text{ in.} \rightarrow \ 3.387 \text{ in.} \end{cases}$$

For the BSE-1E Seismic Hazard Level, iterations for the target displacement are:

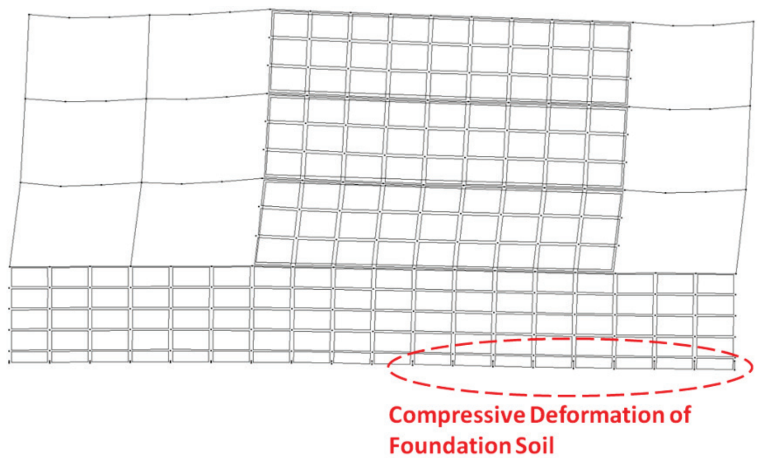
$$\begin{cases} \delta_{t,\,RRS,\text{ E-W}} = 0.233 \text{ in.} \rightarrow \ 0.133 \text{ in.} \rightarrow \ 0.136 \text{ in.} \\ \delta_{t,\,RRS,\text{ N-S}} = 2.011 \text{ in.} \rightarrow \ 1.987 \text{ in.} \end{cases}$$

Table 11-17 shows the target displacements determined using models considering different boundary conditions and soil-structure interaction effects.

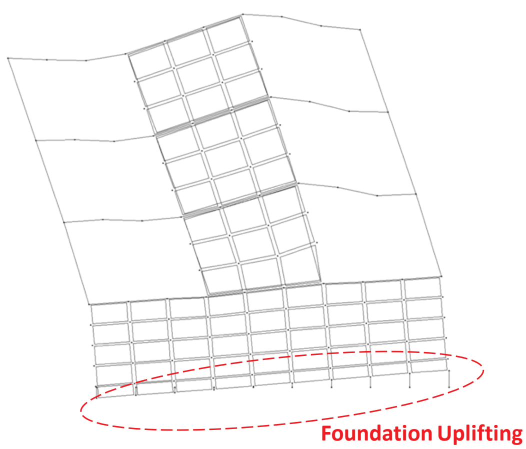
Figure 11-28 shows the deformed shape of Walls 1 and G at the target displacements in the east-west and north-south directions. The foundation deformation including uplifting can be seen in the figure.

Table 11-17 Target Displacement Determined Based on Different Models and Boundary Conditions

Seismic hazard level	Direction	Target displacement (in.)		
		Fixed base without SSI	Flexible base + Kinematic Interaction	Flexible base + Kinematic Interaction + Foundation Damping
BSE-2E	E-W	0.537	0.448	0.278
	N-S	2.264	3.411	3.387
BSE-1E	E-W	0.215	0.233	0.136
	N-S	1.231	2.011	1.987



(a) Deformed shape of Wall 1 at target displacement (magnified by 100 times)



(b) Deformed shape of Wall G at target displacement (magnified by 50 times)

Figure 11-28 Deformed wall shape at target displacement considering flexible base and soil-structure interaction effects (BSE-2E Seismic Hazard Level).

11.7.5 Shear Wall Performance Evaluated Using Flexible-Base Building Model Considering Soil-Structure Interaction Effects

The shear wall responses at the target displacements $\delta_{t,SSI,\text{E-W}}$ and $\delta_{t,SSI,\text{N-S}}$ are listed in Table 11-18 through Table 11-21. For the force-controlled Walls 1 and 4 and deformation-controlled Wall D, the ratio of maximum force or moment to Q_{CL} considering flexible base and soil-structure interaction effects are lower than the equivalent acceptance ratios determined using the fixed-base model. For the deformation-controlled Walls A and G, the acceptance ratios between total shear strain or plastic rotation and acceptance criteria considering flexible base and soil-structure interaction effects are higher than the equivalent fixed-base ratios. All five shear walls meet the CP acceptance criteria and the BSE-2E Seismic Hazard Level and the LS acceptance criteria at the BSE-1E Seismic Hazard Level. This example illustrates that ignoring soil-structure interaction effects is not always conservative.

Table 11-18 Force Demand and Capacity Acceptance Ratios of Walls 1 and 4 Subjected to the BSE-2E Seismic Hazard Level (Force-Controlled Component with Lower-Bound Strength Properties)

Wall location	Level	Action	Max. force/moment (k or k-ft)	Q_{CL} (k or k-ft)	Max. force or moment / Q_{CL}	
					Flexible base+SSI	Fixed base
Grid 1	1	Shear*	649	838	0.77	0.93
		Flexure	19,333	52,440	0.37	0.46
Grid 4	1	Shear*	651	838	0.78	0.93
		Flexure	19,500	52,440	0.37	0.46

* Controlling failure mode

Table 11-19 Force Demand and Capacity Acceptance Ratios of Walls 1 and 4 Subjected to the BSE-1E Seismic Hazard Level (Force-Controlled Component with Lower-Bound Strength Properties)

Wall location	Level	Action	Max. force/moment (k or k-ft)	Q_{CL} (k or k-ft)	Max. force or moment / Q_{CL}	
					Flexible base+SSI	Fixed base
Grid 1	1	Shear*	486	838	0.58	0.78
		Flexure	14,208	52,440	0.27	0.38
Grid 4	1	Shear*	491	838	0.59	0.78
		Flexure	14,417	52,440	0.27	0.38

* Controlling failure mode

Table 11-20 Deformation Demand and Capacity Acceptance Ratios of Walls A, D, and G Subjected to the BSE-2E Seismic Hazard Level (Deformation-Controlled Components with Expected Strength Properties)

Wall location	Level with max. def.	Action	Total shear strain or rotation (unitless/rad)	Elastic rotation limit (rad)	Plastic rotation (rad)	Acceptable shear strain or rotation CP	Deformation acceptance at CP	
							Flexible base+SSI	Fixed base
Grid A	1	Shear	1.62E-03	N/A	N/A	0.02	0.081	0.04
	1	Flexure*	5.65E-03	3.09E-03	2.56E-03	0.02	0.128	0.06
Grid D	2	Shear*	8.40E-04	N/A	N/A	0.02	0.042	0.15
	1	Flexure	4.05E-04	1.50E-03	Elastic	0.015	Elastic	Elastic
Grid G	1	Shear*	1.73E-03	N/A	N/A	0.02	0.087	0.08
	1	Flexure	3.09E-03	1.89E-03	1.20E-03	0.02	0.060	0.06

* Controlling action

Note: Deformation acceptance ratio at CP =Total shear strain or plastic rotation / acceptance criterion at CP.

Table 11-21 Deformation Demand and Capacity of Walls A, D, and G Subjected to BSE-1E Seismic Loads (Deformation-Controlled Components with Expected Strength Properties)

Wall location	Level with max. def.	Action	Total shear strain or rotation (unitless/rad)	Elastic rotation limit (rad)	Plastic rotation (rad)	Acceptable shear strain or rotation LS	Deformation acceptance at LS	
							Flexible base+SSI	Fixed base
Grid A	1	Shear	6.94E-04	N/A	N/A	0.015	0.046	0.02
	1	Flexure*	3.31E-03	3.09E-03	2.2E-04	0.015	0.015	Elastic
Grid D	2	Shear*	3.55E-04	N/A	N/A	0.015	0.024	0.08
	1	Flexure	4.01E-04	1.50E-03	Elastic	0.008	Elastic	Elastic
Grid G	1	Shear*	6.91E-04	N/A	N/A	0.015	0.046	0.03
	1	Flexure	1.66E-03	1.89E-03	Elastic	0.015	Elastic	Elastic

* Controlling action

Note: Deformation acceptance ratio at LS =Total shear strain or plastic rotation / acceptance criterion at LS.

Chapter 12

Unreinforced Masonry Bearing Wall (URM) with Special Procedure

12.1 Overview

Example Summary

Building Type: URM

Performance Objective: Reduced Performance Objective

Risk Category: II

Location: Los Angeles, California

Level of Seismicity: High

Analysis Procedure: Special Procedure

Retrofit Procedure: Special Procedure

Reference Documents:

ASCE 41-13

ACI 318-11

NDS-2012

TMS 402-11

This chapter provides discussion and example application of the evaluation and retrofit of an unreinforced masonry bearing wall (URM) building using the Special Procedure presented in ASCE 41-13 Chapter 15. Although new construction of unreinforced masonry (URM) buildings is prohibited except in regions with very low seismic hazard, an extensive inventory of existing buildings across the United States relies on unreinforced masonry bearing walls for seismic resistance. Existing URM buildings vary in size, age, and occupancy from monumental government offices to small homes and have been subject to varying levels of seismic evaluation and retrofit.

Unreinforced masonry bearing walls have historically been built using clay brick, hollow concrete block, stone, or adobe (unfired clay brick). The most common unreinforced masonry construction consists of two-wythe to four-wythe clay brick exterior walls between one and five stories tall, with wood frame interior partitions and wood frame floors and roof (although concrete floor diaphragms have been used in some larger buildings). It is often possible to distinguish structural masonry walls from exterior veneer construction by the presence of header courses oriented perpendicular to the length of the wall that help connect the wythes.

Characteristic seismic deficiencies in unreinforced masonry buildings include unbraced masonry parapets susceptible to toppling, missing or inadequate wall-to-diaphragm ties, inadequate wall strength to resist either in-plane shear or out-of-plane seismic demands, and inadequate diaphragm strength and stiffness. These deficiencies led the State of California to prohibit the construction of new URM buildings with the passage of the Riley Act, following observations of extensive damage in the 1933 Long Beach Earthquake. However, construction of new URM buildings in regions of moderate to high seismicity continued in other states until the 1990s.

Seismic evaluation of unreinforced masonry bearing wall buildings was codified following extensive research by the ABK Joint Venture (ABK, 1984) conducted in Southern California. The most recent versions of the resulting evaluation procedure are included in Appendix Chapter A1 of the *2015 International Existing Building Code* (ICC, 2015b) and as a Special Procedure in ASCE 41-13 § 15.2. ASCE 41-13 additionally includes a Tier 3 evaluation procedure for unreinforced masonry bearing walls in ASCE 41-13 Chapter 11, which is illustrated in Chapter 13 of this *Guide*.

The example in this chapter illustrates the evaluation and retrofit of an unreinforced masonry bearing wall building using the Special Procedure. Per ASCE 41-13 § 15.2.1, the Special Procedure is consistent with other Tier 2 deficiency-based procedures and is permitted for use with the Reduced Performance Objective for Collapse Prevention Performance (S-5) at the BSE-1E hazard level. The Special Procedure is a stand-alone method, does not reference other procedures in ASCE 41-13, and has not been calibrated to ensure consistent results with the rest of ASCE 41-13. The example calculations for the Special Procedure contained in this chapter are intended to be representative of the evaluation for a typical URM building. Calculations are shown for representative building components and common deficiencies. The example demonstrates the following calculations:

- Calculation of building dead loads, live loads, and seismic weight
- Calculation of spectral acceleration parameters for use with the Special Procedure (ASCE 41-13 § 15.2.1) and Tier 1 screening (ASCE 41-13 § 4.1.2)
- Tier 1 screening (ASCE 41-13 § 4.4 and § 4.5)
- Condition assessment of existing materials (ASCE 41-13 § 15.2.2.1)
- Procedure and calculations for in-place shear testing of masonry walls (ASCE 41-13 § 15.2.2.2)
- Assessment of existing masonry shear strength (ASCE 41-13 § 15.2.2.3)
- Evaluation of existing floor and roof diaphragms (ASCE 41-13 § 15.2.3.2)
- Calculation of in-plane demands on existing masonry walls (ASCE 41-13 § 15.2.3.3.1)
- Evaluation of in-plane capacity of existing masonry walls (ASCE 41-13 § 15.2.3.3.2 and § 15.2.3.3.3) and retrofit for a deficient wall
- Evaluation of existing masonry walls for out-of-plane seismic demands (ASCE 41-13 § 15.2.3.4)

- Retrofit of deficient wall-to-diaphragm shear connection (ASCE 41-13 § 15.2.3.2.6)
- Retrofit of deficient wall-to-diaphragm tension anchorage (ASCE 41-13 § 15.2.3.5)

12.2 Building Description

The building archetype in this example is representative of small commercial buildings commonly constructed on the West Coast from the 1850s to the 1930s. The structure is based on an example originally developed for the 2009 *IEBC SEAOC Structural/Seismic Design Manual* (SEAOC, 2012).

The example building is a two-story, unreinforced masonry bearing wall office building located in Los Angeles with an approximately 30-foot by 60-foot floor area as shown in Figure 12-1 through Figure 12-4. The example building does not represent a specific structure, but it is consistent with prevalent building configurations and has an assumed construction date of 1920.

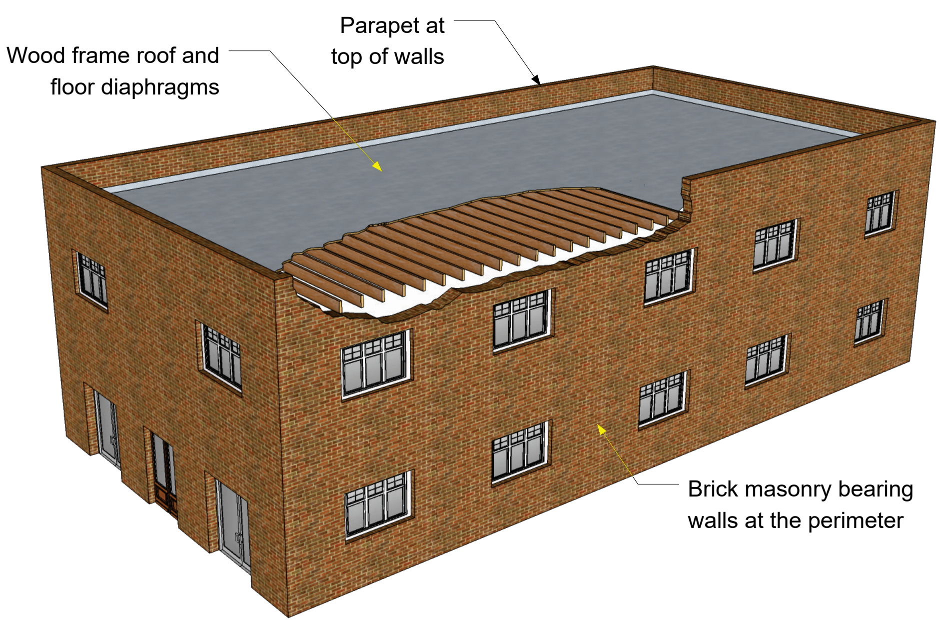


Figure 12-1 Figure showing exterior of example building.

The structural system of the building consists of a wood-frame roof and second floor that are supported by perimeter unreinforced masonry walls and interior wood stud walls. The roof is constructed with 1× straight sheathing over 2×12 wood joists at 24 inches o.c., and the roof covering is applied directly to the straight sheathing. The second floor is constructed with hardwood flooring and 1× straight sheathing over 2×12 wood joists at 16 inches o.c. The 2×12 roof and floor joists were measured at 1-5/8 inches ×

11-1/2 inches, and typical of older buildings, are larger than the current size of 1-1/2 inches by 11-1/4 inches. Both the roof and second floor diaphragms can be treated as flexible. The first floor is a concrete slab-on-grade. The unreinforced masonry walls are located around the exterior of the building, and they measure 13 inches thick at the first and second stories and 9 inches thick at the parapet. There are bearing and nonbearing wood stud walls located on the interior. These wood walls are covered with plaster over wood lath on both sides. The building is founded on continuous concrete strip footings.

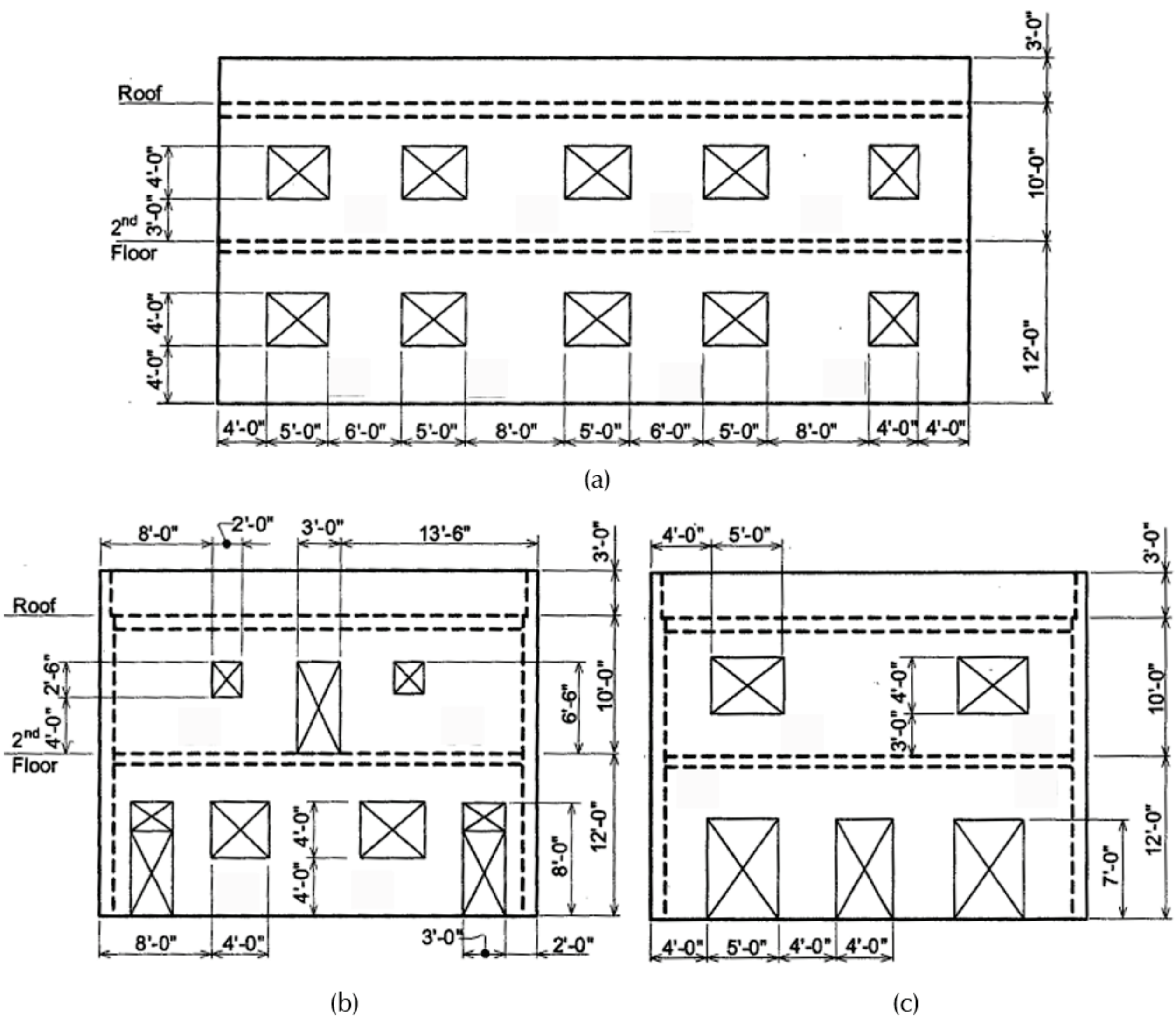


Figure 12-2 Figure showing (a) side wall elevation, (b) rear (north) wall elevation, and (c) front (south) wall elevation.

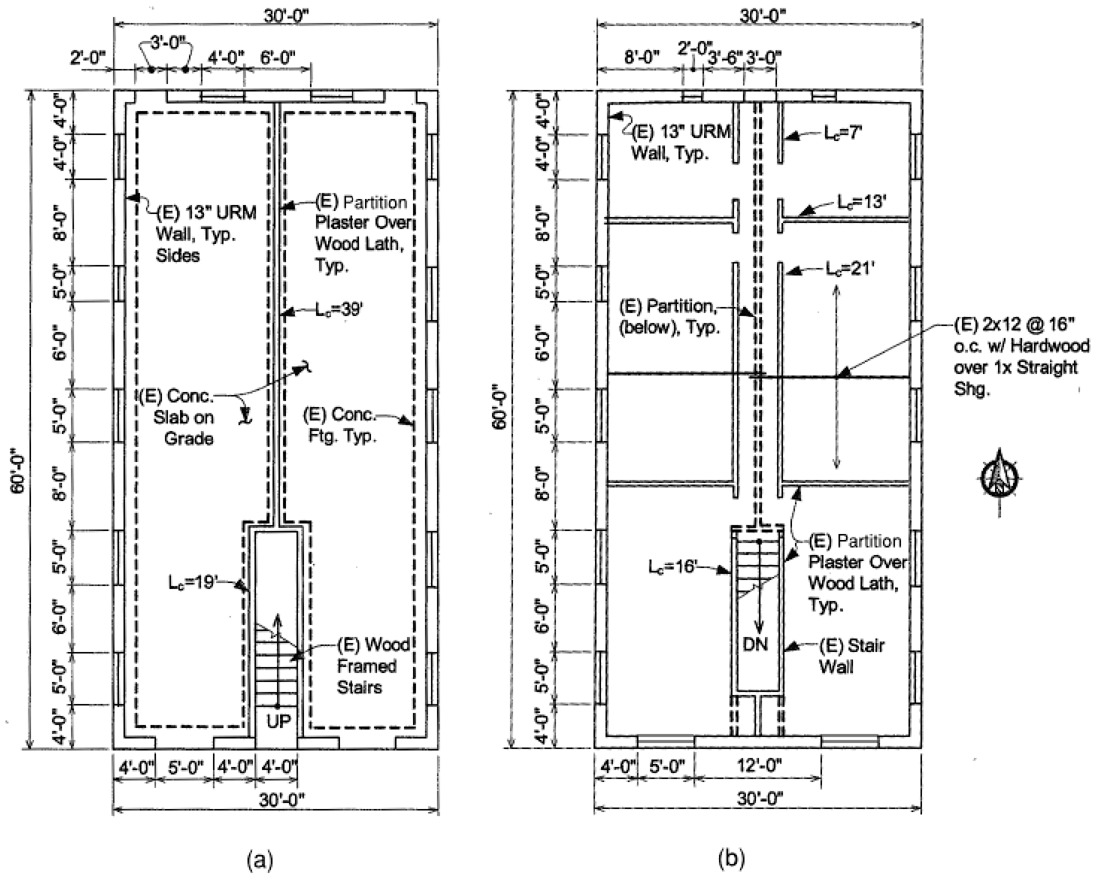


Figure 12-3 Plans for (a) first floor and (b) second floor.

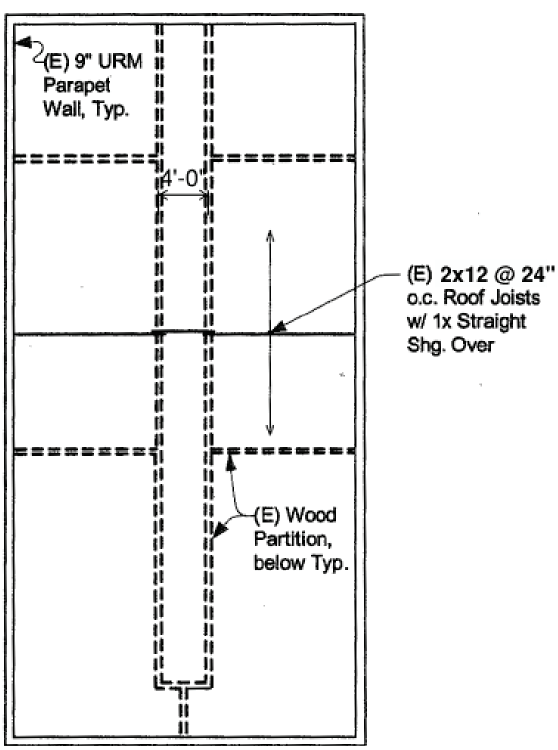


Figure 12-4 Roof plan.

12.3 Dead Loads and Seismic Weight

The flat load tables provided in Table 12-1 and Table 12-2 describe the dead loads and seismic weight tributary to the building flexible diaphragms. Weight of the perimeter masonry walls is treated separately and is not included in the flat loads.

Table 12-1 Flat Loads on Roof

Component	Dead Load (lb/ft^2)	Seismic Weight (lb/ft^2)
Roofing (3 layers rolled roofing)	3.0	3.0
Sheathing (nominal 1×)	2.5	2.5
Framing (2×12 at 24 inches on center)	2.3	2.3
Ceiling (1-inch plaster on wood lath)	10.0	10.0
Partitions	0.0	5.0
MEP/miscellaneous components	1.2	1.2
Total	19.0	24.0

Table 12-2 Flat Loads on Second Floor

Component	Dead Load (lb/ft^2)	Seismic Weight (lb/ft^2)
Flooring (1-inch hardwood)	4.0	4.0
Sheathing (nominal 1× underlayment)	2.5	2.5
Framing (2×12 at 16 inches on center)	3.4	3.4
Ceiling (1-inch plaster on wood lath)	10.0	10.0
Partitions	10.0	8.0
MEP/miscellaneous components	0.6	0.6
Total	30.5	28.5

For the majority of components, the dead load is equal to the seismic weight. Exceptions can occur for components that span vertically between floors (but are simplified as distributed loads in the analysis). For this building, based on an actual load takeoff, there are approximately 10 psf of partitions per of floor area at the second story and approximately 6 psf of partitions at the first story. Note that the ASCE 41-13 § 15.2 Special Procedure and Chapter 7 do not have any requirements like ASCE 7-10 (ASCE, 2010) for use of a minimum partition load in the dead load determination. Since the partitions in the upper levels of the building are generally not continuous to the ground level, the partition load is included in flat load table as dead load for each level that supports the partitions. In other URM buildings, where partitions are continuous, they may not contribute to the dead load that is imposed on the roof or upper floor framing; however, their seismic weight is used in the calculation of wall forces orthogonal to the partitions. In this building, the dead load is carried by the level below the partitions, so there is no partition dead load at the roof. The seismic weight is based on contributions midway above and below the story, so the seismic weight of partitions at the second floor is calculated as 0.5(6 psf + 10 psf) = 8 psf.

The dead load and seismic weight of perimeter masonry walls are shown in Table 12-3. The clay brick used in the structure has a density of 120 lb/ft^3. Weights are shown per square foot of wall area.

Table 12-3 Flat Loads of Perimeter Masonry Walls

Material	Dead Load or Seismic Weight (lb/ft^2)
9-inch thick clay brick at parapet	90
13-inch thick clay brick at first and second stories	130

Seismic weight at the roof level:

A_{Roof} = (30 ft)(60 ft) = 1,800 ft^2

$W_{\text{Diaphragm}}$ = (24 lb/ft^2)(1,800 ft)(1 kip/1,000 lb) = 43 kips

$W_{\text{Side Walls}}$ = Total seismic weight of parapet and second story walls minus wall openings at each side. Half of the height of the second story wall is tributary to the roof level.

= 2{(90 lb/ft^2)(3 ft)(60 ft) + (130 lb/ft^2)[(5 ft)(60 ft) – (2 ft)(24 ft)]}(1 kip/1,000 lb)

= 98 kips

$W_{\text{Rear Wall}}$ = (90 lb/ft^2)(3 ft)(30 ft) + (130 lb/ft^2)[(5 ft)(30 ft) – (1.5 ft)(7 ft)](1 kip/1,000 lb)

= 26 kips

$W_{\text{Front Wall}}$ = (90 lb/ft^2)(3 ft)(30 ft) + (130 lb/ft^2)[(5 ft)(30 ft) – (2 ft)(10 ft)](1 kip/1,000 lb)

= 25 kips

Seismic weight at the second floor:

$A_{\text{Second Floor}}$ = (30 ft)(60 ft) = 1,800 ft^2

$W_{\text{Diaphragm}}$ = (28.5 lb/ft^2)(1,800 ft)(1 kip/1,000 lb) = 51 kips

$W_{\text{Side Walls}}$ = Total seismic weight of first story and second story walls tributary to the second floor diaphragm minus wall openings at each side. Half of the height of the second story wall and half of the height of first story wall is tributary to the second floor.

$$= 2\{(130 \text{ lb/ft}^2)[(5 \text{ ft})(60 \text{ ft}) - (2 \text{ ft})(24 \text{ ft})] + (130 \text{ lb/ft}^2)[(6 \text{ ft})(60 \text{ ft}) - (2 \text{ ft})(24 \text{ ft})]\}(1 \text{ kip/1,000 lb})$$

$$= 147 \text{ kips}$$

$$W_{\text{Rear Wall}} = (130 \text{ lb/ft}^2)[(5 \text{ ft})(30 \text{ ft}) - (1 \text{ ft})(4 \text{ ft}) - (3 \text{ ft})(5 \text{ ft})] + (130 \text{ lb/ft}^2)[(6 \text{ ft})(30 \text{ ft}) - (2 \text{ ft})(14 \text{ ft})](1 \text{ kip/1,000 lb})$$

$$= 37 \text{ kips}$$

$$W_{\text{Front Wall}} = (130 \text{ lb/ft}^2)[(5 \text{ ft})(30 \text{ ft}) - (2 \text{ ft})(5 \text{ ft})] + (130 \text{ lb/ft}^2)[(6 \text{ ft})(30 \text{ ft}) - (1 \text{ ft})(14 \text{ ft})](1 \text{ kip/1,000 lb})$$

$$= 40 \text{ kips}$$

The total seismic weight at each floor level is calculated below and summarized in Table 12-4. Note that for simplicity, diaphragm weights conservatively used the full building width and length. A more accurate takeoff would subtract the wall thickness.

Table 12-4 Seismic Weight Summary

Component	Roof (kips)	Second Floor (kips)
Diaphragm	43	51
Side walls (total for east and west walls)	98	147
Rear (north) wall	26	37
Front (south) wall	25	40
Total	192	275

12.4 Live Loads

Although the Special Procedure is intended to be used independent of the rest of ASCE 41-13, § 15.2 provides no guidance on establishing live loads. In this example, expected live loads are established using ASCE 7-10 and ASCE 41-13. Since the building has an office occupancy, the unreduced live load per ASCE 7-10 is 50 lb/ft^2. The roof is not accessible and has an unreduced live load of 20 lb/ft^2.

ASCE 41-13 §7.2.2 requires that the live load effect be equal to 25% of the unreduced live load per ASCE 7-10 but not less than the actual live load. ASCE 7-10 makes a distinction between floor and roof live loads. Since the roof live load refers to a special, infrequent condition related to reroofing, it is unlikely to coincide with significant earthquake ground shaking and is omitted from earthquake load combinations. ASCE 41-13 does not include any discussion of roof live loads. Therefore, this example follows the logical convention of omitting live load due to reroofing from the earthquake load combinations. The resulting live loads for this evaluation are shown in Table 12-5.

Table 12-5 Live Load Used in Evaluation

Load	Roof (lb/ft²)	Second Floor (lb/ft²)
Live Load	0	13

12.5 Spectral Response Acceleration Parameters

For the Special Procedure, seismic design parameters for the building are determined according to the requirements ASCE 41-13 § 15.2.1 for the BSE-1E Seismic Hazard Level. Similarly, per the requirements of ASCE 41-13 § 4.1.2, the BSE-1E is also used for the component evaluations in Tier 1 screening. There are, however, some triggers in the Special Procedure that reference S_{D1}. The parameter S_{D1} is defined in ASCE 41-13 § 2.5 and uses the S_1 value from the BSE-2N level. The site class, latitude, and longitude for the building are as follows.

- Location: Los Angeles, California
- Latitude: 34.0160º N
- Longitude: 118.2682º W
- Site Class D

The following ground motion parameters are obtained for the BSE-1E and BSE-2N Seismic Hazard Levels using the online tools described in Chapter 3 of this *Guide*:

$S_{XS,BSE\text{-}1E} = 0.913$

$S_{X1,BSE\text{-}1E} = 0.507$

$S_{S,BSE\text{-}2N} = 2.101$, with $F_a = 1.0$

$S_{1,BSE\text{-}2N} = 0.743$, with $F_v = 1.5$

Calculation of S_{DS} and S_{D1}, per ASCE 41-13 § 2.5, are as follows.

$$S_{DS} = (2/3)(F_a)(S_{S,BSE\text{-}2N}) = (2/3)(1.0)(2.101) = 1.401$$

$$S_{D1} = (2/3)(F_v)(S_{1,BSE\text{-}2N}) = (2/3)(1.5)(0.743) = 0.743$$

12.6 Tier 1 Screening (ASCE 41-13 § 4.3 – § 4.5)

In some cases, an initial Tier 1 screening may be a project requirement to identify seismic deficiencies. For a URM bearing wall building, an evaluation would often begin directly with the ASCE 41-13 § 15.2 Special

Procedure described below. For completeness, a summary of Tier 1 screening is provided here.

The Tier 1 evaluation process is shown in ASCE 41-13 Figure 4-1. The first step is to determine if the building meets the Benchmark Building provisions of ASCE 41-13 § 4.3. Older, unretrofitted URM buildings have no benchmark year per ASCE 41-13 Table 4-6 and are thus not exempt from Tier 1 screening. The next step is to select the appropriate checklist to use in the screening. Tier 1 checklists are provided in ASCE 41-13 Chapter 16 and are a function of the Level of Seismicity and the Performance Level. Level of Seismicity is determined per ASCE 41-13 § 2.5 and Table 2-5 and for this example was found to be High. The Life Safety Performance Level is used here. Per ASCE 41-13 Table 4-7 and Chapter 16, a Tier 1 screening would require a Life Safety Basic Configuration Checklist (16.1.2LS), Life Safety Structural Checklist (16.16LS), and Life Safety Nonstructural Checklist (16.17).

For this example, the Nonstructural Checklist is not discussed. Compliance is achieved for the evaluation statements in each of the checklists, except as follows. The Life Safety Nonstructural Checklist is only evaluated here for URM parapets. Remaining items would need to be evaluated in an actual evaluation.

- **Weak/Soft Story:** Per Figure 12-3, on the rear wall, the length of wall through the openings at the second story is 2[(8 ft) + (3.5 ft)] = 23 feet versus 2[(2 ft) + (3 ft)] + (6 ft) = 16 feet at the first story. At the front wall, the length is 2(4 ft) + (12 ft) = 20 feet at the second story versus 4(4 ft) = 16 feet at the first story. Summing the rear and front walls gives (23 ft) + (20 ft) = 43 feet at the second story versus (16 ft) + (16 ft) = 32 feet at the first story. As (32 ft)/(43 ft) = 74% is less than the 80% minimum requirement, a weak story deficiency is identified for the transverse direction. Similarly, assuming stiffness is proportional to length, a soft story deficiency is identified as well since 74% is less than the 80% minimum requirement for the average of the stories above.

- **Shear Stress:** The shear stress check of ASCE 41-13 Checklist 16.16LS references the Quick Check procedure of ASCE 41-13 § 4.5.3.3 for determining demands, applies this to the net area and then compares this against a value of 30 psi for brick clay units. For this example, only the ground story is checked.

  Spectral acceleration is determined as follows:

  $$T = C_t h_n^{\beta} \qquad \text{(ASCE 41-13 Eq. 4-5)}$$

where:

C_t = 0.020 for all other framing systems

h_n = height above base to roof (ft)

β = 0.75 for all other framing systems

T = $(0.020)(22)^{0.75}$ = 0.203

S_a = S_{x1}/T (ASCE 41-13 Eq. 4-4)
= 0.507/0.203 =2.50 > S_{XS} = 0.913, Use 0.913

Base shear per wall line is determined as follows:

V = CS_aW (ASCE 41-13 Eq. 4-1)

where:

C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response per ASCE 41-13 Table 4-8

S_a = Spectral acceleration

V = $(1.0)(0.913)W = 0.913W$
= 0.913 (192 kips at roof from Table 12-4 + 275 kips at second floor)/2 wall lines = 213 kips/wall line

First story shear stress is checked in the north-south direction:

v = $V/A_{n,\text{N-S}}$

where:

V = 213 kips per above

$A_{n,\text{N-S}}$ = [2(4.0 ft) + 2(6.0 ft) + 2(8.0 ft)](12 in./ft)(13 in.)

= 5,616 in.^{2} (see Figure 12-9 and Table 12-7)

v = (213 kips)(1000 lb/kip)/(5,616 in.^{2}) = 38 psi > 30 psi, so there is a shear stress deficiency in the north-south direction

First story shear stress is checked in the east-west direction:

$A_{n,\text{N-S,rear}}$ = [2(2.0 ft) + 2(3.0 ft) + 6.0 ft](12 in./ft)(13 in.) = 2,496 in.^{2}

$A_{n,\text{N-S,front}}$ = [4(4.0 ft)](12 in./ft)(13 in.) = 2,496 in.^{2} (same)

v = (213 kips)(1000 lb/kip)/(2,496 in.^{2}) = 85 psi > 30 psi, so there is also a shear stress deficiency in the east-west direction

- **Wall Anchorage:** Limited anchors connect the walls to the floor and roof diaphragms. The existing anchorage is insufficient to meet the out-of-plane Quick Check procedure of ASCE 41-13 § 4.5.3.7.
- **Wood Ledgers:** The connections between the front and rear walls to the ledgers at the roof and second floor do not have sufficient blocking or ties to prevent cross-grain bending.
- **Masonry Wall Proportions:** For the second story, the h/t ratio is (10 ft)(12 in./ft)/(13 in.) = 9.2 and exceeds the allowable ratio of 9. The first story ratio is (12 ft)(12 in./ft)/(13 in.) = 11.1 and does not exceed the allowable ratio of 15.
- **Cross Ties:** There are no continuous cross ties across the diaphragm between the perimeter masonry walls in either the transverse or longitudinal direction.
- **Straight Sheathing:** Both the roof and the second floor have straight sheathing. In the transverse direction, the nominal dimensions of the buildings have an aspect ratio of 60 feet to 30 feet or 2:1 which is the threshold for the checklist evaluation statement. However, when the stair opening is accounted for, the transverse dimension is reduced and the threshold is exceeded. Even without the diaphragm opening, the actual diaphragm width ratio with the perimeter walls removed is [(60 ft) – (2 walls)(9 in./wall)(1 ft/12 in.)]/[(30 ft) – (2 walls)(9 in./wall)(1 ft/12 in.)] = 2.05 at the roof and [(60 ft) – (2 walls)(13 in./wall)(1 ft/12 in.)]/[(30 ft) – (2 walls)(13 in./wall)(1 ft/12 in.)] = 2.08 at the second floor.
- **Diaphragm Spans:** The nominal diaphragm spans of 60 feet and 30 feet both exceed the 24-foot limit and there is no wood structural panel sheathing or diagonal sheathing, so this results in a noncompliant evaluation statement.
- **URM Parapet:** The height used for a parapet should go from the center line of the roof-to-wall anchor to the top of the parapet, as shown in ASCE 41-13 Figure 13-1. It is assumed that an anchor would be placed at midheight of the 2×12 roof framing, so h = 3’0” to top of roofing + 1-1/2” for sheathing and roofing + 11-1/4”/2 = 3.59’. The h/t ratio for the parapet is (3.59 ft)(12 in./ft)/(9 in.) = 4.8 which exceeds the allowable ratio of 1.5.

Figure 12-5 shows a summary of Tier 1 deficiencies. They are typical of the deficiencies found in most URM buildings. They will be evaluated in greater detail in the Special Procedure evaluation discussed in the sections that follow.

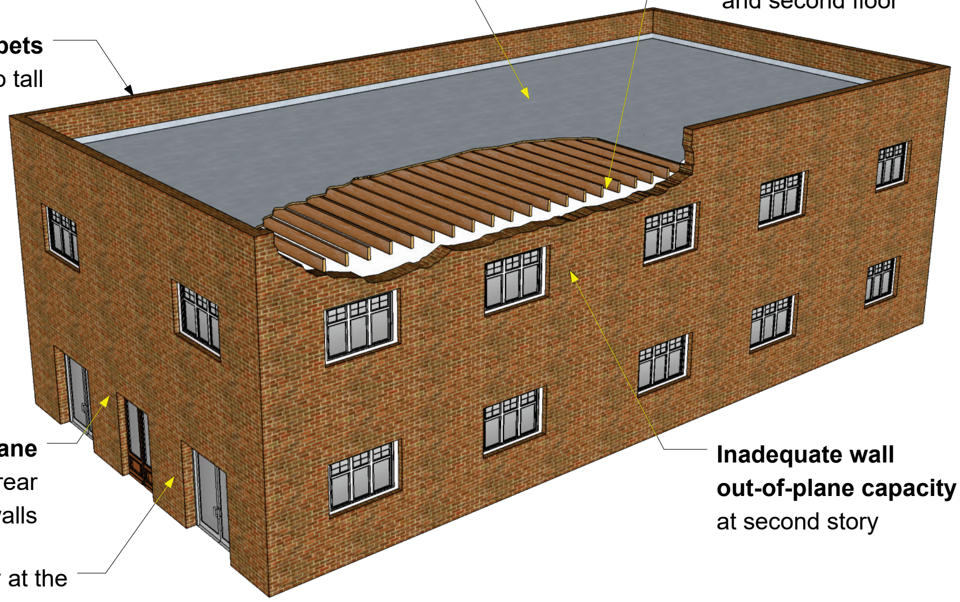


Figure 12-5 Tier 1 screening deficiencies.

12.7 Special Procedure Evaluation and Retrofit Overview (ASCE 41-13 § 15.2)

The Special Procedure for the evaluation of unreinforced masonry bearing walls is based on the results of a testing, evaluation, and reconnaissance program conducted by the ABK Joint Venture in the 1980s. Documentation of the research program is available in a series of reports (ABK, 1981a, 1981b, 1981c, 1981d, and 1984). In general, subsequent anecdotal performance observations after damaging earthquakes have indicated that buildings retrofitted using the Special Procedure have generally met the Collapse Prevention Performance Level under moderate earthquake ground shaking.

The Special Procedure is applicable to unreinforced masonry bearing wall buildings that meet the following criteria (ASCE 41-13 § 15.2.1):

- There are flexible diaphragms at all levels above the seismic base of the structure
- There are a minimum of two lines of walls in each principal direction (except for single-story buildings with an open front)

- There are a maximum of six stories above the seismic base of the structure

Example calculations for the Special Procedure are performed in an order that is consistent with common practice. Since parts of the Special Procedure utilize principles of capacity design based on the strength of floor and roof diaphragms, users of the procedure typically find it valuable to evaluate the diaphragm and then the wall components in order to make a decision as to whether the diaphragm will be strengthened or new vertical force-resisting elements will be added to reduce demands on the diaphragm and walls. Once the diaphragm capacity that will be used in retrofit approach has been established, then a final evaluation of wall components can be made.

12.8 Condition of Materials (ASCE 41-13 § 15.2.2.1)

In order to apply the Special Procedure to a masonry building with multi-wythe walls, ASCE 41-13 requires that the wythes be adequately connected. Specifically, the following conditions must be satisfied:

- At least 10% of the wall area must consist of solid headers extending at least 4 inches into the backing. Figure 12-6 shows a photo of the exterior wall with header courses at every sixth course. Thus, 1/6 = 17% of the wall area has solid headers and meets the 10% requirement.

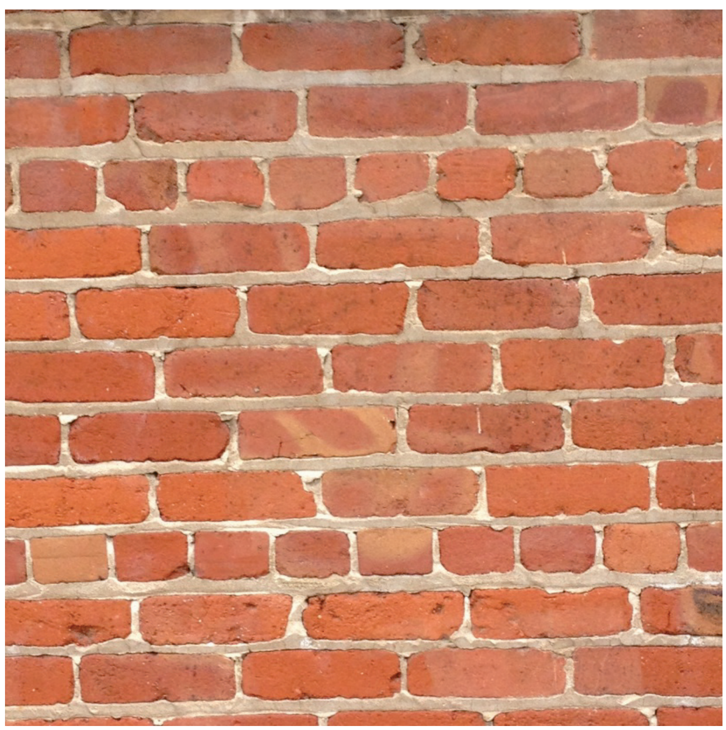


Figure 12-6 Typical URM wall layup.

- For walls where the backing consists of multiple wythes, the header must extend at least 4 inches into the most distant wythe or the wythes must be connected by separate headers meeting the area and spacing requirements of the first bullet. This requirement is satisfied in the example building.
- Walls that fail to satisfy these criteria shall not be considered effective in resisting lateral loads and shall be omitted from the wall thickness for height-to-thickness and shear strength calculations. ASCE 41-13 § 15.2.2.1 lists exceptions for buildings with low spectral response acceleration parameters, which do not apply to this structure.

12.9 In-Place Shear Testing (ASCE 41-13 § 15.2.2.2)

Masonry shear strength is determined by in-plane shear testing for each masonry class, where a masonry class is defined by major variations in strength, quality, materials, or condition over a large area. ASCE 41-13 § 15.2.2.2 defines the minimum number of tests for each masonry class as follows:

- At each of both the first and top stories, not less than two tests per wall or line of wall elements providing a common line of resistance to lateral forces
- At each of all other stories, not less than one test per wall or line of wall elements providing a common line of resistance to seismic forces
- Not less than one test per 1,500 ft^2 of wall surface or less than a total of eight tests

For this structure, assuming only one masonry class is observed, Bullet (1) from above requires (2 stories)(4 walls)(2 tests) = 16 tests, thus the minimum of 8 tests in Bullet (3) is satisfied. Bullet (2) is not applicable, since the building has only two stories. The largest wall area tributary to a single shear test per Bullet (1) occurs at the first story side walls with (60 ft)(12 ft) = 720 ft^2. With 720 ft^2/1500 ft^2 = 0.48 < 2 minimum of Bullet (1), then Bullet (1) governs. Note that in the calculation, the full wall area including openings, is conservatively used.

ASCE 41-13 § 15.2.2.2 presents provisions for two types of masonry testing: in-place mortar tests (often called in-place push tests) and tensile splitting tests. In-place push tests conducted per ASTM 1531 (ASTM, 2009) are typically used in practice and are shown in this example (see Figure 12-7). The test measures the shear strength of the bed joints on the top and bottom of the brick and any contributions from the collar joint mortar behind the loaded brick. In-place push tests involve removing one brick from the outer wythe, placing a jack in the void, and pushing against the remaining bricks on the sides of the void. A head joint is removed from the end opposite the loaded end of one of the bricks being pushed. The load needed to move the brick is recorded. As a part of the investigation, the percentage of collar joint surfaces covered with mortar should be reported. Wall capacity equations have been calibrated assuming a mortared collar joint. In-place push test results in walls without collar joint mortar will be conservative, but a wall without collar joint mortar likely has less out-of-plane capacity. For more information on the derivation of the in-plane push test equations, see FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings* (FEMA, 1998a).

Figure 12-7 Images of in-place test, using a flat jack (left, from Geister, 2013) and a typical hydraulic ram (right, from Almesfer et al., 2014). Typically, the flat jack is more commonly used for historic buildings to avoid the removal of masonry.

Tensile splitting tests can be used with hollow masonry such as hollow clay tile where the push test cannot be used. 2015 IEBC, *International Existing Building Code* (ICC, 2015b), Section A106.3.3 presents more information.

For in-place tests, sample locations should be selected on both the interior and exterior of the building. Environmental effects can cause degradation in exposed exterior mortar. Additionally, it is possible that exterior walls may have been repointed, which replaces the original bed joint and head joint mortar to a given depth. The new mortar may have a different strength or stiffness than the original material. In order to capture a representative sample of the mortar shear capacity, both interior and exterior locations should be tested.

Tests should not be conducted directly beneath window openings, since the masonry does not have well-defined boundary conditions typical of the majority of the wall area, and it is difficult to assess the axial stress on the wall. Per ASCE 41-13 § C15.2.2.2, determination of testing locations should consider factors, such as workmanship at different building heights (upper levels can have poorer workmanship), weathering of exterior surfaces, the condition of interior surfaces, and deterioration of other surfaces. Some tests should also be in close proximity to roof-to-wall and floor-to-wall anchor locations.

Since the on-site investigation for this building showed that masonry quality, condition, and materials were consistent for all walls, only one masonry class was identified, and 16 total tests were conducted. Figure 12-8 shows the locations of the in-place push tests. A good distribution of tests between the inside and outside wall faces, each story and the different wall lines was achieved, but all tests are at 2 feet above the floor levels. As noted above, it would be desirable in practice to have some of the tests closer to roof-to-wall and floor-to-wall locations.

The mortar shear test value is calculated from in-place test results as follows:

$$v_{to} = \frac{V_{\text{test}}}{A_b} - P_{D+L} \qquad \text{(ASCE 41-13 Eq. 15-1)}$$

where:

v_{to} = Mortar shear test value (lb/in.^{2})

V_{test} = Load at first observed movement of brick (lb)

A_b = Total area of bed joints above and below the test specimen (in.^{2}). Note that the collar joint behind the test specimen and head joints adjacent to the test specimen are ignored.

P_{D+L} = Stress on the brick at the test joint resulting from actual dead plus live loads in place at the time of testing (lb/in.^{2})

Field measurements indicate that a typical brick is 4 inches wide, 9 inches long, and 2-5/8 inches high. The bed joint area is:

A_b = 2(4 in.)(9 in.) = 72 in.^{2}

For a typical mortar test location at the second story (# 7 in Figure 12-8a), the stress on the brick is calculated below. Note that since the floor spans east-west, only the side walls (and the interior partition walls) carry tributary floor loads. The north and south walls do not carry tributary floor loads.

P_{D+L} = (Total tributary load)/[(Wall length at bottom of tributary area)(Wall thickness)]

where:

$$P_{\text{Parapet}} = (90 \text{ lb/ft}^2)(11 \text{ ft})(3 \text{ ft}) = 2{,}970 \text{ lb}$$

$$P_{\text{Wall}} = (130 \text{ lb/ft}^2)[(11 \text{ ft})(3 \text{ ft}) + (6 \text{ ft})(4 \text{ ft}) + (6 \text{ ft})(1 \text{ ft}) + 2\{\tan(45°)\}(1 \text{ ft})/2)(1 \text{ ft})] = 8{,}320 \text{ lb}$$

$$P_{\text{Roof}} = (19 \text{ lb/ft}^2 + 0 \text{ lb/ft}^2)(11 \text{ ft})(6.5 \text{ ft}) = 1{,}359 \text{ lb}$$

$$P_{D+L} = (2{,}970 \text{ lb} + 8{,}320 \text{ lb} + 1{,}359 \text{ lb})/[(8 \text{ ft})(12 \text{ in./ft})(13 \text{ in.})] = 10 \text{ lb/in.}^2$$

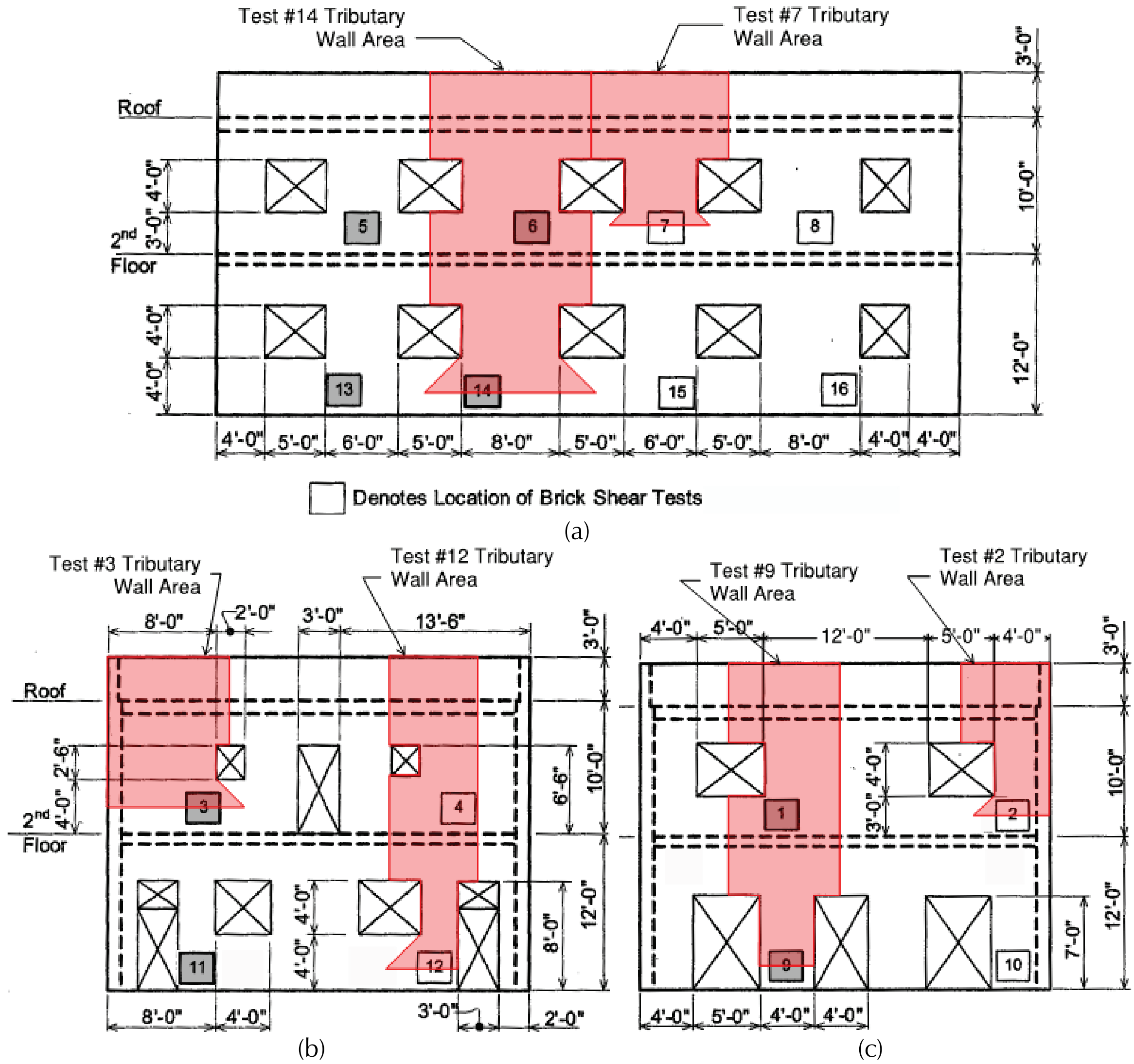


Figure 12-8 Location of in-place shear tests: (a) side wall elevation; (b) rear (north) wall elevation; and (c) front (south) wall elevation. Tributary wall areas used for representative test locations are highlighted. For simplicity, all of the side wall tests are shown on one elevation. In the actual building, half of the tests are conducted on each side wall. The test locations shaded gray are conducted on the inside of the building. Each test is conducted 2′-0″ above the floor level.

In this example, no flange length is assumed for the calculation of loads or strength at corner piers. ASCE 41-13 does not include specific guidance on the evaluation of corner piers or piers with an irregular profile in plan. For piers with an irregular profile, the entire pier area (including protrusions, such as pilasters) should be included in the calculation of gravity loads. The engineer should consider whether the entire profile (including protrusions, such as pilasters) is effective in resisting shear or whether only the effective web area should be included.

A summary of the masonry test results, V_{test}, and calculated shear test value, v_{to}, for this building are shown in Table 12-6, along with a ranking from high values to low.

ASCE 41-13 § 15.2.2.2.1 defines the mortar shear strength, v_{te}, as the shear strength exceeded by 80% of the tests for each masonry class. Since this building has one masonry class, the mortar shear strength is calculated as shown below.

$$\text{80\% Percentile } = 0.8(16 \text{ tests}) = 12.8 \text{ tests} \xrightarrow{\text{roundup}} 13 \text{ tests}$$

Therefore, v_{te} is determined as the fourth lowest test result, so v_{te} = 76 lb/in^2.

Table 12-6 Summary of Mortar Shear Test Results

Test	V_{test} (lb)	P_{D+L} (lb/in.^{2})	v_{to} (lb/in.^{2})	Ranking
1	6,700	9	84	11
2	7,500	10	94	8
3	7,100	7	91	9
4	9,600	7	126	2
5	10,100	10	130	1
6	8,100	10	103	7
7	7,000	10	87	10
8	6,200	10	76	13
9	8,300	32	83	12
10	7,250	25	75	14
11	10,100	20	120	3
12	5,400	20	55	16
13	9,300	19	110	4
14	9,300	20	109	5
15	6,800	20	75	15
16	9,100	19	107	6

* Results below the 80% cutoff have been shaded gray.

Commentary

Limits on mortar shear strength, v_{te}: ASCE 41-13 limits the minimum mortar shear strength to $v_{te} \geq 30$ lb/in^2. For masonry classes not meeting this limit, the walls must be pointed and retested (per § 15.2.2.2.1).

The upper limit on mortar shear strength that exists in other code documents ($v_{te} < 100$ lb/in.^{2}) is not required in ASCE 41-13.

ASCE 41-17 Revision

Definition of v_{te}: The use of the 80th percentile test value for v_{te} was revised to use the mean minus one standard deviation of the test values in ASCE 41-17.

12.10 Masonry Strength (ASCE 41-13 § 15.2.2.3)

The unreinforced masonry strength, v_{me}, is calculated individually for each wall pier as a function of the pier area, the dead load at the top of the pier, and the mortar strength.

$$v_{me} = 0.56 v_{te} + \frac{0.75 P_D}{A_n} \qquad \text{(ASCE 41-13 Eq. 15-4)}$$

where:

v_{me} = Unreinforced masonry strength (lb/in.^{2})

v_{te} = Mortar shear strength (lb/in.^{2})

P_D = Total superimposed dead load at the top of the pier (lb)

A_n = Area of net mortared/grouted section (in.^{2})

For a typical pier on a side wall (#11 in Figure 12-9a), the unreinforced masonry strength is calculated as:

v_{te} = 76 lb/in.^{2} (per Section 12.9 and Table 12-6 of this example)

P_D = Dead load due to parapet and other walls above the top pier and tributary roof and second floor areas
= (90 lb/ft^2)(12.5 ft)(3 ft) + (130 lb/ft^2)[(12.5 ft)(3 ft) + (4 ft)(8 ft) + (12.5 ft)(7 ft)] + (19 lb/ft^2)(12.5 ft)(6.5 ft) + (30.5 lb/ft^2)(12.5 ft)(7.5 ft)
= 28,200 lb

A_n = (depth of pier)(wall thickness)
= (8 ft)(12 in./ft)(13 in.)
= 1,248 in.^{2}

v_{me} = 0.56(76 lb/in.^{2}) + 0.75(28,200 lb)/(1,248 in.^{2})
= 59 lb/in.^{2}

Similar to the calculation of dead load for mortar shear tests, the roof and second floor loads are only considered in the dead load for the side walls. Since the floors span east-west, they do not contribute dead load to piers at the front or rear walls.

A summary of the dead load acting on each wall pier and the resulting unreinforced masonry strength is shown in Table 12-7.

Note that ICC Evaluation Service reports for proprietary anchors typically specify that a minimum average mortar strength of 50 psi is required. The average strength of the values in Table 12-7 is 55 psi, so this criterion is met.

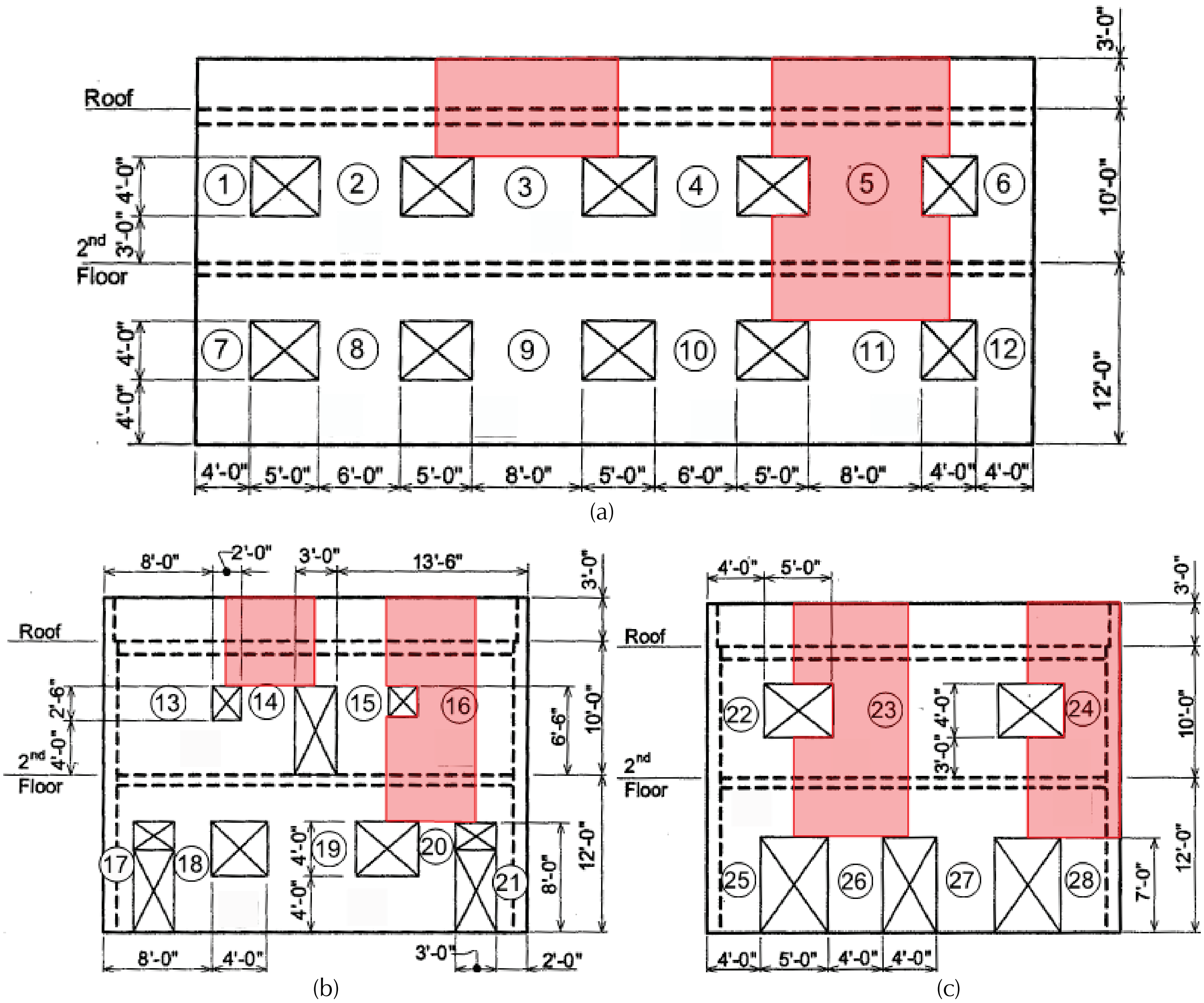


Figure 12-9 Wall pier labels: (a) side wall elevation; (b) rear (north) wall elevation; and (c) front (south) wall elevation. Tributary wall areas for the calculation of P_D are highlighted for representative piers.

Table 12-7 Expected Unreinforced Masonry Strength

Pier	t (in.)	D (ft)	A_n (in.^{2})	P_D (lb)	v_{me} (lb/in.^{2})
1	13	4.0	624	5,100	49
2	13	6.0	936	8,600	49
3	13	8.0	1,248	10,200	49
4	13	6.0	936	8,600	49
5	13	8.0	1,248	9,800	48
6	13	4.0	624	4,700	48
7	13	4.0	624	14,400	60
8	13	6.0	936	23,900	62
9	13	8.0	1,248	29,100	60

Table 12-7 Expected Unreinforced Masonry Strength (continued)

Pier	t (in.)	D (ft)	A_n (in.^{2})	P_D (lb)	v_{me} (lb/in.^{2})
10	13	6.0	936	24,300	62
11	13	8.0	1,248	28,200	59
12	13	4.0	624	13,600	59
13	13	8.0	1,248	6,500	46
14	13	3.5	546	4,400	48
15	13	3.5	546	4,400	48
16	13	8.0	1,248	6,500	46
17	13	2.0	312	7,300	60
18	13	3.0	468	12,800	63
19	13	6.0	936	19,925	58
20	13	3.0	468	12,800	63
21	13	2.0	312	7,300	60
22	13	4.0	624	4,300	48
23	13	12.0	1,872	11,200	47
24	13	4.0	624	4,300	48
25	13	4.0	624	13,100	58
26	13	4.0	624	17,700	64
27	13	4.0	624	17,800	64
28	13	4.0	624	13,400	59

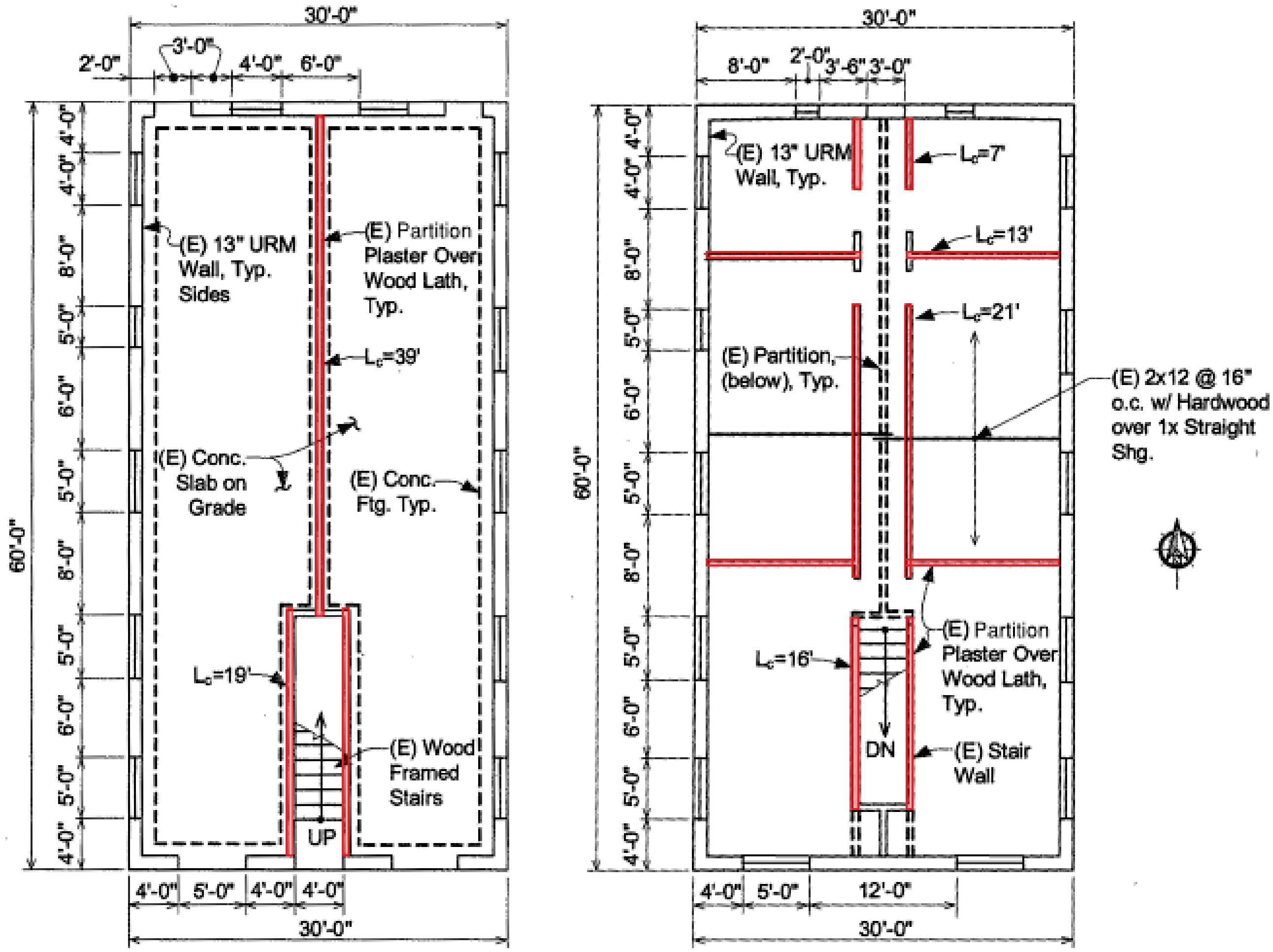
12.11 Diaphragm Evaluation (ASCE 41-13 § 15.2.3.2)

Per ASCE 41-13 § 15.2.3.2.2, since S_{D1} = 0.743 (from Section 12.5 of this *Guide*) exceeds 0.20, evaluation of floor and roof diaphragm demand-capacity ratios is required. The calculation is dependent on the presence or absence of qualifying cross walls. Figure 12-10 shows the floor plans with cross walls.

Cross walls are not considered “shear walls” in the Special Procedure and need not be designed for the tributary loads of the flexible diaphragm(s). Rather, they provide damping of the diaphragm response to earthquake shaking and couple the diaphragm response over multiple floors to reduce the maximum response. Figure 12-11 shows this concept. Cross walls shown on the first floor plan are used in evaluation of the second floor diaphragm. Cross walls shown on the second floor plan are used in evaluation of the second floor and roof diaphragms.

ASCE 41-13 § 15.2.3.1 provides guidance for determining the existence of cross walls. A qualifying cross wall meets the following criteria:

- The wall consists of a wood-framed wall sheathed with the materials defined in ASCE 41-13 Table 15-1.
- Walls are spaced at 40 ft or less measured in the direction perpendicular to loading.



(a) (b)

Figure 12-10 Floor plans showing qualifying cross walls in red: (a) first floor; and (b) second floor.

- Walls extend the full story height and connect to the diaphragms above and below. (Partial connections meeting the exceptions of ASCE 41-13 § 15.2.3.1.1 are acceptable.)
- The ratio of wall length-to-height between openings is greater than or equal to 1.5.
- Walls have strength greater than or equal to 30% of strength of the stronger diaphragm at or above the diaphragm level under consideration.

- At least one of the following criteria are met:
  - Walls are present at all levels (the walls need not be aligned in plan).
  - Walls are present in the top story of a multi-story building (the walls need not be present at lower stories).
  - Walls are present at all levels except within 4 ft of grade in a cripple wall building. In this case, the walls must satisfy the additional requirements of ASCE 41-13 § 15.2.3.1.1.

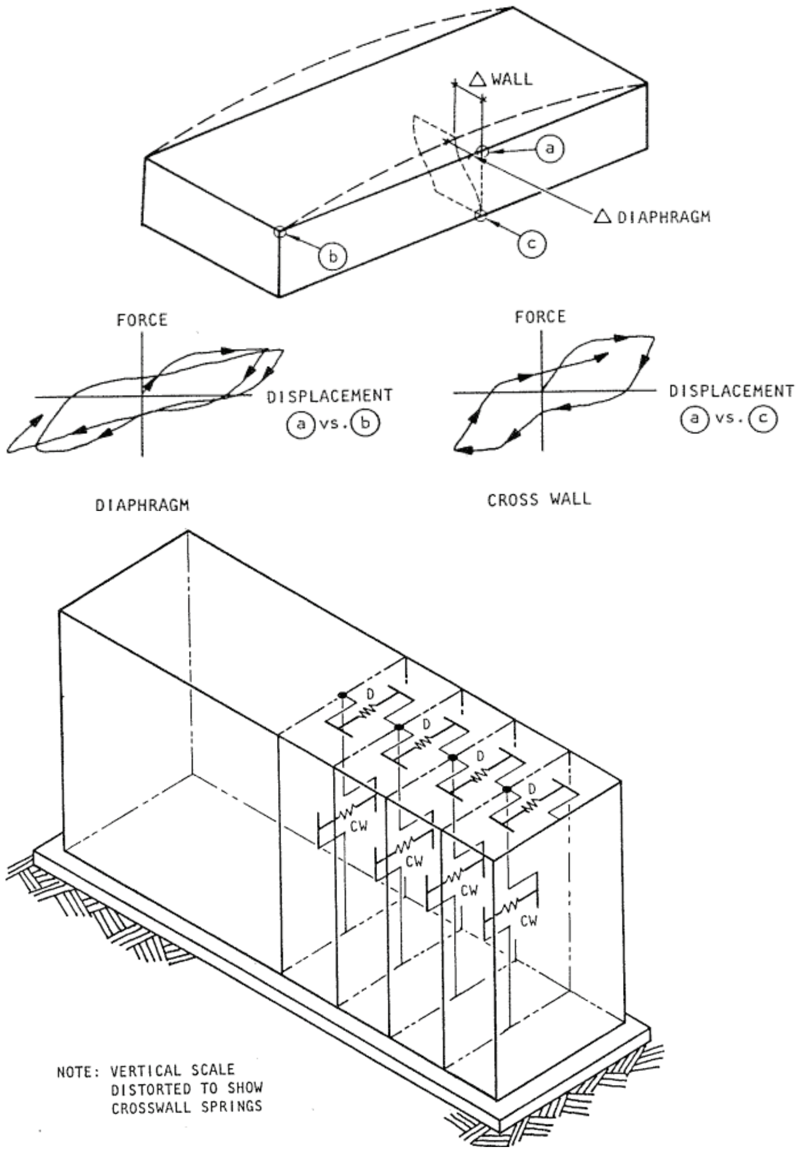


Figure 12-11 Schematic illustration of cross wall as a damping element (ABK, 1984).

Accordingly, the following are checked at the second floor, for the walls in the east-west direction:

- Wall construction using plaster over wood lath is acceptable per ASCE 41-13 Table 15-1
- Lowest ratio of wall length to height is l/h = 13 ft/[(10 ft story height – (11.25 in. for 2×12 – 1 in for sheathing – 1/2 in. for roofing)/12 in./ft] ft at roof diaphragm) = 1.5 ≥ 1.5
- Maximum wall spacing is approximately 24 ft < 40 ft
- Walls connect to the second floor and roof diaphragms
- Walls are present in the top story of a multi-story building

Therefore, the east-west walls at the second story are qualifying cross walls. Similarly, cross walls occurring at the first story and the second story in the north-south direction satisfy the qualifying requirements, except for the 7 ft north walls at the end of the corridor that are too short to meet the $l/h \geq 1.5$ qualifying requirement.

Per ASCE 41-13 § 15.2.3.1.2, the minimum strength of the cross walls needs to be at least 30% of the diaphragm shear strength to effectively reduce diaphragm amplification. The capacity of only one side of the diaphragm connected to the cross wall is used in this check. The check is made for portions of the diaphragm within 40 feet.

$$V_c = v_c L_c \geq 0.3 v_u D$$

where:

V_c = Total shear strength of cross walls (lb)

v_c = Unit shear strength of cross walls (ASCE 41-13 refers to this quantity as seismic shear strength) (lb/ft)

L_c = Total length of cross walls within 40 ft (ft)

v_u = Unit shear strength of the strongest diaphragms at or above the level under consideration (lb/ft)

D = Total length of diaphragm (ft)

In the north-south direction, 40 ft is greater than the 30 ft building width. Therefore, all qualifying cross walls, except the 7 ft long walls at the north end of the corridor, are considered.

Useful Tip

The wall aspect ratio check in ASCE 41-13 § 15.2.3.1 differs from the 2015 IEBC § A111.3.3. In the 2015 IEBC (and previous editions of the IEBC), a qualifying cross wall requires $l/h \geq 0.67$. In ASCE 41-13, the requirement is much more conservative, $l/h \geq 1.5$.

The goal of the check is to omit narrow walls that are unlikely to exhibit the hysteretic energy dissipation necessary to reduce diaphragm acceleration.

ASCE 41-17 Revision

Language was added in ASCE 41-17 clarifying that the capacities that are given in Table 15-1 are per side of a wall.

North-South Strength of Cross Walls Below the Roof

v_c = (2 sides)(600 lb/ft/side) = 1,200 lb/ft (ASCE 41-13 Table 15-1, plaster on wood lath), but Footnote (b) limits the shear strength to 900 lb/ft

L_c = 2(21 ft +16 ft)
= 74 ft

V_c = (74 ft)(900 lb/ft)(1 kip/1,000 lb)
= 67 kips

v_u = 300 lb/ft at roof (per ASCE 41-13 Table 15-2, straight sheathing)

D = 60 ft

$0.3v_uD$ = 0.3(300 lb/ft)(60 ft)(1 kip/1,000 lb)
= 5.4 kips

V_c = 67 kips ≥ 5.4 kips

Therefore, the shear strength of the walls exceeds 30% of the diaphragm shear strength, and the walls can be used in diaphragm strength checks.

North-South Strength of Cross Walls Below the Second Floor:

v_c = (2 sides)(600 lb/ft/side) = 1,200 lb/ft (ASCE 41-13 Table 15-1, plaster on wood lath), but Footnote (b) limits the shear strength to 900 lb/ft

L_c = 39 ft + 2(19 ft)
= 77 ft

V_c = (77 ft)(900 lb/ft)(1 kip/1,000 lb)
= 69 kips

v_u = Greater of 300 lb/ft at roof (per ASCE 41-13 Table 15-2, straight sheathing) or 1,500 lb/ft at second floor (hardwood on straight sheathing)

D = 60 ft

$0.3v_uD$ = 0.3(1,500 lb/ft)(60 ft)(1 kip/1,000 lb)
= 27 kips

V_c = 69 kips ≥ 27 kips

In the east-west direction, there are no cross walls at the first story. At the second story, the southern portion of the building controls, since it has only one cross wall restraining the diaphragm.

East-West Strength of Cross Walls Below the Roof:

v_c = (2 sides)(600 lb/ft/side) = 1,200 lb/ft (per ASCE 41-13 Table 15-1, plaster on wood lath), but Footnote (b) limits the shear strength to 900 lb/ft

L_c = 2(13 ft)
= 26 ft

V_c = (26 ft)(900 lb/ft)(1 kip/1,000 lb)
= 23 kips

v_u = 300 lb/ft at roof (per ASCE 41-13 Table 15-2, straight sheathing)

D = 30 ft

$0.3v_uD$ = 0.3(300 lb/ft)(30 ft)(1 kip/1,000 lb)
= 2.7 kips

V_c = 23 kips ≥ 2.7 kips

Based on these calculations, all of the qualifying cross walls have sufficient strength to be considered in the evaluation of diaphragm demand-capacity ratio.

The demand-capacity ratio of the diaphragms is evaluated according to ASCE 41-13 § 15.2.3.2 as follows:

- Diaphragms without cross walls at levels immediately above or below:

$$\text{DCR} = \frac{2.1S_{X1}W_d}{\sum v_u D} \quad \text{(ASCE 41-13 Eq. 15-7)}$$

- Diaphragms in a one-story building with cross walls

$$\text{DCR} = \frac{2.1S_{X1}W_d}{\sum v_u D + V_{cb}} \quad \text{(ASCE 41-13 Eq. 15-8)}$$

- Diaphragms in a multi-story building with cross walls at all levels:

$$\text{DCR} = \frac{2.1S_{X1}\sum W_d}{\sum(\sum v_u D) + V_{cb}} \quad \text{(ASCE 41-13 Eq. 15-9—Revised)}$$

- Roof diaphragms and the diaphragms directly below if coupled by cross walls:

$$\text{DCR} = \frac{2.1S_{X1}\sum W_d}{\sum\left(\sum v_u D\right)} \quad \text{(ASCE 41-13 Eq. 15-10)}$$

Commentary

The ASCE 41-13 Special Procedure diaphragm provisions use the term "DCR," defined as the demand-capacity ratio, matching the 2015 IEBC and predecessor documents. This should not be confused with the DCR used in ASCE 41-13 Chapter 7, and Equation 7-16 for linear procedures that has a different definition. See Chapter 2 of this *Guide* for details.

Commentary

ASCE 41-13 defines the quantity W_d as the dead load tributary to a diaphragm.

This example makes a distinction between dead load and seismic weight that is implied, but not expressed, in ASCE 41-13 Chapter 15. The check should be performed using tributary seismic weight lumped from midheight above and below each diaphragm level (as opposed to dead load supported by the floor or roof).

ASCE 41-17 Revision

A typographical error in the denominator of ASCE 41-13 Equation 15-9 was corrected in ASCE 41-17, and this revised version is used here.

where:

S_{X1} = BSE-1E spectral response acceleration parameter at a 1-second period

W_d = Total seismic weight tributary to the diaphragm (lb)

ΣW_d = Sum of seismic weight tributary to the diaphragms under evaluation (lb)

V_{cb} = Total shear strength of cross walls in the direction of analysis directly below the (lowest) diaphragm level being evaluated (lb)

v_u = Unit shear strength of diaphragm (lb/ft)

D = Depth of diaphragm (ft)

$\Sigma v_u D$ = Sum of shear capacities of both ends of the diaphragm (lb)

$\Sigma(\Sigma v_u D)$ = Sum of shear capacities of both ends of the diaphragms coupled at and above the level under consideration in ASCE 41-13 Equation 15-9. Sum of shear capacities of both ends of the diaphragms at roof and level below in ASCE 41-13 Equation 15-10 (lb)

ASCE 41-13 Equations 15-7 through 15-10 can be complicated to understand and interpret, and there are issues that have been improved in ASCE 41-17. The following points of commentary are provided regarding application of the diaphragm demand-capacity equations.

- The use of plural for "diaphragms" in ASCE 41-13 Equations 15-7 to 15-10 is because some buildings can have more than one span of diaphragms at any particular level.
- **ASCE 41-13 Equation 15-7:** This equation is applicable to diaphragms without qualifying cross walls directly above or below. Note that non-qualifying walls may be present in the building. Equation 15-7 must be used at all levels in a building without cross walls, unless Equation 15-10 applies to the upper two levels. Equation 15-7 is also used when checking the portion of the diaphragm.
- **ASCE 41-13 Equation 15-8:** The capacity of both ends of the diaphragm and the capacity of the cross wall are combined to calculate the total diaphragm capacity.

- **ASCE 41-13 Equation 15-9:** There is a typographical error in the parentheses in the denominator of Equation 15-9. The correct version is shown above. This was corrected in ASCE 41-17. In a multi-story building with qualifying cross walls at each story, a DCR is calculated using the sum of the demands from all stories above, and the capacities of all the diaphragms plus the capacity of the cross walls below the lowest elevated floor. Per ASCE 41-13 § 15.2.3.2.2, Equation 15-9 is checked at all diaphragm levels. Note that ASCE 41-17 added the statement that the roof diaphragm must meet the requirements of Equation 15-8.
- **ASCE 41-13 Equation 15-10:** This is a special evaluation for the combined resistance of the roof diaphragm and the diaphragm below the roof, as the roof is often weaker and more flexible than diaphragms below. The demand is the sum of the demands for each of the two levels, and the capacity is the sum of the capacity of both diaphragms. Cross wall capacities are not included in this equation. Equation 15-10 applies where cross walls do not exist at all levels. Equation 15-10 is superceded by Equation 15-9 when cross walls exist at all levels. This was clarified in ASCE 41-17. The plural in "diaphragms" in the introduction to Equation 15-10 is used because some buildings can have more than one span of diaphragms at any particular level. It only refers to the diaphragm level directly below the roof.

Check of the Diaphragms for Loads in the North-South Direction:

In the north-south direction, the building has qualifying cross walls at both levels. At the second story, ASCE 41-13 Equation 15-9 is applied since cross walls exist at both the first story and second story. Per ASCE 41-13 § 15.2.3.2.2 both the roof diaphragm and the second floor diaphragms are checked using Equation 15-9. The roof diaphragm is checked first.

$$\text{DCR} = \frac{2.1S_{X1}\sum W_d}{\sum\left(\sum v_u D\right) + V_{cb}} \qquad \text{(ASCE 41-13 Eq. 15-9)}$$

where:

S_{X1} = 0.507

ΣW_d = Seismic weight of roof diaphragm, plus rear (north) and front (south) walls tributary to roof diaphragm
= 43 kips + 26 kips + 25 kips
= 94 kips

$\Sigma(\Sigma v_u D)$ = Shear capacity of roof diaphragm
= 2(300 lb/ft)(60 ft)(1 kip/1,000 lb)
= 36 kips

Useful Tip

The calculation of shear capacity shown is multiplied by two at each level because it represents the sum of the shear capacities at each end wall.

V_{cb} = Total shear strength of cross walls at the second story
= 67 kips (per above)

DCR = 2.1(0.507)(94 kips)/(36 kips + 69 kips) = 0.95

ASCE 41-13 Equation 15-9 also evaluates the combined capacity of the roof diaphragm and second floor diaphragm acting together.

S_{X1} = 0.507

ΣW_d = Seismic weight of roof diaphragm, second floor diaphragm, rear (north) and front (south) walls tributary to roof diaphragm, and rear (north) and front (south) walls tributary to second floor diaphragm;

= 43 kips + 51 kips + 26 kips + 25 kips + 37 kips + 40 kips
= 222 kips

$\Sigma(\Sigma v_u D)$ = Sum of shear capacities of roof diaphragm and second floor diaphragm;
= 2(300 lb/ft)(60 ft)(1 kip/1,000 lb) + 2(1,500 lb/ft)(60 ft)(1 kip/1,000 lb)
= 216 kips

V_{cb} = Total shear strength of cross walls at the first story;
= 69 kips (per above)

DCR = 2.1(0.507)(222 kips)/(216 kips + 69 kips) = 0.83

The higher (more conservative) DCR of 0.95 > 0.83 is used. The diaphragm span of 30 ft is used to evaluate acceptability (see below). The opening in the second floor diaphragm at the stair is judged to have a minimal impact on the behavior of the structure, since it occurs in the middle of the diaphragm span and occupies a small portion of the diaphragm depth (16-foot opening width versus 60-foot diaphragm span).

Check of the Diaphragms for Loads in the East-West Direction

In the east-west direction, the building has qualifying cross walls below the roof (but not below the second floor). In this case, only ASCE 41-13 Equation 15-10 is applicable. The roof and second floor diaphragms span 60 feet between the masonry end walls.

ASCE 41-13 § 15.2.3.2.5 additionally requires that the effects of diaphragm openings be considered in the evaluation. Since the diaphragm opening at the second floor is partially in the end quarter of the diaphragm, two checks of diaphragm capacity are needed. In the first check, the depth of the stair opening is omitted in the calculation of diaphragm depth.

$$\text{DCR} = \frac{2.1 S_{X1} \sum W_d}{\sum\left(\sum v_u D\right)} \qquad \text{(ASCE 41-13 Eq. 15-10)}$$

where:

S_{X1} = 0.507

ΣW_d = Seismic weight of roof diaphragm, second floor diaphragm, side (east and west) walls tributary to roof diaphragm, and side (east and west) walls tributary to second floor diaphragm
= 43 kips + 51 kips + 98 kips + 147 kips
= 339 kips

$\Sigma(\Sigma v_u D)$ = Sum of shear capacities of roof diaphragm and second floor diaphragm. Note that the length at the second floor is not multiplied by two, because each end of the diaphragm has a different length (due to the floor opening)
= 2(300 lb/ft)(30 ft)(1 kip/1,000 lb) + (1,500 lb/ft)(30 ft)(1kip/1,000 lb) + (1,500 lb/ft)(25 ft)(1 kip/1,000 lb)
= 101 kips

DCR = 2.1(0.507)(339 kips)/(101 kips)
= 3.57

Additionally, the portion of the diaphragm adjacent to the opening should be checked separately per ASCE 41-13 § 15.2.3.2.5. In this case, it is appropriate to apply ASCE 41-13 Equation 15-7 for the portion of the diaphragm adjacent to the opening since there are no cross walls above or below in this section of the diaphragm. Since the opening is symmetric, only the portion of the diaphragm on the east side of the stair is checked.

$$\text{DCR} = \frac{2.1 S_{X1} W_d}{\sum v_u D} \qquad \text{(ASCE 41-13 Eq. 15-7)}$$

where:

S_{X1} = 0.507

W_d = Seismic weight of second floor diaphragm over the length of the opening and the portion of the side (east) wall tributary to second floor diaphragm over the length of the opening
= (1/2)(51 kips + 147 kips)(16 ft/60 ft)
= 26 kips

Commentary

When an opening occurs in the end quarter of a diaphragm (measured parallel to the span), the stiffness of the diaphragm can be affected. The depth of the opening (measured perpendicular to the span, in the direction of loading) should be omitted in the calculation of net diaphragm depth. Since the DCR calculation is also a measure of diaphragm deflection, this calculation is a surrogate for a more complicated evaluation of strength and stiffness. From the 2015 IEBC commentary, if the diaphragm satisfies this check, the diaphragm adjacent to the opening can be assumed to have no shear stress (so no collector is needed for load transfer) at the edge of the opening.

Additionally, the continuous portion of the diaphragm should be checked regardless of where the opening occurs. In this case, the diaphragm load, W_d, is the portion occurring over the length of the opening, and the span is the length of the opening parallel to the diaphragm span. The depth, D, is the distance from the closest out-of-plane masonry wall to the opening.

$\Sigma v_u D$ = Shear capacity of second floor diaphragm using the net depth of the diaphragm at the opening;
= (1,500 lb/ft)[(30 ft – 4 ft opening)/2)](1 kip/1,000 lb)
= 19.5 kips

DCR = 2.1(0.507)(26 kips)/(19.5 kips)
= 1.42

Diaphragm Acceptability Criteria

Useful Tip

The diaphragm span plotted in Figure 12-12 is the distance between masonry walls. The presence of qualifying cross walls does not reduce the diaphragm spans because cross walls are not considered to be shear walls.

Acceptability criteria for the diaphragms are defined by ASCE 41-13 § 15.2.3.2.3 using Figure 15-1. The plot is reproduced below as Figure 12-12. Diaphragms with DCR and span within the solid line are acceptable; on the other hand, diaphragms outside the line may experience excessive deflection and must be strengthened. The results of the diaphragm evaluation in this example are plotted, showing that in each direction, the diaphragms satisfy the evaluation criteria.

The regions indicated in Figure 12-12 are used to assess the out-of-plane allowable height-to-thickness ratio of the unreinforced masonry walls per ASCE 41-13 Table 15-4 when $S_{X1} \geq 0.4$. In Region 1, cross walls are expected to modify the response of diaphragms, and the higher h/t ratios in ASCE 41-13 Table 15-4 Column A may be used provided cross walls are present in all stories in the direction of loading under consideration, certain mortar strengths minimums are met, and collar joints have at least 50% mortar coverage.

Commentary

ASCE 41-13 § 15.2.3.2.4 notes that a check of diaphragm chords in the Special Procedure is not required.

Post-earthquake observation has not indicated that lack of a formal diaphragm chord is critical to the behavior of wood diaphragms in URM buildings. Additionally, most buildings have some form of edge restraint suitable to resist the diaphragm flexural demands.

In Region 2, h/t ratios of Column A may be used, whether or not cross walls are present as long as mortar strength and collar joint coverage minimums are met. In Region 3, cross walls are not considered to provide benefits of reduced response, and the lower h/t ratios of Column B shall be used, whether or not cross walls are present.

The solid line represents a diaphragm displacement of approximately 5 inches (ABK, 1981c and 1984).

Per ASCE 41-13 § 15.2.3.2.4, the Special Procedure does not require an explicit check of diaphragm chords. Since the masonry walls are continuous around the perimeter of the building, a check of diaphragm collectors is also not required.

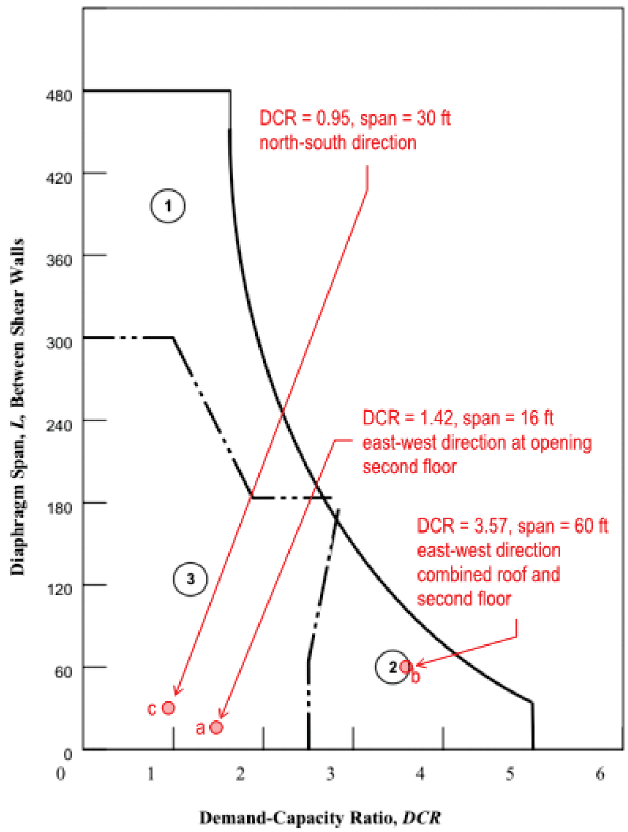


Figure 12-12 Acceptable diaphragm span as a function of DCR (ASCE 41-13 Figure 15-1) with results of the diaphragm evaluations plotted. Printed with permission from ASCE.

12.12 In-Plane Demand on Shear Walls (ASCE 41-13 § 15.2.3.3.1)

ASCE 41-13 § 15.2.3.3.1 requires evaluation of the in-plane strength of masonry shear walls if $S_{X1} > 0.133$. S_{X1} is 0.507 according to Section 12.5 of this *Guide*. Similar to the calculation of diaphragm capacity, wall demand is dependent on the presence or absence of qualifying cross walls. In either case, the wall demand is capped based on the shear strength of the roof or floor diaphragms. Unlike lateral force procedures for rigid diaphragm structures, forces are calculated for wall lines, not stories. Also, the wall story shear at each story is not determined by a vertical distribution (such as an inverted triangle) based on a dynamic analysis as is commonly done with other evaluation and design procedures. Rather, the story shear at each story is simply the sum of the story forces.

Useful Tip

The in-plane demand on masonry walls is dependent on the diaphragm strength.

Therefore, the diaphragm capacity and any required diaphragm strengthening should be evaluated before performing the in-plane wall checks.

As shown in Section 12.11 of this *Guide*, the building has qualifying cross walls at both levels in the north-south direction. The story force is determined as the minimum of:

$$F_{wx} = 0.75S_{X1}(W_{wx} + 0.5W_d) \qquad \text{(ASCE 41-13 Eq. 15-15)}$$

$$F_{wx} = 0.75S_{X1}\left[W_{wx} + \sum W_d \frac{v_u D}{\sum\left(\sum v_u D\right)}\right] \qquad \text{(ASCE 41-13 Eq. 15-16)}$$

$$F_{wx} = 0.75S_{X1}W_{wx} + v_u D \qquad \text{(ASCE 41-13 Eq. 15-17)}$$

where:

F_{wx} = In-plane story force assigned to an individual wall line (lb)

S_{X1} = BSE-1E spectral response acceleration parameter at a 1-second period

W_{wx} = Seismic weight of an unreinforced masonry wall assigned to Level x, taken from mid-story below Level x to mid-story above Level x (lb). Refer to Table 12-4 of this *Guide*.

W_d = Total seismic weight tributary to the diaphragm (lb). Refer to Table 12-4 of this *Guide*.

ΣW_d = Sum of total seismic weight tributary to the diaphragms at and above Level x (lb)

v_u = Unit shear strength of diaphragm (lb/ft)

D = Depth of diaphragm (ft)

v_uD = Shear capacity of one end of the diaphragm at the level under consideration (lb)

$\Sigma(\Sigma v_uD)$ = Sum of shear capacities of both ends of the diaphragms coupled at and above the level under consideration (lb)

Useful Tip

The 0.5 factor in front of W_d in ASCE 41-13 Equation 15-15 indicates that the shear wall being evaluated at each end of the diaphragm resists half of the seismic weight. A flexible diaphragm, tributary area analysis assumption is made.

The second term in ASCE 41-13 Equation15-16 adjusts the loading at each diaphragm level to be proportional to the strength of the diaphragm at that level divided by the strength of all diaphragm levels. It is similar to the distribution of a common force to a series of masonry piers by relative rigidity (SEAOC, 1992).

The equations with cross walls have a small reduction on the multiplier from 0.80 to 0.75 due the ability of the cross walls to transfer some seismic loads to grade.

In the north-south direction, the following story forces are calculated for each of the side walls.

Roof:

S_{X1} = 0.507

W_{wr} = 1/2(98 kips) = 49 kips

W_d = Seismic weight of roof diaphragm plus tributary seismic weight of rear (north) and front (south) walls
= 43 kips + 26 kips + 25 kips = 94 kips

ΣW_d = Same as W_d = 94 kips

v_uD = Shear capacity of one end of the roof diaphragm
= (300 lb/ft)(60 ft)(1 kip/1,000 lb)
= 18 kips

$\Sigma(\Sigma v_uD)$ = Sum of shear capacities of both ends of roof diaphragm
= 2(300 lb/ft)(60 ft)(1 kip/1,000 lb)
= 36 kips

F_{wr} = MIN{0.75(0.507)[49 kips + 0.5(94 kips)], 0.75(0.507)[49 kips + 94 kips(18 kips/36 kips)], 0.75(0.507)(49 kips) + 18 kips}
= MIN(37 kips, 37 kips, 37 kips)
= 37 kips

Second floor:

S_{X1} = 0.507

W_{w2} = 1/2(147 kips) = 74 kips

W_d = Seismic weight of second floor diaphragm plus tributary seismic weight of rear (north) and front (south) walls
= 51 kips + 40 kips + 37 kips = 128 kips

ΣW_d = Seismic weight of roof diaphragm, second floor diaphragm, rear (north) and front (south) walls tributary to roof diaphragm, and rear (north) and front (south) walls tributary to second floor diaphragm
= 43 kips + 51 kips + 26 kips + 25 kips + 37 kips + 40 kips
= 222 kips

v_uD = Shear capacity of one end of the second floor diaphragm
= (1,500 lb/ft)(60 ft)(1 kip/1,000 lb)
= 90 kips

$\Sigma(\Sigma v_uD)$ = Sum of shear capacities of both ends of roof diaphragm and second floor diaphragm
= 2(300 lb/ft)(60 ft)(1 kip/1,000 lb) + 2(1,500 lb/ft)(60 ft)(1 kip/1,000 lb)
= 216 kips

F_{w2} = MIN{0.75(0.507)[74 kips + 0.5(128 kips)], 0.75(0.507)[74 kips + 222 kips(90 kips/216 kips)], 0.75(0.507)(74 kips) + 90 kips}
= MIN(52 kips, 63 kips, 118 kips)
= 52 kips

In the east-west direction, there are no cross walls at the first story. Therefore, the story force is determined as the minimum of:

$$F_{wx} = 0.8S_{X1}(W_{wx} + 0.5W_d) \quad \text{(ASCE 41-13 Eq. 15-13)}$$

$$F_{wx} = 0.8S_{X1}W_{wx} + v_uD \quad \text{(ASCE 41-13 Eq. 15-14)}$$

where variables are as defined for ASCE 41-13 Equations 15-15 to 15-17 above.

In the east-west direction, the front (south) wall is subject to the following story forces:

Roof:

S_{X1} = 0.507

W_{wr} = 25 kips

W_d = Seismic weight of roof diaphragm plus tributary seismic weight of side (east and west) walls
= 43 kips + 98 kips = 141 kips

v_uD = Shear capacity of one end of the roof diaphragm
= (300 lb/ft)(30 ft)(1 kip/1,000 lb)
= 9 kips

F_{wr} = MIN{0.8(0.507)[25 kips + 0.5(141 kips)], 0.8(0.507)(25 kips) + 9 kips}
= MIN(39 kips, 19 kips) = 19 kips

Second floor:

S_{X1} = 0.507

W_{w2} = 40 kips

W_d = Seismic weight of second floor diaphragm plus tributary seismic weight of side (east and west) walls
= 51 kips + 147 kips = 198 kips

v_uD = Shear capacity of one end of the second floor diaphragm;
= (1,500 lb/ft)(30 ft)(1 kip/1,000 lb)
= 45 kips

F_{w2} = MIN{0.8(0.507)[40 kips + 0.5(198 kips)], 0.8(0.507)(40 kips) + 45 kips}
= MIN(56 kips, 61 kips)
= 56 kips

Useful Tip

Note that the shear capacity of the diaphragm is conservatively based on the full length of the diaphragm. If the opening were directly abutting the shear wall, the capacity could be reduced to the net diaphragm length.

Story forces on the rear (north) wall are calculated similarly. A summary of the story forces is shown in Table 12-8.

Table 12-8 Wall In-Plane Story Force Summary

Wall	Roof (kips)	Second Floor (kips)	Total Wall Shear (kips)
Side walls	37	52	89
Front (south) wall	19	56	75
Rear (north) wall	20	55	75

12.13 In-Plane Capacity of Shear Walls (ASCE 41-13 § 15.2.3.3.2 and § 15.2.3.3.3) of a Wall with Sufficient Capacity

ASCE 41-13 § 15.2.3.3.2 and § 15.2.3.3.3 describe the requirements for the evaluation of in-plane strength for unreinforced masonry walls. The procedure is based on research and testing by the ABK Joint Venture undertaken in the early 1980s. The research program included static and dynamic testing of full scale unreinforced masonry construction and evaluation of the performance of URM buildings in past earthquakes. The project produced the *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings* (ABK, 1984).

The methodology determines the expected mode of deformation and shear resistance for each wall line at each floor. Walls are either governed by a rocking controlled mode or a shear controlled mode. For the rocking controlled mode, the bed joint at the top and bottom of each pier cracks and the pier rotates as a rigid body. Restoring force is provided by the moment due to eccentricity between the gravity axial load on the pier and the toe of the pier (point of rotation). This point is assumed to be at 90% of the length of the pier. In the shear controlled mode, piers resist shear based on the bed joint sliding strength of mortar joints. Toe crushing and diagonal tension actions are not addressed by the ASCE 41-13 Special Procedure.

A summary of the procedure for the in-plane evaluation of unreinforced masonry walls is shown in the flowchart in Figure 12-13.

To determine which behavior mode governs, the rocking capacity at each pier is compared to the shear strength of the pier (Check #1 in the Figure 12-13 flowchart, signified by the circled number "1").

$V_r < V_a$ at every pier

$$V_a = 0.67 v_{me} D t \quad \text{(ASCE 41-13 Eq. 15-19)}$$

$$V_r = 0.9 P_D \frac{D}{H} \text{ for walls with openings} \qquad \text{(ASCE 41-13 Eq. 15-21)}$$

$$V_r = 0.9\left(P_D + 0.5 P_w\right)\frac{D}{H} \text{ without openings} \qquad \text{(ASCE 41-13 Eq. 15-20)}$$

where:

V_a = Wall shear strength (lb)

v_{me} = Unreinforced masonry strength (lb/in.^{2}). See Section 12.9 of this *Guide*

D = In-plane masonry pier dimension (in.)

t = Pier thickness (in.)

P_D = Superimposed dead load at the top of the pier (lb)

H = Least clear height of opening on either side of pier (in.)

P_w = Weight of the wall (lb)

Commentary

The Special Procedure of ASCE 41-13 § 15.2 defines wall pier height as the least clear height. The pier height used in the Tier 3 calculation of rocking in ASCE 41-13 Chapter 11 is more complex, depending on the direction of loading. In addition, the Special Procedure does not make a distinction between interior wall piers and corner piers with flanged returns. For simplicity, only the rectangular portion of the corner pier on the in-plane wall line is used.

Refer to Chapter 13 of this *Guide* for further discussion of pier height and corner piers using the Tier 3 evaluation.

For the wall line to exhibit a rocking mode, each pier must have shear strength in excess of the rocking capacity. For walls that meet this criterion, the piers are expected to dissipate energy due to rigid body rotation. However, the piers must have sufficient total strength to resist the expected shear demand (Check #2 in Figure 12-13):

$$0.7 V_{wx} < \sum V_r \qquad \text{(ASCE 41-13 Eq. 15-22)}$$

where:

V_{wx} = Wall story shear (lb)

Walls without adequate capacity to satisfy this check must be strengthened.

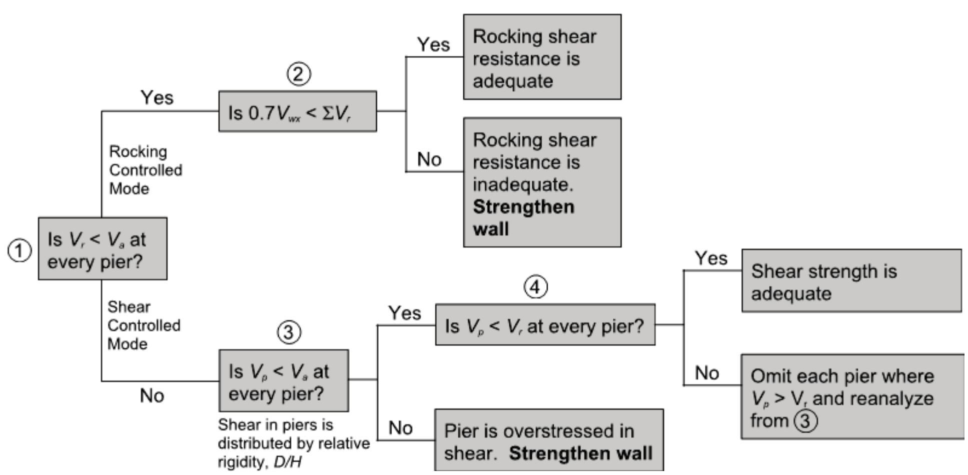


Figure 12-13 Flowchart for analysis of URM wall in-plane shear force. The circled numbers refer to the numbered checks in the following calculations. Variables are defined in the text below.

For walls where the axial load on any pier is large enough to prevent rocking until the shear strength of the mortar is exceeded (Check #1 in Figure 12-13), the shear controlled mode will govern, and pier strength is based on the bed joint sliding capacity. Shear force is distributed to individual piers on the basis of relative shear rigidity. The contribution of flexural stiffness is typically small for shear governed piers and is ignored (piers with a large proportion of flexural to shear rigidity commonly display rocking behavior). The expected pier shear demand is compared to the pier strength (Check #3 in Figure 12-13):

$V_p < V_a$ at every pier (ASCE 41-13 Eq. 15-23)

$$V_p = V_{wx} \frac{\frac{D}{H}}{\sum\left(\frac{D}{H}\right)}$$

where:

V_p = Shear force assigned to a pier on the basis of relative shear rigidity (lb)

Walls that fail to meet this criterion do not have adequate strength and must be retrofitted. However, for walls that satisfy this check, it is still necessary to confirm that rocking behavior will not occur at individual piers at the distributed demand (Check #4 in Figure 12-13):

$V_p < V_r$ at every pier (ASCE 41-13 Eq. 15-24)

If all piers meet this requirement, the wall is judged to have adequate shear strength. If any pier fails this check, its contribution to shear resistance should be omitted and the wall should be reanalyzed (from Check #3 in Figure 12-13).

12.13.1 Second Story Side Walls

For Pier 3 in Figure 12-9:

$V_a = 0.67 v_{me} Dt$ (ASCE 41-13 Eq. 15-19)

v_{me} = 49 lb/in.^{2} (per Table 12-7 of this *Guide*)

D = 96 in (per Table 12-7 of this *Guide*)

t = 13 in (per Table 12-7 of this *Guide*)

V_a = 0.67(49 lb/in.^{2})(96 in.)(13 in.)(1 kip/1,000 lb)
= 41 kips

ASCE 41-17 Revision

The bed joint sliding strength in ASCE 41-13 Equation 15-19 was revised in ASCE 41-17 to $V_a = 0.67 v_{mL} Dt$ where v_{mL} is the lower bound masonry shear strength based on the mean minus one standard deviation of the in-place mortar test shear values.

Since all of the walls have openings, the pier rocking capacity, V_r, is calculated as:

$$V_r = 0.9 P_D \frac{D}{H} \qquad \text{(ASCE 41-13 Eq. 15-21)}$$

P_D = 10,200 lb (per Table 12-7)

H = 48 in.

V_r = 0.9(10,200 lb)(96 in.)(1 kip/1,000 lb)/(48 in.)
= 18 kips

The capacities of other piers at the second story of the side wall are calculated similarly. The results are summarized in Table 12-9.

The behavior of the side wall at the second story is determined by comparing the shear capacity of each pier, V_a, to the rocking shear capacity, V_r. Since $V_r < V_a$ at every pier (per Table 12-9), the wall is expected to exhibit a rocking mode (Check #1 in Figure 12-13).

Table 12-9 Side Wall Second Story Pier Rocking Summary

Pier	V_a (kip)	V_r (kip)
1	20	5
2	31	12
3	41	18
4	31	12
5	40	18
6	20	4
∑	183	68

Since the rocking mode couples the response over all of the second story side wall piers, the rocking capacity is checked using the summation of pier rocking capacities (Check #2 in Figure 12-13):

$$0.7V_{wx} < \sum V_r \qquad \text{(ASCE 41-13 Eq. 15-22)}$$

V_{wx} = 37 kip (per Table 12-8)

ΣV_r = 68 kip (per Table 12-9)

0.7(37) = 26 kips < 68 kips

Therefore, the walls have sufficient rocking resistance and do not need to be strengthened per ASCE 41-13.

12.13.2 First Story Side Walls

The first story side walls are evaluated similarly. Calculations for representative Pier 11 (Figure 12-9) are shown below.

$V_a = 0.67 v_{me} D t$ (ASCE 41-13 Eq. 15-19)

v_{me} = 59.4 lb/in.^{2} (basis in Table 12-7)

D = 96 in (per Table 12-7)

t = 13 in (per Table 12-7)

$$V_a = 0.67(59.4 \text{ lb/in.}^2)(96 \text{ in.})(13 \text{ in.})(1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 50 \text{ kips}$$

The pier rocking capacity, V_r, is calculated as:

$$V_r = 0.9 P_D \frac{D}{H}$$ (ASCE 41-13 Eq. 15-21)

P_D = 28,200 lb (per Table 12-7)

H = 48 in.

$$V_r = 0.9(28{,}200 \text{ lb})(96 \text{ in.})(1 \text{ kip}/1{,}000 \text{ lb})/(48 \text{ in.})$$
$$= 51 \text{ kips}$$

Since $V_r > V_a$, the shear mode controls the wall behavior (Check #1 in Figure 12-13). Therefore, pier shear demand is distributed on the basis of relative shear rigidity. Although the expression is not shown in ASCE 41-13, the pier shear demand is calculated as a function of the story shear, V_{wx}. At the first story, the story shear is the sum of the roof and second story shear forces.

For Pier 11:

$$V_p = V_{wx} \frac{\frac{D}{H}}{\sum\left(\frac{D}{H}\right)}$$

V_{wx} = 89 kip (per Table 12-8)

D/H = 2.00 (per Table 12-10)

$\Sigma D/H$ = 9.00 (per Table 12-10)

$$V_p = (89 \text{ kips})\left(\frac{2}{9}\right)$$
$$= 20 \text{ kips}$$

Similar calculations are performed for the other wall piers at the first story side wall. Results are summarized in Table 12-10. Piers that fail the check for rocking behavior (Check #1 in Figure 12-13) are highlighted.

Table 12-10 Side Wall First Story Pier Rocking Summary

Pier	V_a (kip)	V_r (kip)	D/H	V_p (kip)
7	25	13	1.00	10
8	39	32	1.50	15
9	50	52	2.00	20
10	39	33	1.50	15
11	50	51	2.00	20
12	25	12	1.00	10
$\sum$	227	193	9.00	89

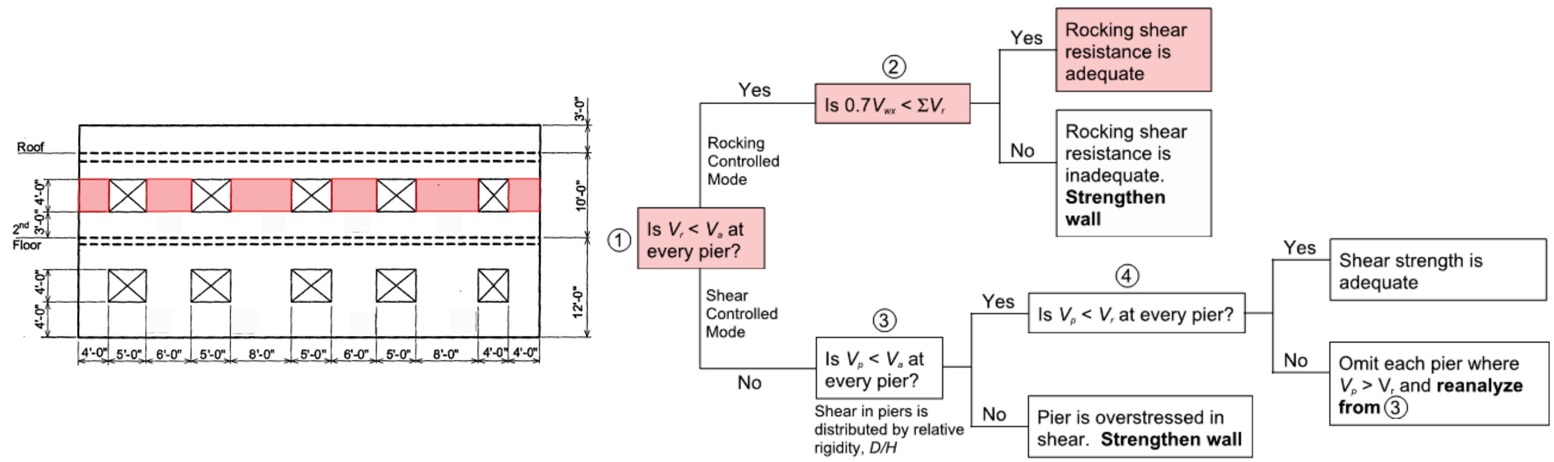
Note: Shaded cells indicate piers that fail the check for rocking behavior.

Since the shear mode controls the overall wall behavior, the shear strength at each pier is compared to the shear demand (Check #3 in Figure 12-13). From Table 12-10, $V_p < V_a$ (ASCE 41-13 Equation 15-23) at every pier, indicating that the piers have adequate shear strength.

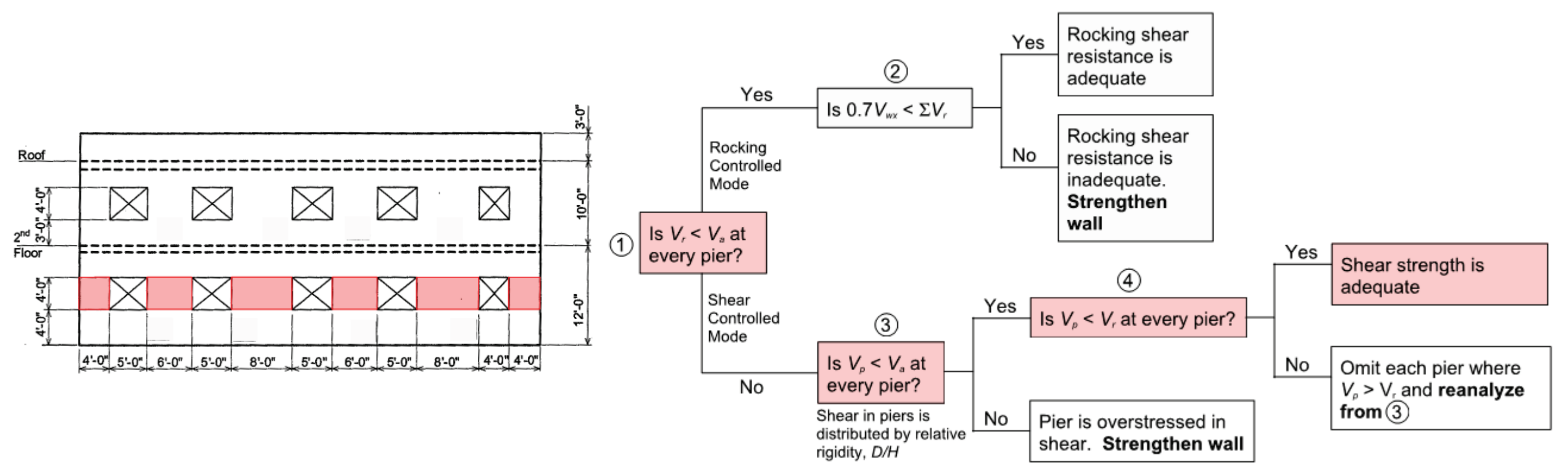
However, to check whether any individual piers are controlled by a rocking mode, the shear demand is compared to the rocking capacity (Check #4 in Figure 12-13). Table 12-10 shows that at every pier $V_p < V_r$ (ASCE 41-13 Equation 15-24). Thus, the previous check of strength based on shear behavior using all piers is accurate. The existing wall is sufficient to resist seismic forces without retrofit. The calculations are summarized in Figure 12-14.

12.14 In-Plane Capacity of Shear Walls (ASCE 41-13 § 15.2.3.3.2 and § 15.2.3.3.3) of a Wall without Sufficient Capacity

For the rear (north) wall at the second story, the pier rocking capacities and shear strengths are shown below in Table 12-11. By inspection, the rocking capacity is lower than the shear strength at every pier, so a rocking mechanism controls. From Table 12-8, 70% of the story shear is 14 kips. The sum of pier rocking capacities is much greater than the shear demand, so the rear (north) wall at the second story is deemed to have adequate shear resistance.



Second Story



First Story

Figure 12-14 Side wall in-plane calculation summary for the first story (bottom) and second story (top). Pier heights used are shaded on the wall evaluation. The flowchart is shaded to show the analysis procedure that was used for each story.

Pier rocking capacities for the rear (north) wall at the first story are shown in Table 12-12. Again, the piers are all controlled by rocking behavior. Since the sum of the pier rocking capacity is less than 70% of the story shear demand (0.7 × 75 kips = 53 kips), the piers have inadequate shear resistance and require strengthening of the rear (north) wall at the first story.

Table 12-11 Rear (North) Wall Second Story Pier Rocking Summary

Pier	V_a (kip)	V_r (kip)
13	39	19
14	18	6
15	18	6
16	39	19
∑	113	49

Table 12-12 Rear (North) Wall First Story Pier Rocking Summary

Pier	V_a (kip)	V_r (kip)
17	13	2
18	20	9
19	36	25
20	20	9
21	13	2
∑	100	45

A summary of the calculations for rear (north) wall is shown in Figure 12-15. Similar calculations performed for the front (south wall) are not shown, but the resulting requirements are the same (strengthening at the first story).

12.15 Retrofit of a Wall with Insufficient Capacity Using Shotcrete

Several options exist for strengthening the rear (north) wall to increase in-plane shear resistance. Several common approaches are discussed in ASCE 41-13 § C11.3.1.3. The retrofit techniques include the following:

- Infilling wall openings
- Enlarging wall openings (to encourage rocking behavior in shear deficient walls)
- Addition of a supplemental concrete or shotcrete wall against a deficient masonry wall
- Adding coatings such as fiber-reinforced polymer overlays

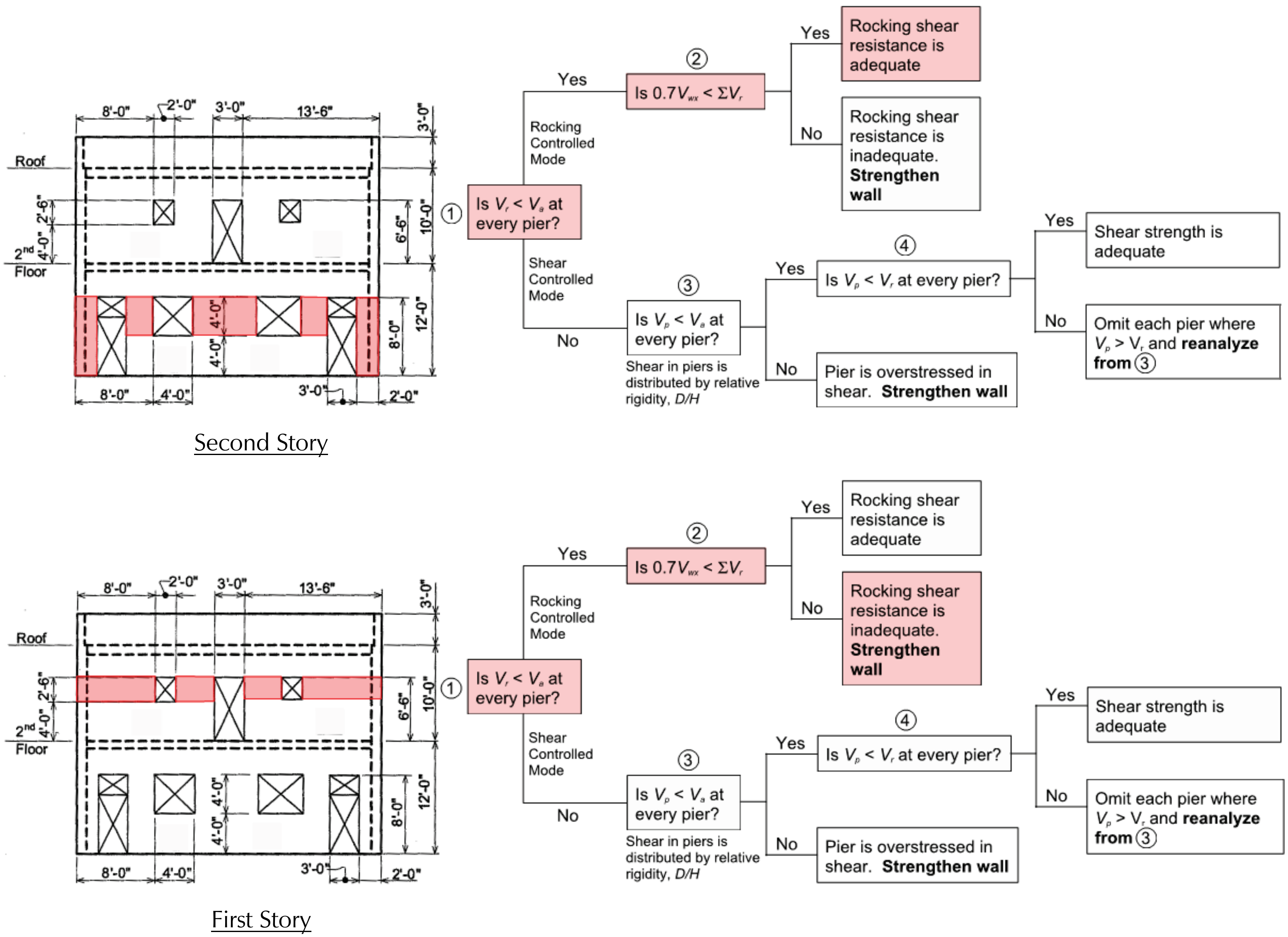


Figure 12-15 Rear (north) wall in-plane calculation summary for the first story (bottom) and second story (top). Pier heights used are shaded on the wall evaluation. The flowchart is shaded to show the analysis procedure that was used for each story.

- Adding reinforced cores with grouted mild steel or prestressed steel reinforcement
- Adding grout injection to fill voids and cracks
- Repointing areas with deteriorated mortar

These techniques are presented in greater detail in FEMA 547, *Techniques for the Seismic Rehabilitation of Existing Buildings* (FEMA, 2006), which describes a variety of considerations and discusses sample details.

> **Commentary**
>
> Although shotcrete is only applied to the first floor walls in this example, engineers should consider whether isolated strengthening of a single story would facilitate other undesirable behaviors like concentrated displacement demands at other levels.

For this building, a retrofit consisting of the addition of a shotcrete wall has been selected (see Figure 12-16). Shotcrete will be applied to the inside face of the wall. In some circumstances, it is possible to apply shotcrete to the exterior face of a masonry wall but that option is not possible for this example due to aesthetic issues. Where shotcrete will be applied to the exterior of a wall, special consideration should be given to ties to the existing diaphragm. ASCE 41-13 Chapter 11 requires a direct load path from the diaphragm to the new shotcrete wall. For this building, because floor joists run east-west, supplemental support for the second floor is not required during construction. A new ledger on the inside face of the shotcrete will connect the diaphragm directly to the wall.

Since the analysis shows adequate strength in the second story walls, shotcrete will only be applied at the first story. The vertical extent of the shotcrete will be from the existing foundation to just below the straight sheathing at the second floor diaphragm. The corner Piers 17 and 21 have a length of only 11 inches between the existing doors and the perpendicular walls. Therefore, these piers will be designed as enlarged pilasters with closed ties using a thickness greater than the rest of the shotcrete wall.

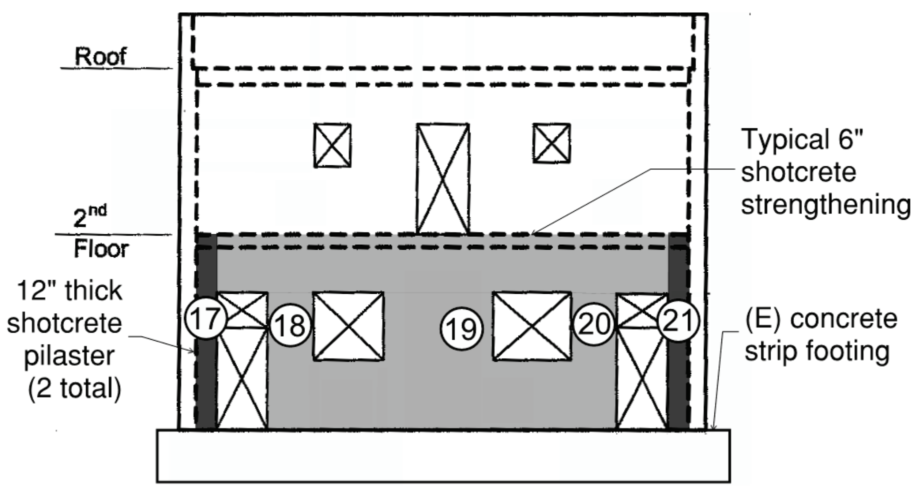


Figure 12-16 Schematic illustration of rear (north) wall shotcrete retrofit. Circled numbers indicate piers.

In order to design the new shotcrete wall, it is necessary to make an assumption about the behavior of the composite wall system. Typically, the design involves assigning shear demands to concrete and masonry piers using a force-design philosophy. The following approaches are commonly used:

- **Method #1:** Assume the shotcrete wall carries 100% of the shear demand and ignore any contribution to strength or stiffness from the existing masonry.
- **Method #2:** Design the shotcrete wall for 100% of the shear but check the masonry for shear demands based on the relative rigidity of the masonry piers.
- **Method #3:** Design each concrete and masonry pier for a portion of the total wall shear based on the relative rigidity of each component.

The approaches are discussed further in FEMA 547. Using Method #2 typically results in the most conservative design for the new shotcrete. Given the limited displacement capacity of the existing masonry, it is possible that if only Method #1 is used, the existing piers will be significantly damaged at the design shear force (particularly with flexure-critical concrete piers). For minor retrofits, like the one in this example, the cost difference between the approaches may be limited. In larger structures requiring extensive retrofits, the selected design criteria can have significant impact on retrofit cost. In this example, all three approaches are shown for completeness.

12.15.1 Method #1

In order to assign shear force to the piers, the additional seismic mass of the shotcrete wall is calculated to determine the revised story shear demand. The preliminary design assumes a typical shotcrete thickness of 6 inches. At the pilasters for Piers 17 and 21, the shotcrete thickness is assumed to be 12 inches.

$W_{\text{Shotcrete}}$ = Weight of 6-inch shotcrete wall (Area 1) minus the wall openings (Area 2) plus 6 inches of additional concrete at the pilasters (Area 3). One half of the 12'-0" first story height is tributary to the second floor diaphragm for this calculation.

Area 1 $= (6\text{ ft})\{30\text{ ft} - 2[(13\text{ in.})/(12\text{ in./ft})]\} = 167\text{ ft}^2$

Area 2 $= (2\text{ ft})(3\text{ ft} + 4\text{ ft} + 4\text{ ft} + 3\text{ ft}) = 28\text{ ft}^2$

Area 3 $= 2(6\text{ ft})[(11\text{ in.})/(12\text{ in./ft})] = 11\text{ ft}^2$

Area $= 167 - 28 + 11 = 150\text{ ft}^2$

$W_{\text{Shotcrete}}$ = (150 ft^2)[(6 in.)/(12 in./ft)](150 lb/ft3)
= 11 kips

The resulting shear force is calculated using ASCE 41-13 § 15.2.3.3.1, similar to Section 12.12 of this example. The resulting story force is F_{w2} = 60 kips and the story shear is V_{w2} = 80 kips.

For this design using Method #1, only the relative rigidity of the individual shotcrete piers is needed. Since the aspect ratio of the piers varies, rigidity is computed considering both shear and flexural components. Given the deep spandrel above the piers, the rigidity is approximated using an assumption of full restraint against rotation at the top and bottom of each pier.

$$k_{\text{Shotcrete}} = \frac{1}{\dfrac{h_{\text{eff}}^{\ 3}}{12E_cI_g} + \dfrac{h_{\text{eff}}}{A_vG_c}}$$

where:

$k_{\text{Shotcrete}}$ = Shotcrete pier stiffness (kip/in.)

h_{eff} = Pier height (in.)

E_c = Concrete elastic modulus (kip/in.^{2})

I_g = Moment of inertia of gross section (in.^{4})

A_v = Shear area (in.^{2})

G_c = Concrete shear modulus (kip/in.^{2})

The retrofit will use concrete with a nominal compressive strength f_c' = 4,000 lb/in.^{2}. Note that the ASCE 41-13 Special Procedure provides no guidance on what stiffness values to use. For this example, nominal material properties and gross sections are used to derive the stiffness values. For Pier 17, the stiffness (or rigidity) is calculated as shown below.

h_{eff} = 96 in

E_c $= 57\sqrt{f_c'}$
= 3,605 kip/in.^{2}

I_g = (12 in.)(11in.)^3/12
= 1,331 in.^{4}

A_v = (11 in.)(12 in.) = 132 in.^{2}

G_c $= \dfrac{E_c}{2(1+\nu)}$
= (3,605 kip/in.^{2})/[2(1+0.17)]
= 1,541 kip/in.^{2}

ν = Poisson's ratio for concrete
= 0.17

$$k_{\text{Shotcrete}} = \left[\left(\frac{(96\text{ in.})^3}{12\left(3{,}605\text{ kip/in.}^2\right)\left(1{,}331\text{ in.}^4\right)} \right) + \left(\frac{96\text{ in}}{\left(132\text{ in.}^2\right)\left(1{,}541\text{ kip/in.}^2\right)} \right) \right]^{-1}$$

$= 63$ kip/in.

Similar calculations are performed for Piers 18 through 21. The resulting stiffnesses are summarized in Table 12-13.

Table 12-13 Method #1 Design Calculations for Concrete Piers at Rear (North) Wall

Pier	k (kip/in.)	F_{Concrete} (kip)	v_{Concrete} (lb/in.^{2})	v_{Concrete} ($\sqrt{f'_c}$)
17	63	0	2	0.0
18	3,940	16	74	1.2
19	11,652	47	109	1.7
20	3,940	16	74	1.2
21	63	0	2	0.0
$\sum$	19,658	80	-	-

Since the piers are in series, the total stiffness for the wall line is the sum of the pier stiffnesses. Shear is distributed to the piers based on relative rigidity, as shown below for Pier 18.

$$F_{\text{Concrete}} = V_{w2}\left(\frac{k}{k_{\text{Total}}}\right)$$

F_{Concrete} = (80 kips)(3,940 kip/in.)/(19,658 kip/in.)
= 16 kips

Calculations for the remaining piers are shown in Table 12-13. Additionally, the table shows the shear stress in each pier.

12.15.2 Methods #2 and #3

Method #2 represents an intermediate approach that implements results from Method #1 (where forces are based on 100% of shear in concrete) and Method #3 (where shear is distributed to shear and masonry based on relative rigidity). Therefore, no separate calculations are required for Method #2, and the results can be inferred from calculations using the other two methods.

For Method #3, both the rigidity of the shotcrete retrofit piers and the existing piers are needed. Testing has shown that the complete wall assembly behaves as a composite unit. The calculation of shotcrete pier stiffness is identical to Method #1. For masonry piers, the stiffness is calculated as shown below.

$$k = \frac{1}{\dfrac{h_{\text{eff}}^{3}}{12E_m I_g} + \dfrac{h_{\text{eff}}}{A_v G_m}} \qquad \text{(ASCE 41-13 Eq. C11-2)}$$

where:

k = Masonry pier stiffness (kip/in.)

E_m = Masonry elastic modulus (kip/in.^{2})

G_m = Masonry shear modulus (kip/in.^{2})

> **Useful Tip**
>
> Note that TMS 402-11 uses the notation E_v for shear modulus instead of G_m per ASCE 41-13.

Per Table 11-2(a) of ASCE 41-13, the default lower bound masonry compressive strength is assumed to be f'_m = 600 lb/in.^{2}. The lower bound strength is used to be consistent with the nominal strength used for the concrete. ASCE 41-13 § 11.2.3.4 and § 11.2.3.7 reference TMS 402-11 (TMS, 2011) Section 1.8.2.2 for standard equations to calculate the elastic modulus and shear modulus of unreinforced masonry materials.

$$\begin{aligned} E_m &= 700 f'_m && \text{(TMS 402-11 § 1.8.2.2.1)} \\ &= 700(600 \text{ lb/in.}^2)(1 \text{ kip}/1{,}000 \text{ lb}) \\ &= 420 \text{ kip/in.}^2 \end{aligned}$$

$$\begin{aligned} G_m &= 0.4E_m && \text{(TMS 402-11 § 1.8.2.2.2)} \\ &= 0.4(420 \text{ kip/in.}^2) \\ &= 168 \text{ kip/in.}^2 \end{aligned}$$

The equation in TMS 402-11 Section 1.8.2.2.2 implies Poisson's ratio ν = 0.25, as $G_m = E_m/[2(1 + \nu)] = E_m/[2(1+0.25)] = 0.4E_m$. For Pier 18, the masonry stiffness is computed as shown below.

h = 48 in.

$$\begin{aligned} I_g &= (13 \text{ in.})(36 \text{ in.})^3/12 \\ &= 50{,}544 \text{ in.}^4 \end{aligned}$$

A_v = (36 in.)(13 in.) = 468 in.^{2}

$$\begin{aligned} k &= \left(\frac{(48 \text{ in.})^3}{12\left(420 \text{ kip/in.}^2\right)\left(50{,}544 \text{ in.}^4\right)} \right) + \left(\frac{48 \text{ in.}}{\left(468 \text{ in.}^2\right)\left(168 \text{ kip/in.}^2\right)} \right)^{-1} \\ &= 957 \text{ kip/in.} \end{aligned}$$

Stiffnesses for the remainder of the masonry piers are calculated similarly and the concrete piers are calculated according to Method #1 (Section 12.14.1 of this example). The results are summarized in Table 12-14.

Shear is distributed to each of the concrete and masonry piers based on their relative contribution to the cumulative stiffness.

$$k_{Total} = \Sigma k_{Concrete} + \Sigma k_{Masonry}$$
$$= 19{,}658 \text{ kip/in} + 4{,}844 \text{ kip/in}$$
$$= 24{,}501 \text{ kip/in}$$

Table 12-14 Method #3 Design Calculations for Concrete and Masonry Piers at Rear (North) Wall

Pier	$k_{Concrete}$ (kip/in.)	$k_{Masonry}$ (kip/in.)	$F_{Concrete}$ (kip)	$F_{Masonry}$ (kip)	$v_{Concrete}$ (lb/in.^{2})	$v_{Concrete}$ ($\sqrt{f'_c}$)	$v_{Masonry}$ (lb/in.^{2})
17	63	74	0	0	2	0.0	1
18	3,940	957	13	3	59	0.9	7
19	11,652	2,782	38	9	88	1.4	10
20	3,940	957	13	3	59	0.9	7
21	63	74	0	0	2	0.0	1
∑	19,658	4,844	64	16	-	-	-

Shear is distributed to the concrete and masonry components of Pier 18 based on relative rigidity.

$$F_{Concrete} = V_{w2}\left(\frac{k_{Concrete}}{k_{Total}}\right)$$

$$F_{Concrete} = (80 \text{ kips})(3{,}940 \text{ kip/in.})/(24{,}501 \text{ kip/in.})$$
$$= 13 \text{ kips}$$

$$F_{Masonry} = V_{wx}\left(\frac{k_{Masonry}}{k_{Total}}\right)$$

$$F_{Masonry} = (80 \text{ kips})(957 \text{ kip/in.})/(24{,}501 \text{ kip/in.})$$
$$= 3 \text{ kips}$$

The summary of the values for both concrete and masonry piers are shown in Table 12-14.

Shear demands and shear stresses for the remaining piers are shown in Table 12-14 of this *Guide*. For Methods #2 and #3, the shear in the masonry pier needs to be compared to the capacity. Pier 19 has the highest demand at 10 psi. The capacity from Table 12-12 is V_a/A_n = 36,000 lb/936 in.^{2} = 38 psi. Thus, the shotcrete attracts sufficient load away from the masonry such that the masonry pier capacity of 38 psi is sufficient to resist the remaining demand of 10 psi.

A comparison between the design forces that would result for each method is shown in Table 12-15. Given the relatively low story shear in this example, the pier forces are similar for each of the three methods. However, in large multistory buildings, it is common for the methods to result in substantially different designs.

Table 12-15 Rear (North) Wall First Story Pier Design Force Summary

Pier	Method #1: Design concrete for 100% shear		Method #2: Design concrete for 100% shear and check masonry by rigidity		Method #3: Distribute shear to concrete and masonry by rigidity	
	$F_{Concrete}$ (kip)	$F_{Masonry}$ (kip)	$F_{Concrete}$ (kip)	$F_{Masonry}$ (kip)	$F_{Concrete}$ (kip)	$F_{Masonry}$ (kip)
17	0	0	0	0	0	0
18	16	0	16	3	13	3
19	47	0	47	9	38	9
20	16	0	16	3	13	3
21	0	0	0	0	0	0
$\sum$	80	0	80	16	64	16

12.15.3 Shotcrete Wall Design

For this example, the wall retrofit will be designed using the shear demands from Method #2. Figure 12-17 shows an elevation of the proposed retrofit.

Commentary

Although ASCE 41-13 § 15.2 does not explicitly define the shear force demand intended for use in the design of in-plane wall retrofits, this example recommends using the shear force consistent with the wall evaluation, V_{wx}.

The design of the shotcrete wall retrofit is based on the requirements of ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2011), for new concrete walls (per ASCE 41-13 § 10.3.1.1). Minimum reinforcement requirements for the new shotcrete walls are discussed in ACI 318-11 Section 21.9.2. For piers with shear stress demand greater than $2\sqrt{f'_c}$, two curtains of reinforcement are required. Additionally, for piers with shear stress demand greater than $\sqrt{f'_c}$, the minimum vertical and horizontal reinforcement ratio is 0.0025.

ASCE 41-17 Revision

Demands, such as wall in-plane demands for design of new elements used for retrofit were revised in ASCE 41-17.

From Table 12-13, all of the piers have shear stress demand less than $2\sqrt{f'_c}$, so only one curtain of reinforcement is required. The retrofit wall design consists of 6 inches of shotcrete and one curtain of distributed horizontal reinforcement and one curtain of distributed vertical reinforcement.

To provide additional ductility capacity, the piers will be designed for flexure-controlled behavior, although this is not required by either ACI

318-11 or ASCE 41-13. Thickened corner pilasters will have four vertical bars confined by closed column ties. The preliminary wall design calls for distributed horizontal reinforcement using #4 bars at 12 inches on center. The typical reinforcement ratio is checked below.

$\rho_{\min} = 0.0025$ (ACI 318-11)

$\rho = (0.2 \text{ in.}^2)/[(6 \text{ in.})(12 \text{ in.})]$
$= 0.0028 > 0.0025$

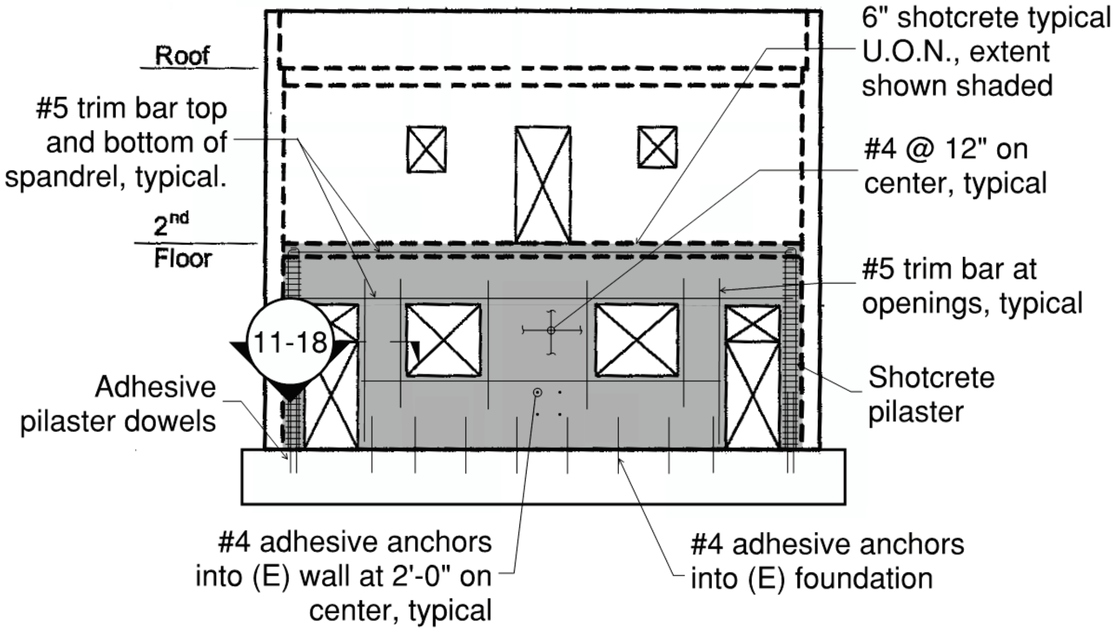


Figure 12-17 Elevation view showing rear (north) wall shotcrete retrofit.

The design of Piers 18 and 20 are shown in this example. The shear capacity is calculated using the typical horizontal reinforcement:

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y\right) \quad \text{(ACI 318-11 Eq. 21-7)}$$

where:

A_{cv} = Gross shear area of concrete (in.^{2})

α_c = Coefficient defining relative contribution of concrete strength to nominal wall shear strength

λ = Modification factor for lightweight concrete

f'_c = Compressive strength of concrete (lb/in.^{2})

ρ_t = Ratio of gross distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement

f_y = Yield strength of steel reinforcement (lb/in.^{2})

Useful Tip

Design of retrofits by ASCE 41-13 does not require the use of a reduction factor, ϕ, in calculations of component strength. See, for example, ASCE 41-13 § 11.3 for masonry and ASCE 41-13 § 10.3 for concrete.

Although retrofit design is not included in ASCE 41-13 § 15.2, this example follows the same general convention used in the remainder of ASCE 41-13.

The code and commentary for 2015 IEBC Section A108.1 also state that capacity reduction factors need not be used for newly added components.

The contribution of concrete strength to wall shear strength depends on the pier aspect ratio, h_w/l_w, per ACI 318-11 Section 21.9.4.1. For Piers 18 and 20, h_w/l_w = (48 in.)/(36 in.) = 1.3. Since the ratio is less than 1.5, α_c = 3.0. The shear strength of the piers is calculated below.

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_y\right) \quad \text{(ACI 318-11 Eq. 21-7)}$$

where:

A_{cv} = (36 in.)(6 in.)
= 216 in.^{2}

α_c = 3.0

λ = 1.0 for normal weight shotcrete

f_c' = 4,000 lb/in.^{2}

ρ_t = (0.2 in.^{2})/[(6 in.)(12 in.)]
= 0.0028 or 0.28 %

f_y = 60,000 lb/in.^{2} nominal reinforcement strength

V_n = (216 in.^{2})[3.0(1.0)(√4,000 lb/in.^{2}) + (0.0028)(60,000 lb/in.^{2})](1 kip)/(1,000 lb)
= 77 kips

DCR = 16 kips (per Table 12-14)/77 kips
= 0.21

To ensure flexure-controlled behavior, the moment demand corresponding to the shear strength is used to set an upper bound for the pier flexural capacity. For this calculation, it is conservative to assume that the inflection point for the pier occurs at midheight, since it results in the lowest moment demand.

$M_{\max}$ = $V_n H/2$
= (77 kips)(48 in.)/2
= 1,850 kip-in
= 154 kip-ft

Similarly, a lower bound for the pier moment capacity is established using the design shear force. The pier should have flexural capacity greater than or equal to the shear demand to satisfy minimum strength requirements. In this case, it is conservative to assume that the lower portion of the wall is stiffer than the spandrel and that the inflection point occurs at a third point on the height of the pier (since it results in the greatest moment demand).

$M_{\min}$ = $2V_U H/3$
= 2(16 kips)(48 in.)/3
= 510 kip-in.

= 43 kip-ft

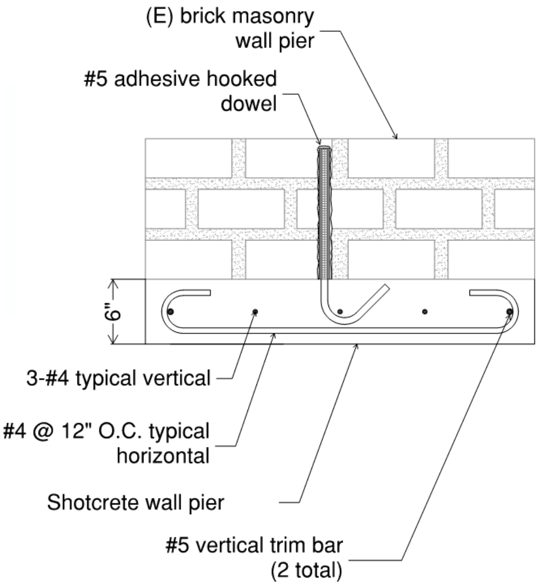
The corresponding gravity load includes the shotcrete wall self-weight. The critical section for each pier is assumed to be at the bottom of the adjacent window or door opening, so the gravity load is estimated at that elevation. For Piers 18 and 20, the critical section occurs at the base of the window opening, 4 inches above the base of the wall.

P = Weight of shotcrete wall above the critical section for the pier
= (6 in.)/(12 in.)(150 lb/ft^3)[(6.5 ft)(4 ft) + (3 ft)(4 ft)](1 kip/1,000 lb)
= 2.9 kips

The preliminary design of Piers 18 and 20, uses one curtain of #4 bars at 12 inches on center. In order to allow the hooked dowel (see Figure 12-18) to engage a vertical bar, the vertical reinforcing spacing is reduced to approximately 7-3/4 inches on center. Additionally, ACI 318-11 Section 14.3.7 requires #5 trim bars at door and window openings, so the end bars are replaced with #5 vertical reinforcement. The resulting axial force-moment interaction diagrams are shown in Figure 12-18. Both the nominal strength diagram and the expected strength diagram are plotted.

Useful Tip

Seismic axial force due to global overturning is ignored in the design of this retrofit. Engineers should consider whether seismic axial demands will have a significant effect on behavior for each structure. It could also be argued that in order for the pier to flexurally yield, it will have to pick up the URM portion doweled to the concrete above the pier. This increase in the axial load would increase the associated moment capacity for this lightly loaded pier as shown in Figure 12-18.



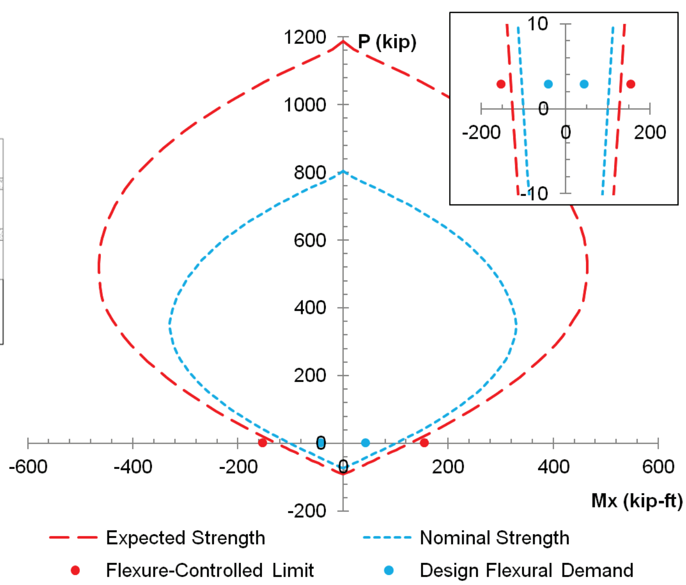


Figure 12-18 Pier 18 and 20 section showing proposed reinforcement (left) and axial force-moment interaction diagram (right) for the pier. The inset image at the top right shows a magnified view of the interaction diagram near the upper and lower bound strength limits (centered on the origin).

Since the point corresponding to the design flexural demand is contained within the nominal strength interaction surface, the pier has sufficient strength to resist the design forces. The point corresponding to the flexure-controlled limit (the flexural demand associated with the shear strength of the pier) is outside of the expected strength surface. That implies that the pier will yield in flexure before failing in diagonal shear, indicating flexure-controlled behavior.

ACI 318-11 Section 21.9.6 describes the requirements for wall boundary elements. ACI 318-11 Section 21.9.6.2 applies to vertically continuous wall piers that deflect in single curvature. Since Piers 18 and 20 are bounded by stiff spandrel beams, this criterion does not apply. The expected deflected shape for the pier is shown in Figure 12-19. ACI 318-11 Section 21.9.6.3 provides an alternate boundary element check based on the maximum extreme fiber compressive stress assuming linear elastic response of the gross concrete section. For piers with compressive stress greater than $0.2 f_c'$, boundary elements are required.

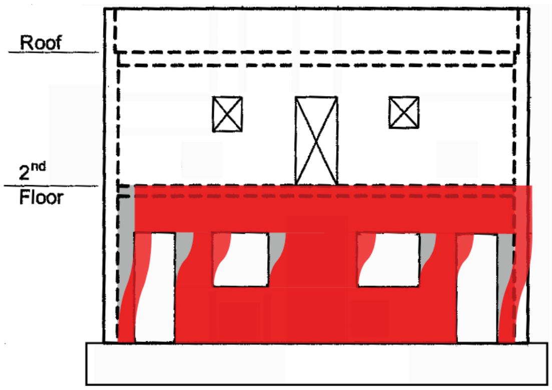


Figure 12-19 Assumed deflected shape in double curvature for the first story.

σ_{min} $= 0.2 f_c'$ (ACI 318-11)
$= 0.2(4{,}000 \text{ lb/in.}^2)$
$= 800 \text{ lb/in.}^2$

σ_g = Compressive stress due to gravity loads
= (2.9 kips)/(6 in.)(36 in.)
$= 13 \text{ lb/in.}^2$

M_{min} = 43 kip-ft (lower bound capacity calculated above)

σ_e = Compressive stress due to bending design force level
$= My/I$
$= M_{min}y/I$
$= (43 \text{ kip-ft})(12 \text{ in./ft})(18 \text{ in.})(1{,}000 \text{ lb/kip})/[(1/12)(6 \text{ in.})(36 \text{ in.})^3]$
$= 398 \text{ lb/in.}^2$

σ_{Total} = Compressive stress due to gravity loads plus compressive stress due to bending
$= 13 \text{ lb/in.}^2 + 398 \text{ lb/in.}^2$
$= 411 \text{ lb/in.}^2$

$\sigma_{Total} < \sigma_{min}$

Useful Tip

Unlike typical concrete design, the boundary element check performed in this example does not use the concrete compression block and tension resistance of steel reinforcement. The check assumes that concrete is a homogenous isotropic material with a linear stress and strain distribution.

Since the extreme fiber compressive stress is less than the ACI limit, boundary elements are not required. Additional wall reinforcement requirements are included in ACI 318-11 Chapter 14. According to the requirements of ACI 318-11 Section 14.3.5, reinforcement should not be spaced farther apart than three times the wall thickness. The 12-inch spacing for typical reinforcement satisfies this requirement.

> **Commentary**
>
> This example uses "Method # 2" where the shotcrete is designed to take 100% of the load. If "Method # 3" were considered for a structure and boundary zones were found to be required, the engineer should reconsider whether such an approach should be implemented since the method relies on the URM piers resisting load, and they would not have confinement and thus similar ductility in compression as the shotcrete.

Additional checks for the design and detailing of the shotcrete wall retrofit are not included in this example. Adhesive anchors spaced at 2'-0" on center will connect the shotcrete retrofit to the existing masonry wall and ensure composite behavior (and strengthen the wall against out-of-plane forces as shown in Section 12.16 of this *Guide*). Transfer of shear force from the shotcrete wall to the foundation system should be considered. This design uses adhesive dowels into the existing foundation which are cast into the shotcrete wall. Calculations for the length of the dowel both into the footing and into the wall are not provided here. Additionally, this example does not illustrate design calculations to verify the adequacy of the existing continuous wall footing.

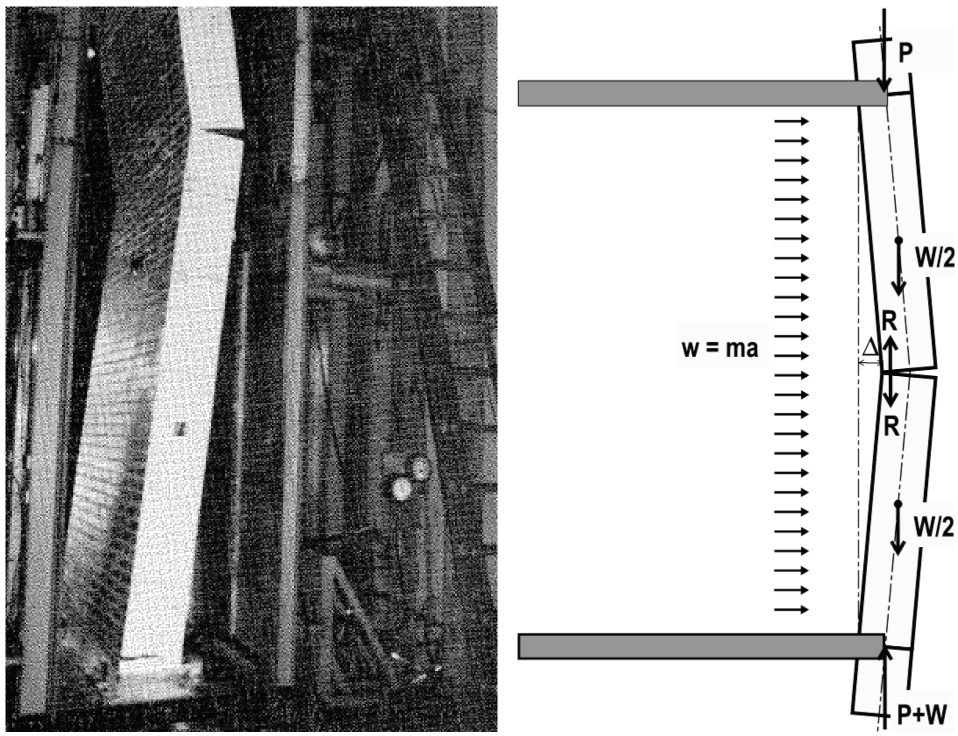
Foundation and bearing checks are not required by ASCE 41-13 Chapter 15. Overturning failures have rarely been observed in damaged, retrofitted URM buildings with squat aspect ratios. However, qualitative consideration of potential weakness in vertical shear strength where wall openings align vertically up the height of walls should be considered. For this particular example, alignments are not detrimental by inspection.

12.16 Out-of-Plane URM Wall Checks and Strengthening (ASCE 41-13 § 15.2.3.4)

Per ASCE 41-13 § 15.2.3.4, out-of-plane checks of masonry walls are required when $S_{X1} \geq 0.133$. The evaluation of URM walls for out-of-plane forces using the Special Procedure is not based on a structural analysis using the flexural capacity of the wall. Instead, the procedure provides prescriptive limits for the ratio of wall height-to-thickness (h/t) based on the results of testing and previous analytical studies. URM walls that satisfy the prescriptive limits are considered stable, while walls that are taller and narrower than allowed require supplemental out-of-plane support.

> **ASCE 41-17 Revision**
>
> The trigger for requiring out-of-plane checks of walls was revised in ASCE 41-17 to use $S_{D1} \geq 0.133$.

The allowable h/t limits are based on a series of dynamic (shake table) tests of unreinforced masonry walls conducted in *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-of-Plane* (ABK, 1981d). The tests determined that unreinforced masonry walls resist out-of-plane forces due to nonlinear elastic arching action that occurs as bed joints crack at connections of the walls to the diaphragms and at midheight of the walls between diaphragms. The wall segments rotate as rigid bodies, as shown in Figure 12-20. Initially, the eccentricity between the reaction at the center of the wall and the applied vertical force at each story level generates a restoring moment. As the displacement increases, the restoring moment decreases until the reaction moves outside of the line of action between the applied vertical forces (at the story levels). Under any further displacement, the wall is unstable and will fail out-of-plane. Priestley and Pauley created a model of the overall force-deformation behavior based on the test results (Priestley, 1985; Pauley, 1992). Figure 12-21 shows the acceleration-displacement curve and the moment-curvature plot for out-of-plane behavior.



(a) (b)

Figure 12-20 URM walls under out-of-plane forces: (a) Image of out-of-plane wall behavior from shake table testing (left) by ABK (1981d); (b) Free-body diagram for a multi-story unreinforced masonry wall spanning vertically under out-of-plane forces (center) based on Pauley (1992).

Allowable height-to-thickness ratios are shown in ASCE 41-13 Table 15-4. For sites where $S_{X1} > 0.4$, the h/t ratio is dependent on the presence or absence of qualifying cross walls and the results of the diaphragm analysis (shown in Figure 12-12 of this *Guide* and ASCE 41-13 Figure 15-1). In addition, the footnotes of ASCE 41-13 Table 15-4 also have minimum strength requirements in order to use certain h/t ratios. The requirements for Column A in ASCE 41-13 Table 15-4 require a minimum v_{te} of 100 psi or v_{te}

of 60 psi with 50% collar joint coverage. In Section 12.9 of this *Guide*, v_{te} was found to be 76 psi. Collar joints were found to be fully filled during in-place shear testing; thus, the ASCE 41-13 Table 15-4 footnote requirement is met and Column A values can be used. For buildings where diaphragm strengthening is required, the *h*/*t* ratio limit should be based on the retrofitted condition.

- In Region 1 (of ASCE 41-13 Figure 15-1), *h*/*t* ratios from Column A of Table 15-4 may be used if qualifying cross walls are present in all stories.
- In Region 2, *h*/*t* ratios from Column A of Table 15-4 may be used whether or not qualifying cross walls are present.
- In Region 3, *h*/*t* ratios from Column B of Table 15-4 shall be used whether or not qualifying cross walls are present.

Commentary

For determining wall height-to-thickness limits the presence or absence of qualifying cross walls are evaluated in each direction. Cross walls should be present in the direction perpendicular to the masonry walls under consideration.

The diaphragm demand-capacity ratio used to determine *h*/*t* ratios is for the loading direction perpendicular to the walls under consideration (i.e., the out-of-plane direction).

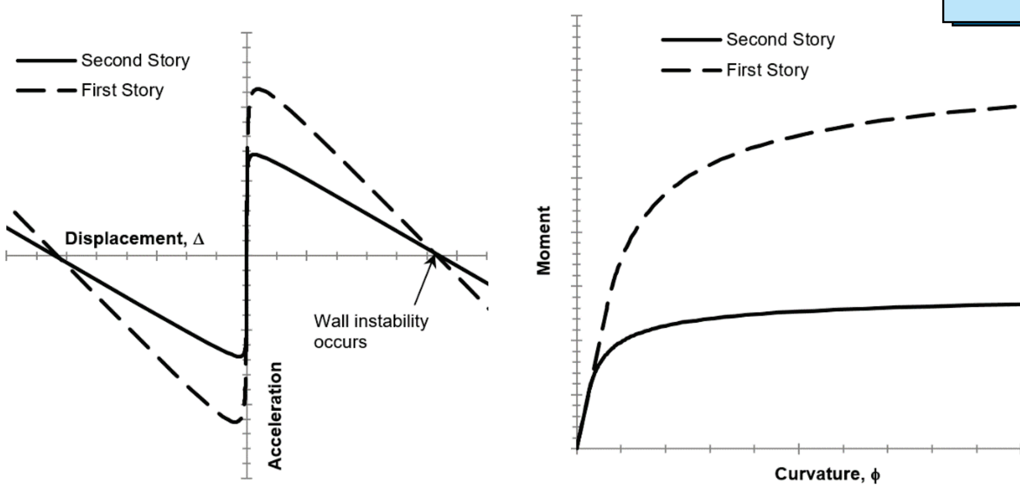


Figure 12-21 Acceleration-displacement curve for out-of-plane behavior (left) and moment-curvature (right). Since the first story wall has greater gravity load, it has a higher resistance to out-of-plane forces. The behavior is nonlinear elastic, and the wall loads and unloads along the same path. The trend of increasing resistance with increasing gravity axial load is implied in ASCE 41-13 Table 15-4.

Since S_{X1} = 0.507 (per Section 12.5 of this *Guide*), the *h*/*t* limits will be based on the results of the diaphragm evaluation.

For the side walls, the diaphragm evaluation in the east-west direction is used. From Section 12.6.5 of this *Guide*, qualifying cross walls are present at the second story, but not at the first story.

S_{X1} = 0.507 > 0.4

The second story DCR is in Region 3 (based on the more conservative of Points “a” and “b” in Figure 12-22). Therefore, *h*/*t* ratios under Column B in Table 15-4 must be used.

h/t = 9 (top story of multi-story building)

The first story DCR is in Region 3 (based on Point “a” in Figure 12-22). Therefore, *h*/*t* ratios under Column B in Table 15-4 must be used.

h/t = 15 (first story of multi-story building)

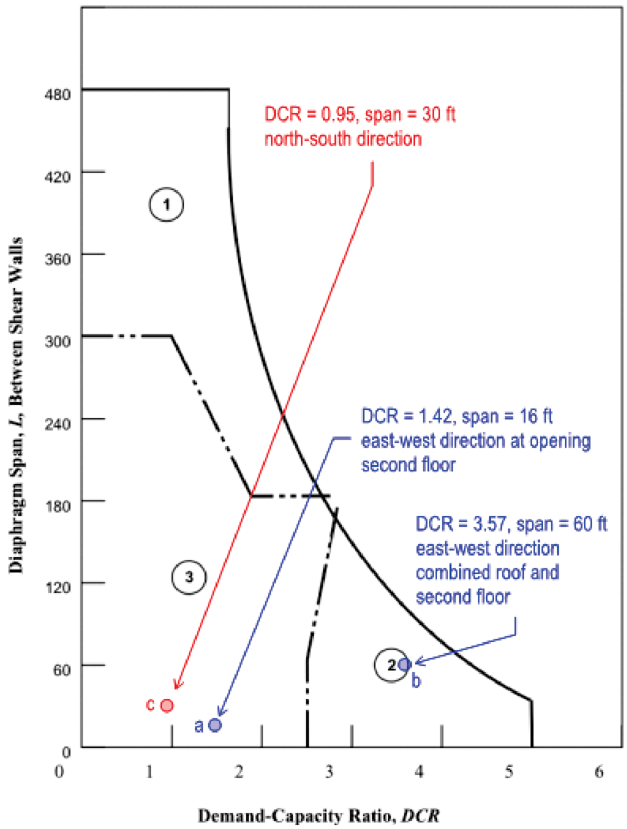


Figure 12-22 Diaphragm demand-capacity ratios (reprinted from Figure 12-12). Blue text refers to the east-west diaphragm evaluation which affects the side walls. Red text refers to the north-south evaluation, affecting the rear (north) and front (south) walls.

The *h*/*t* ratios for the side walls are calculated as shown below.

h/t (Second Story) = (10 ft)(12 in./ft)/(13 in.) = 9.2

h/t (First Story) = (12 ft)(12 in./ft)/(13 in.) = 11.1

Since the second story *h*/*t* ratio exceeds the allowable limit, a retrofit to provide additional bracing is required (although it is very close). The height-to-thickness ratio for the side walls at the first story is lower than the allowable limits, so the walls are considered adequate to resist out-of-plane seismic forces under the Special Procedure.

For the rear (north) wall and front (south) wall, qualifying cross walls are present at the first story and the second story (per Section 12.11 of this *Guide*).

$S_{X1} = 0.507 > 0.4$

The second and first story DCRs are in Region 3 (based on Point "c" in Figure 12-22). Therefore, h/t ratios under Column B in Table 15-4 must be used.

h/t (Second Story) = 9 (top story of multi-story building)

h/t (First Story) = 15 (first story of multi-story building)

The height-to-thickness ratios are the same as the ratios for the side walls. Since the second story h/t ratio exceeds the allowable limit, a retrofit to provide additional bracing is required.

12.16.1 Brace Design

To strengthen the second story walls, steel tube braces will be provided, spanning vertically from the second floor to the roof diaphragm. This section provides an example of the strengthening at the rear (north) wall only. The posts will be supported on the top edge of the new shotcrete wall at the second floor.

Demands for the retrofit bracing are based on ASCE 41-13 § 7.2.11.2 at the Collapse Prevention Performance Level.

$$F_p = 0.4 S_{XS} \chi W_p \qquad \text{(ASCE 41-13 Eq. 7-13)}$$

$$F_{p,\min} = 0.1 \chi W_p \qquad \text{(ASCE 41-13 Eq. 7-14)}$$

where:

F_p = Out-of-plane design force for walls (kips)

S_{XS} = Short period design acceleration parameter
= 0.913 (per Section 12.5 of this *Guide*)

χ = Factor from ASCE 41-13 Table 7-2 based on selected Performance Level
= 1.0 (for Collapse Prevention)

W_p = Unit weight of wall (lb/ft^2)
= 130 lb/ft^2

$$F_p = 0.4(0.913)(1.0)(130) \geq 0.1(1.0)(130)$$
$$= 47 \text{ lb/ft}^2 \geq 13 \text{ lb/ft}^2$$

Commentary

The out-of-plane demand on URM walls is affected by both the diaphragms above and below the story under consideration (except at the first story, where only the diaphragm above should be considered).

ASCE 41-13 does not explicitly state how the h/t limit should be determined when diaphragms above and below a wall have a different DCR. However, consistent with other codes (e.g., 2015 IEBC) and guidelines, this example recommends using the lower (more conservative) h/t limit that would result from the DCR of the diaphragm above and below the wall under consideration.

Commentary

ASCE 41-13 § 15.2 does not provide demands for the design of out-of-plane bracing for deficient masonry walls.

In this example, the demands are taken from ASCE 41-13 § 7.2.11.2 at the Collapse Prevention Performance Level. The Performance Level is consistent with ASCE 41-13 § 15.2.1 of the Special Procedure.

ASCE 41-17 Revision

Wall bracing demands were clarified in ASCE 41-17 and detailing requirements were added to be consistent with the 2015 IEBC. In addition, χ was revised from 1.0 to 0.8 in ASCE 41-17.

ASCE 41-13 does not provide guidance on the design of out-of-plane bracing of deficient walls. However, similar provisions in the 2015 IEBC reflect a general consensus among practitioners regarding appropriate design and detailing requirements. 2015 IEBC Section A113.5.2 requires the following. Note that these requirements were added to ASCE 41-17.

- The braces should be anchored to the masonry wall.
- Horizontal spacing of braces should not exceed the minimum of one-half of the unsupported wall height or 10 ft.
- Brace deflection under design loads should not exceed one-tenth of the wall thickness.
- Brace reactions should be transferred into the diaphragms (not into the URM wall).

The limit on brace spacing is intended to prevent excessive horizontal spans between stiff braces. For the second story, the maximum brace spacing is s = (10 ft)/2 = 5 ft. The braces will be anchored to the wall at one-third points over the wall height.

ASCE 41-13 does not provide capacities for adhesive wall anchors. Anchor capacities are based on product-specific evaluation reports in accordance with International Code Council (ICC-ES) Acceptance Criteria (AC60) *Acceptance Criteria for Anchors in Unreinforced Masonry Elements* (ICC-ES, 2015a). Current reports have the following requirements that affect placement of the anchors:

- Walls shall be 13 inches or thicker
- Minimum edge distance of 16 inches
- Minimum vertical and horizontal spacing of 16 inches

In addition, while ICC-ES AC60 Section 3.3.2 of requires the in-place mortar shear strength, exceeded by over 80% of the tests, to be over 30 psi, as defined by the 2015 IEBC Section A106 Section 3.3.5, which is the same as ASCE 41-17 § 15.2.2.2.1. For this building, this v_{te} value is 76 psi, which exceeds 30 psi. In addition, current ICC-ES reports also typically require the average mortar shear strength to be at least 50 psi. For this building, this average v_{to} value is 95 psi, which exceeds 50 psi.

An elevation showing the proposed layout of vertical strongback braces and horizontal girts is shown in Figure 12-23. There is no explicit guidance in ASCE 41-13 or the IEBC on how openings impact strongback design. The horizontal girt is added above the central opening based on engineering judgment and because the 6’-8” spacing between strongbacks exceeds the 5 ft width noted above in the IEBC. Note that a modification was added ASCE 41-17 that requires anchors be at least 12 inches away from opening edges. The layout in Figure 12-23 complies with that proposal.

The vertical strongback just to the left of the door opening of Figure 12-23 is designed in this example. The minimum sectional requirements for HSS tube to meet strength and deflection criteria are shown as follows. Tributary width are conservative for simplicity.

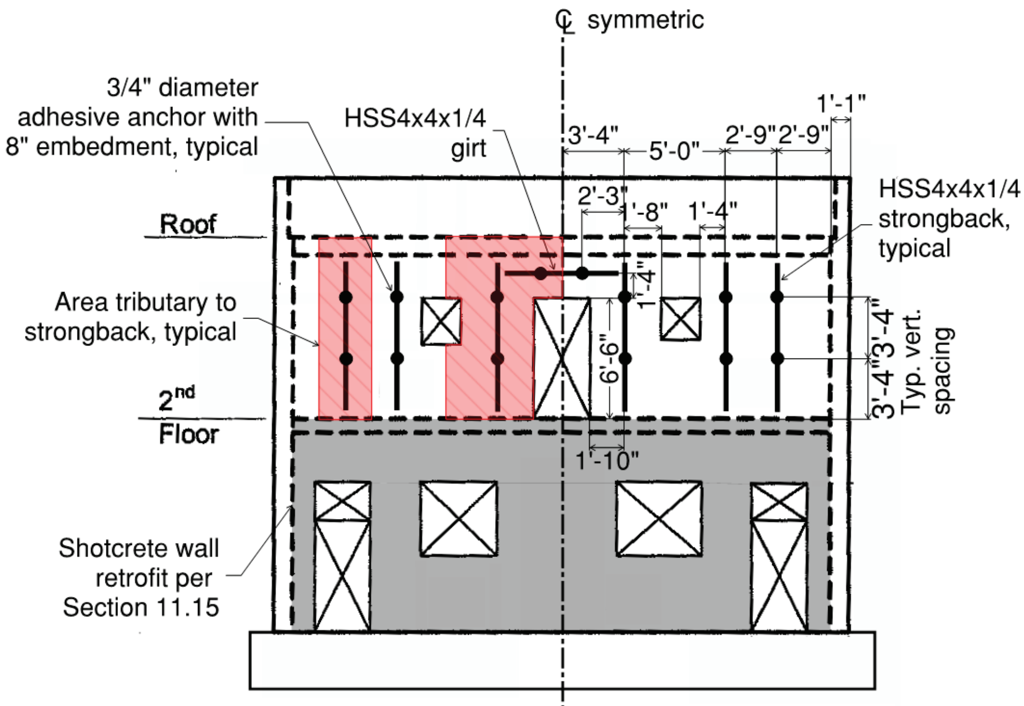


Figure 12-23 Rear (north) wall elevation showing second story out-of-plane retrofit using rectangular steel tube vertical braces and horizontal girts. Red shading shows the area tributary to typical braces.

$$
\begin{aligned}
P_u &= F_p A_{\text{trib}} \\
&< (47 \text{ lb/ft}^2)(3.33 \text{ ft} + 2.5 \text{ ft})(3.33 \text{ ft})/(\text{anchor point}) \\
&< 912 \text{ lb/anchor point} \\
&< 0.91 \text{ kip/anchor point}
\end{aligned}
$$

$$
\begin{aligned}
M_u &= P_u a \\
&= (0.91 \text{ kip})(10 \text{ ft})/3 \\
&= 3.03 \text{ kip-ft} \\
&= 36 \text{ kip-in.}
\end{aligned}
$$

$$
\begin{aligned}
Z_{\min} &\geq \frac{M_u}{F_y} \\
&= (36 \text{ kip-in.})/(46 \text{ kip/in.}^2) \\
&= 0.79 \text{ in.}^3
\end{aligned}
$$

$$I_{min} \geq \frac{P_u l^3}{28 E \Delta_{max}}$$

$$= (0.91 \text{ kip})(120 \text{ in.})^3/[(28)(29{,}000 \text{ kip/in.}^2)(0.1)(13 \text{ in.})]$$

$$= 1.49 \text{ in.}^4$$

Useful Tip

For anchors that will be installed at a wood ledger, to provide a stiffer connection, the adhesive anchor should entirely fill both the hole in the masonry and the wood. Being able to see the adhesive reach the annulus in the wood helps provide confidence that the adhesive filled the masonry annulus well.

Commentary

In a building with masonry walls thicker than 13 inches, the engineer might consider a project specific testing program to justify higher values or the use of straight dowels in tension. Provisions in ICC-ES AC60 would be a starting point for criteria.

ASCE 41-17 Revision

Use of a factor of conversion factor of 3 on ICC-ES report allowable values with a reduction factor, ϕ, of 1.0 was added to ASCE 41-17.

The braces will be constructed using an HSS 4×4×$^1/_4$ tube.

$Z \quad = 4.69 \text{ in.}^3 > 0.79 \text{ in.}^3$

$I \quad = 7.8 \text{ in.}^4 > 1.49 \text{ in.}^3$

ICC-ES AC60 currently allows two connection types for anchors that resist tension forces. The first connection uses a 3/4-inch diameter threaded bent rod. The rod is inserted into an adhesive-filled hole drilled into the masonry at a 22.5° angle. The masonry hole is lined with a proprietary mesh to prevent the loss of adhesive in conditions where collar joints are not completely filled with mortar. An alternative connection uses a 5/8-inch diameter straight dowel, inserted into a hole drilled completely through the masonry wall. A steel bearing plate and a hex nut are connected to the threaded rod on the outside of the wall. A similar mesh lining is used to prevent the loss of adhesive. Both connections are designed to engage an enlarged masonry area to resist pullout. Straight dowels without through plates are not permitted for connections that experience tension forces.

For both connections, ICC-ES AC60 specifies an allowable tension capacity of 1,200 lbs. Explicit instructions for converting to a capacity at the strength design level are not provided. In this example, an approach consistent with traditional URM design is used. In the 1997 *Uniform Code for Building Conservation* (ICC, 1997), anchor capacities were provided at the allowable stress design (ASD) level. Similar provisions in the succeeding code, 2003 *International Existing Building Code* (ICC, 2003), use strength design, with anchor capacities increased by a factor of three.

Using the same convention, this example defines the anchor tension capacity as 3(1,200 lb) = 3,600 lb. Since the anchor demand is 912 lb, the anchors have adequate capacity. In this example, the front and rear walls have higher architectural visibility, so a bent anchor connection as shown in Figure 12-24 will be used at the out-of-plane walls braces.

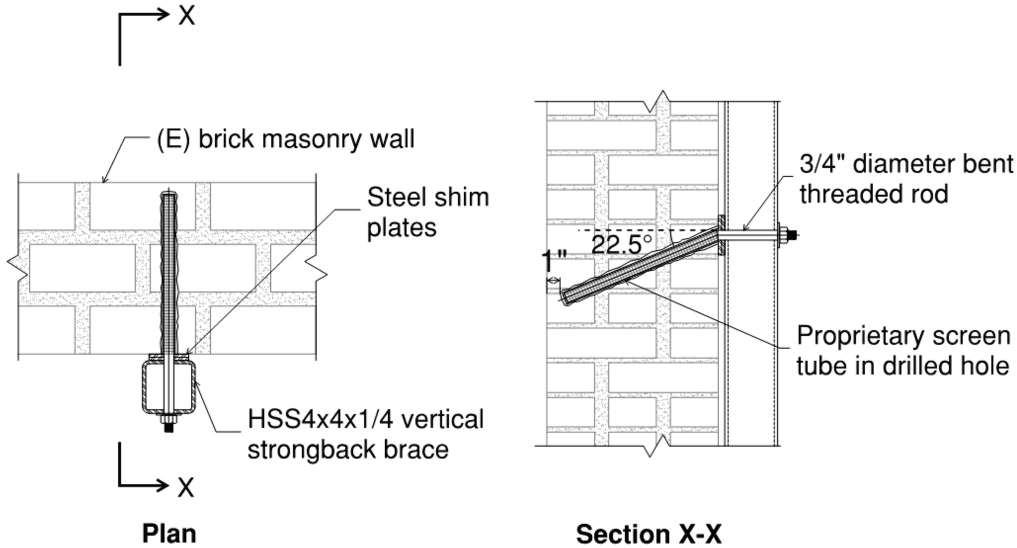


Figure 12-24 Detail of vertical strongback brace anchorage to unreinforced masonry wall using an adhesive anchor.

12.16.2 Parapet Evaluation

Parapet out-of-plane failures have been frequently observed in post-earthquake reconnaissance. Commonly, the parapets have fallen outwards and injured pedestrians. The Special Procedure, as implemented in ASCE 41-13, does not include explicit criteria for evaluating the safety of existing parapets on URM buildings. Instead, parapets are evaluated using the nonstructural component criteria of ASCE 41-13 Chapter 13. ASCE 41-13 § 13.6.5.1 requires force-based evaluation of parapets with $h/t > 1.5$. The height used for a parapet should go from the center line of the roof-to-wall anchor to the top of the parapet, as shown in ASCE 41-13 Figure 13-1. Assume an anchor would be placed at midheight of the 2×12 roof framing, so h = 3'0" to top of roofing + 1-1/2" for sheathing and roofing + 11-1/4"/2 = 3.59'. The h/t ratio for the parapet is (3.59 ft)(12 in./ft)/(9 in.) = 4.8, and exceeds the allowable ratio of 1.5.

Parapets that fail the h/t test should be evaluated for adequate strength using the component force requirements of ASCE 41-13 § 13.4.3. Since the masonry in this building has negligible tension capacity, the parapet is deemed to be inadequate without detailed evaluation.

The existing parapet will be removed, along with the top of the three-wythe perimeter wall. The wall section and parapet will be replaced with a reinforced concrete bond beam. The new beam will serve as a ductile connector element, particularly at the building corners where post-earthquake reconnaissance has revealed vulnerabilities.

12.16.3 Brace Top and Bottom Connection Design

For the design of strongback brace connections at the roof and second floor, the brace reactions are calculated as shown below (for the brace just left of the door opening in Figure 12-23).

$R \quad = 2P_u /2$
$\quad\;\; = 2(0.91 \text{ kip})/2$
$\quad\;\; = 0.91 \text{ kips}$

At the roof, the joists run parallel to the rear (north) and front (south) walls. A saddle plate connection with 3/4-inch diameter bolts provides stability for the post and resists inward out-of-plane loading while a threaded rod resists outward out-of-plane loading and distributes the force into the roof diaphragm (see Figure 12-25).

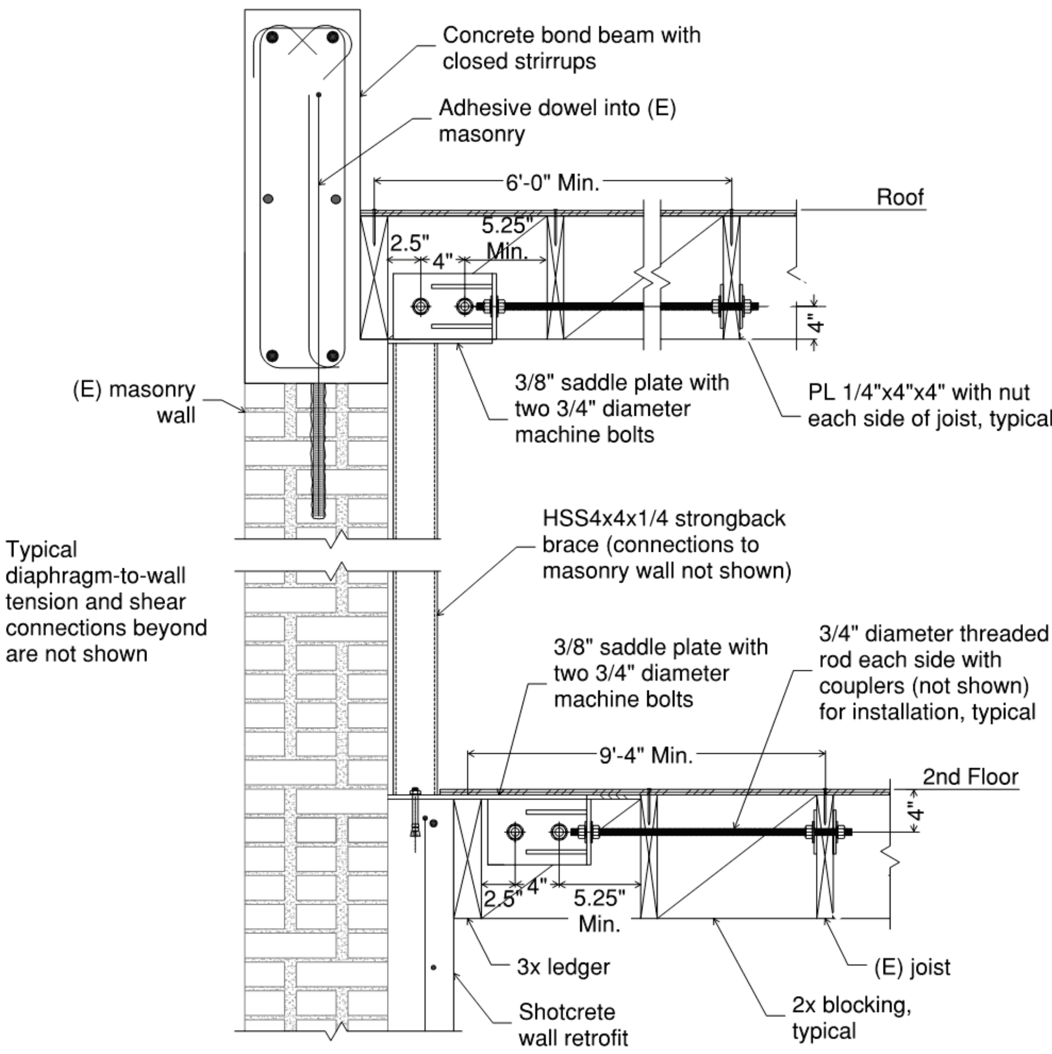


Figure 12-25 URM wall strongback brace connection to roof and second floor diaphragms. See Figure 12-24 for connection of strongback to masonry wall. See Figure 12-26 for typical floor-to-wall connections beyond the brace.

Although the wall of-of-plane force will be distributed between the bolts at the saddle connection and the threaded rod, it is conservative to design the bolts to take the entire force. The new connection capacity is based on the ANSI/AWC NDS-2012, *National Design Specification for Wood Construction* (AWC, 2012):

$$Z' = Z_{||} C_M C_t C_g C_\Delta C_{eg} C_{di} C_{tn} K_F \phi\lambda \qquad \text{(NDS-2012, Table 10.3.1)}$$

where:

Z' = Adjusted lateral design value for single fastener connection (lb)

$Z_{||}$ = Reference lateral design value for single fastener connection load parallel to the grain in double shear with steel side plates (lb)
= 3,340 lb (NDS-2012 Table 11G, Douglas Fir-Larch, 4× blocking)

C_M = Wet service factor
= 1.0 (NDS-2012 Table 10.3.3, Moisture Content ≤ 19%)

C_t = Temperature factor
= 1.0 (NDS-2012 Table 10.3.4, Temperature ≤ 100 °F)

C_g = Group action factor
= 1.0 (NDS-2012 Table 10.3.6C)

where:

A_m = (3.5 in for initial assume 4×)(11.5 in block depth ripped to match existing 1-5/8” wide × 11-1/2” deep joists)
= 40.3 in.^{2}

A_s = 2(0.375 in. side)(6 in.)
= 4.5 in.^{2}

A_m/A_s = (40.3 in.^{2})/(4.5 in.^{2})
= 9

C_Δ = Geometry factor
= 1.0 [NDS-2012 Table 11.5.1A and 11.5.1.B; end distance = (5.25 in.) ≥ 7D = 7(0.75) = (5.25 in.), and spacing = (4 in.) ≥ 4D = 4(0.75 in.) = (3 in.)]. Due to the limited end distance for the left bolt, it is not included for outward loading.

C_{eg} = End grain factor
= 1.0 (NDS-2012 § 11.5.2.2)

C_{di} = Diaphragm factor
= 1.0 (NDS-2012 § 11.5.3)

C_{tn} = Toe-nail factor
= 1.0 (NDS-2012 § 11.5.4)

K_F = Format conversion factor
= 3.32 (NDS-2012 Table 10.3.1)

ϕ = Resistance factor
= 1.0 (see Section 12.15)

λ = Time effect factor
= 1.0 (NDS-2012 Table N.3.3, earthquake load combination)

Z' = (3,340 lb)(3.32) = 11,089 lb per bolt

Similar calculations are performed for 2× blocking, resulting in a capacity of Z' = 3,200 per bolt. Since the reaction R = 0.91 kips < 3.2 kips, 2× blocking will be used.

For outward loading, the anchorage of the blocking depends on the diaphragm shear strength. The threaded rod must extend into the diaphragm far enough to transmit the outward anchorage force to the diaphragm.

L_{Roof} = R/v_u
= (0.91 kips)(1,000 lb/kip)/(300 lb)
= 3.0 ft

L_{Second} = R/v_u
= (0.91 kips)(1,000 lb/kip)/(1,500 lb)
= 0.6 ft

Since the joists are 2'-0" on center at the roof, the threaded rod at the roof would only need to extend 4'-0" into the diaphragm to exceed 3.0 ft. At the second floor, the joists are at 1'-4" on center, and the rod would need to extend only one bay or 1'-4" into the diaphragm. As discussed in Section 12.17 of this *Guide*, development of out-of-plane wall-to-diaphragm anchorage forces into the diaphragm requires larger depths of 6'-0" at the roof and 9'-4" at the second floor. These depths are used for the out-of-plane brace as well.

Figure 12-26 shows the free body diagram for load transfer of the out-of-plane brace at the roof for the case when the rod is in tension, and pulls on the blocking of the most inboard joist bay. Since there are three joist bays resisting the load, one third of the load is transferred into the diaphragm in each bay. The remaining two thirds of the total load is transferred in compression to the second bay. The vertical shear couple in the blocking due to the eccentricity between the threaded rod and the connection of the sheathing to the blocking is calculated for the roof diaphragm as shown below.

V = (912 lb/3 bays)(7.5 in.)/(24 in spacing – 1.625 in existing joist width)
= 102 lbs

Three toe nails will be used at each end of the blocking to resist the connection eccentricity. By inspection, the nails are adequate to resist the vertical shear.

Additional connection calculations should be performed, including checking the strength of steel saddle plate in bending at the threaded rods and the plate bearing against the joist at the interior end of the anchor. The additional calculations are beyond the scope of this example.

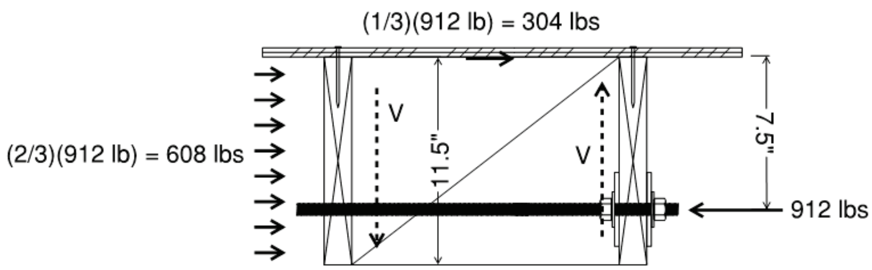


Figure 12-26 Eccentricity at threaded rod diaphragm connection.

12.17 Wall Tension Anchorage Retrofit Design

The model of building behavior that forms the basis for the Special Procedure assumes that out-of-plane walls are adequately connected at each diaphragm level to transfer seismic loads. Out-of-plane walls are assumed to span vertically between floor levels. Walls without sufficient anchorage may pull away from the diaphragm and result in loss of gravity support of the rafters or joists. Post-earthquake damage reports have included numerous examples of buildings where walls have failed due to excessive flexural stresses or where joists have dropped from their supports resulting in an interior collapse of the roof or floor systems.

ASCE 41-13 § 15.2.3.5 requires evaluation of wall tension anchorage when S_{X1} > 0.067. Per Section 12.5 of this *Guide* S_{X1} = 0.507, so evaluation of the wall anchorage is required for the example building. As shown dotted in Figure 12-27, the existing joists are supported in pockets in the masonry wall without a positive tension capacity.

ASCE 41-17 Revision

The trigger for requiring out-of-plane checks of wall was revised ASCE 41-17 to use S_{D1}> 0.067.

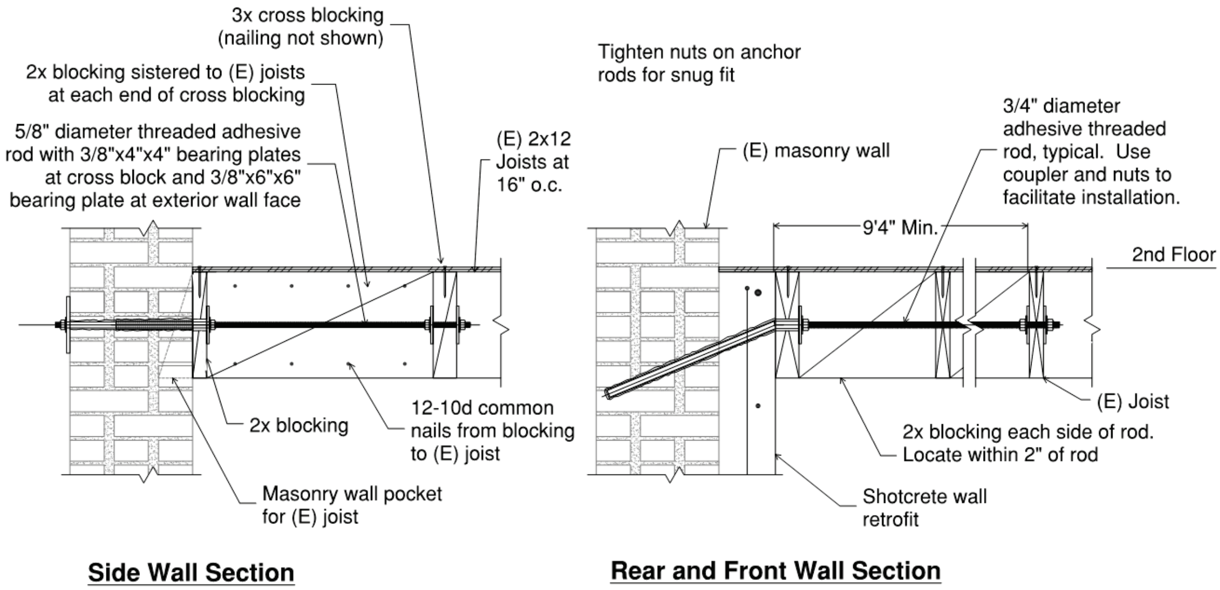


Figure 12-27 Tension anchorage retrofit detail at the second floor.

Along the rear (north) and front (south) walls, the existing ledger anchorage lacks a reliable tension connection to the wall and a load path to deliver force to the existing floor diaphragm. For this example, the capacity of the existing construction is considered negligible, and a new assembly will be designed to resist the entire out-of-plane demand.

ASCE 41-17 Revision

The F_p force in ASCE 41-17 was revised to $F_p = 0.9\ S_{XS}$.

Per ASCE 41-13 § 15.2.3.5, the wall anchorage assembly shall have a capacity equal to the larger of $2.1S_{X1}$ or 200 lb/ft. The demand on the diaphragm from the rear (north) and front (south) walls is calculated below.

$$F_p = 2.1 S_{X1} W$$

$$f_p = \frac{F_P}{L}$$

where:

S_{X1} = 0.507

W_{Roof} = Maximum seismic weight of second floor rear (north) or front (south) wall and parapet tributary to the roof diaphragm (kips)

= 26 kips (rear wall, Table 12-4) – (90 lb/ft^2)(3 ft)(30 ft)(1 kip/1000 lb) for removal of masonry parapet + (150 lb/ft^3)(9 in./12 in./ft)(3 ft)(30 ft)(1 kip/1000 lb)

= 28 kips

W_{Second} = Maximum seismic weight of first and second floor rear (north) or front (south) wall tributary to the second floor diaphragm (kips)

= 40 kips (front wall, Table 12-4) + 11 kips (shotcrete wall overlay per Section 12.15 of this *Guide*)
= 51 kips

L = Wall length (ft)
= 30 ft

F_p = 2.1(0.507)(28 kips) = 30 kips at the roof diaphragm
= 2.1(0.507)(51 kips) = 54 kips at the second floor diaphragm

f_p = (30 kips)(1,000 lb/kip)/(30 ft) = 1,000 lb/ft > 200 lb/ft at the roof diaphragm
= (54 kips)(1,000 lb/kip)/(30 ft) = 1,800 lb/ft > 200 lb/ft at the second floor diaphragm

Similar calculations for the side walls result in tension demands of 940 lb/ft at the roof diaphragm and 1,300 lb/ft at the second floor diaphragm.

To reduce the total number of anchors, tension anchors will be combined with diaphragm shear connections where they are coincident. Since the top of the existing masonry wall and the parapet are being replaced with a concrete bond beam (see Section 12.16 of this *Guide*) and since the focus of this example is on masonry provisions, the design of tension anchors into the bond beam is not included in this example. Anchors between the roof and the bond beam are critical components of the lateral load path, and should be designed according to the requirements of ACI 318-11.

Commentary

Design of combined tension/shear anchors:

Current ICC reports for adhesive anchors qualified under the ICC-ES AC60 acceptance criteria use a tension-shear interaction equation.

The Special Procedure has no explicit requirements to concurrently check tension and shear occurring from seismic loading in orthogonal directions (such as per a 100%/30% orthogonal combination approach), and this is not done in established design practice.

If the retrofit anchor takes shear from gravity loads, then it would be appropriate to combine the shear from gravity loads with tension when the anchor is in tension from out-of-plane seismic loading (as well as separately combining gravity shear with horizontal shear when the anchor is in shear from in-plane seismic loading).

Design of Tension Anchors from Walls to Second Floor

At the second floor, there are two options for anchors that experience both tension and shear demands. At the rear (north) and front (south) walls, preservation of the existing facade is important at this building. Tension anchorage at these walls will use a bent 3/4-inch diameter threaded rod. At the side walls, adhesive 5/8-inch diameter through-bolts with exterior steel plates will be used. For both anchors, the allowable tension capacity is 1,200 lb per anchor consistent with ICC-AC 60. Using the conversion factor of 3 from Section 12.16.1 of this *Guide*, the capacity for strength design is 3,600 lb.

The required spacing for tension anchors to the second floor diaphragm is calculated below.

s = (3,600 lb)/(1,800 lb/ft)
= 2.0 ft at the front and rear walls
= (3,600 lb)/(1,300 lb/ft)
= 2.8 ft at the side walls

From calculations in Section 12.18 ahead, the required spacing of shear anchors is 3’-9” at the side walls and 2 feet at the front and rear walls. Since the required spacing for tension connections is lower at walls, all of the anchors will be combined tension-shear anchors.

At the side walls, it is convenient to match the anchor spacing to the spacing of existing joists. The anchors can be spaced at 32 inches (2’-8”) on center and placed in every other joist bay.

At the rear (north) and front (south) walls, the joists are oriented parallel to the wall. The anchors for these walls will be spaced at 2 feet on center.

The design of the tension anchor at the side walls is shown in this example (see Figure 12-27). The bending demand on the cross blocking at the end of the tension rod is calculated below. The force demand on the cross blocking is conservatively defined by the anchor tension capacity.

$M_u = PL/4$
$= (3{,}600 \text{ lb})(16 \text{ in spacing} - 1.625 \text{ in width of existing joist})/4$
$= 12{,}940$ lb-in.

$S = bd^2/6$
$= (11.25 \text{ in.})(3.5 \text{ in.})^2/6$ (for new 4× cross blocking)
$= 23 \text{ in.}^3$
$= (11.25 \text{ in.})(2.5 \text{ in.})^2/6$ (for new 3× cross blocking)
$= 12 \text{ in.}^3$

$f_b = M_u/S$ (NDS-2012 Eq. 3.3-1)
$= (12{,}940 \text{ lb-in.})/(23 \text{ in.}^3)$
$= 560 \text{ lb/in.}^2$ (for 4× cross blocking)
$= 1{,}080 \text{ lb/in.}^2$ (for 3× cross blocking)

The capacity of Douglas Fir blocking is calculated according to NDS-2012.

$F_b' = F_b C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi \lambda$ (NDS-2012)

where:

F_b' = Adjusted bending design value (lb/in.²)

F_b = Reference bending design value (lb/in.²)
= 1,000 lb/in.² (NDS-2012 Supplement Table 4A, Douglas Fir-Larch, No.1)

C_M = Wet service factor
= 1.0 (NDS-2012 Supplement Table 4A, Moisture Content ≤ 19%)

C_t = Temperature factor

= 1.0 (NDS-2012 Table 2.3.3, Temperature ≤ 100 °F)

C_L = Beam stability factor

= 1.0 (NDS-2012 Section 3.3.3, $d \leq b$)

C_F = Size factor

= 1.0 (wide face, NDS-2012 Supplement Table 4A)

C_{fu} = Flat use factor

= 1.1 (use lesser of 1.1 for 4× and 1.2 for 3×NDS-2012 Supplement Table 4A)

C_i = Incising factor

= 1.0 (NDS-2012 Section 4.3.8)

C_r = Repetitive member factor

= 1.0 (NDS-2012 Section 4.3.9)

K_F = Format conversion factor

= 2.54 (NDS-2012 Table 4.3.1)

ϕ = Resistance factor

= 1.0 (see Section 12.15)

λ = Time effect factor

= 1.0 (NDS-2012 Table N.3.3, earthquake load combination)

F_b' = (1,000 lb/in.^{2})(1.1)(2.54) = 2,790 lb/in.^{2} > 560 lb/in.^{2} for 4× and 1,080 lb/in.^{2} for 3×

Given the short length of the cross block, the shear demand is checked similarly.

V_u = $P/2$

= (3,600 lb)/2

= 1,800 lb

f_v = $3V_u/(2bd)$ (NDS-2012 Eq. 3.4-2)

= 3(1,800 lb)/[2(11.25 in.)(3.5 in.)] (for 4× cross blocking)

= 69 lb/in.^{2}

= 3(1,800 lb)/[2(11.25 in.)(2.5 in.)] (for 3× cross blocking)

= 96 lb/in.^{2}

$$F_v' = F_v C_M C_t C_i K_F \phi \lambda \quad \text{(NDS-2012)}$$

where:

F_v' = Adjusted shear design value (lb/in.^{2})

F_v = Reference shear design value (lb/in.^{2})

= 180 lb/in.^{2} (NDS-2012 Supplement Table 4A, Douglas Fir-Larch)

C_M = 1.0 (NDS-2012 Supplement Table 4A, Moisture Content ≤ 19%)

C_t = 1.0 (NDS-2012 Table 2.3.3, Temperature ≤ 100 °F)

C_i = 1.0 (NDS-2012 Section 4.3.8)

K_F = Format conversion factor
= 2.88 (NDS-2012 Table 4.3.1)

ϕ = Resistance factor
= 1.0 (see Section 12.15 of this *Guide*)

λ = Time effect factor
= 1.0 (NDS-2012 Table N.3.3, earthquake load combination)

F_v' = (180 lb/in.^{2})(2.88) = 520 lb/in.^{2} > 69 lb/in.^{2} for 4× and 96 lb/in.^{2} for 3×

Compression perpendicular to the grain will also be checked at the bearing plate on the cross block. It governs over compression where the cross block bears on the blocking. A 4 inches × 4 inches bearing plate is assumed.

$f_{c\perp}$ = P/A_b
= (3,600 lb)/[(4 in.)(4 in.)]
= 225 lb/in.^{2}

$$F_{c\perp}' = F_{c\perp} C_M C_t C_i C_b K_F \phi \qquad \text{(NDS-2012)}$$

where:

$F_{c\perp}'$ = Adjusted compression design value perpendicular to the grain (lb/in.^{2})

$F_{c\perp}$ = Reference compression design value perpendicular to the grain (lb/in.^{2})
= 625 lb/in.^{2} (NDS-2012 Supplement Table 4A, Douglas Fir-Larch)

C_M = 1.0 (NDS-2012 Supplement Table 4A, Moisture Content ≤ 19%)

C_t = 1.0 (NDS-2012 Table 2.3.3, Temperature ≤ 100 °F)

C_i = 1.0 (NDS-2012 Section 4.3.8)

C_b = Bearing area factor
= 1.0 (NDS-2012 Section 3.10.4)

K_F = Format conversion factor
= 1.67 (NDS-2012 Table 4.3.1)

ϕ = Resistance factor
= 1.0 (see Section 12.15 of this *Guide*)

$F'_{c\perp}$ = (625 lb/in.^{2})(1.67) = 1,040 lb/in.^{2} > 225 lb/in.^{2}

Based on these calculations, 3× cross blocking is adequate. Since the tension load is transferred from the blocking into the joists, the length of the threaded rod is dependent on the nailing from the blocking to the joists (not the diaphragm shear capacity which was used at the rear and front walls in Section 12.11 of this *Guide*). The nailing demand and capacity are calculated below. The nail tables in the NDS-2012 assume a penetration 10 times the diameter of the nail into the main member. For 2× blocking, a 3-inch long nail is required so 10d common nails will be used.

Load = V_u
= 1,800 lb

$$Z' = ZC_M C_t C_g C_\Delta C_{eg} C_{di} C_{tn} K_F \phi \lambda \quad \text{(NDS-2012)}$$

where:

Z = 118 lb (NDS-2012 Table 11N, Douglas Fir-Larch, 1-1/2 inch side member, 10d common nail)

C_M = 1.0 (NDS-2012 Table 10.3.3, Moisture Content ≤ 19%)

C_t = 1.0 (NDS-2012 Table 10.3.4, Temperature ≤ 100 °F)

C_g = 1.0 (NDS-2012 Section 10.3.6, D < 1/4 inch)

C_Δ = 1.0 (NDS-2012 Section 11.5.1.1, D < 1/4 inch)

C_{eg} = 1.0 (NDS-2012 Section 11.5.2.2)

C_{di} = 1.0 (NDS-2012 Section 11.5.3)

C_{tn} = 1.0 (NDS-2012 Section 11.5.4)

K_F = 3.32 (NDS-2012 Table 10.3.1)

ϕ = 1.0 (see Section 12.15)

λ = 1.0 (NDS-2012 Table N.3.3, earthquake load combination)

Z' = (118 lb)(3.32) = 392 lb per nail

Use 12 - 10d nails for a total capacity 12(392 lb/nail) = 4,700 lb > 1,800 lbs.

There are no explicit requirements for cross ties and subdiaphragms in ASCE 41-13 § 15.2 although there is a check for continuous cross ties in Tier 1 Checklist 16.16LS. However, it is good practice to develop the wall-to-diaphragm anchor back into the diaphragm. At the side walls, the joists are perpendicular to the wall, so they can serve as cross ties. They lap at the interior partitions, so bolting of the joists at several laps is recommended. At the front and rear walls, however, the joists are parallel to the wall, making the development of cross ties more critical.

For the second floor, it is assumed that the interior first story partition lines will be developed to serve as cross ties. The rear wall is more critical as there is only one interior partition line, running down the middle of the diaphragm. It is assumed that a subdiaphragm is developed between the masonry side walls and each side of the central cross tie, making a subdiaphragm length of (30 ft)/2 = 15 feet. The required depth of the subdiaphram would then be as follows:

D = (1,800 lb/ft)(15 ft subdiaphragm length/2 ends)/(1,500 lb/ft diaphragm capacity)
= 9 ft (use 16 in./bay × 7 bays = 9 ft 4 in given the 16 inches o.c. joist spacing)

At the second story there is a central pair of partitions separated by 4 feet. It is assumed that these are developed to serve as cross ties. Then, the subdiaphragm length is [(30 ft) – (4 ft)]/2 = 13 feet.

The required depth of the subdiaphram would then be as follows:

D = (1,000 lb/ft)(13 ft subdiaphragm length/2 ends)/(300 lb/ft diaphragm capacity)
= 22 ft

Either extensive nested subdiaphragms would need to be developed, or straight sheathing at the end would need to be strengthened. For this example, the existing straight sheathing will be removed and replaced with wood structural panel sheathing. Values are not given in ASCE 41-13 Table 15-2 for wood structural panel sheathing. The procedure from ASCE 41-13 § 12.5.3.6.2 is used. This uses expected strengths of the diaphragm, defined as 1.5 times the yield values (with $\phi = 1.0$) in SDPWS-2008, *Special Design Provisions for Wind and Seismic Standard with Commentary* (AWC, 2008). Assuming 3/4 inch plywood, a blocked diaphragm, 2× members, and 10d nails at 6 inches o.c. at typical panel edges and 4 inches at diaphragm boundaries, then the expected strength (per SDPWS-2008 Table 4.2A) is 1.5 × 850 lb/ft = 1,275 lb/ft. This reduces the required depth to a more reasonable value as follows:

D = (1,000 lb/ft)(13 ft subdiaphragm length/2 ends)/(1,275 lb/ft diaphragm capacity)
= 5.09 ft (use 6 feet as the roof joists are at 2 ft o.c.)

The strengthening of the roof diaphragm locally for the subdiaphragm has a ripple effect as the strength that the diaphragm can deliver to the wall is now larger than that of the original straight sheathing. This can impact the demand used to evaluate the in-plane capacity of the wall and the value used for retrofitting a deficient wall. As a result, the in-plane demands at the roof in Section 12.12 of this *Guide* should be rechecked conservatively using the upgraded diaphragm strength. The weight of the bond beam should be included as well. With the stronger roof diaphragm and bond beam, the in-plane story force in the governing rear wall rises from 19 kips to 42 kips. Per Section 12.14 of this *Guide*, for checking the wall, the demand is 70% of 42 kips = 29 kips and the capacity is 49 kips. Thus, the second story wall capacity is still adequate, even with the locally strengthened diaphragm and bond beam.

> **Useful Tip**
>
> Strengthening to develop out-of-plane wall anchorage through a subdiaphragm can impact in-plane wall demands and shear transfer from the diaphragm to the wall. It is important to iterate back to confirm the wall is still adequate.

The resulting connection at the second floor side walls is shown in Figure 12-27. Detailed design calculation for rear (north) and front (south) walls are not included in this example, but the sample tension anchorage detail is shown in Figure 12-27.

12.18 Wall Shear Transfer Retrofit Design

Since S_{X1} = 0.507 > 0.133, ASCE 41-13 § 15.2.3.2.6 requires evaluation of the ability for diaphragms to transfer shear to the masonry walls. The diaphragms shall transfer the minimum of:

$$V_d = 1.25 S_{X1} C_p W_d \qquad \text{(ASCE 41-13 Eq. 15-11)}$$

$$V_d = v_u D \qquad \text{(ASCE 41-13 Eq. 15-12)}$$

where:

V_d = Required shear capacity of diaphragm connection to masonry walls (lb)

S_{X1} = BSE-1E spectral response acceleration parameter at a 1 second period

C_p = Horizontal force factor (ASCE 41-13 Table 15-3)

W_d = Total seismic weight tributary to the diaphragm (lb)

v_u = Unit shear strength of diaphragm (lb/ft)

D = Total length of diaphragm (ft)

> **Useful Tip**
>
> ASCE 41-13 Equation 15-11 has a typographical error it in where S_{X1} is mistakenly identified as S_{DX1}. The corrected version is shown here.

> **Useful Tip**
>
> The demands on diaphragm shear connections are dependent on any wall in-plane retrofits that may be required (because the shear demand depends on wall seismic weight).
>
> Therefore, wall in-plane evaluation and any required retrofit design should be performed before designing the wall shear connections.

Shear Anchors at the Roof-to-Wall Connections for North-South Loads

In the north-south direction, the required shear capacity (at each of the east and west walls) at the roof is:

V_d = MIN($1.25S_{X1}C_pW_d$, v_uD)

where:

S_{X1} = 0.507

C_p = 0.50 (ASCE 41-13 Table 15-3, roof with straight sheathing as this is the majority of the diaphragm)

W_d = One half of the seismic weight of roof diaphragm and rear (north) and front (south) walls tributary to roof diaphragm including the concrete bond beam as shown in Section 12.17 of this *Guide*
= (43 kips + 28 kips + 27 kips)/2
= 49 kips

v_uD = Shear capacity of roof diaphragm (conservatively using the strengthened value triggered by the subdiaphragm analysis in Section 12.17 of this *Guide* where it applies)
= [(1,275 lb/ft)(2 ends)(6 ft) + (300 lb/ft)(60 ft – 12 ft)](1 kip /1,000 lb)
= 30 kips

V_d = MIN[(1.25)(0.507)(0.50)(49 kips), 30 kips)]
= 16 kips (or 16,000 lbs/60 ft = 267 lb/ft)

Shear Anchors at the Second Floor-to-Wall Connections for North-South Loads

In the north-south direction, the required shear capacity (at each of the east and west walls) at the second story diaphragm is:

V_d = MIN($1.25S_{X1}C_pW_d$, v_uD)

where:

S_{X1} = 0.507

C_p = 0.75 (ASCE 41-13 Table 15-3, diaphragm with double layers of boards)

W_d = One half of the seismic weight of the second story diaphragm, rear (north) and front (south) walls tributary to second story diaphragm, and the shotcrete wall retrofit (21 kips total);
= (51 kips + 37 kips + 40 kips + 21 kips)/2
= 75 kips

v_uD = Shear capacity of second story diaphragm
= (1,500 lb/ft)(60 ft)(1 kip/1,000 lb)
= 90 kips

V_d = MIN[(1.25)(0.507)(0.75)(75 kips), 90 kips]
= 36 kips (or 36,000 lbs/60 ft = 600 lb/ft)

Shear Anchors at the Diaphragm-to-Wall Connections for East-West Loads

Similar calculations performed in the east-west direction provide shear demands on the rear (north) and front (south) walls. The shear demand at the roof level is V_d = 34 kips (conservatively using the locally strengthened roof capacity), and the shear demand at the second floor level is V_d = 47 kips.

Similar to many URM buildings, the existing diaphragm in the example building lacks a positive shear connection to the perimeter walls. Joist ends are supported on pockets in the east and west masonry walls. The ledger on the rear (north) and front (south) walls has only a nominal gravity connection. As a result, a retrofit to provide positive shear resistance at the edges of the diaphragm is required. Shear demands per unit of diaphragm length are shown in Table 12-16.

Commentary

The current ICC-ES AC60 acceptance criteria limit the allowable shear capacity of through-bolt anchors to 750 lb per anchor.

Although the stated shear capacity is lower than alternative connections, it provides a redundant, physical tie that may be superior to anchors that rely solely on an adhesive. For that reason, through-bolts are used as combined tension-shear diaphragm anchors in this example.

Table 12-16 Diaphragm Shear Demands

Level	Side (East and West) Walls (lb/ft)	Rear (North) and Front (South) Walls (lb/ft)
Roof	267	1,133
Second Floor	600	1,500

As shown in Section 12.16.2 of this *Guide*, the top of the existing masonry wall will be replaced at the roof (see Figure 12-25). Adhesive dowels will be provided to transfer shear from a new ledger into the bond beam according to the requirements of ACI 318-11 Appendix D. Detailed design calculations for the adhesive anchors are beyond the scope of this example.

From the ICC-ES AC 60 report for adhesive anchors in unreinforced masonry, there are three options for resisting shear loading. In addition to the angled anchor and through-dowel described in Section 12.16.1 of this *Guide*, a straight 3/4-inch diameter embedded dowel is allowed for shear resistance (but not for resisting tension). However, as noted in Section 12.17 of this *Guide*, all of the anchors will be used for both tension and shear, so the straight shear dowel option will not be used. The angled anchor has a shear capacity of 1,000 lb at the ASD level. Using the conversion factor of 3 from Section 12.16.1 of this *Guide*, the capacity for strength design is 3,000 lb.

At the side walls, combined anchors will use the through bolt connection with an allowable shear capacity of 750 lb per anchor. The capacity for

Commentary

Similar to the design of out-of-plane wall bracing (Section 12.16.1 of this *Guide*), it is preferable to transfer diaphragm shear forces into the shotcrete overlay (rather than the existing masonry wall). In this case, it is assumed that a majority of the resistance is actually provided by the portion of the anchor embedded in the concrete. Additionally, the design provides numerous supplemental anchors (located in the main body of the wall, not at the ledger) to further ensure composite behavior.

strength design is 3(750 lb) = 2,250 lb per anchor. At the rear (north) and front (south) walls, preservation of the existing facade precludes the use a through-bolt. At these locations, the bent adhesive anchor with a strength design capacity of 3,000 lb per anchor will be used.

For the side walls, the spacing of shear anchors at the second floor is calculated below.

$$s = (2{,}250 \text{ lb})/(600 \text{ lb/ft})$$
$$= 3.75 \text{ ft} = 3 \text{ feet } 9 \text{ inches}$$

Note that it is good practice to limit the maximum spacing of shear connections to 6 feet on center. This requirement, which has been included in other implementations of the Special Procedure (see, for example, the 2015 IEBC) is not explicitly required in ASCE 41-13.

Since the joists are spaced at 16 inches on center, a convenient spacing for the side wall anchors is 2(16 in.) = 32 in on center or every other joist bay. This matches the tension dowel requirement of Section 12.17 of this *Guide*, so no change is needed.

At the second floor front and rear walls, the joists run parallel to the wall, so the spacing of anchors need not match the joist spacing interval. However, these shear anchors occur at the retrofitted shotcrete wall. Since the shotcrete thickness is inadequate to fully develop the anchor, the rod will pass through the shotcrete and into the masonry. The adhesive will connect the anchor to both components of the wall, as shown in Figure 12-27. It is conservative to use the shear capacity of the anchors for unreinforced masonry to determine the anchor spacing.

$$s = (3{,}000 \text{ lb})/(1{,}500 \text{ lb/ft}) = 2 \text{ ft}$$

For a 3/4-inch diameter threaded rod, the ledger capacity in horizontal shear parallel to the grain is calculated according the NDS-2012.

$$Z' = Z_{||}\, C_M C_t C_g C_\Delta C_{eg} C_{di} C_{tn} K_F \phi\lambda \qquad \text{(NDS-2012 Table 10.3.1)}$$

where:

$Z_{||}$ = 1,270 lb (NDS-2012 Table 11E, Douglas Fir-Larch, 2× ledger)

C_M = 1.0 (NDS-2012 Table 10.3.3, Moisture Content ≤ 19%)

C_t = 1.0 (NDS-2012 Table 10.3.4, Temperature ≤ 100 °F)

C_g = 1.0 (NDS-2012 Table 10.3.6C)

C_Δ = 1.0 (NDS-2012 Table 11.5.1A and Table 11.5.1.B, end distance > 7D and spacing > 4D)

C_{eg} = 1.0 (NDS-2012 Section 11.5.2.2)

C_{di} = 1.0 (NDS-2012 Section 11.5.3)

C_{tn} = 1.0 (NDS-2012 Section 11.5.4)

K_F = 3.32 (NDS-2012 Table 10.3.1)

ϕ = 1.0 (see Section 12.15 of this *Guide*)

λ = 1.0 (NDS-2012 Table N.3.3, earthquake load combination)

Z' = (1,270 lb)(3.32) = 4,220 lb per bolt > 3,000 lb

Therefore, the strength of the existing 2× ledger material exceeds the design strength for the shear anchors. The existing ledgers will be used along the side walls (at the second floor).

At the rear (north) and front (south) walls, construction of the shotcrete overlay will require removal of the existing ledger and the first interior joist to provide sufficient clearance. The ledger will be replaced with 3× material mounted on the shotcrete overlay to facilitate diaphragm nailing. Calculation of the required nailing is beyond the scope of this example. Note also that the ledger anchors connected to the shotcrete wall will take gravity shear and this should be combined using the ICC AC60 interaction and evaluation report interaction equations when combining with seismic loads, but the tributary gravity load is very small here, so this is neglected.

12.19 Summary of Special Procedure Retrofit Measures

The retrofit measures required by the Special Procedure are shown in Figure 12-28. They can be compared to the deficiencies identified in the Tier 1 screening in Figure 12-5.

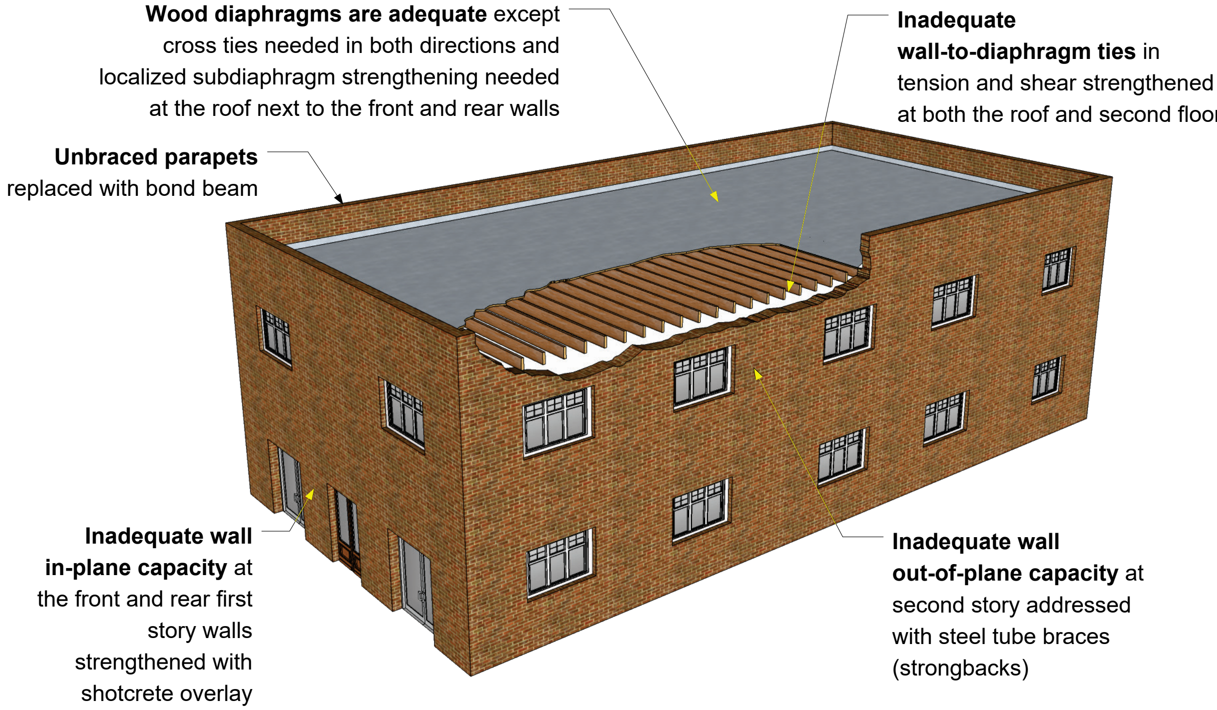


Figure 12-28 Retrofit measures required by the Special Procedure.

12.20 Additional Areas of Revision in ASCE 41-17

Additional revisions were incorporated into ASCE 41-17 that are not directly relevant to the design example, but are presented here for convenience. Although ASCE 41-13 § 15.1 notes that the Special Procedure is a stand-alone procedure, revisions in ASCE 41-17 § 15.2 include the following references to requirements in other sections of ASCE 41-17:

- Continuity requirements in Chapter 7
- Structures sharing common elements in Chapter 7
- Building separations in Chapter 7
- Anchored veneer in Chapter 13
- URM partitions in Chapter 13
- Truss and beam supports in Chapter 7 (and the trigger was revised to S_{D1} > 0.3g

Chapter 13

Unreinforced Masonry Bearing Wall (URM) with Tier 3 Procedure

13.1 Overview

This chapter provides discussion and example application of the Tier 3 systematic evaluation and retrofit procedures of ASCE 41-13 (ASCE, 2014) of the same unreinforced masonry (URM) bearing wall building analyzed in Chapter 12 of this *Example Application Guide* using the Special Procedure of ASCE 41-13 Chapter 15. Review of the design example provided in Chapter 12 is recommended, even though it uses the Special Procedure in ASCE 41-13 § 15.2. The following information relevant to this design example can be found in Chapter 12 of this *Guide* and is not repeated here for brevity: building description (Section 12.2), calculation of building dead loads and seismic weight (Section 12.3), calculation of building live loads (Section 12.4), calculation of spectral response acceleration parameters (Section 12.5), Tier 1 screening in accordance with ASCE 41-13 § 4.4 and § 4.5 (Section 12.6), and in-place shear testing (Section 12.9).

Example Summary

Building Type: URM

Performance Objective: BPOE

Risk Category: II

Location: Los Angeles, California

Level of Seismicity: High

Analysis Procedures: Linear Static (LSP)

Evaluation Procedure: Tier 3

Reference Documents:

ACI 318-11

NDS-2012

SDPWS-2008

TMS 402-11

For this example, the Risk Category II building will be evaluated to the Basic Performance Objective (BPOE) using the Tier 3 linear static procedure (LSP). Per ASCE 41-13 Table 2-1, with Risk Category II and Tier 3, analysis of the building structure is required for both the Life Safety Structural Performance Level at the BSE-1E Seismic Hazard Level and the Collapse Prevention Structural Performance Level at the BSE-2E Seismic Hazard Level. For the purposes of condensing this design example, only the BSE-1E Seismic Hazard Level will be examined. Results for the BSE-1E level can be compared against those for the Special Procedure example in Chapter 12 of this *Guide* as it also uses the BSE-1E Seismic Hazard Level.

This example illustrates the following:

- **Section 13.3:** Condition assessment of existing materials (ASCE 41-13 § 11.2.2)
- **Section 13.4:** Calculations for in-place shear testing of masonry walls (ASCE 41-13 § 11.2.3)

- **Section 13.5:** Assessment of existing masonry shear strength (ASCE 41-13 § 11.2.3.6)
- **Section 13.6:** Evaluation of in-plane shear capacity of existing masonry walls using the LSP (ASCE 41-13 § 11.3.2.2) and review of the LSP limitations ASCE 41-13 § 7.3.1.1
- **Section 13.7:** Evaluation of existing floor and roof diaphragms and retrofitted floor and roof diaphragms (ASCE 41-13 § 12.5)
- **Section 13.8:** Evaluation of existing masonry walls for out-of-plane seismic demands (ASCE 41-13 § 11.3.3)

The example does not include retrofit design of the URM walls for in-plane or out-of-plane demands and evaluation or retrofit design of wall-to-diaphragm tension and shear ties. These items are discussed in detail in Chapter 12 of this *Guide*.

13.2 Introduction to Tier 3 Evaluation and Retrofit

The seismic evaluation process outlined in ASCE 41-13 consists of the following three tiers: Tier 1 screening procedure, Tier 2 deficiency-based evaluation procedure, and Tier 3 systematic evaluation procedure. Tier 1 screening relies on the completion of checklists of evaluation statements that identify potential deficiencies in a building based on performance of similar buildings in past earthquakes. The Tier 1 screening procedure summary for this example can be viewed in Section 12.6 of this *Guide*. If a building does not conform to the Common Building Type in ASCE 41-13 § 3.2.1 and height limit from ACSE 41-13 Table 3-2, a Tier 3 evaluation must be performed. Alternatively, the evaluation process may proceed directly to the Tier 3 systematic evaluation as an option.

For the example outlined in this chapter, the building, shown in Figure 13-1, is a two-story URM building to be analyzed for both the Life Safety Structural Performance Level at the BSE-1E Seismic Hazard Level (S-3) and the Collapse Prevention Performance Level at the BSE-2E Seismic Hazard Level (S-5). According to ASCE 41-13 Table 3-2, the height limit for URM buildings for a High Level of Seismicity at the S-3 level is four stories. Therefore, a Tier 3 evaluation is not required. However, the example provided in this chapter outlines the procedure for a Tier 3 evaluation of a URM building, should the engineer choose to perform one.

The Tier 3 systematic evaluation performed in this chapter differs from the Special Procedure outlined ASCE 41-13 § 15.2 in that it is not self contained within a single chapter of ASCE 41-13. The Special Procedure represents a holistic approach to evaluating a URM building whereas the typical requirements for seismic evaluation in ASCE 41-13 are component based, including those for URM buildings. Unlike the Special Procedure, to perform a Tier 3 systematic evaluation of a URM building using ASCE 41-13 Chapter 11, the engineer will need to utilize multiple chapters to complete the analysis, evaluation, and retrofit. For example, the scope of the structural analysis is in accordance with ASCE 41-13 § 7.2; criteria for masonry walls are included in ASCE 41-13 Chapter 11; criteria for wood diaphragms are defined in ASCE 41-13 Chapter 12; out-of-plane wall anchorage is in ASCE 41-13 § 7.2.11.1 and ASCE 41-13 § 11.5; out-of-plane wall strength is in ASCE 41-13 § 7.2.11.2 and ASCE 41-13 § 11.3; and many other items use the general analysis provisions in ASCE 41-13 Chapter 7.

Refer to Section 12.2 of this *Guide* for a detailed building description. This example was drawn from the URM bearing wall example in the *2009 IEBC SEAOC Structural/Seismic Design Manual* (SEAOC, 2012).

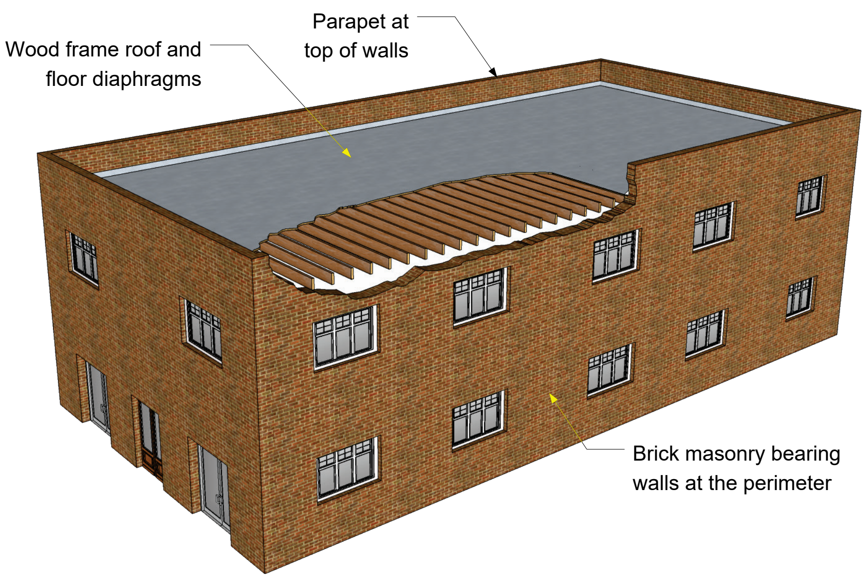


Figure 13-1 Figure showing exterior of example building.

Figure 13-2 through Figure 13-4 repeat plans and elevations first shown in Chapter 12 of this *Guide* for convenience and reference for the calculations preformed in this chapter.

13.3 Condition of Materials (ASCE 41-13 § 11.2.2)

The procedure for assessing masonry condition is defined in ASCE 41-13 § 11.2.2. Mechanical properties of the masonry are based on available drawings, specifications, and other documents for the existing construction in accordance with the requirements of ASCE 41-13 § 6.2. For this example, design drawings have been obtained and usual testing has been performed.

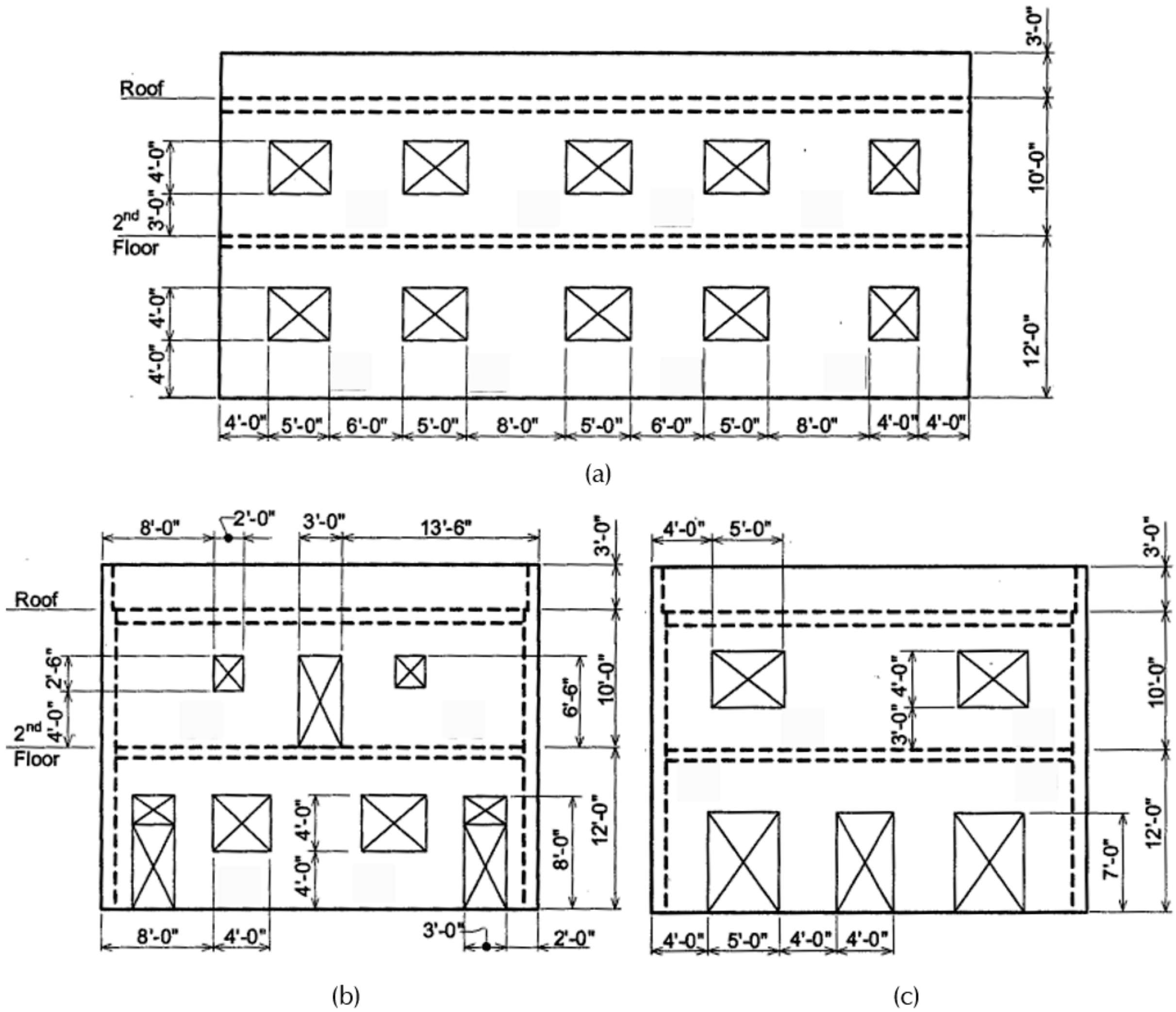


Figure 13-2 Figure showing (a) side wall elevation, (b) rear (north) wall elevation, and (c) front (south) wall elevation.

Therefore, per ASCE 41-13 Table 6-1, the level of knowledge falls under the category of usual, and the knowledge factor, κ, is equal to 1.0.

Where design drawings are available, information is verified through a visual condition assessment as described in ASCE 41-13 § 11.2.2. For this example, based on visual assessment, it is concluded that:

- The physical condition of the primary and secondary components does not display any significant degradation or deterioration due to weathering.
- The presence and configuration of components and their connections and the continuity of load paths have been verified.
- Other conditions that may influence building performance have been identified and documented.

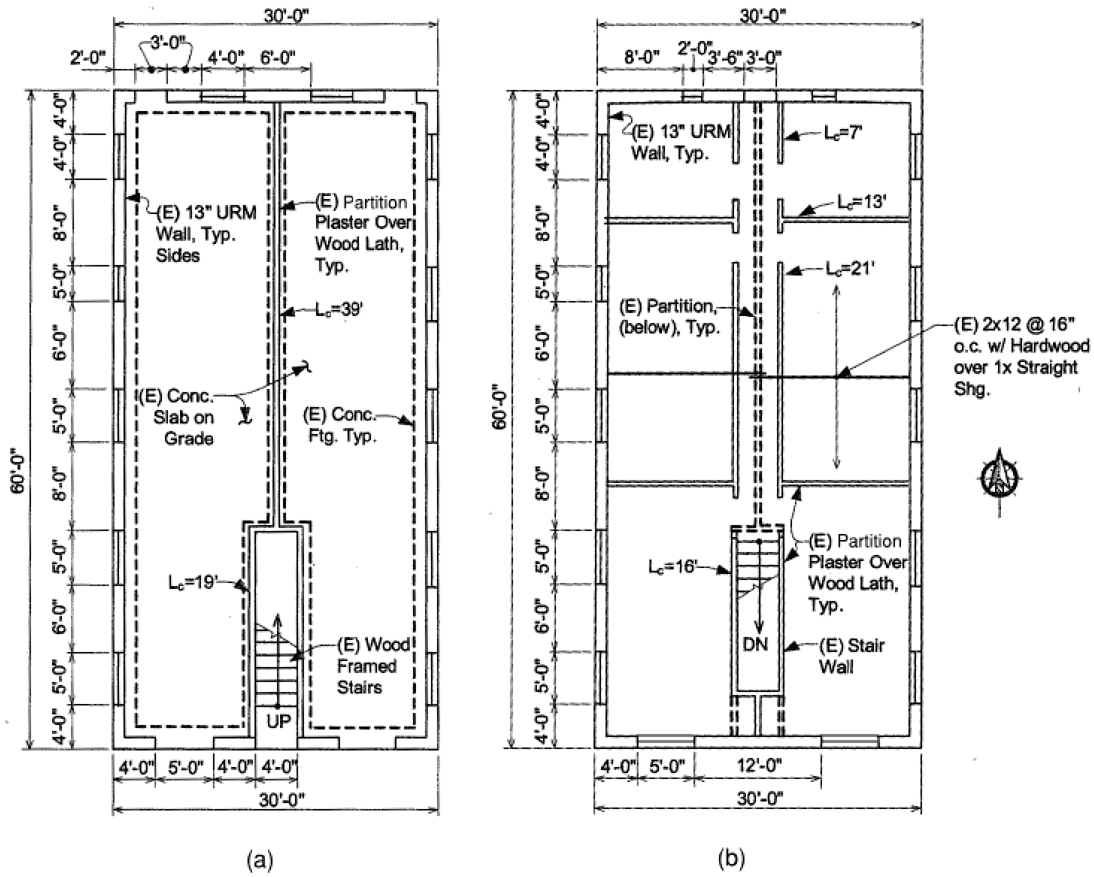


Figure 13-3 Plans for (a) first floor and (b) second floor.

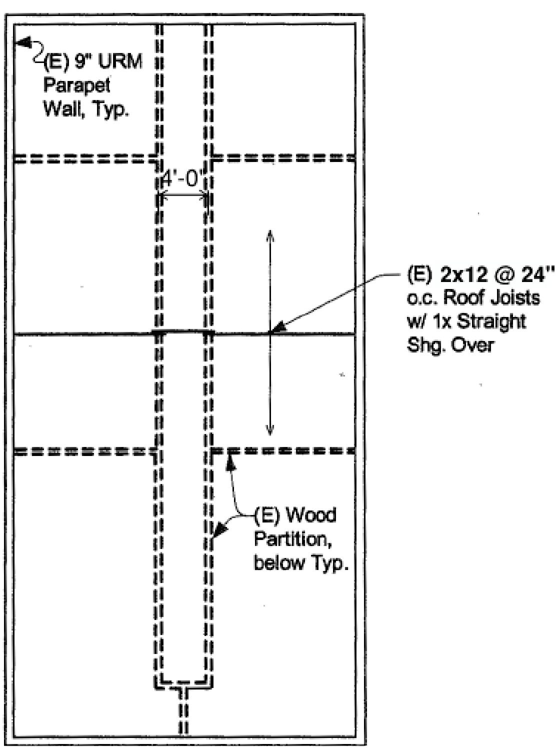


Figure 13-4 Roof plan.

Masonry found during condition assessment displayed no visible cracking, deterioration, or damage. Thus, the existing masonry condition is classified as good condition.

13.4 In-Place Shear Testing (ASCE 41-13 § 11.2.3)

Masonry shear strength is determined by in-place shear testing. ASCE 41-13 § 11.2.3.9 defines the minimum number of tests for each masonry class. For the usual level of testing, the minimum requirements are as follows:

- At the top and first stories, no fewer than two tests per wall or line of wall elements providing a common line of resistance to lateral forces
- At each of all other stories, no fewer than one test per wall or line of wall elements providing a common line of resistance to lateral forces
- No fewer than one test per 1,500 ft^2 of wall surface and no fewer than a total of eight tests

The ASCE 41-13 § 11.2.3.9 requirements are the same as those in ASCE 41-13 § 15.2.2.2 for the Special Procedure. Thus, the Tier 3 push test requirements are satisfied with the in-place push tests documented in Section 12.9 of this *Guide*. It should be noted that ASCE 41-13 § 11.2.3.6 and § 15.2.2.2 discuss two types of masonry tests: in-plane mortar testing, and tensile splitting testing. This example is performed assuming in-place mortar testing has been performed. Refer to Section 12.9 of this *Guide* for a description of tests performed, calculation of v_{to}, bed-joint shear stress from a single test, and mortar shear test results summary. Refer to Table 12-6 of this *Guide* for mortar shear test results.

ASCE 41-13 § 11.2.3.6 defines the mortar shear strength, v_{te}, as the average of the bed-joint shear strength test values. Since this building has one masonry class, the mortar shear strength is calculated by averaging the tests performed in Chapter 12 of this *Guide*. Therefore, the average of the test results from Table 12-6 yields v_{te} = 95 lb/in^2.

> **Commentary**
>
> In-place test material properties and strengths for the Tier 3 procedure differ in subtle ways from the ASCE 41-13 Special Procedure.
>
> **Differences in mortar shear strength, v_{te}:** Per ASCE 41-13 Equation 11-2, v_{te} is defined as the average of the bed-joint shear strength test values.
>
> Per ASCE 41-13 Equation 15-1, v_{te} is defined as the shear strength exceeded by 80% of the tests for each masonry class.
>
> **Differences in expected masonry capacity, v_{me}:** There is an additional 1/1.5 factor applied to ASCE 41-13 Equation 11-2. The 1.5 factor is introduced in the pier shear capacity of ASCE 41-13 Equation 15-19 with 0.67 (= 1/1.5), which is also identical to IEBC Equation A1-20. Thus, the Tier 3 evaluation, the Special Procedure, and IEBC/A1 all have the 1.5 factor; however, ASCE 41-13 Equation 11-2 applies the 1/1.5 factor to the masonry strength, while ASCE 41-13 Chapter 15 and IEBC/A1 apply the 1/1.5 factor to the pier capacity. ASCE 41-13 C11.2.3.6.1 notes that the 1/1.5 factor is to translate tested shear capacity of a single brick to average shear on a wall or pier.

13.5 Expected Masonry Strength for Bed-Joint Sliding (ASCE 41-13 § 11.2.3.6)

The expected unreinforced masonry bed-joint sliding strength, v_{me}, is calculated individually for each wall pier as a function of the pier area, the dead load at the top of the pier, and the mortar strength. As stated in Section 12.9 of this *Guide*, this building has one masonry class. Therefore, the expected masonry shear strength is calculated as shown below:

$$v_{me} = \frac{0.75\left(0.75v_{te} + \dfrac{P_D}{A_n}\right)}{1.5} \qquad \text{(ASCE 41-13 Eq. 11-2)}$$

where:

v_{me} = Expected unreinforced masonry bed-joint sliding strength (lb/in.^{2})

v_{te} = Average of bed-joint shear strength test values (lb/in.^{2})

P_D = Superimposed dead load at the top of the pier (lb)

A_n = Area of net mortared/grouted section of the pier (in.^{2})

It should be noted that ASCE 41-13 § 11.2.3.6.1 permits the 0.75 factor on v_{te} to be 1.0 if the mortar in the collar joint is not present of in poor condition.

Figure 12-9 of this *Guide* displays the wall labels corresponding to each pier and has been replicated as Figure 13-5. Walls analyzed in Chapter 12 of this *Guide* have the same label for the evaluation in this chapter. For a typical pier on a side wall (Pier 11 in Figure 13-5a), the unreinforced masonry strength is calculated as:

v_{te} = 95 lb/in.^{2}

P_D = Dead load due to parapet and other walls above the top pier and tributary roof and second floor areas

= (9" brick wall weight lb/ft^2)(pier tributary width ft)(parapet height ft) + (13" brick wall weight lb/ft^2)[(pier tributary width ft)(tributary wall height above pier ft) + (pier height ft)(pier width ft) + (pier tributary width ft)(tributary wall height below pier ft)] + (roof flat load lb/ft^2)(pier tributary width ft)(roof tributary width ft) + (second floor flat load lb/ft^2)(pier tributary width ft)(second floor tributary width ft)

= (90 lb/ft^2)(12.5 ft)(3 ft) + (130 lb/ft^2)[(12.5 ft)(3 ft) + (4 ft)(8 ft) + (12.5 ft)(7 ft)] + (19 lb/ft^2)(12.5 ft)(6.5 ft) + (30.5 lb/ft^2)(12.5 ft)(7.5 ft)

= 28,200 lb

A_n = (depth of pier)(wall thickness)

= (8 ft)(12 in/ft)(13 in.)

= 1,248 in.^{2}

v_{me} = (0.75 [0.75(95 lb/in.^{2}) + (28,200 lb)/(1,248 in.^{2})])/1.5
= 47.0 lb/in.^{2}

Commentary

Limits on the average bed-joint shear strength test value, v_{te}: Per ASCE 41-13 § 11.2.3.6, v_{te} shall not exceed 100 lb/in^2.

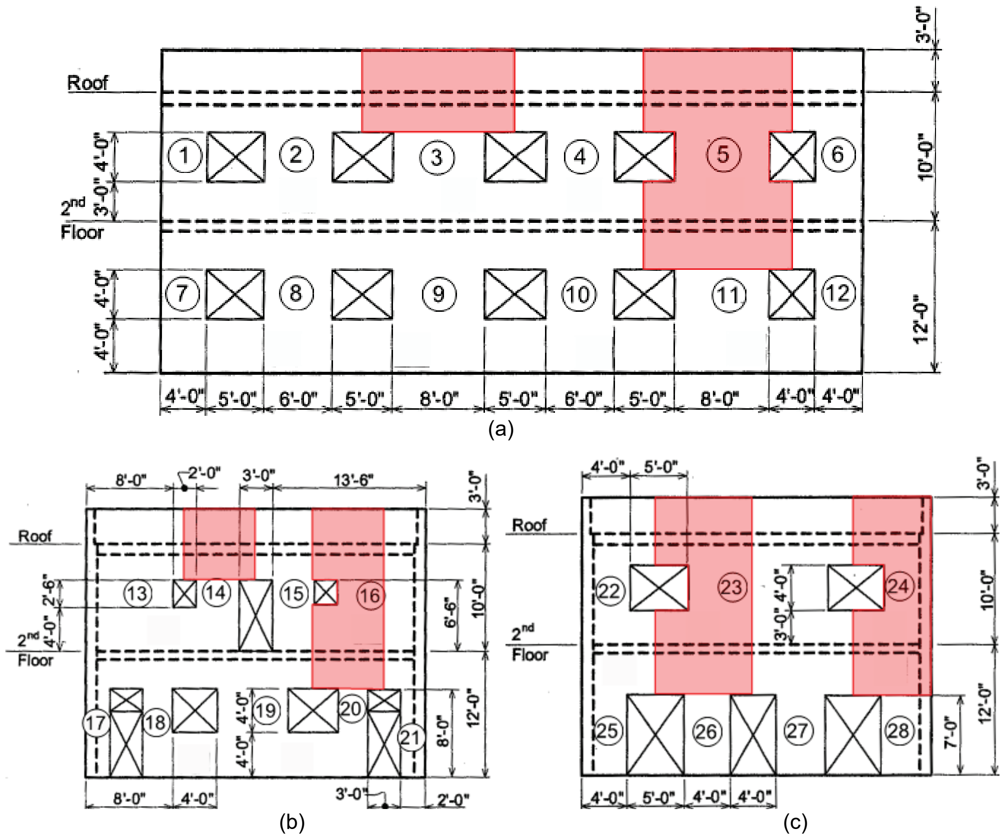


Figure 13-5 Wall pier labels: (a) side wall elevation; (b) rear (north) wall elevation; and (c) front (south) wall elevation. Tributary wall areas for the calculation of P_D are highlighted for representative piers.

Similar to the calculation of dead load for mortar shear tests, the roof and second floor loads are only considered in the dead load for the side walls. Since the floors span east-west, they do not contribute dead load to piers at the front or rear walls.

A summary of the dead load acting on each wall pier and the resulting unreinforced masonry strength is shown in Table 13-1 of this *Guide*.

Note that ICC Evaluation Service reports for proprietary anchors typically specify that a minimum average mortar strength of 50 psi is required, but this is as calculated in accordance with the *International Existing Building Code* (IEBC; ICC, 2015). The IEBC uses the same approach as ASCE 41-13 § 15.2.2.3. The average of the strengths in Table 12-7 of this *Guide* is 55 psi, which is over 50 psi.

Table 13-1 Expected Unreinforced Masonry Strength

Pier	t (in.)	D (ft)	A_n (in.^{2})	P_D (lb)	v_{me} (lb/in.^{2})
1	13	4	624	5,100	39.8
2	13	6	936	8,600	40.3
3	13	8	1,248	10,200	39.8
4	13	6	936	8,600	40.3
5	13	8	1,248	9,800	39.7
6	13	4	624	4,700	39.5
7	13	4	624	14,400	47.3
8	13	6	936	23,900	48.5
9	13	8	1,248	29,100	47.4
10	13	6	936	24,300	48.7
11	13	8	1,248	28,200	47.0
12	13	4	624	13,600	46.6
13	13	8	1,248	6,500	38.4
14	13	3.5	546	4,400	39.8
15	13	3.5	546	4,400	39.8
16	13	8	1,248	6,500	38.4
17	13	2	312	7,300	47.4
18	13	3	468	12,800	49.4
19	13	6	936	19,925	46.4
20	13	3	468	12,800	49.4
21	13	2	312	7,300	47.4
22	13	4	624	4,300	39.2
23	13	12	1,872	11,200	38.7
24	13	4	624	4,300	39.2
25	13	4	624	13,100	46.2
26	13	4	624	17,700	49.9
27	13	4	624	17,800	50.0
28	13	4	624	13,400	46.5

13.6 In-plane Capacity (ASCE 41-13 § 11.3.2.2) and Demand (ASCE 41-13 § 7.4.1) of Shear Walls with Linear Static Procedure

ASCE 41-13 § 11.3.2.2 indicates that when calculating the strength of URM walls subject to in-plane actions using the linear static procedure, inclusion of the effects of wall flanges, spandrels, and vertical components of seismic

Commentary

The Special Procedure of ASCE 41-13 § 15.2 defines wall pier height as the least clear height. The pier height used in the Tier 3 calculation of rocking in ASCE 41-13 Chapter 11 is more complex, depending on the direction of loading. In addition, the Special Procedure does not make a distinction between interior wall piers and corner piers with flanged returns. For simplicity, only the rectangular portion of the corner pier on the in-plane wall line and a constant wall height are used in Section 13.6.2 of this *Guide*.

See Section 13.6.3 and Section 13.6.4 of this *Guide* for examples where pier height refinements and flange effects are incorporated.

ASCE 41-13 Chapter 11 focuses on pier behavior and assumes solid spandrels are not the weak link in wall in-plane behavior.

loading is required. The following section contains three different examples outlining why the consideration of these elements is critical and can produce significantly different results. This section illustrates the following:

- **Section 13.6.1:** Procedure for calculating the linear static procedure capacity and demand of a URM shear wall using ASCE 41-13 § 11.3.2.2 and demand using ASCE 41-13 § 7.4.1
- **Section 13.6.2:** Procedure for calculating in-plane capacity and demand of shear without considering the alternative pier height effect (i.e., the effect on pier height of different opening heights on the sides of the pier) and ignoring the effect of wall flanges
- **Section 13.6.3:** Procedure for calculating in-plane capacity and demand of shear walls considering the alternative pier height effect but not wall flanges
- **Section 13.6.4:** Procedure for calculating in-plane capacity and demand of shear walls with linear static procedure considering the alternative pier height effect and wall flanges
- **Section 13.6.5:** A comparison of results for the three different sets of assumptions and the Special Procedure used in Chapter 12
- **Section 13.6.6:** Review of the limitations for using the LSP per ASCE 41-13 § 7.3.1.1

13.6.1 Procedure Using m-Factors in ASCE 41-13 Table 11-3

The following section contains a procedure for calculating the demand and the capacity of a URM shear wall using the linear static procedure in ASCE 41-13.

13.6.1.1 Determination of Capacity

ASCE 41-17 Revision

ASCE 41-17 Chapter 11 added provisions for evaluating spandrel behavior as part of in-plane wall evaluations.

ASCE 41-13 § 11.3.2.2 describes four different failure mechanisms that can occur from in-plane loading of unreinforced masonry shear walls. Expected in-plane strength of URM walls is the lesser of the rocking strength in ASCE 41-13 § 11.3.2.2.1 or the bed-joint sliding strength in ASCE 41-13 § 11.3.2.2.2. Both of these two failure mechanisms are deformation-controlled actions. The lower-bound in-plane strength of URM walls is the lesser of the toe crushing strength described in ASCE 41-13 § 11.3.2.2.3 or the diagonal tension strength in ASCE 41-13 § 11.3.2.2.4. Both of these two failure mechanisms are force-controlled actions. The lowest capacity for all four URM wall failure modes is the governing failure mechanism for each wall pier.

Per ASCE 41-13 § 7.5.2, the deformation-controlled actions for the LSP have the following acceptance criteria:

$$Q_{UD} = Q_G + Q_E \qquad \text{(ASCE 41-13 Eq. 7-34)}$$

where:

Q_{UD} = Deformation-controlled action caused by gravity loads in combination with earthquake forces

Q_G = Action caused by gravity loads

Q_E = Action caused by seismic loads at the selected Seismic Hazard Level

and:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

where:

m = Component capacity modification factor to account for the expected ductility at the selected Structural Performance Level listed in ASCE 41-13 Table 11-3 for URM walls

Q_{CE} = Expected strength of component deformation-controlled action of an element at the deformation level under consideration

κ = 1.0 per Section 13.3 of this *Guide*

Q_{UD} = In-plane shear demand

Per ASCE 41-13 § 7.5.2, the force-controlled actions for the LSP have the following demand capacity ratio (DCR) acceptance criteria:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \qquad \text{(ASCE 41-13 Eq. 7-35)}$$

where:

Q_{UF} = Force-controlled action caused by gravity loads in combination with earthquake forces

C_1 = Modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response per ASCE 41-13 § 7.4.1.3.1

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response per ASCE 41-13 Equation 7-21

J = Force delivery reduction factor, greater than or equal to 1.0, taken as the smallest DCR of the components in the load path delivering force to the component in question calculated per ASCE 41-13 § 7.4.1.3.1

and:

$$\kappa Q_{CL} > Q_{UF} \quad \text{(ASCE 41-13 Eq. 7-37)}$$

where:

Q_{UF} = per above

Q_{CL} = Lower-bound strength for the force-controlled action at the deformation level under consideration

κ = 1.0 per Section 13.3 of this *Guide*

Like the Special Procedure used in Chapter 12, the Tier 3 evaluation depends on the failure mechanisms of other wall piers in the same wall line. Each wall or wall pier is evaluated on an individual basis. According to ASCE 41-13 § 11.3.2.3, the wall piers are considered:

> *Deformation-controlled if the expected lateral rocking strength or bed-joint sliding strength of each wall or wall pier in the line of resistance, as specified in Sections 11.3.2.2.1 and 11.3.2.2.2, is less than the lower-bound strength of each wall or wall pier limited by diagonal tension or toe crushing, as specified in Sections 11.3.2.2.3 and 11.3.2.2.4. URM walls that do not meet the criteria for deformation-controlled components shall be considered force-controlled components. Expected rocking strength, V_r, as specified in Section 11.3.2.2.1, shall be neglected in lines of resistance not considered deformation controlled.*

However, unlike the Special Procedure, it is not explicitly stated that the resistance of all wall piers should be summed to calculate the resistance of the entire wall line. Additionally, this is a requirement for linear procedures only. Nonlinear procedures do not neglect the lateral strength associated with rocking.

For the deformation-controlled actions, the expected in-plane rocking strength is calculated as follows:

$$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5P_w)L/h_{\text{eff}} \quad \text{(ASCE 41-13 Eq. 11-8)}$$

where:

Q_{CE} = Expected lateral strength of URM walls or wall piers

V_r = Strength of wall based on rocking (lb)

α = Factor equal to 1.0 for a fixed-fixed wall pier (the typical situation) or 0.5 for a fixed-free cantilever wall

P_D = Superimposed dead load at the top of the pier (lb)

P_w = Self weight of the wall pier (lb)

L = Length of wall or wall pier (ft)

h_{eff} = Height to the resultant seismic force (ft)

ASCE 41-13 distinguishes between initial in-plane bed-joint sliding strength and final in-plane bed joint sliding strength. The initial strength represents the URM shear strength before the mortar has cracked. The shear resistance due to bed-joint sliding is a function of bond strength and frictional resistance. The bond strength decreases while the mortar is cracking, until only the frictional component remains. The expected initial in-plane bed joint sliding strength is calculated as follows:

$$Q_{CE} = V_{bjs1} = v_{me} A_n \qquad \text{(ASCE 41-13 Eq. 11-9)}$$

where:

V_{bjs1} = Expected initial strength of wall or pier based on bed-joint sliding shear strength (lb)

v_{me} = Expected bed-joint sliding strength per Section 13.5 (lb/in.^{2})

A_n = Area of net mortared or grouted section of wall or wall pier (in.^{2})

The final strength represents the URM shear strength after the mortar has cracked and only the frictional component resists the lateral forces. The expected final in-plane bed joint sliding strength is calculated as follows:

$$Q_{CE,f} = V_{bjs2} = 0.5P_D \qquad \text{(ASCE 41-13 Eq. 11-10)}$$

where:

V_{bjs2} = Expected final strength of wall or pier based on bed-joint sliding (lb)

P_D = Superimposed dead load at the top of the pier or wall (lb)

ASCE 41-13 § 11.3.2.3.1 indicates that if a wall's in-plane behavior is governed by bed-joint sliding, V_{bjs1} is used when assessing component behavior at the Immediate Occupancy Structural Performance Level. For all other Performance Levels, V_{bjs2} is to be used. V_{bjs2} is the theoretical in-plane strength of the wall after all the mortar has cracked and the only resistance for shear is provided by friction between the masonry units. The V_{bjs2}

concept first appeared in FEMA 306 (FEMA, 1998a), and the approach in ASCE 41-13 is a significant simplification of the methodology in FEMA 306. More background on V_{bjs2} and its application is contained in FEMA 306. Using V_{bjs2} for the expected in-plane bed-joint sliding strength ignores the potential effects of residual mortar strength and can result in a very conservative estimate of in-plane wall capacity, as will be shown in the example in Section 13.6.1.2 of this *Guide*. ASCE 41-17 has revised the requirement in § 11.3.2.3.1 and instead allows for V_{bjs1} to be used as the expected strength of the wall or wall pier based on bed-joint sliding. The ASCE 41-17 approach will be used in this example to avoid the very conservative estimate and ultimately inaccurate use of V_{bjs2} in the approach in ASCE 41-13. Thus, for the deformation-controlled actions, V_r is compared to V_{bjs1}, not V_{bjs2}.

The behavior mode that governs the force-controlled actions will be either the toe crushing strength or the diagonal tension strength of each wall or wall pier. The toe crushing failure occurs at the base of rocking walls where the bottom corner is under compression and leads to crushing of the masonry units at the base. Diagonal tension cracking is a result of the tensile strength of the masonry being exceeded when subjected to the applied stress state during earthquake loading. Unlike bed-joint sliding where cracks typically stair step in the mortar bed and head joints, diagonal tension cracking typically goes directly through the masonry units and can be identified by diagonal "X" cracks.

For the force-controlled actions, the expected lower bound in-plane toe crushing strength is calculated as follows:

$$Q_{CL} = V_{tc} = \left(\alpha P_D + 0.5 P_w\right)\left(\frac{L}{h_{\text{eff}}}\right)\left(1 - \frac{f_a}{0.7 f'_m}\right) \qquad \text{(ASCE 41-13 Eq. 11-11)}$$

where: P_D, P_w, L, h_{eff}, and α are the same as those given in ASCE 41-13 Equation 11-8

Q_{CL} = Lower-bound lateral strength of URM walls or wall piers

V_{tc} = Lower-bound shear strength based on toe crushing for a wall or pier (lb)

f_a = Axial compressive stress caused by gravity loads (psi)

f'_m = Lower-bound masonry compressive strength (psi)

The lower bound in-plane diagonal tension strength is calculated as follows:

$$Q_{CL} = V_{dt} = f'_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f'_{dt}}} \qquad \text{(ASCE 41-13 Eq. 11-12)}$$

where:

Q_{CL} and f_a are per above

V_{dt} = Lower-bound shear strength based on diagonal tension stress for wall or pier (lb)

f'_{dt} = Lower-bound masonry diagonal tension strength (lb/in.^{2})

A_n = Area of net mortared or grouted section of wall or wall pier

β = 0.67 for $L/h_{\text{eff}} < 0.67$; L/h_{eff} for $0.67 < L/h_{\text{eff}} \leq 1.0$; or 1.0 for $L/h_{\text{eff}} > 1.0$

In contrast to the Special Procedure, the in-plane shear capacity for URM walls and wall piers used in the Tier 3 evaluation is dependent on direction of loading and inclusion of wall flanges at interior and exterior corner walls. The effective height of the resultant seismic force used in the Tier 3 calculation for rocking depends on direction of loading based on ASCE 41-13 § C11.3.2.2. In addition, the Special Procedure does not make a distinction between interior wall piers and corner wall piers with flanged returns.

As a reference point, only the rectangular portion of the corner pier on the in-plane wall line and a constant wall height are used in the example in Section 13.6.2 of this *Guide*. This example contains worked through calculations for wall capacity for Pier 1 and Pier 3. The example provided in Section 13.6.3 of this *Guide* illustrates the height difference and how it affects loading to all piers as well as the individual failure mechanism. Section 13.6.3 contains worked through example calculations for wall capacity for Pier 1 only. The example provided in Section 13.6.4 of this *Guide* demonstrates the difference in wall capacity achieved with the inclusion of wall flanges. Section 13.6.4 contains worked example calculations for wall capacity for Pier 1 only. Finally, Section 13.6.5 of this *Guide* includes a summary, comparing the results from the three variations in Sections 13.6.2, 13.6.3, and 13.6.4 with the results from the Special Procedure performed in Chapter 12.

13.6.1.2 Determination of Demand

Per ASCE 41-13 § 6.3, a Tier 3 systematic evaluation shall include an analysis performed in accordance with ASCE 41-13 Chapter 7 for structural systems. ASCE 41-13 § 7.4.1 describes the linear static analysis procedure. A pseudo lateral force is utilized to predict nonlinear displacements using a linear analysis method. The vertical distribution of the pseudo seismic force (per ASCE 41-13 Equation 7-21) is based on a dynamic analysis and is determined by ASCE 41-13 Equation 7-24 and ASCE 41-13 Equation 7-25.

In order to use the LSP, the building must comply with a set of criteria provided in ASCE 41-13. ASCE 41-13 § 7.3.1.1 states, "If a component DCR exceeds the lesser of 3.0 and the *m*-factor for the component action and any irregularity described in Section 7.3.1.1.3 or Section 7.3.1.1.4 is present, then linear procedures are not applicable and shall not be used." These limitations will be checked at the end of this LSP example in Section 13.6.6 of this *Guide*.

The pseudo seismic force is calculated per ASCE 41-13 Equation 7-21.

$$V = C_1 C_2 C_m S_a W \qquad \text{(ASCE 41-13 Eq. 7-21)}$$

where:

C_1 = Modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response

$C_1 C_2$ = Alternate values from ASCE 41-13 Table 7-3

C_m = Effective mass factor to account for higher modal mass participation effects obtained from ASCE 41-13 Table 7-4

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration using the procedure specified in ASCE 41-13 § 2.4

W = Effective seismic weight of building, including total dead load and applicable portions of other loads listed in ASCE 41-13 § 7.4.1.3.1

A first step in calculating several of the above values is to determine the fundamental period for the building. Per ASCE 41-13 Chapter 7, there are three different methods to determine the fundamental period for the linear static procedure. For this example, Method 2, the empirical method, will be used. The fundamental period of the building is determined as follows:

$$T = C_t h_n^{\beta} \qquad \text{(ASCE 41-13 Eq. 7-18)}$$

where:

T = Fundamental period in the direction under consideration

C_t = 0.020 for all other framing systems

h_n = Height above the base to the roof level (ft)

β = 0.75 for all other framing systems

$T = (0.020)(22)^{0.75}$
$= 0.203$ seconds

Response spectrum acceleration, at the fundamental period, is determined in accordance with ASCE 41-13 § 2.4:

S_{X1} and S_{XS} spectral acceleration parameters are determined for 20% in 50 year period for a BSE-1E Seismic Hazard Level, and are used for evaluation of the building at the Life Safety Performance Level (S-3). See Section 12.5 of this *Guide* for the spectral response acceleration parameters.

$T_S = S_{X1}/S_{XS}$ (ASCE 41-13 Eq. 2-9)
$= (0.506)/(0.913)$
$= 0.555$ seconds $> T = 0.203$ seconds

$T_0 = 0.2T_S$ (ASCE 41-13 Eq. 2-10)
$= (0.2)(0.555$ seconds$)$
$= 0.111$ seconds $< T = 0.203$ seconds

$B_1 = 4/[5.6 - \ln(100\beta)]$ (ASCE 41-13 Eq. 2-11)

where:

β = Effective viscous damping ratio
$= 0.050$

$B_1 = 4/[5.6 - \ln(5)]$
$= 1.0$

$S_a = S_{XS}/B_1$ for $T_0 < T < T_S$ (ASCE 41-13 Eq. 2-6)
$= 0.913/1.0 = 0.913$

The alternate values of ASCE 41-13 Table 7-3 will be used to establish C_1C_2. This requires determination of m_{max}, the largest m-factor for all primary elements of the building in the direction under consideration. As a placeholder, the following approach is taken of using the values in ASCE 41-13 Table 11-3.

For rocking, $m = 1.5 \leq 3h_{eff}/L \leq 3.75$ (for primary elements at the Life Safety Performance Level)

Example Pier 1: $3h_{eff}/L = 3(4\text{ ft})/(4\text{ ft}) = 3$

Therefore $3 \leq m_{max} \leq 3.75$

For bed-joint sliding, $m = 3$

Therefore, for either failure mode, $3 \leq m_{max} \leq 3.75$.

Per ASCE 41-13 Table 7-3, with $T = 0.203$ seconds < 0.3, and $2 \leq m_{max} < 6$, then

$$C_1C_2 = 1.4$$

Per ASCE 41-13 Table 7-4: $C_m = 1.0$ (Other building system, 1-2 stories)

Refer to Table 12-4 of this *Guide* for the seismic weight summary for W.

$$V = C_1C_2C_mS_aW \qquad \text{(ASCE 41-13 Eq. 7-21)}$$
$$= (1.4)(1.0)(0.913)(192 \text{ kips} + 275 \text{ kips})$$
$$= 597 \text{ kips}$$

Commentary

Vertical force distribution: Per ASCE 41-13 § 7.4.1.3.2, the vertical distribution of seismic forces is based on ASCE 41-13 Equation 7-25. This method differs from the Special Procedure, where forces are calculated for wall lines, not stories, and the story shear at each story is simply the sum of the story forces.

The vertical distribution of seismic forces is calculated per ASCE 41-13 § 7.4.1.3.2 and summarized in Table 13-2.

$$F_x = C_{vx}V \qquad \text{(ASCE 41-13 Eq. 7-24)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(ASCE 41-13 Eq. 7-25)}$$

where:

C_{vx} = Vertical distribution factor

V = Pseudo seismic force from ASCE 41-13 Equation 7-21 (kips)

k = 1.0 for $T = 0.203$ seconds ≤ 0.5 seconds

w_i = Portion of the effective seismic weight W located on or assigned to level i (kips)

w_x = Portion of the effective seismic weight W located on or assigned to level x (kips)

h_i = Height from the base to level i (ft)

h_x = Height from the base to level x (ft)

Table 13-2 Vertical Distribution of Seismic Forces

Level	W (kips)	h_i (ft)	h_i^k (ft)	wh_i^k	C_v	F_x (kips)
Roof	192	22	22	4,224	0.56	335
Second Story	275	12	12	3,300	0.44	262
Total				7,524		597

Example calculation at roof level:

C_{vr} = (192 kips)(22 ft)$^{1.0}$/[(192 kips)(22 ft)$^{1.0}$+(275 kips)(12 ft)$^{1.0}$]
= 0.56

F_r = (0.56)(597 kips)
= 335 kips

The above ASCE 41-13 Chapter 7 vertical distribution of seismic forces is approximately triangular. The force distribution using the Special Procedure is rectangular. Per ASCE 41-13 Chapter 7, forces are determined for the entire story. With the Special Procedure, forces are calculated for individual wall lines.

For linear procedures, the shear force assigned to each pier is based on its relative rigidity. The lateral in-plane stiffness of solid shear walls is calculated by using the following equation:

For wall piers fully restrained:

$$k = \frac{1}{\dfrac{h_{\text{eff}}^3}{12E_m I_g} + \dfrac{h_{\text{eff}}}{A_v G_m}}$$

Commentary

Calculation of wall pier stiffness, *k*: Per ASCE 41-13 § C11.3.2.1 the stiffness depends on the boundary conditions of the wall. See ASCE 41-13 Eq. C11-1 for a solid cantilevered shear wall.

For this example, f'_m = 1,000 psi × 0.6 per the default lower-bound unreinforced masonry strength listed in ASCE 41-13 Table 11-2(a).

f'_m = (1000 psi)(0.6)
= 600 psi

To translate the lower-bound masonry strength into expected strengths, refer to ASCE 41-13 Table 11-1. The factor to translate lower-bound shear strength to expected shear strength is 1.3. Therefore, the expected masonry shear strength is calculated as:

f_{me} = (600 psi)(1.3)
= 780 psi

ASCE 41-13 § 11.3.2.1 indicates that the expected elastic modulus, E_{me}, in compression should be used and is calculated per ASCE 41-13 § 11.2.3.4, which then references TMS 402-11, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries* (TMS, 2011). However, TMS 402-11 does not make a distinction between the expected elastic modulus, E_{me}, and the modulus of elasticity for masonry, E_m. For this example, E_{me} is assumed to be the modulus of elasticity found using f_{me}. Per ASCE 41-13 § 11.2.3.7, the shear modulus can be taken from TMS 402-11 § 1.8.2.2.1 as follows:

$E_m = 700 f'_m$

$E_{me} = 700 f_{me}$
$= 700(780 \text{ psi})$
$= 546{,}000 \text{ psi}$

$G_{me} = 0.4 E_{me}$
$= 218{,}400 \text{ psi}$

13.6.2 Procedure without Considering the Alternative Pier Height Effect and Wall Flanges

As a useful comparative point of reference, the following examples illustrate a simplified approach to in-plane shear wall capacity by ignoring the alternative pier height effect and wall flanges.

13.6.2.1 Determination of Capacity

Example: Pier 3

Pier 3 from Figure 13-5 of this *Guide* is a non-end wall pier at the second story of the side wall. Deformation-controlled actions for Pier 3 are determined as follows.

The pier rocking capacity, V_r, is calculated as:

$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5 P_w) L / h_{\text{eff}}$ (ASCE 41-13 Eq. 11-8)

where:

$\alpha = 1.0$

$P_D = 10{,}200$ lb (per Table 13-1 of this *Guide*)

$P_w = (120 \text{ lb/ft}^3)[13 \text{ in}/(12 \text{ in/ft})](4 \text{ ft})(8 \text{ ft})$
$= 4{,}160 \text{ lb}$

$h_{\text{eff}} = 4$ ft (per Figure 13-5)

$L = 8$ ft (per Figure 13-5)

$V_r = 0.9[(1.0)(10{,}200 \text{ lb}) + (0.5)(4{,}160 \text{ lb})](8 \text{ ft})/(4 \text{ ft})$
$= 22.1 \text{ kips}$

Note that, for this example of Section 13.6.2.1 of this *Guide*, h_{eff} is based on the smallest pier height either side of the opening and not the h_{eff} defined in ASCE 41-13 Figure C11-3. Also note that ASCE 41-13 Table 11-3 Footnote (b) requires $f_a / f'_m < 4\%$ for rocking modes unless it can be demonstrated by a moment curvature analysis or other method that toe crushing does not occur at the expected pier drift. Otherwise, the pier shall be considered force-controlled.

f_a = (10,200 lb + 4,160 lb)/(1,248 in.^{2})
= 11.5 psi

f'_m = 1,000 psi per above.

f_a / f'_m = 11.5/1,000 = 1.5% < 4%

Thus, the ASCE 41-13 Table 11-3 Footnote (b) requirement is met, and V_r = 22 kips still holds. At the lower level, the dead load is greater. For Pier 9:

f_a = (29,100 lb + 4,160 lb)/(1,248 in.^{2})
= 26.7 psi

f'_m = 600 psi per above.

f_a / f'_m = 26.7/600 = 4.5% > 4% No Good

The Footnote (b) requirement is not explicitly met, although it is quite close. A strict interpretation would trigger an analysis using moment curvature, or some other method, to prove toe crushing would not occur at the expected pier drift. It is assumed for this example that lightly loaded piers such as these ones would not exhibit toe crushing.

The pier initial bed-joint sliding capacity, V_{bjs1}, is calculated as:

$V_{bjs1} = v_{me}A_n$ (ASCE 41-13 Eq. 11-9)

where:

v_{me} = 39.8 lb/in.^{2} (per Table 13-1)

A_n = 1,248 in.^{2} (per Table 13-1)

V_{bjs1} = (39.8 lb/in.^{2})(1,248 in.^{2})
= 49.7 kips

Therefore, rocking failure controls the deformation-controlled actions.

The pier final bed joint sliding capacity, V_{bjs2}, is calculated as:

$V_{bjs2} = 0.5P_D$ (ASCE 41-13 Eq. 11-10)
= (0.5)(10,200 lb)
= 5.1 kips

As stated previously, for the Life Safety Performance Level, ASCE 41-13 § 11.3.2.3.1 indicates that V_{bjs2} is to be used in lieu of V_{bjs1} for the bed joint sliding capacity. However, this is a very conservative approach. Using Pier 3 as an example, V_{bjs2} yields a wall capacity that is more than 4 times lower than the rocking capacity and nearly 10 times lower than the bed-joint sliding capacity using V_{bjs1}. This was revised in ASCE 41-17 to allow use of V_{bjs1}.

For the purposes of this example, V_{bjs1} is used to calculate the expected in-plane capacity for a bed-joint sliding failure mechanism.

Considering the force-controlled actions for Pier 3 in Figure 13-5, the pier toe crushing capacity, V_{tc}, is calculated as:

$$V_{tc} = \left(\alpha P_D + 0.5 P_w\right)\left(\frac{L}{h_{\text{eff}}}\right)\left(1-\frac{f_a}{0.7 f'_m}\right) \qquad \text{(ASCE 41-13 Eq. 11-11)}$$

f_a = (10,200 lb + 4,160 lb)/(1,248 in.^{2})
= 11.5 psi

f'_m = 600 psi per above

V_{tc} = [(1.0)(10,200 lb)+(0.5)(4,160 lb)](8 ft/4ft)[1-(11.5 psi)/[(0.7)(600 psi)]]
= 23.9 kips

In order to calculate the wall or wall pier diagonal tension capacity, f'_{dt}, the diagonal tension strength, is required. ASCE 41-13 allows the engineer to substitute f'_{dt} with v_{mL}, the lower-bound bed-joint shear strength. v_{mL} is determined using the in-plane shear test results performed in Chapter 12 of this *Guide* as well as ASCE 41-13 Equation 11-6. The pier diagonal tension capacity, V_{dt}, is calculated as:

$$V_{dt} = f'_{dt} A_n \beta \sqrt{1+\frac{f'_a}{f'_{dt}}} \qquad \text{(ASCE 41-13 Eq. 11-12)}$$

where:

$$f'_{dt} = v_{mL} = \frac{0.75\left(0.75 v_{tL} + \dfrac{P_D}{A_n}\right)}{1.5} \qquad \text{(ASCE 41-13 Eq. 11-6)}$$

where:

v_{tL} = mean minus one standard deviation of the bed-joint shear strength test values, v_{to}, given in Table 12-6
= 95 psi – 21 psi
= 74 psi

f'_{dt} = (0.75)[(0.75)(74 psi) + (10,200 lb)/(1248 in.^{2})]/1.5
= 32 psi

L/h_{eff} = 8 ft/4 ft
= 2 > 1.0, so β = 1.0

$$V_{dt} = (32\text{ psi})(1248\text{ in.}^2)(1.0)\sqrt{1+(11.5\text{ psi})/(32\text{ psi})}$$
= 46.4 kips

Therefore, toe crushing failure controls the force-controlled actions.

The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of *m*-factors for ductility. For Pier 3:

V_r = 22.1 kips

V_{bjs1} = 49.7 kips

V_{tc} = 23.9 kips

V_{dt} = 46.4 kips

V_r is the lowest shear capacity and is the governing failure mode for Pier 3.

Example: Pier 1

Pier 1 from Figure 13-5 is an end wall pier at the second story side of the side wall. Deformation-controlled actions for Pier 1 are determined as follows:

The Pier 1 rocking capacity, V_r, is calculated as:

$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5P_w)L/h_{\text{eff}}$ (ASCE 41-13 Eq. 11-8)

where:

α = 1.0

P_D = 5,100 lb (per Table 13-1 of this *Guide*)

P_w = $(120\ \text{lb/ft}^3)[13\ \text{in}/(12\text{in/ft})](4\ \text{ft})(4\ \text{ft})$
= 2,080 lb

h_{eff} = 4 ft (per Figure 13-5)

L = 4 ft (per Figure 13-5)

V_r = 0.9[(1.0)(5,100 lb)+(0.5)(2,080 lb)](4 ft)/(4 ft)
= 5.5 kips

The pier initial bed joint sliding capacity, V_{bjs1}, is calculated as:

$V_{bjs1} = v_{me}A_n$ (ASCE 41-13 Eq. 11-9)

v_{me} = 39.8 lb/in.^{2} (per Table 13-1)

A_n = 624 in.^{2} (per Table 13-1)

V_{bjs1} = (39.8 lb/in.^{2})(624 in.^{2})
= 24.9 kips

Therefore, rocking failure controls the deformation-controlled actions.

Considering the force-controlled actions for Pier 1 in Figure 13-5, the pier toe crushing capacity, V_{tc}, is calculated as:

$$V_{tc} = \left(\alpha P_D + 0.5P_w\right)\left(\frac{l}{h_{\text{eff}}}\right)\left(1 - \frac{f_a}{0.7f'_m}\right) \qquad \text{(ASCE 41-13 Eq. 11-11)}$$

f_a = (5,100lb + 2,080 lb)/(624 in.^{2})
= 11.5 psi

f'_m = 600 psi per above

V_{tc} = [(1.0)(5,100 lb)+(0.5)(2,080 lb)](4 ft/4ft)[1-(11.5 psi)/[(0.7)(600 psi)]]
= 6.0 kips

The pier diagonal tension capacity, V_{dt}, is calculated as:

$$V_{dt} = f'_{dt}A_n\beta\sqrt{1 + \frac{f_a}{f'_{dt}}} \qquad \text{(ASCE 41-13 Eq. 11-12)}$$

$$f'_{dt} = v_{mL} = \frac{0.75\left(0.75v_{tL} + \dfrac{P_D}{A_n}\right)}{1.5} \qquad \text{(ASCE 41-13 Eq. 11-6)}$$

where:

v_{tL} = 74 psi (per Pier 3 example above)
= (0.75)[(0.75)(74 psi) + (5,100 lb)/(624 in.^{2})]/1.5
= 32 psi

L/h_{eff} = 4 ft/4 ft
= 1; thus β = 1.0

$$V_{dt} = (32\text{ psi})(624\text{ in.}^2)(1.0)\sqrt{1 + (11.5\text{ psi})/(32\text{ psi})}$$
= 23.2 kips

The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of *m*-factors for ductility. For Pier 1:

V_r = 5.5 kips

V_{bjs1} = 24.9 kips

V_{tc} = 6.0 kips

V_{dt} = 23.2 kips

The rocking failure mechanism, V_r, has the lowest shear capacity and is the governing failure mode for Pier 1.

A summary of the shear capacities for all other wall piers for the deformation-controlled and force-controlled actions is included in Table 13-3 through Table 13-8.

A summary of the four failure modes of each wall pier of the second story side wall piers and their respective shear capacities is shown in Table 13-3.

Table 13-3 Second Story Side Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
1	5.5	24.9	6.0	23.2	Deformation-Controlled: Rocking
2	13.7	37.8	14.8	35.7	Deformation-Controlled: Rocking
3	22.1	49.7	23.9	46.4	Deformation-Controlled: Rocking
4	13.7	37.8	14.8	35.7	Deformation-Controlled: Rocking
5	21.4	49.5	23.1	46.0	Deformation-Controlled: Rocking
6	5.2	24.7	5.6	22.8	Deformation-Controlled: Rocking

A summary of the four failure modes of each wall pier of the first story side wall piers and their respective shear capacities is shown in Table 13-4.

Table 13-4 First Story Side Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
7	13.9	29.5	14.5	31.7	Deformation-Controlled: Rocking
8	34.4	45.4	35.6	49.6	Deformation-Controlled: Rocking
9	56.1	59.2	58.4	63.7	Deformation-Controlled: Rocking
10	34.9	45.6	36.1	50.0	Deformation-Controlled: Rocking
11	54.5	58.7	56.8	62.9	Deformation-Controlled: Rocking
12	13.2	29.1	13.8	31.0	Deformation-Controlled: Rocking

A summary of the four failure modes of each wall pier of the second story front wall piers and their respective shear capacities is shown in Table 13-5.

Table 13-5 Second Story Front Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
22	4.8	24.5	5.2	22.4	Deformation-Controlled: Rocking
23	38.7	72.5	42.0	65.7	Deformation-Controlled: Rocking
24	4.8	24.5	5.2	22.4	Deformation-Controlled: Rocking

A summary of the four failure modes of each wall pier of the second story rear wall piers and their respective shear capacities is shown in Table 13-6.

Table 13-6 Second Story Rear Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
13	22.5	47.9	24.5	42.2	Deformation-Controlled: Rocking
14	6.3	21.7	6.8	19.9	Deformation-Controlled: Rocking
15	6.3	21.7	6.8	19.9	Deformation-Controlled: Rocking
16	22.5	47.9	24.5	42.2	Deformation Controlled: Rocking

A summary of the four failure modes of each wall pier of the first story front wall piers and their respective shear capacities is shown in Table 13-7.

Table 13-7 First Story Front Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
25	7.7	28.9	8.0	31.1	Deformation-Controlled: Rocking
26	10.0	31.2	10.2	35.3	Deformation-Controlled: Rocking
27	10.1	31.2	10.3	35.3	Deformation-Controlled: Rocking
28	7.8	29.0	8.1	31.4	Deformation-Controlled: Rocking

A summary of the four failure modes of each wall pier of the first story rear wall piers and their respective shear capacities is shown in Table 13-8.

Table 13-8 First Story Rear Wall Pier Capacities

Pier	Q_{CE}		Q_{CL}		Governing Failure Mode
	V_r kips	V_{bjs1} kips	V_{tc} kips	V_{dt} kips	
17	1.88	14.8	1.94	16.3	Deformation-Controlled: Rocking
18	9.2	23.1	9.4	25.6	Deformation-Controlled: Rocking
19	29.0	43.4	30.3	46.0	Deformation-Controlled: Rocking
20	9.2	23.1	9.4	25.6	Deformation-Controlled: Rocking
21	1.88	14.8	1.94	16.3	Deformation-Controlled: Rocking

The expected lateral rocking strength of each wall pier is less than the lower-bound strength of each wall or wall pier limited by diagonal tension or toe crushing. Therefore, the URM wall qualifies as a deformation-controlled component, and none of the wall piers are neglected in the analysis.

Example: Application of *m*-Factors, Pier 3 and Pier 1

Using Pier 3 as an example again, the following equation must be satisfied:

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level, the equation for *m*-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 22.1 kips

m = $1.5 \le 3h_{\text{eff}}/L \le 3.75$ (ASCE 41-13 Table 11-3)
= 3(4 ft)/(8 ft)
= 1.5

$m\kappa Q_{CE}$ = (1.5)(1.0)(22.1 kips)
= 33.2 kips

For the end wall, Pier 1:

V_r = 5.5 kips

m = $1.5 \le 3h_{\text{eff}}/L \le 3.75$ (ASCE 41-13 Table 11-3)
= 3(4 ft)/(4 ft)
= 3

$m\kappa Q_{CE}$ = (3.0)(1.0)(5.2 kips)
= 15.6 kips

13.6.2.2 Determination of Demand

For Pier 3 in Figure 13-5:

k = 1/[(48 in.)^3/[(12)(546,000 psi)(13 in.)(96 in.)^3/12]+48 in/[(13 in.)(96 in.)(218,400 psi)])]
= 5,162 kip/in.

The two side walls are symmetric and the diaphragm is a flexible diaphragm. Therefore, the story force distributed to each wall line should be one half of the story force. The same is true for the front wall and rear wall. For the second story side walls, the force to each wall line, F_x, is equal to 335 kips /2 = 167 kips.

> **Commentary**
>
> **Distribution of lateral forces**: The shear force distribution per ASCE 41-13 Chapter 11 differs from the Special Procedure. The Special Procedure incorporates failure mechanisms and relative rigidity with the force distribution whereas the Tier 3 evaluation is based solely on relative rigidity.

Example calculation of shear demand for Pier 3 in Figure 13-5:

$$V_p = \frac{k}{\Sigma k}(F_x)$$

= [(5,162 kip/in.)/(21,612 kip/in.)](167 kips)
= 40 kips

A summary of the stiffness of each wall pier of the second story side walls and its respective shear force is shown in Table 13-9.

For Pier 11 in Figure 13-5:

k = 1/(48 in.^{3}/[(12)(546,000 psi)(13 in.)(96 in.)^3/(12in/ft)]+48 in/[(13 in.)(96 in.)(218,400 psi)])
= 5,162 kip/in.

The two side walls are symmetric and the diaphragm is a flexible diaphragm. Therefore, the story force distributed to each wall line should be one half of the story force. The same is true for the front wall and rear wall. For the first story side walls, the force to each wall line, F_x, is equal to 597 kips /2 = 298 kips.

Table 13-9 Second Story Side Wall Pier Stiffnesses and Shears

Pier	K kips/in.	$k/\Sigma k$	V_p kips
1	2,028	0.09	16
2	3,616	0.17	28
3	5,162	0.24	40
4	3,616	0.17	28
5	5,162	0.24	40
6	2,028	0.09	16
∑	21,612	1.00	167

Example calculation of shear demand for Pier 11 in Figure 13-5:

$$V_p = \frac{k}{\Sigma k}\left(F_x\right)$$

= [(5,162 kip/in.)/(21,612 kip/in.)](298 kips)
= 71 kips

A summary of the stiffness of each wall pier of the first story side walls and its respective shear force is shown in Table 13-10.

A summary of the stiffness of each wall pier of the second story front wall and its respective shear force is shown in Table 13-11.

A summary of the stiffness of each wall pier of the first story front wall and its respective shear force is shown in Table 13-12.

A summary of the stiffness of each wall pier of the second story rear wall and its respective shear force is shown in Table 13-13.

A summary of the stiffness of each wall pier of the first story rear wall and its respective shear force is shown in Table 13-14.

Table 13-10 First Story Side Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
7	2,028	0.09	28
8	3,616	0.17	50
9	5,162	0.24	71
10	3,616	0.17	50
11	5,162	0.24	71
12	2,028	0.09	28
∑	21,612	1.00	298

Table 13-11 Second Story Front Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
22	2,028	0.17	28
23	8,155	0.67	112
24	2,028	0.17	28
∑	12,211	1.00	167

Table 13-12 First Story Front Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
25	729	0.25	75
26	729	0.25	75
27	729	0.25	75
28	729	0.25	75
∑	2,917		298

Table 13-13 Second Story Rear Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
13	8,744	0.36	61
14	3,301	0.14	23
15	3,301	0.14	23
16	8,744	0.36	61
∑	24,090	1.00	167

Table 13-14 First Story Rear Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
17	96	0.02	5
18	1,245	0.20	59
19	3,616	0.57	171
20	1,245	0.20	59
21	96	0.02	5
∑	6,297	1.00	298

13.6.2.3 Determination of Acceptance Ratio

Pier 3 from Figure 13-5 is a non-end wall pier at the second story side of the side wall. The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of *m*-factors for ductility. For Pier 3, from Section 13.6.2.1 above:

V_r = 22.1 kips

V_{bjs1} = 49.7 kips

V_{tc} = 23.9 kips

V_{dt} = 46.4 kips

V_r is the lowest shear capacity and is the governing failure mode for Pier 3. Now that deformation-controlled rocking has been determined as the governing mode, the acceptance criterion is given as follows:

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level the equation for *m*-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 22.1 kips

m = 1.5 (as shown above)

$m\kappa Q_{CE}$ = 33.2 kips

$Q_{UD} = Q_G + Q_E$ (ASCE 41-13 Eq. 7-34)

where:

Q_G = 0 kips

$Q_E = V_p$ = 40 kips (per Table 13-9 of this *Guide*)

= 0 kips + 40 kips

= 40 kips

$Q_{UD}/m\kappa Q_{CE}$ = (40 kips)/(33.2 kips)
= 1.21

The term $Q_{UD}/m\kappa Q_{CE}$ is defined in this *Guide* as the acceptance ratio (see Chapter 2 for additional details). Acceptance ratios over 1.0 indicate the component has inadequate capacity. Therefore, with an acceptance ratio here of 1.21, there is insufficient rocking capacity.

Pier 1 from Figure 13-5 is an end wall pier at the second story side of the side wall. For Pier 1:

V_r = 5.5 kips

V_{bjs1} = 24.9 kips

V_{tc} = 6.0 kips

V_{dt} = 23.2 kips

V_r is the lowest shear capacity and is the governing failure mode for Pier 1.

V_r = 5.5 kips

m = 3.0 (as shown above)

$m\kappa Q_{CE}$ = 16.6 kips

$Q_{UD} = Q_G + Q_E$ (ASCE 41-13 Eq. 7-34)

Q_G = 0 kips

$Q_E = V_p$ = 16 kips (Per Table 13-9)
= 0 kips + 16 kips
= 16 kips

$Q_{UD}/m\kappa Q_{CE}$ = (16 kips)/(16.6 kips)
= 0.95

Therefore, there is approximately sufficient rocking capacity.

A summary of the acceptance ratios for all other wall piers for the deformation-controlled actions is included in Table 13-15 through Table 13-20.

A summary of the acceptance ratios for the deformation-controlled actions of each wall pier of the second story side wall is shown in Table 13-15.

Table 13-15 Second Story Side Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
1	17	16	0.95	Rocking controlled
2	27	28	1.02	Rocking controlled
3	33	40	1.21	Rocking controlled
4	27	28	1.02	Rocking controlled
5	32	40	1.25	Rocking controlled
6	15	16	1.01	Rocking controlled

A summary of the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for the deformation-controlled actions of each wall pier of the first story side wall is shown in Table 13-16.

Table 13-16 First Story Side Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
7	42	28	0.67	Rocking controlled
8	69	50	0.73	Rocking controlled
9	84	71	0.85	Rocking controlled
10	70	50	0.71	Rocking controlled
11	82	71	0.87	Rocking controlled
12	40	28	0.71	Rocking controlled

A summary of the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for the deformation-controlled actions of each wall pier of the second story front wall is shown in Table 13-17.

Table 13-17 Second Story Front Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
22	14	28	1.93	Rocking controlled
23	58	112	1.93	Rocking controlled
24	14	28	1.93	Rocking controlled

A summary of the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for the deformation-controlled actions of each wall pier of the first story front wall is shown in Table 13-18.

Table 13-18 First Story Front Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
25	29	75	2.59	Rocking controlled
26	38	75	1.98	Rocking controlled
27	38	75	1.97	Rocking controlled
28	29	75	2.54	Rocking controlled

A summary of the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for the deformation-controlled actions of each wall pier of the second story rear wall is shown in Table 13-19.

Table 13-19 Second Story Rear Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
13	34	61	1.80	Rocking controlled
14	13	23	1.71	Rocking controlled
15	13	23	1.71	Rocking controlled
16	34	61	1.80	Rocking controlled

A summary of the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for the deformation-controlled actions of each wall pier of the first story rear wall is shown in Table 13-20.

Table 13-20 First Story Rear Wall Acceptance Ratio

Pier	$m\kappa Q_{CE}$ $m\kappa V_r$ kips	Q_{UD} kips	$Q_{UD}/m\kappa Q_{CE}$	Status
17	7	5	0.65	Rocking controlled
18	34	59	1.72	Rocking controlled
19	58	171	2.95	Rocking controlled
20	34	59	1.72	Rocking controlled
21	7	5	0.65	Rocking controlled

13.6.3 Procedure Considering the Alternative Pier Height Effect and Ignoring the Effect of Wall Flanges

The following examples illustrate a simplified approach to in-plane shear wall capacity by ignoring the effects of wall flanges. The effective height of the resultant seismic force used in the Tier 3 calculation for rocking depends on direction of loading based on ASCE 41-13 § C11.3.2.2. The commentary states that, "the effective height to be used should be the height over which a diagonal compression strut is most likely to develop in the wall pier at the steepest possible angle that provided lateral resistance" and provides Figure C11-3 (shown below as Figure 13-6). This figure comes from research by Moon (2004), but the source document provides relatively little additional specific guidance on how to determine the effective height.

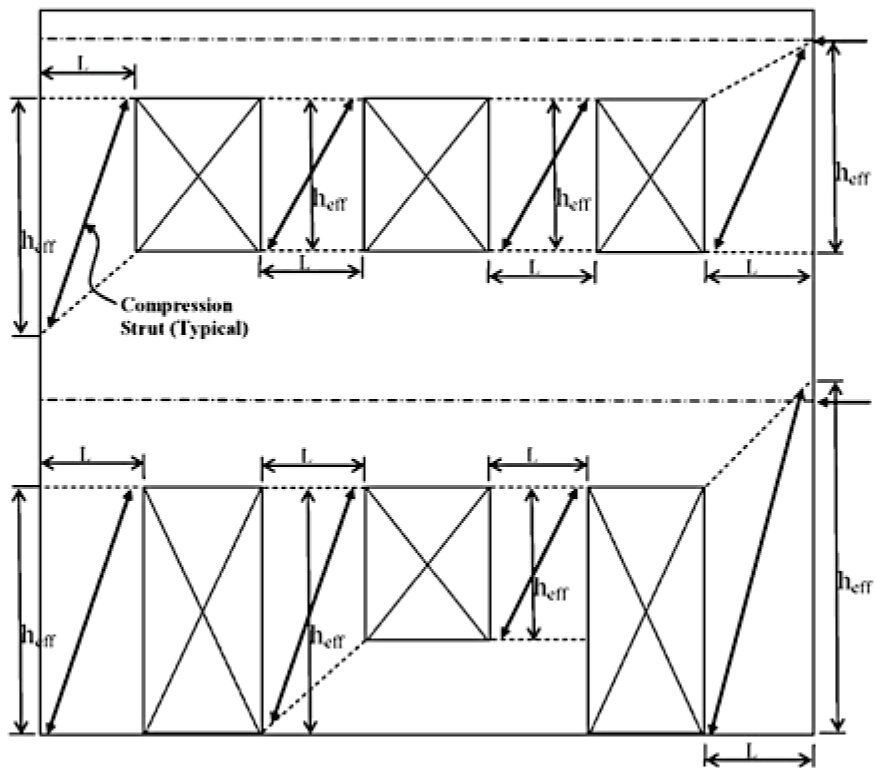


Figure 13-6 Wall pier effective height depending on direction of loading (ASCE 41-13 Figure C11-3). Printed with permission from ASCE.

ASCE 41-13 § C11.3.2.2.1 does mention "the angles at pier hinges generally depend on bed and head joint dimensions and stair-step cracking along mortar joints." For typical brick walls comprised of standard brick units in a typical running bond layout, the angle would be shallower than 45 degrees from horizontal based on the fact that the geometry of the brick as the brick unit height and associated head joint height are typically a bit less than the horizontal distance along bed joints between adjacent head joints.

The commentary in ASCE 41-17 § C11.3.2.2 provides additional discussion, a modified version of the ASCE 41-13 figure (ASCE 41-17 Figure C11-5a), and an alternative figure adapted from Dolce (1989) shown in ASCE 41-17 Figure C11-5b. ASCE 41-17 Figure C11-5a schematically shows the stair stepping described above. ASCE 41-17 Figure C11-5b simplifies the situation with the assumption of a pier height based on using an angle of 30 degrees from horizontal in either direction of loading, resulting in a lower effective pier height.

For the purposes of this example, the steepest possible angle that provides lateral resistance is assumed to be a 45 degree angle until either the edge of wall is reached horizontally or the diaphragm is reached vertically. Neither ASCE 41-13 nor ASCE 41-17 discusses any limitations regarding the diaphragm. Theoretically, though, the lateral forces are traveling through the diaphragm in the walls. Therefore, the application of the lateral force, and in turn the effective height, should not extend above or below the diaphragm levels.

13.6.3.1 Determination of Capacity

The effective height of each pier is dependent on the direction of loading. When the force is acting in one direction, the effective height extends upward into the spandrel above, and in the other direction the effective height extends downward into the spandrel below. For the purposes of this example, the direction of loading is from south to north and the east side wall is analyzed in depth. See Figure 13-7 for the direction of loading.

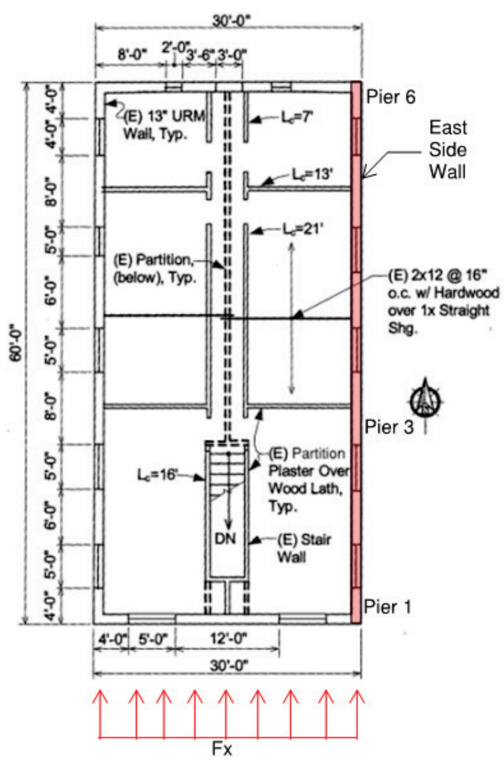


Figure 13-7 Direction of seismic loading and wall of interest, second story plan.

Pier 3 is in a non-end wall; therefore, its effective height does not change with the alternative pier height effect. Because Pier 1 is an end wall, the alternative pier height procedure increases the effective height of the wall pier, altering its rocking capacity and the toe crushing capacity of the pier. See Figure 13-8 for an elevation of the side wall depicting the increase in effective height for Pier 1 and Pier 6.

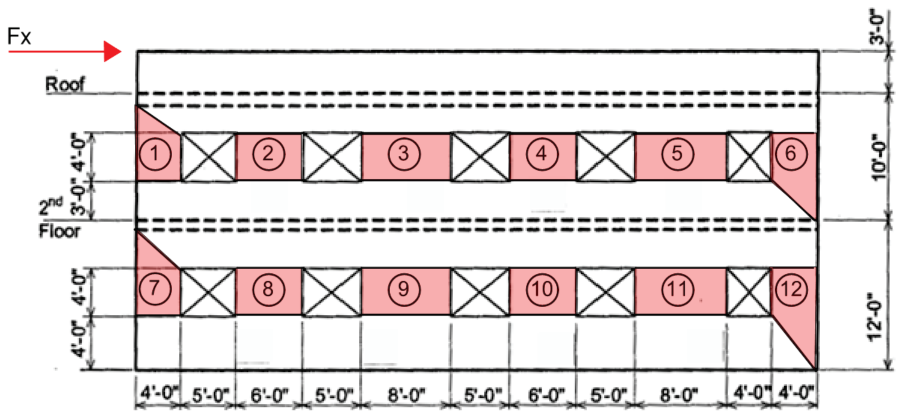


Figure 13-8 Direction of seismic loading and change in effective height of wall piers, east side wall elevation.

In Sections 13.6.3 and 13.6.4 of this *Guide*, only Pier 1 is described in detail for its change in capacity.

13.6.3.1 Example: Pier 1

Figure 13-9 illustrates the direction of loading and how the effective height of Pier 1 increases. Mechanics of materials dictate that for uniaxial tension the maximum shear stress occurs at a 45-degree angle from the line of action of the load. However, at a 45-degree incline, the boundary line for Pier 1 would extend 4 ft vertically, or 1 ft above the diaphragm. Engineering judgment indicates that the seismic forces are transferred through the diaphragm to the vertical lateral force-resisting elements. Therefore, the effective height of the pier should not extend above the diaphragm and is limited to an additional 3 ft for Pier 1. This is not explicitly stated in ASCE 41-13 or ASCE 41-17; however, for this particular example of the *Guide,* it is the assumption used.

Considering how direction of loading changes the effective height of Pier 1, the rocking capacity, V_r, is calculated as:

$$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5P_w)L/h_{\text{eff}} \qquad \text{(ASCE 41-13 Eq. 11-8)}$$

where:

α = 1.0

P_D = 5,100 lb (per Table 13-1 of this *Guide*)

P_w = (120 lb/ft^3)[13 in/(12 in/ft)](4 ft)(4 ft)
= 2,080 lb

h_{eff} = 4 ft + 3 ft
= 7 ft

L = 4 ft (per Figure 13-5)

V_r = 0.9[(1.0)(5,100 lb)+(0.5)(2,080 lb)](4 ft)/(7 ft)
= 3.2 kips

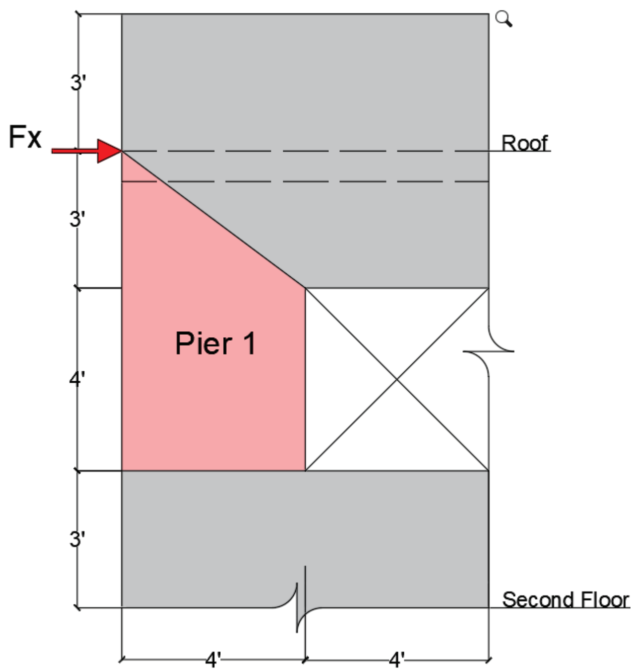


Figure 13-9 Elevation of Pier 1 and the change in effective height.

Note that, for simplicity, the same geometries used previously are assumed in the calculation of P_D and P_w, even though it could be argued that the triangular portion at the top of the pier in Figure 13-9 is now part of the pier (and thus P_w) and no longer part of the P_D from the spandrel above it.

The pier initial bed joint sliding capacity, V_{bjs1}, and the pier final bed joint sliding capacity, V_{bjs2}, do not change with effective height and are the same as calculated in Section 13.6.2.1 of this *Guide*.

Although rocking behavior governs Pier 1 in this example, the force-controlled actions are demonstrated below:

Considering the force-controlled actions for Pier 1 in Figure 13-5 of this *Guide*, the pier toe crushing capacity, V_{tc}, is calculated as:

$$V_{tc} = \left(\alpha P_D + 0.5P_w\right)\left(\frac{L}{h_{\text{eff}}}\right)\left(1 - \frac{f_a}{0.7f'_m}\right) \quad \text{(ASCE 41-13 Eq. 11-11)}$$

where:

f_a = (5,100 lb + 2,080 lb)/(624 in.^{2})
= 11.5 psi

f'_m = 600 psi per above

V_{tc} = [(1.0)(5,100 lb)+(0.5)(2,080 lb)](4 ft/7 ft)[1-(11.5 psi)/[(0.7)(600 psi)]]
= 3.4 kips

The pier diagonal tension capacity, V_{dt}, is calculated as:

$$V_{dt} = f'_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f'_{dt}}} \quad \text{(ASCE 41-13 Eq. 11-12)}$$

where:

$$f'_{dt} = v_{mL} = \frac{0.75\left(0.75v_{tL} + \dfrac{P_D}{A_n}\right)}{1.5} \quad \text{(ASCE 41-13 Eq. 11-6)}$$

where:

v_{tL} = 74 psi (as shown before)

f'_{dt} = (0.75)[(0.75)(74 psi) + (5,100 lb)/(624 in.^{2})]/1.5
= 32 psi

L/h_{eff} = 4 ft/7 ft
= 0.57 < 0.67; thus, β = 0.67

$$V_{dt} = (32 \text{ psi})(624 \text{ in.}^2)(0.67)\sqrt{1 + (11.5 \text{ psi})/(32 \text{ psi})}$$
= 15.5 kips

The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of m-factors for ductility. For Pier 1:

V_r = 3.2 kips

V_{bjs1} = 24.9 kips

V_{tc} = 3.4 kips

V_{dt} = 15.5 kips

V_r is the lowest shear capacity and is the governing failure mode for Pier 1. Therefore, rocking still governs the behavior. The following equation must be satisfied:

$m\kappa Q_{CE} > Q_{UD}$ (ASCE 41-13 Eq. 7-36)

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level the equation for *m*-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 3.2 kips

m = 3.75

$m\kappa Q_{CE}$ = (3.75)(1.0)(3.2 kips)
= 11.8 kips

13.6.3.2 Determination of Demand

The effective height of the resultant seismic force used in the Tier 3 calculation for rocking depends on direction of loading based on ASCE 41-13 § C11.3.2.2. The direction of loading is from south to north per Figure 13-7. For linear procedures, the shear force assigned to each pier is based on its relative rigidity. The change in effective height will alter the stiffness of the walls and thus the distribution of the lateral shear forces accordingly. The lateral in-plane stiffness of solid shear walls is calculated by using the following equation:

Example: Pier 1

For wall piers fully restrained:

$$k = \frac{1}{\dfrac{h_{\text{eff}}}{12E_m I_g} + \dfrac{h_{\text{eff}}}{A_v G_m}}$$ (ASCE 41-13 Eq. C11-2)

where:

E_m = 546,000 psi

G_m = 218,400 psi

h_{eff} = 7 ft

k = 1/[(84 in.)^3/[(12)(546,000 psi)(13 in.)(48 in.)^3/(12)]+84 in/[(13 in.)(48 in.)(218,400 psi)]
= 729 kip/in.

Example calculation of shear demand for Pier 1 in Figure 13-5:

$$V_p = \frac{k}{\Sigma k}(F_x)$$

= [(729 kip/in.)/(19,015 kip/in.)](167 kips)
= 6.4 kips

A summary of the stiffness of each wall pier of the second story side walls and its respective shear force is shown in Table 13-21.

Table 13-21 Second Story Side Wall Pier Stiffnesses and Shears

Pier	$k$kips/in.	$k/\sum k$	V_p kips
1	729	0.04	6.4
2	3,616	0.19	31.8
3	5,162	0.27	45.5
4	3,616	0.19	31.8
5	5,162	0.27	45.5
6	729	0.04	6.4
∑	19,015	1.00	167.5

The reduction in stiffness of Pier 1 and Pier 6 due to the change in effective height decreases the demands on the end wall piers significantly. In turn, the relative stiffness of the piers that do not have a change in height increases along with the shear demand. The capacity of Pier 3 does not change; however, the demand increases significantly. While the ratio of $Q_{UD}/m\kappa Q_{CE}$ for Piers 1 and 6 may decrease, it is important to note the ratio of $Q_{UD}/m\kappa Q_{CE}$ of Pier 3 and other non-end wall piers will increase.

The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of m-factors for ductility. For Pier 1:

V_r = 3.2 kips

V_{bjs1} = 24.9 kips

V_{tc} = 3.4 kips

V_{dt} = 15.5 kips

V_r is the lowest shear capacity and is the governing failure mode for Pier 1. Therefore, rocking still governs the behavior. The following equation must be satisfied:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level the equation for m-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 3.2 kips

m = 3.75

$m\kappa Q_{CE}$ = 11.8 kips

Q_{UD} = 6.4 kips

$$Q_{UD}/m\kappa Q_{CE} = (6.4 \text{ kips})/(11.8 \text{ kips})$$
$$= 0.54$$

Therefore, there is sufficient rocking capacity for Pier 1. With the alternative pier height effect, the acceptance ratio of $Q_{UD}/m\kappa Q_{CE}$ for Pier 1 decreases from 0.95 to 0.54 (as compared to Table 13-15), though this cannot be attributed to an increase in capacity. Rather, it is the decrease in relative stiffness that lowers the demand on the pier. The piers in the same wall line that do not have a change in effective height attract more load, and thus have higher acceptance ratios of $Q_{UD}/m\kappa Q_{CE}$.

See Table 13-22 for the change in stiffness and the change in demand for the wall piers of the second story side wall.

Table 13-22 Change in Second Story Side Wall Pier Stiffnesses and Shears

Pier	k_1 kips/in.	k_2 kips/in.	V_{p1} kips	V_{p2} kips	% Change in Stiffness	% Change in Demand
1	2,600	935	15.7	6.4	-64%	-59%
2	4,636	4,636	28.0	31.8	0%	14%
3	6,618	6,618	40.0	45.5	0%	14%
4	4,636	4,636	28.0	31.8	0%	14%
5	6,618	6,618	40.0	45.5	0%	14%
6	2,600	935	15.7	6.4	-64%	-59%
$\sum$	27,708	24,378	167.5	167.5	-12%	0%

Note: k_1 and V_{p1} are the wall pier stiffness and shear demand from Section 13.6.2.
k_2 and V_{p2} are the wall pier stiffness and shear demand from Section 13.6.3.

See Table 13-23 for the final $Q_{UD}/m\kappa Q_{CE}$ acceptance ratios for the second story side wall.

Table 13-23 Change in Second Story Side Wall Acceptance Ratios

Pier	Governing $Q_{UD}/m\kappa Q_{CE1}$	Governing $Q_{UD}/m\kappa Q_{CE2}$	% Change in $Q_{UD}/m\kappa Q_{CE}$
1	0.95	0.54	-43%
2	1.02	1.16	14%
3	1.21	1.37	14%
4	1.02	1.16	14%
5	1.25	1.42	14%
6	1.01	0.58	-43%

Note: $Q_{UD}/m\kappa Q_{CE1}$ is from Section 13.6.2; $Q_{UD}/m\kappa Q_{CE2}$ is from Section 13.6.3.

Although the $Q_{UD}/m\kappa Q_{CE}$ acceptance ratios for Pier 1 and Pier 6 decrease by 43%, the acceptance ratios of $Q_{UD}/m\kappa Q_{CE}$ for the non-end walls all increase by 14%.

13.6.4 Procedure Considering the Alternative Pier Height Effect and Wall Flanges

ASCE 41-13 § C11.3.2.2 indicates that engineers should include flanged returns when determining wall pier shear capacities. The commentary states that one commonly used approach to model wall flanges is to assume the length of flanges acting in compression is equal to six times the thickness of the in-plane walls or the actual length of flanges, whichever is less. Flanged walls can have a significantly higher rocking strength when compared to walls where the flanges are ignored.

13.6.4.1 Determination of Capacity

Example: Pier 1

The direction of loading is from the south to north as illustrated in Figure 13-7. In this direction, the flange for Pier 6 is Pier 13 and the flange for Pier 1 is Pier 24. For Pier 1, the actual length of the flange is 48 inches whereas for Pier 6, the length of flange is limited to six times the thickness, or 6(13 in.) = 78 in. Refer to the plan in Figure 13-3 of this *Guide* for clarification. The change in effective height in the inclusion of the flanges will alter the stiffness of the walls and thus distribute the lateral shear forces accordingly. Figure 13-10 presents the revised section of Pier 1.

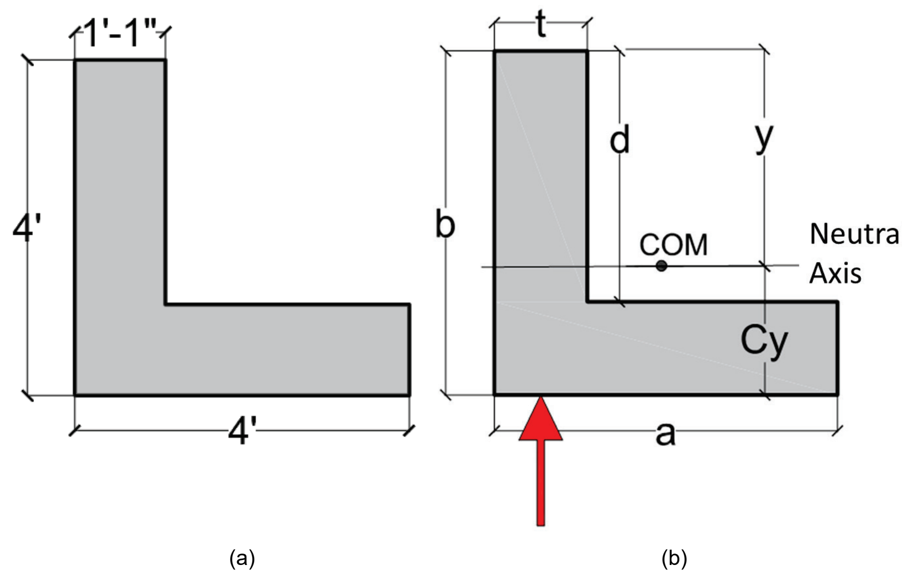


Figure 13-10 Figure showing (a) Pier 1 cross section with flange, (b) Pier 1 cross section with variables for equations.

The change in dead load also affects the unreinforced masonry shear strength. The inclusion of the flange changes both the weight of the pier, P_w, and the dead load above it resisted by the pier, P_D. See Figure 13-11 for a 3D representation of the dead loads tributary to Pier 1.

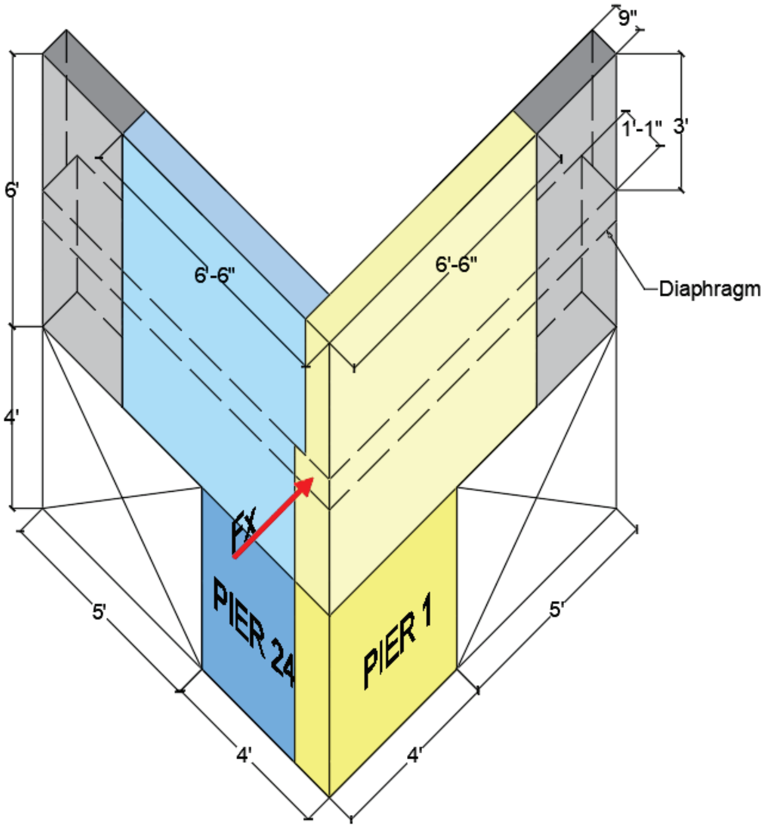


Figure 13-11 Pier 1, its flange (Pier 24), and the dead load tributary to the pier.

In Figure 13-11, Pier 1 is represented in dark yellow, the dead load above tributary to Pier 1 is light yellow, the flange for Pier 1 is dark blue, and the dead load tributary to the wall flange is light blue. Figure 13-11 also illustrates the complexity of each analysis method used in Section 13.6 of this *Guide*. As noted previously, ASCE 41-13 § 11.3.2.2 requires the inclusion of the effects of wall flanges, spandrels, and vertical components of seismic loading when calculating the strength of URM walls subject to in-plane actions using the linear static procedure. The ASCE 41-13 § C11.3.2.2 commentary notes that the effect of global overturning to impart vertical components of seismic loading on the piers depends in turn on the effectiveness of the spandrels in transmitting shear and bending. However, no provisions are provided in ASCE 41-13 for assessing the capacity of the spandrels. This effect is addressed in more detail in ASCE 41-17. For simplicity, global overturning effects are not considered in this design example, but flange effects are considered.

The initial analysis provided in Section 13.6.2 of this *Guide* ignores spandrels and wall flanges and is represented diagrammatically with the dark yellow in Figure 13-11. This is the same area used for the ASCE 41-13 § 15.2 Special Procedure. Section 13.6.3 of this *Guide* included the alternative pier height effect and the extent of the pier considered is all the yellow areas in Figure 13-11. Finally, this section includes wall flanges, and the pier considered is all of the yellow and blue areas in Figure 13-11. Ultimately, ASCE 41-13 § 11.3.2.2 requires the designer to consider the entire pier in Figure 13-11 with all its yellow and blue components for a complete Tier 3 linear static analysis.

Conservatively, the only area considered to provide shear resistance is the wall pier itself (the "web" portion), not the flange. However, the flange does affect the moment of inertia, and thus the stiffness of the piers. Additionally, the dead load includes the dead from both the pier and the flange and the area over which it acts is the area of the pier and the flange. Therefore, the area that provides shear resistance is not the same as the area that the dead load acts on.

$$v_{me} = \frac{0.75\left(0.75v_{te} + \dfrac{P_D}{A_n}\right)}{1.5} \qquad \text{(ASCE 41-13 Eq. 11-2)}$$

where:

P_D = the dead load from Pier 1 and the dead load from Pier 24
= 5,100 lb + 4,300 lb (from Table 13-1 of this *Guide*)
= 9,400 lb

$A_{n,w/f}$ = the area that provides dead load resistance with the flange included
= (48 in.)(13 in.) + (35 in.)(13 in.)
= 1,079 in.^{2}

v_{te} = 95 lb/in.^{2}

v_{me} = (0.75 [0.75(95 lb/in.^{2}) + (9,400 lb)/(1,079 in.^{2})])/1.5
= 40.0 lb/in.^{2}

Commentary

ASCE 41-13 requires the inclusion of flanges; however, it does not distinguish between the area that resists dead loads and the area that provides shear resistance. It is important to be careful when applying the respective areas. In order to distinguish between the two, here $A_{n,w/f}$ (i.e., with flange) is used to describe the area that provides dead load resistance and A_n is used for the area that provides shear resistance.

Considering how changes to the effective height based on the alternative pier height effect and flanges alter the capacity of Pier 1, V_r, is calculated as follows. The small increase in h_{eff} when the effective pier base is lower is not included in the wall weight.

$Q_{CE} = V_r = 0.9(\alpha P_D + 0.5P_w)L/h_{\text{eff}}$ (ASCE 41-13 Eq. 11-8)

where:

α = 1.0

P_D = 9,400 lb

P_w = (120 lb/ft^3)(4 ft high)[1,079 in.^{2}/(12 in/ft)^2]
= 3,600 lb

h_{eff} = 4 ft + 3 ft
= 7 ft

L = 4 ft (per Figure 13-5)

V_r = 0.9[(1.0)(9,400 lb)+(0.5)(3,600 lb)](4 ft)/(7 ft)
= 5.8 kips

The pier initial bed joint sliding capacity, V_{bjs1}, is calculated as follows. Any contribution of the flange to the in-plane shear area, A_n, is conservatively ignored. Additionally, since the tributary weight has increased, thus altering v_{me}, the expected unreinforced masonry strength must be recalculated as well.

$V_{bjs1} = v_{me}A_n$ (ASCE 41-13 Eq. 11-9)

A_n = Area that provides shear resistance
= (48 in.)(13 in.)
= 624 in.^{2}

v_{me} = (0.75 [0.75(95 lb/in.^{2}) + (9,400 lb)/(624 in.^{2})])/1.5
= 43.3 lb/in.^{2}

V_{bjs1} = (43.3 lb/in.^{2})(624 in.^{2})
= 27.0 kips

Therefore, rocking failure controls the deformation controlled actions.

Considering the force-controlled actions or Pier 1 in Figure 13-5, the pier toe crushing capacity, V_{tc}, is calculated as:

$$V_{tc} = \left(\alpha P_D + 0.5P_w\right)\left(\frac{L}{h_{\text{eff}}}\right)\left(1 - \frac{f_a}{f'_m}\right) \quad \text{(ASCE 41-13 Eq. 11-11)}$$

f_a = (9,400 lb + 3,600 lb)/(1,079 in.^{2})
= 12 psi

f'_m = 600 psi per above

V_{tc} = [(1.0)(9,400 lb)+(0.5)(3,600 lb)](4 ft/7 ft)[1 – (12 psi)/[(0.7)(600 psi)]]
= 6.2 kips

The pier diagonal tension capacity, V_{dt}, is calculated as:

$$V_{dt} = f'_{dt} A_n \beta \sqrt{1 + \frac{f_a}{f'_{dt}}} \qquad \text{(ASCE 41-13 Eq. 11-12)}$$

where:

$$f'_{dt} = v_{mL} = \frac{0.75\left(0.75 v_{te} + \frac{P_D}{A_n}\right)}{1.5} \qquad \text{(ASCE 41-13 Eq. 11-6)}$$

where:

v_{tL} = 74 psi from before

f'_{dt} = 0.75[(0.75)(74 psi) + (9,400 lb)/(1,079 in.^{2})]/1.5
= 32.1 psi

L/h_{eff} = 4 ft/7 ft
= 0.57 < 0.67; thus, β = 0.67

$$V_{dt} = (32.1 \text{ psi})(624 \text{ in.}^2)(0.67)\sqrt{1 + (12 \text{ psi})/(32.1 \text{ psi})}$$
= 15.7 kips

Therefore, rocking still governs the behavior of Pier 1. The following equation must be satisfied:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level the equation for *m*-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 5.8 kips

m = 3.75

$m\kappa Q_{CE}$ = (3.75)(1.0)(5.8 kips)
= 21.6 kips

13.6.4.2 Determination of Demand

Example: Pier 1

Using Figure 13-10, for wall piers with a flange, the neutral axis and moment of inertia are calculated as follows:

Neutral axis: $C_y = \dfrac{t(2d+a)+d^2}{2(d+a)}$

Moment of Inertia, $I = \dfrac{1}{3}\left[ty^3 + a(b-y)^3 - (a-t)(b-y-t)^3\right]$

Note: $y = b - C_y$

where:

b = the length of pier (in.)

For Pier 1:

C_y = [(13)[(2)(35)+(48)] + (35)^2]/[2(35 + 48)]
= 16.6 in.

y = 48 in. – 16.6 in.
= 31.4 in.

I = (1/3)[(13)(31.4)^3 + (48)(48 – 31.4)^3 – (48 – 13)(48 – 31.4 – 13)^3]
= 206,800 in.^{4}

For wall piers fully restrained:

$$k = \frac{1}{\dfrac{h_{\text{eff}}^3}{12E_m I_g} + \dfrac{h_{\text{eff}}}{AvG_m}}$$

where:

E_m = 546,000 psi

G_m = 218,400 psi

h_{eff} = 7 ft

I_g = 206,800 in.^{4}

A_v = 624 in.^{2}

k = 1/[(84 in.)^3/[(12)(546,000 psi)(206,800 in.^{3}]+48 in./[(624 in.^{2})(218,400 psi)]])]
= 949 kip/in.

Example calculation of shear demand for Pier 1 in Figure 13-5:

$V_p = \dfrac{k}{\Sigma k}(F_x)$
= [(949 kip/in.)/(19,515 kip/in.)](167 kips)
= 8.1 kips

A summary of the stiffness of each wall pier of the second story side walls and its respective shear force is shown in Table 13-24.

Table 13-24 Second Story Side Wall Pier Stiffnesses and Shears

Pier	k kips/in.	$k/\sum k$	V_p kips
1	949	0.05	8.1
2	3,616	0.19	31.0
3	5,162	0.26	44.3
4	3,616	0.19	31.0
5	5,162	0.26	44.3
6	1,009	0.05	8.7
$\sum$	19,515		167.5

The increase in stiffness of Pier 1 and Pier 6 due to the inclusion of the wall flanges increases the demands on the wall piers significantly. In turn, the relative stiffness of the piers that do not change due to the flange and height effects decreases along with the shear demand.

The governing behavior is the lowest shear capacity value out of the four modes, prior to the application of m-factors for ductility. For Pier 1:

V_r = 5.8 kips

V_{bjs1} = 27.0 kips

V_{tc} = 6.2 kips

V_{dt} = 15.7 kips

Therefore, rocking still governs the behavior of Pier 1. The following equation must be satisfied:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

Per ASCE 41-13 Table 11-3, at a Life Safety Structural Performance Level the equation for m-factors for URM in-plane walls and wall piers with a rocking behavioral mode is as follows:

V_r = 5.8 kips

m = 3.75

$m\kappa Q_{CE}$ = 21.6 kips

Q_{UD} = 8.1 kips

$Q_{UD}/m\kappa Q_{CE}$ = (8.1 kips)/(21.6 kips)
= 0.38

Therefore, there is sufficient rocking capacity for Pier 1. With the alternative pier height effect and wall flanges, the $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio for Pier 1 decreases from 0.95 to 0.38.

See Table 13-25 for the change in stiffness and the change in demand for the wall piers of the second story east side wall. It is worth noting that while the effect of including the flange increases the moment of inertia, the increase in stiffness is offset by an increase in effective height associated with the "alternate effective height formulation." The net effect is a reduction in stiffness when compared to the stiffness associated with the clear height, and ignoring the flange. See Table 13-26 for the final acceptance ratios of $Q_{UD}/m\kappa Q_{CE}$ for the second story side wall.

Table 13-25 Change in Second Story East Side Wall Pier Stiffnesses and Shears

Pier	k_1 kips/in.	k_3 kips/in.	V_{p1} kips	V_{p3} kips	% Change in Stiffness	% Change in Demand
1	2,028 →	949	15.7 →	8.1	-53%	-48%
2	3,616	3,616	28.0	31.0	0%	11%
3	5,162	5,162	40.0	44.3	0%	11%
4	3,616	3,616	28.0	31.0	0%	11%
5	5,162	5,162	40.0	44.3	0%	11%
6	2,028	1,009	15.7	8.7	-50%	-45%
∑	21,612	19,515	167.5	167.5	-10%	0%

Note: k_1 and V_{p1} are the wall pier stiffness and shear demand from Section 13.6.2
k_3 and V_{p3} are the wall pier stiffness and shear demand from Section 13.6.4

Table 13-26 Change in Second Story East Side Wall Acceptance Ratios

Pier	Governing $Q_{UD}/m\kappa Q_{CE1}$	Governing $Q_{UD}/m\kappa Q_{CE13}$	% Change in $Q_{UD}/m\kappa Q_{CE}$
1	0.95	0.38	-60%
2	1.02	1.13	11%
3	1.21	1.34	11%
4	1.02	1.13	11%
5	1.25	1.38	11%
6	1.01	0.33	-68%

Note: $Q_{UD}/m\kappa Q_{CE1}$ is from Section 13.6.2; $Q_{UD}/m\kappa Q_{CE3}$ is from Section 13.6.4

Although the DCRs for Pier 1 and Pier 6 decrease by 60%-68%, the $Q_{UD}/m\kappa Q_{CE}$ acceptance ratios for the non-end wall piers all increase by 11%. The most literal interpretation of ASCE 41-13 would be to use the highest individual $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio as the summary value (1.38 in this case) as the governing value. Another approach for deformation-controlled wall lines, however, would be to sum the capacities for all the piers and divide by the full shear demand on that wall line, as is done in the Special Procedure. This is shown in the next section.

13.6.5 Comparison of 13.6.2, 13.6.3, 13.6.4 and the Special Procedure

Analysis of Pier 1 is performed in three ways in this section, as well as in Chapter 12 of this *Guide* using the Special Procedure. A comparison of the DCRs for the governing failure mode using the methods described can be seen in Table 13-27.

Table 13-27 Pier 1 Comparison

Failure Mechanism	DCR			
	Special Procedure	Section 13.6.2	Section 13.6.3 [1]	Section 13.6.4 [2]
Rocking	0.38	0.95	0.54	0.37

[1] Alternative pier height

[2] Alternative pier height and flanges

Table 13-27 illustrates that the Special Procedure is not always the most conservative method for analyzing URM shear wall buildings with flexible diaphragms, at least when looking at an individual pier. In fact, depending on how fine the initial assumptions and the specific geometry, the results could be significantly different. For example, when ignoring the effects of flanges and the alternative pier height in the Tier 3 analysis, Pier 1 just barely has sufficient capacity to resist lateral loads. However, a more refined analysis that takes the flanges and the alternative pier height into account yields a $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio of 0.37.

The wall line, though, has other piers, and the decreased demand on Wall Pier 1 when considering spandrels and flanges also increases the demand on the other walls. See Table 13-28 for a comparison of the $Q_{UD}/m\kappa Q_{CE}$ acceptance ratios of all the piers in the wall line for all three analyses performed in Section 13.6 of this *Guide* and the Special Procedure performed in Chapter 12 of this *Guide*.

The Special Procedure requires the sum of the deformation-controlled capacities for all the piers to be divided by the full shear demand, resulting in a $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio of 0.38 for the second story side wall. This approach is shown in the last row of Table 13-28 for each of the three sets of Tier 3 pier assumptions. The increase in pier height for the Section 13.6.3 of this *Guide* approach reduces the rocking capacities of the end piers and leads to an overall reduction in wall line rocking capacity and thus a small increase in the wall line $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio. When the added weight from the flanges is included in Section 13.6.4, the overall wall line rocking capacity rises but the wall line $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio is reduced below the initial simpler assumptions. Using this approach and depending on the depth of analysis, an individual pier may have a $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio greater than 1, but the whole wall line can have sufficient capacity.

Table 13-28 Second Story East Side Wall Piers Comparison

Pier	Failure Mechanism	$Q_{UD}/m\kappa Q_{CE}$ ratios			
		Special Procedure	Section 13.6.2	Section 13.6.3 [(1)]	Section 13.6.4 [(2)]
1	Rocking	0.38	0.95	0.54	0.38
2	Rocking	0.38	1.02	1.16	1.13
3	Rocking	0.38	1.21	1.37	1.34
4	Rocking	0.38	1.02	1.16	1.13
5	Rocking	0.38	1.25	1.42	1.38
6	Rocking	0.38	1.01	0.58	0.33
Wall Line	Rocking	0.38	1.10	1.17	1.00

[(1)] Alternative pier height
[(2)] Alternative pier height and flanges

It is apparent that, for this example, the Tier 3 results are significantly more conservative than those of the Special Procedure. It is important to note that the Special Procedure has a reduced Performance Objective that is below the BPOE so it should be expected to have a lower $Q_{UD}/m\kappa Q_{CE}$ acceptance ratio than the Tier 3 analysis. However, for at least this particular design example and wall line, the differences are perhaps more significant than engineering judgment would expect.

13.6.6 LSP Limitations per ASCE 41-13 § 7.3.1.1

ASCE 41-13 § 7.3.1.1 states, "If a component DCR exceeds the lesser of 3.0 and the *m*-factor for the component action and any irregularity described in Section 7.3.1.1.3 or Section 7.3.1.1.4 is present, then linear procedures are not applicable and shall not be used." Note that "DCR" here is defined by ASCE 41-13 Equation 7-16 as Q_{UD}/Q_{CE} and thus does not include the *m*-factor or knowledge factor, κ. It is a measure of implied ductility.

Per ASCE 41-13 § 7.3.1.1:

$$DCR = Q_{UD}/Q_{CE} \qquad \text{(ASCE 41-13 Eq. 7-16)}$$

In the North-South Direction:

The maximum DCR in the north-south direction is Pier 6:

$$
\begin{aligned}
\text{DCR} &= (16\text{ kips})/(5.2\text{ kips}) \\
&= 3.04
\end{aligned}
$$

The *m*-factor is calculated as follows:

$$
\begin{aligned}
m &= 1.5 \le 3h_{\text{eff}}/L \le 3.75 && \text{(ASCE 41-13 Table 11-3)} \\
&= 3(4\text{ ft})/(4\text{ ft}) \\
&= 3.0
\end{aligned}
$$

Therefore, $m = 3.0 < \text{DCR} = 3.04$. No Good, but quite close.

In the East-West Direction:

The maximum DCR in the east-west direction is Pier 25:

$$
\begin{aligned}
\text{DCR} &= (75\text{ kips})/(8\text{ kips}) \\
&= 9.72
\end{aligned}
$$

The *m*-factor is calculated as follows:

$$
\begin{aligned}
m &= 1.5 \le 3h_{\text{eff}}/L \le 3.75 && \text{(ASCE 41-13 Table 11-3)} \\
&= 3(7\text{ ft})/(4\text{ ft}) \\
&= 5.25
\end{aligned}
$$

Therefore, $m = 3.75 < \text{DCR} = 9.72$. No Good.

ASCE 41-13 § 7.3.1.1.3 describes a weak story irregularity that exists if the ratio of the average shear DCR for elements in any story to that of an adjacent story in the same direction exceeds 125%. The average DCR is calculated as follows:

$$
\overline{\text{DCR}} = \frac{\sum_{1}^{n} \text{DCR}_i V_i}{\sum_{1}^{n} V_i} \qquad \text{(ASCE 41-13 Eq. 7-17)}
$$

Because both directions exhibit a component DCR that exceeds the lesser of 3.0 and the *m*-factor for the component action, the weak story irregularity constraint must be evaluated.

For the east-west direction, see Table 13-29 for a summary of the average DCRs for the second story walls:

Table 13-29 Second Story East-West Walls Average DCR

Pier	Shear kips	Governing DCR	$(DCR_i)(V_i)$	*m* factor
22	27.8	5.79	161.0	3.00
23	111.8	2.89	323.1	1.50
24	27.8	5.79	161.0	3.00
13	60.8	2.71	164.8	3.75
14	23.0	3.67	84.4	3.75
15	23.0	3.67	84.4	3.75
16	60.8	2.71	164.8	3.75
∑	335		1,144	
		Avg. DCR $\Sigma(DCR_i)(V_i)/\Sigma(V_i)$ =	3.41	

See Table 13-30 for a summary of the average DCRs for the first story east-west walls:

Table 13-30 First Story East-West Walls Average DCR

Pier	Shear kips	Governing DCR	$(DCR_i)(V_i)$	*m factor*
25	108.3	9.72	1052.7	1.50
26	40.9	7.43	303.9	2.14
27	40.9	7.43	303.9	2.14
28	108.3	9.53	1,032.1	1.50
17	4.5	2.42	10.9	3.75
18	59.0	6.43	379.4	3.75
19	171.3	5.91	1,012.4	2.00
20	59.0	6.43	379.4	3.75
21	4.5	2.42	10.9	3.75
∑	597		4,486	
		Avg. DCR $\Sigma(DCR_i)(V_i)/\Sigma(V_i)$ =	7.51	

The ratio of the average DCR for the wall piers in the first story to that of the second story is:

$$\overline{DCR} = 7.51/3.41 = 2.20$$
$$= 202\% > 125\%$$

Therefore, a weak story irregularity per ASCE 41-13 § 7.3.1.1.3 does exist in the east-west direction. Because a component DCR exceeds the lesser of 3.0

and the *m*-factor for the component action and the irregularity described in ASCE 41-13 § 7.3.1.1.3 is present, linear procedures are not applicable in this direction. The east-west walls must be strengthened and reevaluated, or a nonlinear procedure must be used to assess this example two-story URM building in the east-west direction per the letter of the ASCE 41-13 provisions. Note that nonlinear evaluations of URM bearing wall buildings are highly unusual in typical engineering retrofit practice. Evaluation by the ASCE 41-13 § 15.2 Special Procedure is a possible alternative to avoid this, though it has a lower Performance Objective.

13.7 Evaluation of Existing Floor and Roof Diaphragms

The wood floor and roof diaphragms of the URM building are evaluated per ASCE 41-13 § 12.5 using the LSP. ASCE 41-13 § 12.5.2 categorizes common wood diaphragms into a number of different types. As described in Section 12.2 of this *Guide*, the roof is constructed with 1× straight sheathing over 2×12 wood joists at 24 inches o.c., and the roof covering is applied directly to the straight sheathing. The second floor is constructed with hardwood flooring and 1× straight sheathing over 2×12 wood joists at 16 inches o.c. For both the roof and second floor, the diaphragms fall into the category of “Single Straight Sheathing,” as defined in ASCE 41-13 § 12.5.2.1.1.

As described in Section 12.2 of this *Guide*, both the roof and second floor diaphragms can be treated as flexible. Traditionally, analysis of flexible diaphragms assumes a uniform distribution of acceleration along the length of the diaphragm. However, ASCE 41-13 Equation C7-1 and Figure C7-1 (reprinted here as Figure 13-12) propose a parabolic distribution of horizontal inertial forces:

$$f_d(x) = \frac{1.5F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right] \qquad \text{(ASCE 41-13 Eq. C7-1)}$$

where:

f_d = Inertial load per foot

F_d = Total inertial load on a flexible diaphragm

x = Distance from the center line of the flexible diaphragm

L_d = Distance between lateral support points for the diaphragm

For the second floor diaphragm of the example building, Figure 13-13 and Figure 13-14 show the distribution of seismic inertial force distribution and shear force diagrams for loads in the east-west and north-south directions.

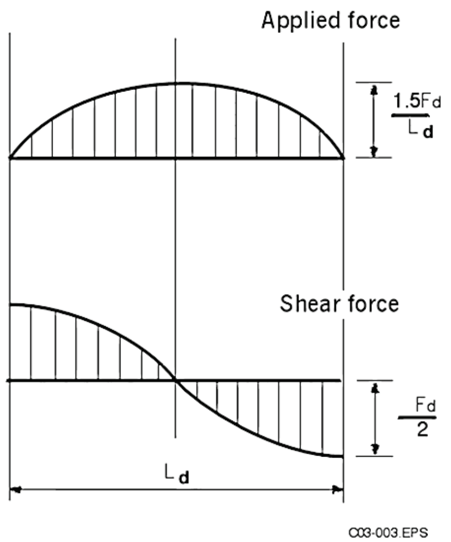


Figure 13-12 Plausible force distribution in a flexible diaphragm (ASCE 41-13 Figure C7-1). Printed with permission from ASCE.

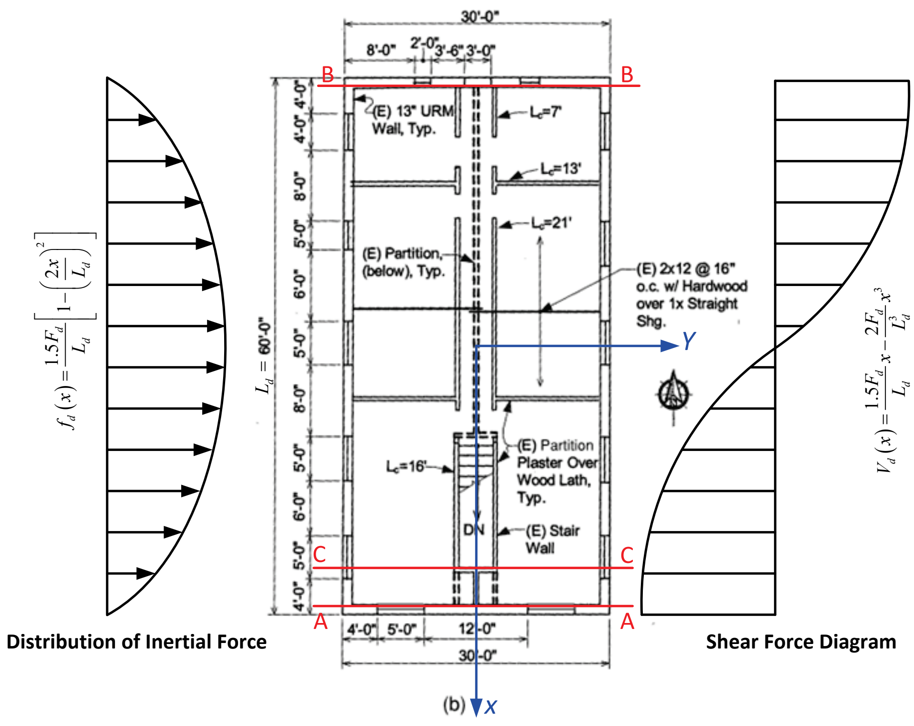


Figure 13-13 Distribution of east-west inertial force and shear force diagram for Level 2 per ASCE 41-13 Figure C7-1.

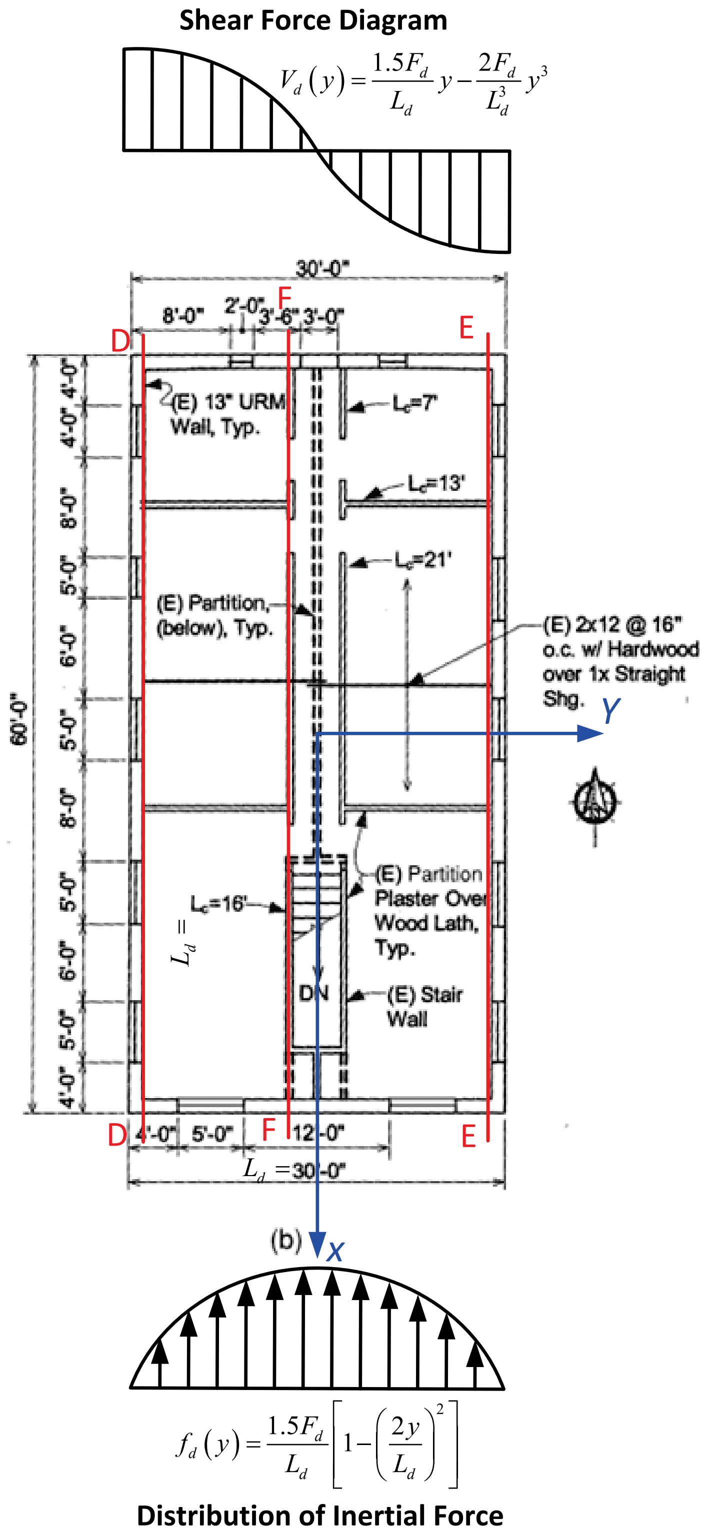


Figure 13-14 Distribution of north-south inertial force and shear force diagram for Level 2 per ASCE 41-13 Figure C7-1.

The formula for the shear force diagram is obtained by taking integration of ASCE 41-13 Equation C7-1 and is shown as follows:

$$V_d(x) = \int f_d\left(x\right)\,\mathrm{d}x = \frac{1.5F_d}{L_d}x - \frac{2F_d}{L_d^3}x^3$$

where F_d, L_d, and x are the same as those used in ASCE 41-13 Equation C7-1.

13.7.1 In-Plane Shear Demand on Wood Diaphragms

Diaphragm forces for the LSP are calculated per ASCE 41-13 § 7.4.1.3.4 and ASCE 41-13 Equation 7-26.

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_x \qquad \text{(ASCE 41-13 Eq. 7-26)}$$

where:

F_{px} = Diaphragm inertia force at level x

F_i = Lateral inertia force applied at level i given by ASCE 41-13 Equation 7-24

w_i = Portion of the effective seismic weight W located on or assigned to floor level i

w_x = Portion of the effective seismic weight W located on or assigned to floor level x

As presented in Section 12.11 of this *Guide*, when the diaphragm is evaluated against seismic loads in the north-south direction, the seismic weight of the two side walls (east and west walls) is neglected. Similarly, the seismic weight of the front and rear walls (north and south walls) is neglected for evaluation against east-west seismic loads. This consideration is reasonable because seismic forces induced by the seismic weight of shear walls in the in-plane loading direction are directly taken by these walls rather than the diaphragm, while seismic forces caused by the weight of shear walls spanning perpendicular to the seismic loading direction are taken by the diaphragm. Using the seismic mass summarized in Table 12-4 of this *Guide* and seismic forces listed in Table 13-2 of this *Guide*, the diaphragm inertia forces are calculated.

For seismic forces in the north-south direction:

$$\begin{cases} F_{p,\text{Roof}}^{(\text{N-S})} = \dfrac{335\text{ k}}{192\text{ k}}\left(192\text{ k} - \underbrace{98\text{ k}}_{\text{Side walls}}\right) = 164.01\text{ k} = 164{,}010\text{ lbs} \\ F_{p,\text{Level 2}}^{(\text{N-S})} = \dfrac{335\text{ k} + 262\text{ k}}{192\text{ k} + 275\text{ k}}\left(275\text{ k} - \underbrace{147\text{ k}}_{\text{Side walls}}\right) = 163.63\text{ k} = 163{,}630\text{ lbs} \end{cases}$$

For seismic forces in the east-west direction:

$$\begin{cases} F_{p,\text{Roof}}^{(\text{E-W})} = \dfrac{335\text{ k}}{192\text{ k}}\left(192\text{ k} - \underbrace{26\text{ k}}_{\text{Rear wall}} - \underbrace{25\text{ k}}_{\text{Front wall}}\right) = 246.02\text{ k} = 246{,}020\text{ lbs} \\ F_{p,\text{Level 2}}^{(\text{E-W})} = \dfrac{335\text{ k} + 262\text{ k}}{192\text{ k} + 275\text{ k}}\left(275\text{ k} - \underbrace{37\text{ k}}_{\text{Rear wall}} - \underbrace{40\text{ k}}_{\text{Front wall}}\right) \\ \qquad = 253.12\text{ k} = 253{,}120\text{ lbs} \end{cases}$$

In the above four equations, the lateral inertia forces, 335 k and 262 k are obtained from Table 13-2 of this *Guide*. The weights of the walls are obtained from Table 12-4 of this *Guide*.

The maximum in-plane diaphragm shear demand occurs at the diaphragm edges. For the east-west seismic case, the maximum diaphragm shear demand occurs at Sections A-A and B-B shown in Figure 13-13. For the north-south seismic cases, the maximum shear demand occurs at Sections D-D and E-E shown in Figure 13-14.

$$\begin{cases} V_{E,\text{Roof}}^{(\text{N-S})} = \dfrac{F_{p,\text{Roof}}^{(\text{N-S})}}{2} = \dfrac{164{,}010\text{ lbs}}{2} = 82{,}005\text{ lbs} \\ V_{E,\text{Level 2}}^{(\text{N-S})} = \dfrac{F_{p,\text{Level 2}}^{(\text{N-S})}}{2} = \dfrac{163{,}630\text{ lbs}}{2} = 81{,}815\text{ lbs} \end{cases}$$

$$\begin{cases} V_{E,\text{Roof}}^{(\text{E-W})} = \dfrac{F_{p,\text{Roof}}^{(\text{E-W})}}{2} = \dfrac{246{,}020\text{ lbs}}{2} = 123{,}010\text{ lbs} \\ V_{E,\text{Level 2}}^{(\text{E-W})} = \dfrac{F_{p,\text{Level 2}}^{(\text{E-W})}}{2} = \dfrac{253{,}120\text{ lbs}}{2} = 126{,}560\text{ lbs} \end{cases}$$

Using the building lengths as the nominal diaphragm lengths, the maximum shear demand distributed to one foot of diaphragm is:

$$\begin{cases} v_{E,\text{Roof}}^{(\text{N-S})} = V_{E,\text{Roof}}^{(\text{N-S})} / 60\text{ ft} = 82{,}005\text{ lbs} / 60\text{ ft} = 1{,}367\text{ lb/ft} \\ v_{E,\text{Roof}}^{(\text{E-W})} = V_{E,\text{Roof}}^{(\text{E-W})} / 30\text{ ft} = 123{,}010\text{ lbs} / 30\text{ ft} = 4{,}100\text{ lb/ft} \end{cases}$$

$$\begin{cases} v_{E,\text{Level 2}}^{(\text{N-S})} = V_{E,\text{Level 2}}^{(\text{N-S})} / 60\text{ ft} = 81{,}815\text{ lbs} / 60\text{ ft} = 1{,}364\text{ lb/ft} \\ v_{E,\text{Level 2}}^{(\text{E-W})} = V_{E,\text{Level 2}}^{(\text{E-W})} / 30\text{ ft} = 126{,}560\text{ lbs} / 30\text{ ft} = 4{,}219\text{ lb/ft} \end{cases}$$

Since gravity load does not cause in-plane shear demand in the roof and floor diaphragms, the gravity term, Q_G, in ASCE 41-13 Equation 7-34 is zeroed out and:

$$\begin{cases} v_{UD,\text{Roof}}^{(\text{N-S})} = v_{E,\text{Roof}}^{(\text{N-S})} = 1{,}367 \text{ lb/ft} \\ v_{UD,\text{Roof}}^{(\text{E-W})} = v_{E,\text{Roof}}^{(\text{E-W})} = 4{,}100 \text{ lb/ft} \end{cases}$$

$$\begin{cases} v_{UD,\text{Level 2}}^{(\text{N-S})} = v_{E,\text{Level 2}}^{(\text{N-S})} = 1{,}364 \text{ lb/ft} \\ v_{UD,\text{Level 2}}^{(\text{E-W})} = v_{E,\text{Level 2}}^{(\text{E-W})} = 4{,}219 \text{ lb/ft} \end{cases}$$

In addition to the check of the diaphragm strength at full-depth cross-section against maximum shear demands, shear demands at the Level 2 diaphragm opening are also calculated. For the east-west seismic case shown in Figure 13-13, Section C-C is critical for the opening, and the shear demand along Section C-C is calculated as:

$$\begin{aligned} & V_{E,\text{Level 2,C-C}}^{(\text{E-W})}\left(x = 30 \text{ ft} - 4 \text{ ft}\right) \\ & = \left.\frac{1.5F_d}{L_d}x - \frac{2F_d}{L_d^3}x^3\right|_{x=30-4 \text{ ft}} \\ & = \frac{1.5\left(253{,}120 \text{ lbs}\right)}{60 \text{ ft}}\left(30-4 \text{ ft}\right) - \frac{2\left(253{,}120 \text{ lbs}\right)}{\left(60 \text{ ft}\right)^3}\left(30-4 \text{ ft}\right)^3 \\ & = 123{,}335 \text{ lbs} \end{aligned}$$

For the north-south seismic case shown in Figure 13-14, Section F-F is critical for the opening, and the shear demand along Section F-F is calculated as:

$$\begin{aligned} & V_{E,\text{Level 2,F-F}}^{(\text{N-S})}\left(y = 4/2 \text{ ft}\right) \\ & = \left.\frac{1.5F_d}{L_d}y - \frac{2F_d}{L_d^3}y^3\right|_{y=4/2 \text{ ft}} \\ & = \frac{1.5\left(163{,}630 \text{ lbs}\right)}{30 \text{ ft}}\left(2 \text{ ft}\right) - \frac{2\left(163{,}630 \text{ lbs}\right)}{\left(30 \text{ ft}\right)^3}\left(2 \text{ ft}\right)^3 \\ & = 16{,}266 \text{ lbs} \end{aligned}$$

13.7.2 In-Plane Shear Capacity of Wood Diaphragms

ASCE 41-13 § 12.5.3.1.2, *Strength of Single Straight Sheathing Diaphragms*, indicates "The expected strength of straight-sheathed diaphragms shall be determined in accordance with Section 12.2.2."

ASCE 41-13 § C12.2.2.1.4 indicates "Actions associated with wood and CFS (cold-formed steel) light-frame components generally are deformation controlled; thus, expected strength material properties are used most often." Accordingly, the in-plane shear capacity of wood diaphragms is calculated using expected material properties. ASCE 41-13 § 12.2.2.5 states that "Use of default properties to determine component strengths shall be permitted in conjunction with the linear analysis procedures of Chapter 7," and "Default expected strength and stiffness values for wood diaphragm assemblies shall be taken from Table 12-2." As listed in ASCE 41-13 Table 12-2, the expected strength Q_{CE} of "Single Straight Sheathing" is 120 lb/ft (in ASCE 41-13 Table 12-2, the "lb/in." following "Expected Strength (Q_{CE})" is a typo, which should be corrected to "lb/ft," and which has been done in ASCE 41-17). ASCE 41-13 Table 12-2 Footnote (b) indicates that "For single straight sheathing, expected strength shall be multiplied by 1.5 where built-up roofing is present. The value for stiffness shall not be changed." Typical built-up roofing consists of laminated bitumen and reinforcing felts that form a roofing membrane. The bitumen is commonly asphalt or coal tar pitch. The reinforcing felts can be glass fiber, organic fibers, or non-woven synthetic fibers. The expected strength of the single straight sheathing roof diaphragm with built-up roofing material is

$$v_{CE,\text{Roof}} = 1.5(120\text{ lb/ft}) = 180\text{ lb/ft}$$

As described in Section 12.2 of this *Guide*, the single straight sheathing diaphragm at second floor has hardwood flooring on its top. However, the contribution of flooring to the diaphragm strength is not included in ASCE 41-13 Table 12-2, although it is included in ASCE 41-13 Table 15-2. ASCE 41-13 Table 12-2 does have a value of 600 lb/ft for double straight sheathing, and this is used as an estimation of the expected strength of single straight sheathing diaphragm with hard wood flooring (Table 12-2 of this *Guide* indicates that there is one-inch hardwood flooring on Level 2).

$$v_{CE,\text{Level2}} = 600\text{ lb/ft}$$

13.7.3 Acceptance of In-Plane Shear of Wood Diaphragms

Per ASCE 41-13 § 7.5.2.2.1, deformation-controlled actions in primary and secondary components shall satisfy the following:

$$m\kappa Q_{CE} > Q_{UD} \qquad \text{(ASCE 41-13 Eq. 7-36)}$$

As introduced in Section 13.3 of this *Guide*, the level of knowledge falls under the category of usual, and the knowledge factor, κ, is equal to 1.0. ASCE 41-13 § 12.5.3.1.3, *Acceptance Criteria for Single Straight Sheathing Diaphragms*, indicates "For linear procedures, m-factors for use with deformation-controlled actions shall be taken from Table 12-3." ASCE 41-13 § C7.2.9 notes that "buildings with solid structural walls on all sides often do not require diaphragm chords." As the roof and floor diaphragms are connected to perimeter masonry walls, the diaphragms are treated as chorded. The aspect ratio of the diaphragms of the URM building is 60 ft/30 ft = 2.0 < 3.0. In ASCE 41-13 Table 12-3, for chorded single straight sheathing with a Length/Width ratio not greater than 3.0, the *m*-factor for primary components and the Life Safety (LS) Performance Objective is 2.0. For the diaphragms, the acceptance ratios of $Q_{UD}/m\kappa Q_{CE}$ are calculated as follows:

$$\begin{cases} \text{Acceptance Ratio}_{\text{Roof}}^{\text{(N-S)}} = \dfrac{v_{UD,\text{Roof}}^{\text{(N-S)}}}{m\kappa v_{CE,\text{Roof}}} = \dfrac{1{,}367\ \text{lb/ft}}{2.0(1.0)(180\ \text{lb/ft})} = 3.8 \\ \text{Acceptance Ratio}_{\text{Roof}}^{\text{(E-W)}} = \dfrac{v_{UD,\text{Roof}}^{\text{(E-W)}}}{m\kappa v_{CE,\text{Roof}}} = \dfrac{4{,}100\ \text{lb/ft}}{2.0(1.0)(180\ \text{lb/ft})} = 11.4 \end{cases}$$

$$\begin{cases} \text{Acceptance Ratio}_{\text{Level 2}}^{\text{(N-S)}} = \dfrac{v_{UD,\text{Level 2}}^{\text{(N-S)}}}{m\kappa v_{CE,\text{Level 2}}} = \dfrac{1{,}364\ \text{lb/ft}}{2.0(1.0)(600\ \text{lb/ft})} = 1.1 \\ \text{Acceptance Ratio}_{\text{Level 2}}^{\text{(E-W)}} = \dfrac{v_{UD,\text{Level 2}}^{\text{(E-W)}}}{m\kappa v_{CE,\text{Level 2}}} = \dfrac{4{,}219\ \text{lb/ft}}{2.0(1.0)(600\ \text{lb/ft})} = 3.5 \end{cases}$$

These acceptance ratios greater than unity indicate that the diaphragms are inadequate at the locations of maximum in-plane shear demands. Additionally, adequacy of the Level 2 diaphragm around the stair opening is checked using demands calculated in Section 13.7.1 of this *Guide* and capacities of net cross sections.

For the east-west seismic case, shear demand along Section C-C is 123,335 lbs, as shown in Section 13.7.1 of this *Guide*. The corresponding shear capacity and acceptance ratio are:

$$m\kappa Q_{CE,\text{Level 2,C-C}} = 2.0(1.0)(600\ \text{lb/ft})(30\ \text{ft} - 4\ \text{ft}) = 31{,}200\ \text{lbs}$$

$$\text{Acceptance Ratio}_{\text{Level 2,E-E}}^{\text{(E-W)}} = \frac{V_{E,\text{Level 2, C-C}}^{\text{(E-W)}}}{m\kappa V_{CE,\text{Level 2,C-C}}} = \frac{123{,}335\ \text{lbs}}{31{,}200\ \text{lbs}} = 4.0$$

For the north-south seismic case, shear demand along Section F-F is 16,266 lbs, as shown in Section 13.7 of this *Guide*. The corresponding shear capacity and acceptance ratio are

$$m\kappa Q_{CE,\text{Level 2,F-F}} = 2.0(1.0)(600\ \text{lb/ft})(60\ \text{ft} - 16\ \text{ft}) = 52{,}800\ \text{lbs}$$

$$\text{Acceptance Ratio}_{\text{Level 2,F-F}}^{\text{(N-S)}} = \frac{V_{E,\text{Level 2,F-F}}^{\text{(N-S)}}}{m\kappa V_{CE,\text{Level 2,F-F}}} = \frac{16{,}266\ \text{lbs}}{52{,}800\ \text{lbs}} = 0.31$$

Therefore, around the stair opening at Level 2, the diaphragm is adequate along the north-south direction but inadequate along the east-west direction.

13.7.4 Retrofit of Wood Diaphragms

To address the inadequate diaphragms on the roof and second floor, vertical lateral force-resisting elements such as shear walls or braced frames could be added to reduce the diaphragm span or the diaphragm could be strengthened. For this design example, the diaphragms are strengthened. The existing single straight sheathing is replaced with wood structural panels. ASCE 41-13 § 12.5.3.6.2 states that "Expected strengths shall be permitted to be based on 1.5 times yield strengths of wood structural panel diaphragms. Yield strengths shall be determined using LRFD procedures contained in AWC SDPWS, except that the resistance factor, ϕ, shall be taken as 1.0 and expected material properties shall be determined in accordance with Section 12.2.2."

Table 4.2A of the SDPWS-2008*, Special Design Provisions for Wind & Seismic* (AWC, 2008) provides nominal unit shear capacities for blocked wood structural panel diaphragms. Assuming 15/32-in. minimum nominal panel thickness, sheathing grade of Structural I, 2 in. minimum nominal width of nailed face at adjoining panel edges and boundaries, 10d nails at 2-1/2" spacing at diaphragm boundaries, continuous panel edges parallel to load, and at all panel edges, 4" spacing at other panel edges, the nominal unit shear capacity, v_s, is 1,280 plf. The expected strength is $v_{CE,\text{retrofit}}$ = 1.5(1,280 plf) = 1,920 plf. As shown in ASCE 41-13 Table 12-3, the m-factor for blocked and chorded wood structural panel with an aspect ratio not greater than 3.0 for primary components and the Life Safety (LS) Performance Level is 3.0. The demand-to-capacity ratios for the new diaphragms are calculated. As shown in Table 12-1 and Table 12-2 of this *Guide*, the wood sheathing takes only about 10% of the total seismic weight of the diaphragm. The 15/32 in. plywood structural panels with blocking have approximately the same weight as the original 1× single straight sheathing, and, thus, the seismic weights and seismic forces of the retrofitted diaphragms remain the same as the original ones.

$$\begin{cases} \text{Acceptance Ratio}_{\text{Roof, retrofit}}^{\text{(N-S)}} = \dfrac{v_{UD,\text{Roof}}^{\text{(N-S)}}}{m\kappa v_{CE,\text{retrofit}}} = \dfrac{1{,}367 \text{ lb/ft}}{3.0(1.0)(1{,}920 \text{ lb/ft})} = 0.24 < 1 \\ \text{Acceptance Ratio}_{\text{Roof, retrofit}}^{\text{(E-W)}} = \dfrac{v_{UD,\text{Roof}}^{\text{(E-W)}}}{m\kappa v_{CE,\text{Roof}}} = \dfrac{4{,}100 \text{ lb/ft}}{3.0(1.0)(1{,}920 \text{ lb/ft})} = 0.71 < 1 \end{cases}$$

$$\begin{cases} \text{Acceptance Ratio}^{(\text{N-S})}_{\text{Level 2, retrofit}} = \dfrac{v^{(\text{N-S})}_{UD,\text{Level 2}}}{m\kappa v_{CE,\text{retrofit}}} = \dfrac{1{,}364 \text{ lb/ft}}{3.0(1.0)(1{,}920 \text{ lb/ft})} = 0.24 < 1 \\ \text{Acceptance Ratio}^{(\text{E-W})}_{\text{Level 2, retrofit}} = \dfrac{v^{(\text{E-W})}_{UD,\text{Level 2}}}{m\kappa v_{CE,\text{retrofit}}} = \dfrac{4{,}219 \text{ lb/ft}}{3.0(1.0)(1{,}920 \text{ lb/ft})} = 0.73 < 1 \end{cases}$$

At the Level 2 stair opening,

$$\begin{aligned} \text{Acceptance Ratio}^{(\text{E-W})}_{\text{Level 2,C-C, retrofit}} &= \frac{V^{(\text{E-W})}_{E,\text{Level 2,C-C}}}{m\kappa V_{CE,\text{Level 2,C-C, retrofit}}} \\ &= \frac{123{,}335 \text{ lbs}}{3.0(1.0)(1{,}920 \text{ lb/ft})(30-4 \text{ ft})} \\ &= 0.82 < 1 \end{aligned}$$

$$\begin{aligned} \text{Acceptance Ratio}^{(\text{N-S})}_{\text{Level 2,F-F, retrofit}} &= \frac{V^{(\text{N-S})}_{E,\text{Level 2,F-F}}}{m\kappa V_{CE,\text{Level 2,F-F, retrofit}}} \\ &= \frac{16{,}266 \text{ lbs}}{3.0(1.0)(1{,}920 \text{ lb/ft})(60-16 \text{ ft})} \\ &= 0.06 < 1 \end{aligned}$$

The selected structural panels are adequate for retrofitting the roof and Level 2 diaphragms.

13.8 Evaluation of Unreinforced Masonry Walls Subject to Out-of-Plane Actions (ASCE 41-13 §11.3.3)

For the Collapse Prevention Structural Performance Level, per ASCE 41-13 § 11.3.3.3, out-of-plane stability is checked per ASCE 41-13 Table 11-5. This table is similar to the ASCE 41-13 Special Procedure Table 15-4. It relates the spectral acceleration, the wall *h*/*t* ratios, and the story. In ASCE 41-13 Table 11-5, for "Walls in top story of multistory building," the permissible *h*/*t* ratio for URM subjected to out-of-plane actions" subjected to S_{X1} = 0.507g > 0.37g is 9. At Story 2,

$(h/t)_{\text{Story2}}$= (120 in./13 in.) = 9.23 > 9

So the walls at Story 2 need be checked for out-of-plane stability at the Collapse Prevention (CP) Structural Performance Level. In ASCE 41-13 Table 11-5, for "First-story wall of multistory building," the permissible *h*/*t* ratio for URM subjected to out-of-plane actions" subjected to S_{X1} = 0.507g > 0.37g is 15. At Story 1,

$(h/t)_{\text{Story1}}$= (144 in./13 in.) = 11.1 < 15

Thus, the walls at Story 1 need not be checked for out-of-plane stability at the CP Structural Performance Level.

As noted at the start of the design example, though, the focus of the example is on the Life Safety Structural Performance Level. For the Life Safety Structural Performance Level, the evaluation is more complicated. ASCE 41-13 § 11.3.3.3 states that "For the Life Safety Structural Performance Level, flexural cracking in URM walls caused by out-of-plane inertial loading shall be permitted, provided that cracked wall segments remain stable during dynamic excitation. Stability shall be checked using analytical time-step integration models considering acceleration time histories at the top and base of a wall panel." This provision appeared in FEMA 273 (FEMA, 1997a) and was carried on to FEMA 356 (FEMA, 2000g) and finally to ASCE 41-13. As noted by Simsir et al. (2004), methods for creating the analytical time-step integration models and conducting the dynamic analysis for checking stability are not specified and are left to the user. This provision has been replaced by a more straightforward and practical method in ASCE 41-17 §11.3.3.3.2, "Life Safety Acceptance Criteria for URM Walls Subject to Out-of-Plane Actions." It is based on a procedure developed by Penner and Elwood (2016), and ASCE 41-17 § C11.3.3.3.2 provides commentary on modification made to the procedure and discusses some of the limitations.

The first step of using this method is to evaluate the diaphragm stiffness using fundamental period of the diaphragm, T_{DIAPH}. ASCE 41-13 Equation 7-20 calculates the fundamental period of unreinforced masonry buildings with single-span flexible diaphragms six stories or fewer high. As indicated by ASCE 41-13 § C7.4.1.2.3, this equation assumes wall deformations to be negligible compared with diaphragm deflections, so the period calculated by ASCE 41-13 Equation 7-20 is also close to the fundamental period of the diaphragm. ASCE 41-06 Equation 3-8 (same as ASCE 41-13 Equation 7-20) is cited by Wilson et al. (2013) as a method to calculate diaphragm period.

$$T = \sqrt{0.078\Delta_d} \qquad \text{(ASCE 41-13 Eq. 7-20)}$$

where Δ_d is the maximum in-plane diaphragm displacement in inches due to a lateral force in the direction under consideration equal to the weight tributary to the diaphragm. Wilson et al. (2013) propose a modified method for calculating natural period of flexible timber diaphragms. As the result of this method, the fundamental period of diaphragms is calculated as:

$$T = \sqrt{0.066\Delta_d}$$

which is close to ASCE 41-13 Eq. 7-20.

Wilson et al. (2013) also provide an equation for calculating Δ_d:

$$\Delta_d = \frac{3mgL}{16GA} = \frac{3mgL}{16G_dB}$$

where mg is the tributary seismic weight of the diaphragm, L is the diaphragm span, G_d is the in-plane shear stiffness of the diaphragm, and B is the diaphragm width. In this example, ASCE 41-13 Equation 7-20 is used to determine the diaphragm period. Substituting Δ_d by ($3mgL/16G_dB$) into ASCE 41-13 Equation 7-20 yields:

$$T = \sqrt{0.078\Delta_d} = \sqrt{0.0146\frac{mgL}{G_d B}}$$

where the unit of length variables is inch.

For the second story, the period of the roof diaphragm is calculated as follows:

$$\begin{cases} T_{\text{Roof}}^{(\text{E-W})} = \sqrt{0.0146\dfrac{(192{,}000\text{ lbs-}51{,}000\text{ lbs})(60\text{ ft})}{(24{,}000\text{ lb/ft})(30\text{ ft})}(12\text{ in./ft})} = 1.43\text{ s} \\ T_{\text{Roof}}^{(\text{N-S})} = \sqrt{0.0146\dfrac{(192{,}000\text{ lbs-}98{,}000\text{ lbs})(30\text{ ft})}{(24{,}000\text{ lb/ft})(60\text{ ft})}(12\text{ in./ft})} = 0.59\text{ s} \end{cases}$$

where the seismic weight 192,000 lbs is obtained from Table 12-4 of this *Guide*, and the shear stiffness 24,000 lb/ft is obtained from ASCE 41-13 Table 12-2 for single straight sheathing.

Since both $T_{\text{Roof}}^{(\text{E-W})}$ and $T_{\text{Roof}}^{(\text{N-S})}$ are greater than 0.5 s, the roof diaphragm is considered flexible for both directions, per ASCE 41-17 §11.3.3.3.2.

Per ASCE 41-17 §11.3.3.3, for the Life Safety Structural Performance Level, a cracked wall shall be considered stable during dynamic excitation if $h/t \leq 8$ or ASCE 41-17 Equation 11-27a is satisfied.

$$\left(\frac{h}{t}\right)_{\text{Story 2}} = \left(\frac{120\text{ in.}}{13\text{ in.}}\right) = 9.23 > 8$$

ASCE 41-17 requires that a cracked wall with an h/t ratio over 8 be further checked against S_{X1} using Equation 11-27a for determining its out-of-plane stability at the Life Safety Performance Level. An h/t ratio of 9.23 > 9 is also not permissible for URM subject to out-of-plane actions at the Collapse Prevention Performance Level per ASCE 41-13 §11.3.3.3 and Table 11-5.

Since $\left(\frac{h}{t}\right)_{\text{Story 2}} > 8$, ASCE 41-17 Equation 11-27a is further checked.

$$S_{X1} \leq C_a C_t C_g C_{pl} S_{a\text{DIAPH}}(1) \qquad \text{(ASCE 41-17 Eq. 11-27a)}$$

where per ASCE 41-17 Equation 11-27b:

$$S_{a\text{DIAPH, Story 2}}(1)=\begin{cases}\dfrac{4}{h/t}=\dfrac{4}{9.23}=0.43 \text{ for stiff diaphragm}\\[2ex] \dfrac{1.8}{\left(h/t\right)^{0.75}}=\dfrac{1.8}{9.23^{0.75}}=0.34 \text{ for flexible diaphragm}\end{cases}$$

Because of the flexible roof diaphragm:

$$S_{a\text{DIAPH, Story 2}}^{(\text{E-W})}(1)=S_{a\text{DIAPH, Story 2}}^{(\text{N-S})}(1)=0.34$$

Per ASCE 41-17 Equation 11-27c:

$$C'^{(\text{E-W})}_{a,\text{Story 2}}=C'^{(\text{N-S})}_{a,\text{Story 2}}=0.2 \text{ for flexible diaphragms.}$$

Using the component weight listed in Table 12-4 of this *Guide*, P_D, "vertical load acting on the wall in lb/ft (not including the self-weight of the wall at the story under consideration," is calculated as follows:

$$\begin{cases}P_{D,\text{Story2, side walls}}=\underbrace{\dfrac{43{,}000 \text{ lbs}}{60 \text{ ft}\times 2}}_{\text{Roof diaphragm}}+\underbrace{90 \text{ psf}\left(3 \text{ ft}\right)}_{\text{Roof parapet}}=628 \text{ lb/ft}\\[3ex] P_{D,\text{Story2, front \& rear walls}}=\underbrace{90 \text{ psf}\left(3 \text{ ft}\right)}_{\text{Roof parapet}}=270 \text{ lb/ft}\end{cases}$$

$$\begin{cases}\begin{aligned}C^{(\text{E-W})}_{a,\text{Story 2,side}}&=1+C'^{(\text{E-W})}_{a,\text{Story 2}}\left(P_{D,\text{Story 2,side}}/685\right)\left(1-\frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 2}}-8\right)\right)\\&=1+0.2\left(\frac{628}{685}\right)\left(1-\frac{1}{12}(9.23-8)\right)\\&=1.16\\C^{(\text{N-S})}_{a,\text{Story 2,front\&rear}}&=1+C'^{(\text{N-W})}_{a,\text{Story 2}}\left(P_{D,\text{Story 2,front\&rear}}/685\right)\left(1-\frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 2}}-8\right)\right)\\&=1+0.2\left(\frac{270}{685}\right)\left(1-\frac{1}{12}(9.23-8)\right)\\&=1.07\end{aligned}\end{cases}$$

$$C_{t,\text{Story 2}}=0.2+\frac{t}{15.7}=0.2+\frac{13}{15.7}=1.028>1.0\Rightarrow C_{t,\text{Story 2}}=1.0$$

$C_{g,\text{Story 2}}$ = 1.0 for walls not on the ground

$C_{pl,\text{Story 2}}$ = 0.9 for the Life Safety Performance Level

For the two side walls of Story 2:

$$\begin{aligned} S_{X1} = 0.507 &> C_{a,\text{Story 2,side}}^{(\text{E-W})} C_{t,\text{Story 2}} C_{g,\text{Story 2}} C_{pl,\text{Story 2}} S_{a\text{DIAPH, Story 2}}^{(\text{E-W})}(1) \\ &> 1.16(1.0)(1.0)(0.9)(0.34) \\ &> 0.36 \end{aligned}$$

For the front and rear walls of Story 2:

$$\begin{aligned} S_{X1} = 0.507 &> C_{a,\text{Story 2,side}}^{(\text{N-S})} C_{t,\text{Story 2}} C_{g,\text{Story 2}} C_{pl,\text{Story 2}} S_{a\text{DIAPH, Story 2}}^{(\text{N-S})}(1) \\ &> 1.07(1.0)(1.0)(0.9)(0.34) \\ &> 0.33 \end{aligned}$$

Therefore, both criteria specified in ASCE 41-17 §11.3.3.3.2 indicate that the URM walls on the second story are not considered stable during dynamic excitation.

For the first story, period of the Level 2 diaphragm is

$$\begin{cases} T_{\text{Level 2}}^{(\text{E-W})} = \sqrt{0.0146 \dfrac{(275{,}000 \text{ lbs-}77{,}000 \text{ lbs})(60 \text{ ft})}{(180{,}000 \text{ lb/ft})(30 \text{ ft})}(12 \text{ in./ft})} = 0.62 \text{ s} \\ T_{\text{Level 2}}^{(\text{N-S})} = \sqrt{0.0146 \dfrac{(275{,}000 \text{ lbs-}147{,}000 \text{ lbs})(30 \text{ ft})}{(180{,}000 \text{ lb/ft})(60 \text{ ft})}(12 \text{ in./ft})} = 0.25 \text{ s} \end{cases}$$

where the seismic weight 275,000 lbs is obtained from Table 12-4 of this *Guide*, the shear stiffness 180,000 lb/ft is obtained from ASCE 41-13 Table 12-2 for a double straight sheathing diaphragm (since single straight sheathing with hardwood flooring is not included in Table 12-2, the stiffness of double straight sheathing is used as an approximation).

Per ASCE 41-17 §11.3.3.3.2, the diaphragm is flexible under seismic load in the east-west direction (0.62 s > 0.5 s), while it is between stiff and flexible under seismic load in the north-south direction (0.2 s < 0.25 s < 0.5 s). Linear interpolation is used to calculate parameters for the semi-rigid diaphragm, per ASCE 41-17 §11.3.3.3.2.

$$\left(\frac{h}{t}\right)_{\text{Story 1}} = \left(\frac{144 \text{ in.}}{13 \text{ in.}}\right) = 11.1 > 8$$

Note that although ASCE 41-17 requires that a cracked wall with an h/t = 11.1 > 8 be further checked against S_{X1} using Equation 11-27a for determining its out-of-plane stability at the Life Safety Performance Level, an h/t ratio of 11.1 < 15 is permissible for a URM wall subject to out-of-plane actions at the Collapse Prevention Performance Level per ASCE 41-13 §11.3.3.3 and Table 11-5.

$$S_{a\text{DIAPH, Story 1}}(1)=\begin{cases}\dfrac{4}{h/t}=\dfrac{4}{11.1}=0.36 \text{ for stiff diaphragm}\\ \dfrac{1.8}{(h/t)^{0.75}}=\dfrac{1.8}{11.1^{0.75}}=0.30 \text{ for flexible diaphragm}\end{cases}$$

$$\begin{cases}S^{(\text{E-W})}_{a\text{DIAPH, Story 1}}(1)=0.30\\ S^{(\text{N-S})}_{a\text{DIAPH, Story 1}}(1)=0.36-(0.36-0.30)\dfrac{0.25\text{ s - }0.2\text{ s}}{0.5\text{ s - }0.2\text{ s}}=0.35\end{cases}$$

$$\begin{cases}C'^{(\text{E-W})}_{a,\text{Story 1}}=0.2\\ C'^{(\text{N-S})}_{a,\text{Story 1}}=0.5-(0.5-0.2)\dfrac{0.25-0.2}{0.5-0.2}=0.45\end{cases}$$

Using the component weight listed in Table 12-4 of this *Guide*, P_D, "vertical load acting on the wall in lb/ft (not including the self-weight of the wall at the story under consideration," is calculated as follows:

$$\begin{cases}P_{D,\text{Story1, side wall}}=\underbrace{628\text{ lb/ft}}_{\text{From roof}}+\underbrace{\dfrac{51{,}000\text{ lbs}}{60\text{ ft}\times 2}}_{\text{Level 2 diaphragm}}+\underbrace{\dfrac{98{,}000\text{ lbs}}{60\text{ ft}}}_{\text{Full-height story 2 side wall}}=2{,}686\text{ lb/ft}\\ P_{D,\text{Story1, front wall}}=\underbrace{270\text{ lb/ft}}_{\text{From roof}}+\underbrace{\dfrac{2(25{,}000\text{ lbs})}{30\text{ ft}}}_{\text{Full-height story 2 front wall}}=1{,}937\text{ lb/ft}\\ P_{D,\text{Story1, rear wall}}=\underbrace{270\text{ lb/ft}}_{\text{From roof}}+\underbrace{\dfrac{2(26{,}000\text{ lbs})}{30\text{ ft}}}_{\text{Full-height story 2 rear wall}}=2{,}003\text{ lb/ft}\end{cases}$$

$$C_{t,\text{Story 1}}=0.2+\frac{t}{15.7}=0.2+\frac{13}{15.7}=1.028>1.0\Rightarrow C_{t,\text{Story 1}}=1.0$$

$$\begin{cases}C_{g,\text{Story 1,side}}=1.1\\ C_{g,\text{Story 1,front}}=C_{g,\text{Story 1,rear}}=1.0+(1.1-1.0)\dfrac{0.25\text{ s }-0.2\text{ s}}{0.5\text{ s }-0.2\text{ s}}=1.02\end{cases}$$

$C_{pl,\text{Story 1}} = 0.9$ for Life Safety Performance Level

$$\begin{cases} C_{a,\text{Story 1,side}}^{(\text{E-W})} = 1 + C_{a,\text{Story 1}}'^{(\text{E-W})}\left(P_{D,\text{Story 1,side}} / 685\right)\left(1 - \frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 1}} - 8\right)\right) \\ \quad = 1 + 0.2\left(\frac{2{,}686}{685}\right)\left(1 - \frac{1}{12}(11.1 - 8)\right) \\ \quad = 1.58 \\ C_{a,\text{Story 1,front}}^{(\text{N-S})} = 1 + C_{a,\text{Story 1}}'^{(\text{N-S})}\left(P_{D,\text{Story 1,front}} / 685\right)\left(1 - \frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 1}} - 8\right)\right) \\ \quad = 1 + 0.45\left(\frac{1937}{685}\right)\left(1 - \frac{1}{12}(11.1 - 8)\right) \\ \quad = 1.94 \\ C_{a,\text{Story 1,rear}}^{(\text{N-S})} = 1 + C_{a,\text{Story 1}}'^{(\text{N-S})}\left(P_{D,\text{Story 1,rear}} / 685\right)\left(1 - \frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 1}} - 8\right)\right) \\ \quad = 1 + 0.45\left(\frac{2003}{685}\right)\left(1 - \frac{1}{12}(11.1 - 8)\right) \\ \quad = 1.98 \end{cases}$$

For the two side walls of Story 1:

$$\begin{aligned} S_{X1} = 0.507 > C_{a,\text{Story 1,side}}^{(\text{E-W})} C_{t,\text{Story 1}} C_{g,\text{Story 1}} C_{pl,\text{Story 1}} S_{a\text{DIAPH, Story 1}}^{(\text{E-W})}(1) \\ = 1.58(1.0)(1.1)(0.9)(0.30) \\ = 0.47 \end{aligned}$$

For the front wall of Story 1:

$$\begin{aligned} S_{X1} = 0.507 < C_{a,\text{Story 1,front}}^{(\text{N-S})} C_{t,\text{Story 1}} C_{g,\text{Story 1}} C_{pl,\text{Story 1}} S_{a\text{DIAPH, Story 1}}^{(\text{N-S})}(1) \\ = 1.94(1.0)(1.02)(0.9)(0.35) \\ = 0.62 \end{aligned}$$

For the rear wall of Story 1:

$$\begin{aligned} S_{X1} = 0.507 < C_{a,\text{Story 1,rear}}^{(\text{N-S})} C_{t,\text{Story 1}} C_{g,\text{Story 1}} C_{pl,\text{Story 1}} S_{a\text{DIAPH, Story 1}}^{(\text{N-S})}(1) \\ = 1.98(1.0)(1.02)(0.9)(0.35) \\ = 0.64 \end{aligned}$$

Therefore, both criteria specified in ASCE 41-17 §11.3.3.3.2 indicate that the side walls of the first story are not considered stable during dynamic excitation. The front and rear walls of the first story, on the other hand, are considered stable during dynamic excitation since ASCE 41-17 Equation 11-27a is satisfied.

The same procedure is applied to the retrofitted diaphragms with structural wood panels. As default properties of blocked and chorded wood structural panel sheathing are not included in ASCE 41-13 Table 12-2, properties of the plywood structural panel used to replace the single straight sheathing specified in SDPWS-2008 Table 4-2A are used in the following calculation. The tabulated diaphragm shear stiffness for the selected blocked wood structural panel is 15,000 lb/in. As shown in Table 12-1 and Table 12-2 of this *Guide*, the wood sheathing takes only about 10% of the total seismic weight of the diaphragm. The 15/32 in. plywood structural panels with blocking have approximately the same weight as the original 1× single straight sheathing and, thus, the seismic weights and seismic forces of the retrofitted diaphragms remain the same as the original ones.

For the second story, the period of the roof diaphragm is calculated as follows:

$$\begin{cases} T^{(\text{E-W})}_{\text{Roof, retrofit}} = \sqrt{0.0146\dfrac{(192{,}000\text{ lbs-}51{,}000\text{ lbs})(60\text{ ft})}{(180{,}000\text{ lb/ft})(30\text{ ft})}(12\text{ in./ft})} \\ \qquad = 0.52\text{ s} \\ T^{(\text{N-S})}_{\text{Roof, retrofit}} = \sqrt{0.0146\dfrac{(192{,}000\text{ lbs-}98{,}000\text{ lbs})(30\text{ ft})}{(180{,}000\text{ lb/ft})(60\text{ ft})}(12\text{ in./ft})} \\ \qquad = 0.21\text{ s} \end{cases}$$

Since $T^{(\text{E-W})}_{\text{Roof, retrofit}} > 0.5$ s and 0.2 s < $T^{(\text{N-S})}_{\text{Roof, retrofit}} < 0.5$ s, the retrofitted roof diaphragm is considered flexible under east-west seismic forces and semi-stiff under north-south seismic forces.

Per ASCE 41-17 § 11.3.3.3, for the Life Safety Structural Performance Level, a cracked wall shall be considered stable during dynamic excitation if $h/t \leq 8$ or ASCE 41-17 Equation 11-27a is satisfied.

$$\left(\frac{h}{t}\right)_{\text{Story 2}} = \left(\frac{120\text{ in.}}{13\text{ in.}}\right) = 9.23 > 8$$

Since $\left(\dfrac{h}{t}\right)_{\text{Story 2}} > 8$, ASCE 41-17 Equation 11-27a is further checked.

$$S_{X1} \leq C_a C_t C_g C_{pl} S_{a\text{DIAPH}}(1) \qquad \text{(ASCE 41-17 Eq. 11-27a)}$$

Per ASCE 41-17 Equation 11-27b:

$$S_{a\text{DIAPH, Story 2}}(1) = \begin{cases} \dfrac{4}{h/t} = \dfrac{4}{9.23} = 0.43 \text{ for stiff diaphragm} \\ \dfrac{1.8}{(h/t)^{0.75}} = \dfrac{1.8}{9.23^{0.75}} = 0.34 \text{ for flexible diaphragm} \end{cases}$$

Based on the flexibility of the retrofitted roof diaphragm:

$$\begin{cases} S_{a\text{DIAPH, Story 2, retrofit}}^{(\text{E-W})}(1) = 0.34 \\ S_{a\text{DIAPH, Story 2, retrofit}}^{(\text{N-S})}(1) = 0.43 - (0.43 - 0.34)\dfrac{0.21\text{ s} - 0.2\text{ s}}{0.5\text{ s} - 0.2\text{ s}} = 0.43 \end{cases}$$

Per ASCE 41-17 Equation 11-27c:

$$\begin{cases} C'^{(\text{E-W})}_{a,\text{Story 2,retrofit}} = 0.2 \text{ for flexible diaphragms} \\ C'^{(\text{N-S})}_{a,\text{Story 2,retrofit}} = 0.5 - \left(0.5 - 0.2\right)\dfrac{0.21 - 0.2}{0.5 - 0.2} = 0.49 \end{cases}$$

P_D for the retrofitted roof diaphragm is the same as that for the original roof diaphragm.

$$\begin{cases} P_{D,\text{Story2, side walls}} = 628 \text{ lb/ft} \\ P_{D,\text{Story2, front \& rear walls}} = 270 \text{ lb/ft} \end{cases}$$

$$\begin{cases} \begin{aligned} C^{(\text{E-W})}_{a,\text{Story 2,side,retrofit}} &= 1 + C'^{(\text{E-W})}_{a,\text{Story 2}}\left(P_{D,\text{Story 2,side}} / 685\right)\left(1 - \frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 2}} - 8\right)\right) \\ &= 1 + 0.2\left(\frac{628}{685}\right)\left(1 - \frac{1}{12}\left(9.23 - 8\right)\right) \\ &= 1.16 \\ C^{(\text{N-S})}_{a,\text{Story 2,front\&rear,retrofit}} &= 1 + C'^{(\text{N-S})}_{a,\text{Story 2}}\left(P_{D,\text{Story 2,front\&rear}} / 685\right) \\ &\quad \left(1 - \frac{1}{12}\left(\left(\frac{h}{t}\right)_{\text{Story 2}} - 8\right)\right) \\ &= 1 + 0.49\left(\frac{270}{685}\right)\left(1 - \frac{1}{12}\left(9.23 - 8\right)\right) \\ &= 1.17 \end{aligned} \end{cases}$$

$$C_{t,\text{Story 2}} = 0.2 + \frac{t}{15.7} = 0.2 + \frac{13}{15.7} = 1.028 > 1.0 \Rightarrow C_{t,\text{Story 2}} = 1.0$$

$C_{g,\text{Story 2}} = 1.0$ for walls not on the ground.

$C_{pl,\text{Story 2}} = 0.9$ for the Life Safety Performance Level.

For the two side walls of Story 2:

$$\begin{aligned} S_{X1} = 0.507 &> C^{(\text{E-W})}_{a,\text{Story 2,side,retrofit}} C_{t,\text{Story 2}} C_{g,\text{Story 2}} C_{pl,\text{Story 2}} S^{(\text{E-W})}_{a\text{DIAPH, Story 2,retrofit}}(1) \\ &> 1.16\left(1.0\right)\left(1.0\right)\left(0.9\right)\left(0.34\right) \\ &> 0.36 \end{aligned}$$

For the front and rear walls of Story 2:

$$
\begin{aligned}
S_{X1} = 0.507 &> C^{(\text{N-S})}_{a,\text{Story 2,front\&rear,retrofit}} C_{t,\text{Story 2}} C_{g,\text{Story 2}} C_{pl,\text{Story 2}} S^{(\text{N-S})}_{a\text{DIAPH, Story 2, retrofit}}(1) \\
&> 1.17(1.0)(1.0)(0.9)(0.43) \\
&> 0.45
\end{aligned}
$$

Therefore, both criteria specified in ASCE 41-17 §11.3.3.3.2 indicate that for the building with retrofitted diaphragms, the URM walls on the second story are not considered stable during dynamic excitation, and retrofits are needed to improve the out-of-plane stability of these walls.

For the first story, period of the retrofitted Level 2 diaphragm is:

$$
\begin{cases}
T^{(\text{E-W})}_{\text{Level 2, retrofitted}} = \sqrt{0.0146 \dfrac{(275{,}000 \text{ lbs-}77{,}000 \text{ lbs})(60 \text{ ft})}{(180{,}000 \text{ lb/ft})(30 \text{ ft})}(12 \text{ in./ft})} \\
\qquad = 0.62 \text{ s} \\
T^{(\text{N-S})}_{\text{Level 2, retrofitted}} = \sqrt{0.0146 \dfrac{(275{,}000 \text{ lbs-}147{,}000 \text{ lbs})(30 \text{ ft})}{(180{,}000 \text{ lb/ft})(60 \text{ ft})}(12 \text{ in./ft})} \\
\qquad = 0.25 \text{ s}
\end{cases}
$$

For this particular case, the original and retrofitted diaphragms have the same periods ($T^{(\text{E-W})}_{\text{Level 2}} = T^{(\text{E-W})}_{\text{Level 2, retrofitted}}$, $T^{(\text{N-S})}_{\text{Level 2}} = T^{(\text{N-S})}_{\text{Level 2, retrofitted}}$) and, thus, the same flexibility category. Due to the same flexibility category of the original and retrofitted diaphragms, the calculations for h/t, C_a, C_t, C_g, C_{pl}, and $S_{a\text{DIAPH}}(1)$ are the same as those for the original Level 2 diaphragm. Therefore, the conclusions for Level 2 diaphragm remain the same, which are that the side walls of the first story are not considered stable during dynamic excitation, while the front and rear walls of the first story are considered stable during dynamic excitation. The retrofit for the Level 2 wood diaphragm is adequate for seismic loads, but the side walls of the first story need retrofits for improving their out-of-plane stability.

Appendix A

Other Resources

To gain insight into successful strategies for presenting design examples and to identify the design examples not already presented, the project team reviewed relevant sample design example documents. These are summarized in Table A-1 for existing buildings and Table A-2 for new buildings.

Observations from the review informed the project team on technical content and editorial decisions. The review also identified buildings that could be used in the examples within the *Guide.*

Table A-1 Review of Sample Design Example Documents: Existing Buildings

Document	Reference Date	Eval/ Retrofit?	Tiers Studied			Procedure	Building Type	Nonstructural
			Tier 1	Tier 2	Tier 3			
ATC-40 Appendices[1]	1996	Eval/ Retrofit	Yes	NSP, NDP	Yes	NSP, NDP	C1, C2	No
FEMA 276[2]	1999	Eval/ Retrofit	No	LSP, LSP, NSP, LDP, Simplified	Yes	LSP, LSP, NSP, LDP, Simplified	W2, S1, C1, C2, RM1	Yes
FEMA 307[3]	1999	Eval	No	NSP	Yes	NSP	C2	No
Tuckunder Study for City of San Jose[4]	2000	Eval/ Retrofit	No	Force-based	Yes	Force-based	W1a	No
ASCE/SEI 31-03[5]	2003	Eval	Yes	LSP	Yes	LSP	W1, W2, S1a, S2, C3, RM2, URM	Yes
FEMA 440[6]	2005	Eval	No	NSP	Yes	NSP	C2	No
"Practical Approaches for the Rehabilitation of Existing Buildings" seminar by SEAOW[7]	2008	Retrofit	Yes	LSP	Yes	LSP	URM, C2	No
SEAOC 2009 IEBC Design Manual[8]	2012	Retrofit	Yes	NSP	Yes	NSP	URM, PC1 (TU), W1, W1a, C1	No

Table A-1 Review of Sample Design Example Documents: Existing Buildings (continued)

Document	Reference Date	Eval/ Retrofit?	Tiers Studied			Procedure	Building Type	Nonstructural
			Tier 1	Tier 2	Tier 3			
"Seismic Evaluation and Retrofit of Brick Masonry Buildings Using ASCE 41-13" webinar[9]	2014	Eval	Yes	LSP	Yes	LSP	URM	No
Case Studies of Soft-Story Retrofits Using Different Design Guidelines[10]	2015	Retrofit	Yes	NDP	Yes	NDP	W1a	No

[1] ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996)

[2] FEMA 276, *Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1999)

[3] FEMA 307, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings: Technical Resources* (FEMA, 1998b)

[4] *Seismic Rehabilitation of Three Model Buildings with Tuckunder Parking: Engineering Assumptions and Cost Information* (Rutherford + Chekene, 2000)

[5] ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003)

[6] FEMA 440, *Improvement of Nonlinear Static Seismic Analysis Procedure* (FEMA, 2005)

[7] *Practical Approaches for the Rehabilitation of Existing Buildings* (SEAOW, 2008)

[8] *2009 IEBC SEAOC Structural/Seismic Design Manual, Existing Building Seismic Hazard Reduction Design Examples* (SEAOC, 2012)

[9] *Seismic Evaluation and Retrofit of Brick Masonry Buildings Using ASCE 41-13* (Turner, 2014)

[10] *Case Studies of Soft-Story Retrofits Using Different Design Guidelines* (Buckalew et al., 2013)

Table A-2 Review of Sample Design Example Documents: New Buildings

Document	Reference Date	Procedure	Building Type	Nonstructural
FEMA P-751[1]	2012	LSP, LDP, NSP, NDP	W1a, S1, S2, C1, PC1, PC2a, RM1, RM2	Yes
SEAOC 2012 IBC Design Manual (overall) [2]	2013-2014	-	n/a	Yes
Vol. 1 - Code Application Examples[2]	2013	-	n/a	Yes
Vol. 2 - Light-Frame, Tilt Up, Masonry[2]	2013	LSP	W1, PC1, W1/C1, RM1, PC1	No
Vol. 3 - Concrete Examples[2]	2013	LSP, DSP, NSP	C2, C2, C1, C2, C1, C2	No
Vol. 4 - Steel Examples[2]	2013	LSP	S1, S2, S2, S6, S2, S2,	No
Vol. 5 - Isolated/Damping Examples[3]	2014	LSP, NDP	Isolated, Damped Buildings	No

[1] FEMA P-751, *2009 NEHRP Recommended Seismic Design Provisions: Design Examples* (FEMA, 2012a)

[2] *2012 IBC SEAOC Structural/Seismic Design Manual* (SEAOC, 2013a, b, c, d, 2014)

Appendix B
Changes from ASCE 41-06 to ASCE 41-13

Chapter 1 of this *Guide* provides an overview of the timeline of development of ASCE 41-13 (ASCE, 2014). ASCE 41-13 is based on two standards that were developed separately: ASCE/SEI 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003) and ASCE 41-06, *Seismic Rehabilitation of Existing Buildings* (including Supplement No. 1) (ASCE, 2007). ASCE 41-13 includes technical updates, and resolves inconsistencies between the two standards, e.g., ASCE 31-03 is based on the underlying philosophy that existing buildings should not be required to be evaluated to the same standards as new buildings whereas the retrofit procedures in ASCE 41-06 are based on a Basic Safety Objective providing performance similar to new building standards. Inconsistencies between the analysis procedures and acceptance criteria were also resolved.

An Outline Map of ASCE 31-03 and ASCE 41-06 to ASCE 41-13 is presented on ASCE 41-13 page xxv. In addition, Pekelnicky and Poland (2012) present a summary of technical and consistency updates implemented in the development of ASCE 41-13.

B.1 Chapter 1 General Requirements

ASCE 41-13 Chapter 1 summarizes the new evaluation and retrofit process including the Tier 1 Screening Evaluation, Tier 2 Deficiency-Based Evaluation, and Tier 3 Systematic Evaluation procedures. The process is also outlined in flowcharts shown in ASCE 41-13 Figures C1-1 and C1-2.

> **Useful Tip**
>
> The ASCE 41-13 Figure C1-1 and C1-2 flow charts capture the overall ASCE 41-13 evaluation and retrofit process well. In addition, before delving into ASCE 41-13 Chapter 2, determine if mandatory requirements prevail!

B.2 Chapter 2 Performance Objectives and Seismic Hazards

As discussed in Section 2.2 of this *Guide*, Performance Objectives in ASCE 41-13, such as the Basic Performance Objective for Existing Buildings (BPOE) and the Basic Performance Objective Equivalent to New Building Standards (BPON), consist of a selected Performance Level in combination with a specific Seismic Hazard Level. The selection of the BPOE and BPON is dependent on the Risk Category of the structure. The BPON is provided as a means to demonstrate equivalency to new building design standards.

Key Terms

BSE-1E: 20% in 50 years

BSE-2E: 5% in 50 years

BSE-1N: DE of ASCE 7-10

BSE-2N: MCE_R of ASCE 7-10

Performance Level terminology in ASCE 41-13 remains consistent with ASCE 41-06. A suite of Seismic Hazard Levels and Performance Levels is included in ASCE 41-13. For reasons stated in Section 2.2 of this *Guide*, previous codes and guidelines allowed existing buildings to have a higher risk by applying a reduction factor to the code-level forces used to design the building, such as a 0.75 factor permitted by various jurisdictions. Similarly, ASCE 31-03 allowed for an increase in component capacities (*m*-factors) of approximately 1.33. In lieu of reduction factors, ASCE 41-13 provides predefined Basic Safety Earthquake (BSE) Seismic Hazard Levels for existing buildings, which represent probabilities of exceedance of 20% in 50 years (BSE-1E) and 5% in 50 years (BSE-2E), respectively. BSE-1N and BSE-2N Seismic Hazard Levels are also defined in ASCE 41-13 and are equivalent to the Design Earthquake (DE) and Risk-Targeted Maximum Considered Earthquake (MCE_R) hazard levels specified for new buildings in ASCE 7-10 (ASCE, 2010), respectively. The BSE-1E and BSE-2E are probabilistic hazard levels, while the BSE-1N and BSE-2N Seismic Hazard Levels are based on a risk-targeted approach. The BSE-1E and BSE-2E Seismic Hazard Levels are capped at the BSE-1N and BSE-2N levels, respectively, to prevent existing building design parameters from being greater than that for new buildings and to capture the deterministic caps imposed on the MCE_R. The flowchart in Figure 2-3 of this *Guide* may help to determine the appropriate criteria for a specific building.

The motivation for changing the recurrence interval rather than using a scaling factor on the hazard or acceptance criteria includes the fact that simply reducing the ground motion demand by a factor of 0.75 does not result in a spatially uniform hazard, because of differences in the seismic hazard curves for different locations. For example, reducing the 2% in 50-year ground motion parameter in San Francisco by 25% results in a ground motion parameter with approximately a 5% in 50-year probability of exceedance, whereas the same 25% reduction in the 2% in 50-year ground motion for Memphis results in an approximately 3% in 50-year hazard.

It is important to note that a level of reliability is incorporated into building codes for new buildings, such as ASCE 7-10, with a uniform risk approach to determining Seismic Hazard Levels. ASCE 41-13 (and ASCE 41-17) includes Seismic Hazard Levels calculated based on a uniform hazard approach, which can lead to inconsistencies with ASCE 7-10. For instance, the BSE-1E Seismic Hazard Level in ASCE 41-13 is considerably lower than the Design Earthquake for new buildings in the central and eastern United States.

The nonstructural provisions in ASCE 41-13 are based primarily on those in ASCE 41-06, but with the incorporation of the nonstructural checklist from ASCE 31-03. The Nonstructural Performance Levels were changed, with Life Safety being pared back to only encompass items that had been identified as causing serious injury or death to building occupants, as opposed to paralleling ASCE 7-10 nonstructural requirements. A new Nonstructural Performance Level, Position Retention, was introduced to match the ASCE 7-10 Chapter 13 requirements when I_p = 1.0. It was clarified that the Operational Nonstructural Performance Level was intended to match the ASCE 7-10 Chapter requirements when I_p = 1.5. The Immediate Occupancy and Hazards Reduced Performance Levels were eliminated.

The Level of Seismicity provisions in ASCE 41-13 have also been updated to parallel ASCE 7-10 Seismic Design Categories.

B.3 Chapter 3 Evaluation and Retrofit Requirements

In ASCE 41-13, the use of the deficiency-based procedure (Tier 2) for evaluation and retrofit is permitted for more buildings. Maximum allowable building heights were increased, and the use of some mixed building types is included in ASCE 41-13 Table 3-2, which defines the limitations for the use of Tier 2.

B.4 Chapter 4 Tier 1 Screening

Tier 1 checklists in ASCE 41-13 have less repetition and are thus more user-friendly. Since all building types have common items, a Basic Configuration Checklist was created to address these items. Each type of building also has a unique checklist that includes additional items specific to that building type. The building type checklists are organized by Level of Seismicity, thereby limiting the evaluation to the required items based on the appropriate Level of Seismicity. The checklists are further streamlined with editorial changes, and clarifications were made to better assist the user with correctly assessing checklist items. Some Benchmark Building provisions have been updated to more recent code editions, resulting in Tier 1 screenings for more buildings.

Useful Tip

The flow chart in ASCE 41-13 Figure 4-1 captures the Tier 1 evaluation process well and more buildings can use the Tier 2 deficiency-based procedure.

Key Terms

Screening: Coarse evaluation net to identify obviously compliant buildings.

Benchmark Building: A building that meets a relatively recent standard or code is deemed compliant.

B.5 Chapter 5 Tier 2 Deficiency-Based Evaluation and Retrofit

The ASCE 31-03 Tier 2 evaluation included Deficiency-Only and Full Building Evaluations. The ASCE 41-13 Tier 2 deficiency-based procedure is now intended for evaluation or retrofit, and encompasses the ASCE 31-03

Useful Tip

The flowchart in ASCE 41-13 Figure 5-1 captures the Tier 2 evaluation process well. The flowchart is also useful as a provision checklist regarding analysis limitations and component checks.

deficiency-only Tier 2 evaluation and the ASCE 41-06 simplified retrofit procedure. The ASCE 31-03 full-building Tier 2 evaluation is replaced by the ASCE 41-13 Tier 3 systematic evaluation procedure, which permits the use of linear static (LSP), linear dynamic (LDP), nonlinear static (NSP), or nonlinear dynamic procedures (NDP). Other significant changes include:

- The use of a single-level Seismic Hazard Level (BSE-1E) and Performance Level (Life Safety) is permitted for Tier 2 deficiency-based retrofits and is intended to provide protection at more intense earthquakes such that a building would have some chance of meeting the Collapse Prevention Performance Level at the BSE-2E Seismic Hazard Level.
- A knowledge factor of 0.75 was introduced to the evaluation procedure, and is also used for the Tier 2 retrofit procedure.

B.6 Chapter 6 Systematic Evaluation and Retrofit

The Tier 3 systematic evaluation and retrofit chapter includes the materials testing and other requirements needed to determine the knowledge factor, which significantly impacts component action capacities (ASCE 41-13 Table 6-1). These have been modified to permit use of a higher knowledge factor (lower penalty) when a complete set of drawings is available for the building. This chapter provides the charging language for comprehensive retrofit of a building and now directs the user to the analysis and material chapters.

B.7 Chapter 7 Analysis Procedures and Acceptance Criteria

There were numerous technical changes made in Chapter 7 with some of the key points described below:

- The simplified nonlinear static procedure in ASCE 41-06 consisted of elastic-perfectly-plastic component action behavior (simplified backbone curves), which was intended for an era where hardware and software limitations prevented more complex analysis. Given that the simplified NSP introduced complications to the ASCE 41 procedures, it was eliminated in ASCE 41-13.
- Various analysis and out-of-plane provisions were updated in ASCE 41-13 to provide equivalency to ASCE 7-10.
- ASCE 41-13 Table 7-1 now provides explicit earthquake hazard (ground motion) and post-processing requirements for NDP analyses.

- In lieu of using ASCE 41 Equations 7-5 or 7-6, a provision has been added to point the user to ASCE 41-13 Chapter 8 for foundation overturning assessment.

B.8 Chapter 8 Foundations and Geologic Site Hazards

There were numerous technical changes in Chapter 8, and some of the key points are provided below:

- The derivation of soil properties has been modified.
- Provisions for liquefaction are now included.
- The shallow foundation provisions have new procedures when flexible base modeling assumptions are used.
- The soil-structure interaction (SSI) material from ASCE 41-06 was revised and made more conservative to match findings presented in NIST GCR 12-917-21, *Soil-Structure Interaction for Building Structures* (NIST, 2012).
- Modeling parameters and acceptance criteria for the overturning action of shallow foundations are now included.

B.9 Chapter 9-12 Material Specific Chapters

There are numerous technical changes in these material specific chapters; some of the key points are provided below.

- Buckling Restrained Braced Frames have been added to Chapter 9, Steel, and the braced frame acceptance criteria have been revised.
- Modest technical updates were made to the Chapter 10, Concrete.
- Wood and cold-formed steel provisions have been separated in Chapter 12.
- Initial findings from the 2011 Christchurch, New Zealand earthquakes at the time of ASCE 41-13 balloting resulted in significant changes to the unreinforced masonry provisions in the final standard. These include recognizing bed sliding action as deformation controlled, and revising approaches to URM modeling behavior and acceptance criteria.

References

ABK, 1981a, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Categorization of Buildings*, ABK-TR-01, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates, El Segundo, California.

ABK, 1981b, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Seismic Input*, ABK-TR-02, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates, El Segundo, California.

ABK, 1981c, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing*, ABK-TR-03, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates, El Segundo, California.

ABK, 1981d, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-of-Plane*, ABK-TR-04, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates, El Segundo, California.

ABK, 1984, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: The Methodology*, ABK-TR-08, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates, El Segundo, California.

ACI, 2011, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-11, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2014, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-14, American Concrete Institute, Farmington Hills, Michigan.

ACI, 2017, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, ACI 440.2R-17, American Concrete Institute, Farmington Hills, Michigan.

AISC, 2010a, *Seismic Provisions for Structural Steel Buildings*, AISC 341-10, American Institute of Steel Construction, Chicago, Illinois.

AISC, 2010b, *Specification for Structural Steel Buildings*, AISC 360-10, American Institute of Steel Construction, Chicago, Illinois.

AISC, 2011, *Steel Construction Manual*, Fourteenth Edition, Fourth Printing, American Institute of Steel Construction, Chicago, Illinois.

AISC, 2013, *Seismic Design Manual*, Second Edition, American Institute of Steel Construction, Chicago, Illinois.

AISI, 2009a, *North American Specification for the Design of Cold-Formed Steel Structural Members*, AISI S213-07/S2-10, 2007 Edition with Supplement 2, American Iron and Steel Institute, Washington, D.C.

AISI, 2009b, *North American Standard for Cold-Formed Steel Framing – Lateral Design*, AISI S213-07/S1-09, 2007 Edition with Supplement 1, American Iron and Steel Institute, Washington, D.C.

AITC, 2007, *AITC Technical Note 26, Design Values of Structural Glued Laminated Timber in Existing Structures*, American Institute of Timber Construction, Centennial, Colorado.

Almesfer, N., Dizhur, D., Lumantarna, R., and Ingham, J., 2014, "Material properties of existing unreinforced clay brick masonry buildings in New Zealand," *Bulletin of the New Zealand Society for Earthquake Engineering,* Vol. 47, No. 2, pp. 75-96.

Arnold, E.A., Uang, C-M., Filiatrault, A., 2003, *Cyclic Behavior and Repair of Stucco and Gypsum Sheathed Woodframe Walls*, Structural Systems Research Project, Report No. SSRP-2002/07, funded by the Consortium of Universities for Research in Earthquake Engineering, Department of Structural Engineering, University of California, San Diego.

ASCE, 2003, *Seismic Evaluation of Existing Buildings*, ASCE/SEI 31-03, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASCE, 2007, *Seismic Rehabilitation of Existing Buildings (including Supplement 1)*, ASCE/SEI 41-06, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASCE, 2010, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASCE, 2014, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-13, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASCE, 2017a, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASCE, 2017b, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-17, Structural Engineering Institute of American Society of Civil Engineers, Reston, Virginia.

ASTM, 2008, *Standard Test Methods, Practices and Terminology for Chemical Analysis of Steel Products*, ASTM A751-08, American Society for Testing and Materials, West Conshohocken, Pennsylvania.

ASTM, 2009, *Standard Test Methods for In Situ Measurement of Masonry Mortar Joint Shear Strength Index*, ASTM C1531, American Society for Testing and Materials, West Conshohocken, Pennsylvania.

ASTM, 2017, *Standard Test Method for Brinell Hardness of Metallic Materials*, ASTM E10-17, American Society for Testing and Materials, West Conshohocken, Pennsylvania.

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, ATC-14 Report, Applied Technology Council, Redwood City, California.

ATC, 1996, *Seismic Evaluation and Retrofit of Concrete Buildings*, ATC-40 Report, prepared by the Applied Technology Council for the California Seismic Safety Commission, Redwood City, California.

ATC, 2005, *Field Manual: Postearthquake Safety Evaluation of Buildings, Second Edition*, ATC-20-1 Report, Applied Technology Council, Redwood City, California.

ATC, 2011, *Here Today—Here Tomorrow: The Road to Earthquake Resilience in San Francisco: Post-Earthquake Repair and Retrofit*, ATC-52-4 Report, prepared by the Applied Technology Council for the San Francisco Department of Building Inspection, Redwood City, California.

AWC, 2008, *Special Design Provisions for Wind and Seismic Standard with Commentary,* ANSI/AWC SDPWS-2008 (previously AF&PA SDPWS-08), American Wood Council, Washington, D.C.

AWC, 2012, *National Design Specification for Wood Construction*, ANSI/AWC NDS-2012, American Wood Council, Washington, D.C.

Bentz, E.C., 2000, *Sectional Analysis of Reinforced Concrete Members*, PhD Thesis, Department of Civil Engineering, University of Toronto, 2000, 310 pp.

Buckalew, J., Maison, B., McDonald, B., Schotanus, M., and McCormick, D., 2013, *Case Studies of Soft-Story Retrofits Using Different Design Guidelines*, Structural Engineers Association of Northern California (SEAONC) Special Projects Initiative Report, San Francisco, California.

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the for the Federal Emergency Management Agency, Washington, D.C.

CMACN, 2012, *2012 Design of Reinforced Masonry Structures*, Concrete Masonry Association of California and Nevada, Citrus Heights, California.

CSI, 2013, *PERFORM-3D Version 5.0.1*, Computers and Structures, Inc., Walnut Creek, California.

CSI, 2016, *SAP2000 Version 18.2*, Computers and Structures, Inc., Walnut Creek, California.

Dolce, M., 1989, "Models for in-plane loading of masonry walls," *Corso sul consolidamento degli edifici in muratura in zona sismica*, Ordine degli Ingegneri (in Italian).

FEMA, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, prepared by the Applied Technology Council for the Building Seismic Safety Council and the Federal Emergency Management Agency, Washington, D.C.

FEMA, 1997b, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 274, prepared by the Applied Technology Council for the Building Seismic Safety Council and the Federal Emergency Management Agency, Washington, D.C.

FEMA, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings: Basic Procedures*, FEMA 306, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings: Technical Resources*, FEMA 307, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 1998c, *Handbook for the Seismic Evaluation of Buildings—A Prestandard*, FEMA 310, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 1999, *Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* FEMA 276, prepared by the Applied Technology Council for the Building Seismic Safety Council and the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000a, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, FEMA 350, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000b, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, FEMA 351, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000c, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, FEMA 352, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000d, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, FEMA 353, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000e, *State of the Art Report on Connection Performance*, FEMA 355D, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied

Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000f, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings*, FEMA 355F, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2000g, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings,* FEMA 356, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2005, *Improvement of Nonlinear Static Seismic Analysis Procedure*, FEMA 440, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2006, *Techniques for the Seismic Rehabilitation of Existing Buildings*, FEMA 547, developed by Rutherford + Chekene under a contract with National Institute of Science and Technology, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2012a, 2009 *NEHRP Recommended Seismic Design Provisions: Design Examples*, FEMA 751, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2012b, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories,* FEMA P-807, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2012c, *Seismic Performance Assessment of Buildings, Volume 1 – The Methodology*, FEMA P-58-1 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2015a, *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, Third Edition, FEMA P-154, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2015b, *Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure*, FEMA P-1026, prepared by the Building

Seismic Safety Council of the National Institute of Building Sciences for the Federal Emergency Management Agency, Washington, D.C.

Gajan, S., Raychowdhury, P., Hutchinson, T., Kutter, B.L., and Stewart, J.P., 2010, “Application and validation of practical tools for nonlinear soil-foundation interaction analysis,” *Earthquake Spectra*, Vol. 26, No. 1, pp. 111-129.

Geister, A., 2013, “Investigating masonry structures,” *Structure Magazine,* National Council of Structural Engineering Associations, June.

Harden, C.W. and Hutchinson, T.C., 2009, “Beam-on-nonlinear-Winkler-foundation modeling of shallow, rocking-dominated footings,” *Earthquake Spectra*, Vol. 25, No. 2, pp. 277-300.

Haselton, C.B., Baker, J.W., Stewart, J.P, Whittaker, A.S., Luco, N., Fry, A., Hamburger, R.O., Zimmerman, R.B., Hooper, J.D., Charney, F.A., and Pekelnicky, R.G., 2017a, “Response history analysis for the design of new buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part I – Overview and specification of ground motions,” *Earthquake Spectra,* Vol. 33, No. 2, pp. 373-395.

Haselton, C.B., Baker, J.W., Stewart, J.P, Whittaker, A.S., Luco, N., Fry, A., Hamburger, R.O., Zimmerman, R.B., Hooper, J.D., Charney, F.A., and Pekelnicky, R.G., 2017b, “Response history analysis for the design of new buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part II – Structural analysis procedures and acceptance criteria,” *Earthquake Spectra,* Vol. 33, No. 2, pp. 397-417.

Hibbeler, R.C., 2014, *Structural Analysis, 9th Edition*, Pearson Education, Upper Saddle River, New Jersey.

ICBO, 1982, *1982 Uniform Code for the Abatement of Dangerous Buildings,* International Conference of Building Officials, Whittier, California.

ICBO, 1997, *1997 Uniform Building Code*, International Conference of Building Officials, Whittier, California.

ICC, 2003, *2003 International Existing Building Code,* International Code Council, Whittier, California.

ICC, 2012a, *2012 International Building Code*, International Code Council, Whittier, California.

ICC, 2012b, *2012 International Existing Building Code*, International Code Council, Whittier, California.

ICC, 2015a, *2015 International Building Code*, International Code Council, Whittier, California

ICC, 2015b, *2015 International Existing Building Code*, International Code Council, Whittier, California.

ICC-ES, 2015a, *Acceptance Criteria for Anchors in Unreinforced Masonry Elements*, ICC-Evaluation Service AC 60, International Code Council, Whittier, California.

ICC-ES, 2015b, *Acceptance Criteria for Hold-downs (Tie-downs) Attached to Wood Members*, ICC-Evaluation Service AC 155, International Code Council, Whittier, California.

Ichimura, T., Fujita, K., Quinay, P.E.B., Maddegedara, L., Hori, M., Tanaka, S., Shizawa, Y., Kobayashi, H., and Minami, K., 2015, "Implicit nonlinear wave simulation with 1.08T DOF and 0.270T unstructured finite elements to enhance comprehensive earthquake simulation," *SC'15 Proceedings of the International Conference for High Performance Computing, Networking, Storage and Analysis*, 15-20 November 2015.

Jarret, J.A., Zimmerman, R.B., Charney, F.A., and Jalalian, A., 2017, "Response-history analysis for the design of new buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part IV – A study of assumptions," *Earthquake Spectra,* Vol. 33, No. 2, pp. 449-468.

Kutter, B.L, Moore, M., Hakhamaneshi, M., and Champion, C., 2016, "Rationale for shallow foundation rocking provisions in ASCE/SEI 41-13," *Earthquake Spectra*, Vol. 32, No. 2, pp. 1097-1119.

Mander, J.B., Priestley, M.J.N., and Park, R., 1988, "Theoretical stress-strain model for confined concrete," *Journal of Structural Engineering*, Vol. 144, Issue 8.

NIST, 2009, *NEHRP Seismic Design Technical Brief No. 1: Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers*, NIST GCR 09-917-3, prepared by the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2010, *NEHRP Seismic Design Technical Brief No. 4: Nonlinear Structural Analysis for Seismic Design: A Guide for Practicing Engineers*, NIST GCR 10-917-5, prepared by the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2011, *Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses*, NIST GCR 11-917-15, prepared by the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2012, *Soil-Structure Interaction for Building Structures,* NIST GCR 12-917-21, prepared by the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2015a, *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 1: Special Moment Frames*, NIST TN 1863-1, National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2015b, *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 2: Special Concentrically Braced Frames*, NIST TN 1863-2, National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2015c, *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 3: Eccentrically Braced Frames,* NIST TN 1863-3, National Institute of Standards and Technology, Gaithersburg, Maryland.

NIST, 2017, *Guidelines for Nonlinear Structural Analysis for Design of Buildings, Part IIa – Steel Moment Frames*, NIST GCR 17-917-46v2, prepared by the Applied Technology Council for the National Institute of Standards and Technology, Gaithersburg, Maryland.

Pauley, T. and Priestley, M.J.N., 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, New York.

PCA, 2013, *Notes on ACI 318-11 Building Code Requirements for Structural Concrete with Design Applications*, Portland Cement Association, Skokie, Illinois.

Pekelnicky, R. and Poland, C., 2012, "ASCE 41-13: Seismic evaluation and retrofit of existing buildings," *Structural Engineers Association of California (SEAOC) 2012 Convention Proceedings*, Santa Fe, New Mexico.

Pekelnicky, R., Hagen, G., and Martin, D., 2017, "A summary of significant updates in ASCE 41-17," *Proceedings of the 2017 SEAOC Convention,* Structural Engineers Association of California.

Penner, O. and Elwood, K., 2016, "Out-of-plane dynamic stability of unreinforced masonry walls in one-way bending: Parametric study and assessment guidelines," *Earthquake Spectra*, Vol. 32, No. 3, Oakland, California, pp. 1699-1723.

Priestley, M.J.N., 1985, "Seismic behavior of unreinforced masonry walls," *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 18, No. 2, pp. 191-205.

Pryor, S.E., 2002, "The effects of eccentric overturning restraint in complete shear wall assemblies," *BSSC TS7 Draft*.

Rutherford + Chekene, 2000, *Seismic Rehabilitation of Three Model Buildings with Tuckunder Parking: Engineering Assumptions and Cost Information*, prepared for the Department of Housing and Office of Emergency Services, City of San Jose, California, May.

SEAOC, 1992, *Commentary on Appendix Chapter 1 of the Uniform Code for Building Conservation, Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2006, *2006 IBC Structural/Seismic Design Manual*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2008a, "Limit state design of piles, pile caps, and grade beams," *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2008b, "A brief guide to seismic design factors," *Structure Magazine*, Structural Engineers of California Seismology Committee, National Council of Structural Engineering Associations, September.

SEAOC, 2009, "Anchor bolts in light-frame construction at small edge distances," *SEAOC Blue Book – Seismic Design Recommendations*, Article 12.04.030, Structural Engineers Association of California Seismology Committee.

SEAOC, 2012, *2009 IEBC SEAOC Structural/Seismic Design Manual, Existing Building Seismic Hazard Reduction Design Examples*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2013a, *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 1: Code Application Examples*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2013b, *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 2: Examples for Light-Frame, Tilt-up and Masonry Buildings*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2013c, *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 3: Examples for Concrete Buildings*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2013d, *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 4: Examples for Steel-Framed Buildings*, Structural Engineers Association of California, Sacramento, California.

SEAOC, 2014, *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 5: Examples for Seismically Isolated Buildings and Buildings with Supplemental Damping*, Structural Engineers Association of California, Sacramento, California.

SEAONC, 2001, *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*, Structural Engineers Association of Northern California, San Francisco, California.

SEAW, 2008, *Practical Approaches for the Rehabilitation of Existing Buildings*, Seminar Notes, Structural Engineers Association of Washington Existing Buildings and Education Committees.

Simsir, C.C., Aschheim, M.A., and Abrams, D.P., 2004, "Out-of-plane dynamic response of unreinforced masonry bearing walls attached to flexible diaphragms," *Proceeding of 13th World Conference on Earthquake Engineering*, August 1-6, Vancouver, British Columbia, Canada.

SBCCI, 1979, *Standard Building Code*, Southern Building Code Congress International, Birmingham, Alabama.

StructurePoint, 2016, *spColumn Software*, version 4.60, developed by StructurePoint, LLC, Skokie, Illinois.

TMS, 2011, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries*, TMS 402-11/ACI

530-11/ASCE 5-11, Masonry Standards Joint Committee, American Concrete Institute, American Society of Civil Engineers, and The Masonry Society, Boulder, Colorado.

Turner, F., 2014, *Seismic Evaluation and Retrofit of Brick Masonry Buildings Using ASCE 41-13*, webinar, ASCE Knowledge & Learning Series.

Wilson, A., Quenneville, P., and Ingham, J., 2013, “Natural period and seismic idealization of flexible timber diaphragms,” *Earthquake Spectra*, Volume 29, No. 3, pp. 1003-1019.

Wright, J.K., 2015, *Reinforced Concrete: Mechanics and Design, 7th Edition*, Pearson Education, Inc., Upper Saddle River, New Jersey.

Zimmerman, R.B., Baker, J.W., Hooper, J.D., Bono, S., Haselton, C.B., Engel, A., Hamburger, R.O., Celikbas, A., Jalalian, A., 2017, “Response-history analysis for the design of new buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part III – Example applications illustrating the recommended methodology,” *Earthquake Spectra,* Vol. 33, No. 2, pp. 419-447.

Glossary

This *Guide* uses terminology consistent with ASCE 41-13. The list below is a copy of ASCE 41-13 § 1.2.1, Definitions, printed here with permission from ASCE.

Acceleration-Sensitive Component: A component that is sensitive to, and subject to, damage from inertial loading.

Acceptance Criteria: Limiting values of properties, such as drift, strength demand, and inelastic deformation, used to determine the acceptability of a component at a given Performance Level.

Action: An internal moment, shear, torque, axial force, deformation, displacement, or rotation corresponding to a displacement caused by a structural degree of freedom; designated as force- or deformation-controlled.

Active Fault: A fault for which there is an average historic slip rate of 1 mm per year or more and evidence of seismic activity within Holocene times (the past 11,000 years).

Adjusted Resistance: The reference resistance adjusted to include the effects of applicable adjustment factors resulting from end use and other modifying factors, excluding time-effect adjustments, which are considered separately and are not included.

Aspect Ratio: Ratio of full height to length for concrete and masonry shear walls; ratio of story height to length for wood shear walls; ratio of span to depth for horizontal diaphragms.

Assembly: Two or more interconnected components.

Authority Having Jurisdiction: The organization, political subdivision, office, or individual legally charged with responsibility for administering and enforcing the provisions of this standard.

Balloon Framing: Continuous stud framing from sill to roof, with intervening floor joists nailed to studs and supported by a let-in ribbon.

Base: The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Beam: A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Bearing Wall: A wall that supports gravity loads of at least 200 lb/ft from floors or roofs.

Bed Joint: The horizontal layer of mortar on which a masonry unit is laid.

Benchmark Building: A building designed and constructed or evaluated to a specific performance level using an acceptable code or standard listed in ASCE 41-13 Table 4-6.

Boundary Component: A structural component at the boundary of a shear wall or a diaphragm or at an edge of an opening in a shear wall or a diaphragm that possesses tensile or compressive strength to transfer lateral forces to the seismic-force-resisting system.

BPOE—Basic Performance Objective for Existing Buildings: A series of defined Performance Objectives based on a building's Risk Category meant for evaluation and retrofit of existing buildings.

BPON—Basic Performance Objective Equivalent to New Building Standards: A series of defined Performance Objectives based on a building's Risk Category meant for evaluation and retrofit of existing buildings to achieve a level of performance commensurate with the intended performance of buildings designed to a standard for new construction.

Braced Frame: A vertical seismic-force-resisting element consisting of vertical, horizontal, and diagonal components joined by concentric or eccentric connections.

BSE-1E: Basic Safety Earthquake-1 for use with the Basic Performance Objective for Existing Buildings, taken as a seismic hazard with a 20% probability of exceedance in 50 years, but not greater than the BSE-1N, at a site.

BSE-1N: Basic Safety Earthquake-1 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as two-thirds of the BSE-2N at a site.

BSE-1X: Basic Safety Earthquake-1, either the BSE-1E or BSE-1N.

BSE-2E: Basic Safety Earthquake-2 for use with the Basic Performance Objective for Existing Buildings, taken as a seismic hazard with a 5% probability of exceedance in 50 years, but not greater than the BSE-2N, at a site.

BSE-2N: Basic Safety Earthquake-2 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) per ASCE 7 at a site.

BSE-2X: Basic Safety Earthquake-2, either the BSE-2E or BSE-2N.

Building Performance Level: A limiting damage state for a building, considering structural and nonstructural components, used in the definition of Performance Objectives.

Building Type: A building classification defined in ASCE 41-13 Table 3-1 that groups buildings with common seismic-force-resisting systems and performance characteristics in past earthquakes.

Capacity: The permissible strength or deformation for a component action.

Cast Iron: A hard, brittle, nonmalleable iron–carbon alloy containing 2.0% to 4.5% carbon. Shapes are obtained by reducing iron ore in a blast furnace, forming it into bars (or pigs), and remelting and casting it into its final form.

Cavity Wall: A masonry wall with an air space between wythes.

Checklist: Set of evaluation statements that shall be completed as part of the Tier 1 screening. Each statement represents a potential deficiency based on performance in past earthquakes.

Chord: See **Diaphragm Chord**.

Clay Tile Masonry: Masonry constructed with hollow units made of clay tile.

Clay-Unit Masonry: Masonry constructed with solid, cored, or hollow units made of clay; can be ungrouted or grouted.

Closed Stirrups or Ties: Transverse reinforcement defined in ACI 318 consisting of standard stirrups or ties with 90-degree hooks and lap splices in a pattern that encloses longitudinal reinforcement.

Code Official: The individual representing the authority having jurisdiction who is legally charged with responsibility for administering and enforcing the provisions of a legally adopted regulation, building code, or policy.

Collar Joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and backup construction; can be filled with mortar or grout.

Collector: See **Diaphragm Collector**.

Column (or Beam) Jacketing: A retrofit method in which a concrete column or beam is encased in a steel or concrete "jacket" to strengthen or repair the member by confining the concrete.

Common Building Type: One of the common building types listed and described in ASCE 41-13 Table 3-1.

Component: A part of an architectural, mechanical, electrical, or structural system of a building.

Composite Masonry Wall: Multi-wythe masonry wall acting with composite action.

Composite Panel: A structural panel composed of thin wood strands or wafers bonded together with exterior adhesive.

Concentrically Braced Frame (CBF): Braced frame element in which component work-lines intersect at a single point or at multiple points such that the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint.

Concrete Masonry: Masonry constructed with solid or hollow units made of concrete; can be ungrouted or grouted.

Condition of Service: The environment to which a structure is subjected.

Connection: A link that transmits actions from one component or element to another component or element, categorized by type of action (moment, shear, or axial).

Connection Hardware: Proprietary or custom-fabricated body of a component that is used to link wood components.

Connectors: Nails, screws, lags, bolts, split rings, shear plates, headed studs, and welds used to link components to other components.

Contents: Movable items within the building introduced by the owner or occupants.

Continuity Plates: Column stiffeners at the top and bottom of a panel zone.

Control Node: A node located at the center of mass at the roof of a building used in the nonlinear static procedure (NSP) to measure the effects of earthquake shaking on a building.

Coupling Beam: A component that ties or couples adjacent shear walls acting in the same plane.

Cripple Studs: Short studs between a header and top plate at openings in wall framing, or studs between the base and sill of an opening.

Cripple Wall: Short wall between the foundation and the first floor framing.

Critical Action: The component action that reaches its elastic limit at the lowest level of lateral deflection or loading of the structure.

Cross Tie: A component that spans the width of the diaphragm and delivers out-of-plane wall forces over the full depth of the diaphragm.

Cross Wall: A wood-framed wall sheathed with lumber, structural panels, or gypsum wallboard.

Decay: Decomposition of wood caused by action of wood-destroying fungi. The term "dry rot" is used interchangeably with decay.

Decking: Solid sawn lumber or glue-laminated decking, nominally 2 to 4 in. thick and 4 or more in. wide. Decking may be tongue-and-groove or connected at longitudinal joints with nails or metal clips.

Deep Foundation: Driven piles made of steel, concrete, or wood, cast-in-place concrete piers, or drilled shafts of concrete.

Deformability: The ratio of the ultimate deformation to the limit deformation.

Deformation-Controlled Action: An action that has an associated deformation that is allowed to exceed the yield value of the element being evaluated. The extent of permissible deformation beyond yield is based on component modification factors (m-factors).

Deformation-Sensitive Component: A component that is sensitive to deformation imposed by the drift or deformation of the structure, including deflection or deformation of diaphragms.

Demand: The amount of force or deformation imposed on an element or component.

Design Earthquake: A user-specified earthquake for the evaluation or retrofit of a building that has ground-shaking criteria described in Chapter 2.

Design Professional: The individual in responsible charge of the evaluation or retrofit design being performed using this standard.

Design Resistance (Force or Moment, as appropriate): Resistance provided by a member or connection; the product of adjusted resistance, the resistance factor, and the time-effect factor.

Diagonal Bracing: Inclined components designed to carry axial force, enabling a structural frame to act as a truss to resist lateral forces.

Diaphragm: A horizontal (or nearly horizontal) structural element, such as a floor or roof system, used to transfer inertial lateral forces to vertical elements of the seismic-force-resisting system.

Diaphragm Chord: A boundary component perpendicular to the applied force that is provided to resist tension or compression caused by the diaphragm moment.

Diaphragm Collector: A component parallel to the applied force that transfers lateral forces from the diaphragm of the structure to vertical elements of the seismic-force-resisting system.

Diaphragm Ratio: See **Aspect Ratio**.

Diaphragm Strut: See **Diaphragm Tie**.

Diaphragm Tie: A component parallel to the applied load that is provided to transfer wall anchorage or diaphragm inertial forces within the diaphragm. Also called diaphragm strut. See **Cross Tie**, for case where **Diaphragm Tie** spans the entire diaphragm width.

Differential Compaction: An earthquake-induced process in which soils become more compact and settle in a nonuniform manner across a site.

Dimensioned Lumber: Lumber from nominal 2 through 4 in. thick and nominal 2 or more in. wide.

Displacement-Dependent Energy Dissipation Devices: Devices that have mechanical properties such that the force in the device is related to the relative displacement in the device.

Dowel-Type Fasteners: Bolts, lag screws, wood screws, nails, and spikes.

Drag Strut: See **Diaphragm Collector**.

Dressed Size: The dimensions of lumber after surfacing with a planing machine.

Drift: Horizontal deflection at the top of the story relative to the bottom of the story.

Dry Rot: See **Decay**.

Dry Service: Structures wherein the maximum equilibrium moisture content does not exceed 19%.

Eccentrically Braced Frame (EBF): Braced-frame element in which component work lines do not intersect at a single point and the distance between the intersecting work lines (or eccentricity) exceeds the width of the smallest component connecting at the joint.

Edge Distance: The distance from the edge of the member to the center of the nearest fastener.

Effective Damping: The value of equivalent viscous damping corresponding to the energy dissipated by the building, or element thereof, during a cycle of response.

Effective Stiffness: The value of the lateral force in the building, or an element thereof, divided by the corresponding lateral displacement.

Effective Void Ratio: Ratio of collar joint area without mortar to the total area of the collar joint.

Element: An assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, shear walls, and diaphragms.

Energy Dissipation Device: Non-gravity-load-supporting element designed to dissipate energy in a stable manner during repeated cycles of earthquake demand.

Energy Dissipation System: Complete collection of all energy dissipation devices, their supporting framing, and connections.

Evaluation: An approved process or methodology of evaluating a building for a selected Performance Objective.

Expected Strength: The mean value of resistance of a component at the deformation level anticipated for a population of similar components, including consideration of the variability in material strength as well as strain-hardening and plastic section development.

Fair Condition: Masonry found during condition assessment to have mortar and units intact but with minor cracking.

Fault: Plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Flexible Component: A component, including its attachments, having a fundamental period greater than 0.06 s.

Flexible Connection: A link between components that permits rotational or translational movement without degradation of performance, including universal joints, bellows expansion joints, and flexible metal hose.

Flexible Diaphragm: A diaphragm with horizontal deformation along its length twice or more than twice the average story drift.

Force-Controlled Action: An action that is not allowed to exceed the nominal strength of the element being evaluated.

Foundation System: An assembly of structural components, located at the soil–structure interface, that transfers loads from the superstructure into the supporting soil.

Fundamental Period: The natural period of the building in the direction under consideration that has the greatest mass participation.

Gauge or Row Spacing: The center-to-center distance between fastener rows or gauge lines.

Global System: The primary components of a building that collectively resist seismic forces.

Glulam Beam: Shortened term for glue-laminated beam, which is a wood-based component made up of layers of wood bonded with adhesive.

Good Condition: Masonry found during condition assessment to have mortar and units intact and no visible cracking.

Grade: The classification of lumber with regard to strength and utility, in accordance with the grading rules of an approved agency.

Grading Rules: Systematic and standardized criteria for rating the quality of wood products.

Gypsum Wallboard or Drywall: An interior wall surface sheathing material; can sometimes be considered for resisting lateral forces.

Head Joint: Vertical mortar joint placed between masonry units in the same wythe.

Header Course: A course where the masonry units are oriented perpendicular to those in the course above or below to tie the wythes of the wall together, typically with the masonry unit long dimension perpendicular to the wall.

High-Deformability Component: A component whose deformability is not less than 3.5 when subjected to four fully reversed cycles at the limit deformation.

Hollow Masonry Unit: A masonry unit with net cross-sectional area in every plane parallel to the bearing surface less than 75% of the gross cross-sectional area in the same plane.

Hoops: Transverse reinforcement defined in Chapter 21 of ACI 318 consisting of closed ties with 135-degree hooks embedded into the core and no lap splices.

In-Plane Wall: See **Shear Wall**.

Infill: A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed "isolated infills." Panels that are in full contact with a frame around its full perimeter are termed "shear infills."

Isolation Interface: The boundary between the upper portion of the structure (superstructure), which is isolated, and the lower portion of the structure, which is assumed to move rigidly with the ground.

Isolation System: The collection of structural components that includes all individual isolator units, all structural components that transfer force between components of the isolation system, and all connections to other structural components. The isolation system also includes the wind-restraint system, if such a system is used to meet the design requirements of this section.

Isolator Unit: A horizontally flexible and vertically stiff structural component of the isolation system that permits large lateral deformations under seismic load. An isolator unit shall be used either as part of or in addition to the weight-supporting system of the building.

Joint: An area where ends, surfaces, or edges of two or more components are attached; categorized by type of fastener or weld used and method of force transfer.

King Stud: Full-height studs adjacent to openings that provide out-of-plane stability to cripple studs at openings.

Knee Joint: A joint that in the direction of framing has one column and one beam.

Landslide: A downslope mass movement of earth resulting from any cause.

Level of Seismicity: A degree of expected seismic hazard. For this standard, levels are categorized as very low, low, moderate, or high, based on mapped acceleration values and site amplification factors, as defined in ASCE 41-13 Table 2-5.

Light Framing: Repetitive framing with small, uniformly spaced members.

Lightweight Concrete: Structural concrete that has an air-dry unit weight not exceeding 115 lb/ft^3.

Limit Deformation: Two times the initial deformation that occurs at a load equal to 40% of the maximum strength.

Limited-Deformability Component: A component that is neither a low-deformability nor a high-deformability component.

Linear Dynamic Procedure (LDP): A Tier 2 or Tier 3 response-spectrum-based modal analysis procedure, the use of which is required where the distribution of lateral forces is expected to depart from that assumed for the linear static procedure.

Linear Static Procedure (LSP): A Tier 2 or Tier 3 lateral force analysis procedure using a pseudolateral force. This procedure is used for buildings for which the linear dynamic procedure is not required.

Link Beam: A component between points of eccentrically connected members in an eccentrically braced frame element.

Link Intermediate Web Stiffeners: Vertical web stiffeners placed within a link.

Link Rotation Angle: Angle of plastic rotation between the link and the beam outside of the link, derived using the specified base shear, V.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose shear strength and liquefy as a result of increase in pore-water pressure during earthquake shaking.

Load and Resistance Factor Design: A method of proportioning structural components (members, connectors, connections, and assemblages) using load factors and strength reduction factors such that no applicable limit state is exceeded when the structure is subjected to all design load combinations.

Load Duration: The period of continuous application of a given load, or the cumulative period of intermittent applications of load. See **Time-Effect Factor**.

Load Path: A path through which seismic forces are delivered from the point at which inertial forces are generated in the structure to the foundation and, ultimately, the supporting soil.

Load Sharing: The load redistribution mechanism among parallel components constrained to deflect together.

Load/Slip Constant: The ratio of the applied load to a connection and the resulting lateral deformation of the connection in the direction of the applied load.

Local Component: A specific element or connection in a building's global system.

Low-Deformability Component: A component whose deformability is 1.5 or less.

Lower-Bound Strength: The mean minus one standard deviation of the yield strengths, Q_y, for a population of similar components.

Lumber: The product of the sawmill and planing mill, usually not further manufactured other than by sawing, resawing, passing lengthwise through a standard planing machine, cross-cutting to length, and matching.

Masonry: The assemblage of masonry units, mortar, and possibly grout or reinforcement; classified with respect to the type of masonry unit, including clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Mat-Formed Panel: A structural panel manufactured in a mat-formed process including oriented strand board and waferboard.

Maximum Considered Earthquake, Risk-Targeted (MCE_R): An extreme seismic hazard level set forth in ASCE 7 and determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustments for a targeted risk.

Maximum Displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or elements thereof, excluding additional displacement caused by accidental torsion.

Mean Return Period: The average period of time, in years, between the expected occurrences of an earthquake of specified severity.

Means of Egress: A path for exiting a building, including but not limited to doors, corridors, ramps, and stairways.

Moisture Content: The weight of the water in wood expressed as a percentage of the weight of the oven-dried wood.

Moment-Resisting Frame (MRF): A frame capable of resisting horizontal forces caused by the members (beams and columns) and joints resisting forces primarily by flexure.

Narrow Wood Shear Wall: Wood shear walls with an aspect ratio (height to width) greater than 2:1.

Nominal Size: The approximate rough-sawn commercial size by which lumber products are known and sold in the market. Actual rough-sawn sizes vary from nominal. Reference to standards or grade rules is required to determine nominal to actual finished size relationships, which have changed over time.

Nominal Strength: The capacity of a structure or component to resist the effects of loads, as determined by (1) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics; or (2) field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Nonbearing Wall: A wall that supports gravity loads less than 200 lb/ft.

Noncompact Member: A steel section that has width-to-thickness ratios exceeding the limiting values for compactness specified in AISC 360.

Noncomposite Masonry Wall: Multi-wythe masonry wall acting without composite action.

Nonstructural Component: An architectural, mechanical, or electrical component of a building that is permanently installed in, or is an integral part of, a building system.

Nonstructural Performance Level: A limiting damage state for nonstructural building components used to define Performance Objectives.

Normal Wall: A wall perpendicular to the direction of seismic forces.

Occupancy: The purpose for which a building, or part thereof, is used or intended to be used, designated in accordance with the governing regulation, building code, or policy.

Open Front: An exterior building wall plane on one side only, without vertical elements of the seismic-force-resisting system in one or more stories.

Ordinary Moment Frame: A moment frame system that meets the requirements for ordinary moment frames as defined in seismic provisions for new construction in AISC 341, Chapter 9.

Oriented Strand Board: A structural panel composed of thin, elongated wood strands with surface layers arranged in the long panel direction and core layers arranged in the cross-panel direction.

Out-of-Plane Wall: A wall that resists lateral forces applied normal to its plane.

Overturning: Behavior that results when the moment produced at the base of vertical seismic-force-resisting elements is larger than the resistance provided by the building weight and the foundation resistance to uplift.

Owner: The individual(s) or entity having legal possession or rights to sanction evaluation or retrofit of a building.

P-Δ (P-Delta) Effect: The secondary effect of vertical loads and lateral deflection on the shears and moments in various components of a structure.

Panel: A sheet-type wood product.

Panel Rigidity or Stiffness: The in-plane shear rigidity of a panel; the product of panel thickness and modulus of rigidity.

Panel Shear: Shear stress acting through the panel thickness.

Panel Zone: Area of a column at a beam-to-column connection delineated by beam and column flanges.

Parapet: Portions of a wall extending above the roof diaphragm.

Partially Grouted Masonry Wall: A masonry wall containing grout in some of the cells.

Particleboard: A panel manufactured from small pieces of wood, hemp, and flax, bonded with synthetic or organic binders and pressed into flat sheets.

Perforated Wall or Perforated Infill Panel: A wall or panel not meeting the requirements for a solid wall or infill panel.

Performance Objective: One or more pairings of a selected Seismic Hazard Level with both an acceptable or desired Structural Performance Level and an acceptable or desired Nonstructural Performance Level.

Pier: Vertical portion of a wall between two horizontally adjacent openings. Piers resist axial stresses from gravity forces and bending moments from combined gravity and lateral forces.

Pitch or Spacing: The longitudinal center-to-center distance between any two consecutive holes or fasteners in a row.

Platform Framing: Construction method in which stud walls are constructed one floor at a time, with a floor or roof joist bearing on top of the wall framing at each level.

Ply: A single sheet of veneer, or several strips laid with adjoining edges that form one veneer lamina in a glued plywood panel.

Plywood: A structural panel composed of plies of wood veneer arranged in cross-aligned layers bonded with adhesive cured upon application of heat and pressure.

Pointing: The partial reconstruction of the bed joints of a masonry wall by removing unsound mortar and replacing it with new mortar.

Pole: A round timber of any size or length, usually used with the larger end in the ground.

Pole Structure: A structure framed with generally round, continuous poles that provide the primary vertical frame and lateral-load-resisting system.

Poor Condition: Masonry found during condition assessment to have degraded mortar, degraded masonry units, or significant cracking.

Pounding: The action of two adjacent buildings coming into contact with each other during earthquake excitation as a result of their close proximity and differences in dynamic response characteristics.

Preservative: A chemical that, when suitably applied to wood, makes the wood resistant to attack by fungi, insects, marine borers, or weather conditions.

Pressure-Preservative-Treated Wood: Wood products pressure-treated by an approved process and preservative.

Primary Component: An element that is required to resist the seismic forces and accommodate seismic deformations for the structure to achieve the selected performance level.

Primary (Strong) Panel Axis: The direction that coincides with the length of the panel.

Probability of Exceedance: The chance, expressed as a percentage (%), that a more severe event will occur within a specified period, expressed in number of years.

Pseudo Seismic Force (*V*): The calculated lateral force used for the Tier 1 Quick Checks and for the Tier 2 Linear Static Procedure. The pseudo lateral force represents the force required, in a linear analysis, to impose the expected actual deformation of the structure in its yielded state where subjected to the design earthquake motions.

Punched Metal Plate: A light steel plate fastener with punched teeth of various shapes and configurations that are pressed into wood members to effect force transfer.

Quick Check: Analysis procedure used in Tier 1 screenings to determine if the seismic-force-resisting system has sufficient strength or stiffness.

Redundancy: The quality of having alternative load paths in a structure by which lateral forces can be transferred, allowing the structure to remain stable following the failure of any single element.

Reentrant Corner: Plan irregularity in a diaphragm, such as an extending wing, plan inset, or E-, T-, X-, or L-shaped configuration, where large tensile and compressive forces can develop.

Reinforced Masonry: Masonry with the following minimum amounts of vertical and horizontal reinforcement: vertical reinforcement of at least 0.20 in.^{2} in cross-section at each corner or end, at each side of each opening, and at a maximum spacing of 4 ft throughout. Horizontal reinforcement of at least 0.20 in.^{2} in cross-section at the top of the wall, at the top and bottom of wall openings, at structurally connected roof and floor openings, and at a maximum spacing of 10 ft.

Repointing: A method of repairing cracked or deteriorating mortar joints in which the damaged or deteriorated mortar is removed and the joints are refilled with new mortar.

Required Member Resistance (or Required Strength): Action on a component or connection, determined by structural analysis, resulting from the factored loads and the critical load combinations.

Resistance: The capacity of a structure, component, or connection to resist the effects of loads.

Resistance Factor: A reduction factor applied to member resistance that accounts for unavoidable deviations of the actual strength from the nominal value and for the manner and consequences of failure.

Retrofit: Improving the seismic performance of structural or nonstructural components of a building.

Retrofit Measures: Modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a scheme to rehabilitate a building to achieve a selected Performance Objective.

Retrofit Method: One or more procedures and strategies for improving the seismic performance of existing buildings.

Retrofit Strategy: A technical approach for developing rehabilitation measures for a building to improve seismic performance.

Rigid Component: A component, including attachments, having a fundamental period less than or equal to 0.06 s.

Rigid Diaphragm: A diaphragm with horizontal deformation along its length less than half the average story drift.

Risk Category: A categorization of a building for determination of earthquake performance based on the governing regulation, building code, or policy or in lieu of an applicable regulation, building code, or policy, ASCE 7.

Rough Lumber: Lumber as it comes from the saw before any dressing operation.

Row of Fasteners: Two or more fasteners aligned with the direction of load.

Running Bond: A pattern of masonry where the head joints are staggered between adjacent courses by at least one-quarter of the length of a masonry unit.

Scragging: The process of subjecting an elastomeric bearing to one or more cycles of large-amplitude displacement.

Seasoned Lumber: Lumber that has been dried either by open-air drying within the limits of moisture content attainable by this method, or by controlled air drying.

Secondary Component: An element that accommodates seismic deformations but is not required to resist the seismic forces it may attract for the structure to achieve the selected performance level.

Seismic-Force-Resisting System: Those elements of the structure that provide its basic strength and stiffness to resist seismic forces.

Seismic Hazard Level: Ground-shaking demands of specified severity, developed on either a probabilistic or deterministic basis.

Shallow Foundation: Isolated or continuous spread footings or mats.

Shear Wall: A wall that resists lateral forces applied parallel with its plane; also known as an **In-Plane Wall**.

Sheathing: Lumber or panel products that are attached to parallel framing members, typically forming wall, floor, ceiling, or roof surfaces.

Short Captive Column: A column with a height-to-depth ratio less than 75% of the nominal height-to-depth ratios of the typical columns at that level.

Shrinkage: Reduction in the dimensions of wood caused by a decrease of moisture content.

Site Class: A classification assigned to a site based on the types of soils present and their engineering properties.

Slip-Critical Joint: A bolted joint in which slip resistance of the connection is required.

Solid Masonry Unit: A masonry unit with net cross-sectional area in every plane parallel to the bearing surface equal to 75% or more of the gross cross-sectional area in the same plane.

Solid Wall or Solid Infill Panel: A wall or infill panel with openings not exceeding 5% of the wall surface area. The maximum length or height of an opening in a solid wall must not exceed 10% of the wall width or story height. Openings in a solid wall or infill panel must be located within the middle 50% of a wall length and story height and must not be contiguous with adjacent openings.

Special Moment Frame (SMF): A moment frame system that meets the special requirements for frames as defined in seismic provisions for new construction.

Stack Bond: A placement of masonry units such that the head joints in successive courses are aligned vertically.

Stiff Diaphragm: A diaphragm that is neither flexible nor rigid.

Storage Racks: Industrial pallet racks, movable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members; does not include other types of racks, such as drive-in and drive-through racks, cantilever wall-hung racks, portable racks, or racks made of materials other than steel.

Story: The portion of a structure between the tops of two successive finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the roof structural element.

Story Shear Force: Portion of the pseudo lateral force carried by each story of the building.

Strength: The maximum axial force, shear force, or moment that can be resisted by a component.

Stress Resultant: The net axial force, shear, or bending moment imposed on a cross-section of a structural component.

Strong-Back System: A secondary system, such as a frame, commonly used to provide out-of-plane support for an unreinforced or underreinforced masonry wall.

Strong Column–Weak Beam: A connection where the capacity of the column in any moment frame joint is greater than that of the beams, ensuring inelastic action in the beams.

Structural Component: A component of a building that provides gravity- or lateral-load resistance as part of a continuous load path to the foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections; designated as primary or secondary.

Structural Performance Level: A limiting structural damage state; used in the definition of Performance Objectives.

Structural Performance Range: A range of structural damage states; used in the definition of Performance Objectives.

Structural System: An assemblage of structural components that are joined together to provide regular interaction or interdependence.

Stud: Vertical framing member in interior or exterior walls of a building.

Subassembly: A portion of an assembly.

Subdiaphragm: A portion of a larger diaphragm used to distribute loads between diaphragm ties, struts, or cross ties.

Superstructure: In a building with a seismic isolation system, the portion of the structure above the isolation system.

Target Displacement: An estimate of the maximum expected displacement of the roof of a building calculated for the design earthquake.

Tie: See **Diaphragm Tie**.

Tie-Down: A device used to resist uplift of the chords of light-framed shear walls.

Tie-Down System: For seismically isolated structures, the collection of structural connections, components, and elements that provide restraint against uplift of the structure above the isolation system.

Tier 1 Screening: Completion of checklists of evaluation statements that identifies potential deficiencies in a building based on performance of similar buildings in past earthquakes.

Tier 2 Evaluation: An approach applicable to certain types of buildings and Performance Objectives based on specific evaluation of potential deficiencies to determine if they represent actual deficiencies that may require mitigation. Analysis of the response of the entire building may not be required.

Tier 2 Retrofit: The mitigation of deficiencies identified in the Tier 1 screening.

Tier 3 Evaluation: An approach to evaluation in which complete analysis of the response of the building to seismic hazards is performed, implicitly or explicitly recognizing nonlinear response.

Tier 3 Retrofit: An approach to retrofitting in which complete analysis of the response of the building to seismic hazards is performed, implicitly or explicitly recognizing nonlinear response.

Timber: Lumber of nominal cross-section dimensions of 5 in. or more.

Time-Effect Factor: A factor applied to adjusted resistance to account for effects of duration of load. (See **Load Duration**.)

Total Design Displacement: The design earthquake displacement of an isolation or energy dissipation system, or components thereof, including additional displacement caused by actual and accidental torsion.

Total Maximum Displacement: The maximum earthquake displacement of an isolation or energy dissipation system, or components thereof, including additional displacement caused by actual and accidental torsion.

Transverse Wall: A wall that is oriented transverse to in-plane shear walls and resists lateral forces applied normal to its plane; also known as an out-of-plane wall.

Ultimate Deformation: The deformation at the point where gravity load support cannot be maintained.

Unreinforced Masonry (URM) Bearing Wall: An unreinforced masonry wall that provides vertical support for a floor or roof for which the total superimposed vertical load exceeds 100 lb/ft of wall.

Unreinforced Masonry (URM) Wall: A masonry wall containing less than the minimum amounts of reinforcement as defined for reinforced masonry walls; assumed to resist gravity and lateral loads solely through resistance of the masonry materials.

V-Braced Frame: A concentrically braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span.

Velocity-Dependent Energy Dissipation Devices: Devices that have mechanical characteristics such that the force in the device is dependent on the relative velocity in the device.

Veneer: A masonry wythe that provides the exterior finish of a wall system and transfers out-of-plane load directly to a backing but is not considered to add load-resisting capacity to the wall system.

Vertical Irregularity: A discontinuity of strength, stiffness, geometry, or mass in one story with respect to adjacent stories.

Waferboard: A non-veneered structural panel manufactured from 2to 3-in. flakes or wafers bonded together with a phenolic resin and pressed into sheet panels.

Wall Pier: Vertical portion of a wall between two horizontally adjacent openings.

Wind-Restraint System: The collection of structural components that provides restraint of the seismic-isolated structure for wind loads; may be either an integral part of isolator units or a separate device.

Wood Structural Panel: A wood-based panel product bonded with an exterior adhesive, meeting the requirements of NIST PS 1-95 or PS 2-92, including plywood, oriented-strand board, waferboard, and composite panels.

Wrought Iron: An easily welded or forged iron containing little or no carbon. Initially malleable, it hardens quickly when rapidly cooled.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

X-Braced Frame: A concentrically braced frame in which a pair of diagonal braces crosses near the midlength of the braces.

Y-Braced Frame: An eccentrically braced frame (EBF) in which the stem of the Y is the link of the EBF system.

Yield Story Drift: The lateral displacement of one level relative to the level above or below at which yield stress is first developed in a frame member.

Symbols

This *Guide* uses symbols and notations consistent with referenced standards, primarily ASCE 41-13. This section presents a list of symbols and notations used in this *Guide* and their definitions as presented in the referenced standards. The list is not exhaustive; for comprehensive definition of symbols and notation, please refer to the referenced standards identified within the sections of the *Guide*.

If a symbol or notation is referenced in ASCE 41-13, this is not indicated in the list below, as this constitutes most of the entries. If a symbol or notation is from a different reference document, a citation is provided.

A Cross-sectional area of a pile

A_b Sum of net mortared area of bed joints above and below the test unit

A_{base} Area of foundation footprint if the foundation components are interconnected laterally

A_c Critical contact area of footing required to support vertical loads

Cross-sectional area of concrete in compression member, per ACI 440-2R.17, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI, 2017)

A_{cv} Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, per ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2011)

A_e Effective net area of the horizontal leg

Cross-sectional area of effectively confined concrete section, per ACI 440-2R.17

A_g Gross area of column

A_{gv} Gross area subject to shear, per AISC 360-10, *Specification for Structural Steel Buildings,* (AISC, 2010b)

A_j Effective cross-sectional area of a beam–column joint, in a plane parallel to the plane of reinforcement generating shear in the joint

A_m Gross cross-sectional area of main members, per NDS-2012, *National Design Specification for Wood Construction* (AWC, 2012)

A_n Area of net mortared or grouted section of a wall or wall pier

A_{Na} Projected influence area of a single adhesive anchor or group of adhesive anchors, per ACI 318-11

A_{Nao} Projected influence area of a single adhesive anchor, f or calculation of bond strength in tension if not limited by edge distance or spacing, per ACI 318-11

A_{net} Net section area

A_{rect} Area of the smallest rectangle that covers the footing footprint

A_s Area of non-prestressed tension reinforcement

Sum of gross-sectional areas of side members, per NDS-2012

A_s' Area of compression reinforcement

$A_{se,N}$ Effective cross-sectional area of anchor in tension, per ACI 318-11

A_{st} Total area of non-prestressed longitudinal reinforcement, per ACI 318-11

A_v Area of shear reinforcement

Shear area of masonry wall pier

A_w Summation of the net horizontal cross-sectional area for concrete and masonry wall or length for wood of all shear walls in the direction of loading

B Width of footing, typically taken as the dimension perpendicular to the direction of seismic force unless noted otherwise

B_1 Damping coefficient used to adjust spectral response for the effect of viscous damping

B_2 Multiplier to account for P-Δ effects, per AISC 360-10

B_{bsa} Bessel function used to compute base slab averaging effects

B_f Width of footing, typically taken as the dimension perpendicular to the direction of seismic force *D* unless noted otherwise

C Modification factor to relate expected maximum inelastic displacements calculated for linear elastic response

C_1 Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

C_2 Modification factor to represent the effects of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on the maximum displacement response

C_c Curvature factor for structural glued laminated timber, per NDS-2012

C_d Deflection amplification factor, per ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010)

C_{di} Diaphragm factor for nailed connections, per NDS-2012

C_E Environmental reduction factor, per ACI 440-2R.17

C_{eg} End-grain factor for connections, per NDS-2012

C_F Size factor for sawn lumber, per NDS-2012

C_{fu} Flat use factor, per NDS-2012

C_g Group action factor for connections, per NDS-2012

C_I Stress interaction factor for tapered glued laminated timbers, per NDS-2012

C_i Incising factor for dimension lumber, per NDS-2012

C_L Beam stability factor, per NDS-2012

C_M Wet service factor, per NDS-2012

C_m Effective mass factor to account for higher modal mass participation effects

C_P Column stability factor, per NDS-2012

C_p Horizontal force factor

C_r Repetitive member factor for dimension lumber, prefabricated wood I-joists, and structural composite lumber, per NDS-2012

C_t Numerical value for adjustment of period T

Temperature factor, per NDS-2012

C_{tn} Toe-nail factor for nailed, per NDS-2012

C_V Volume factor for structural glued laminated timber or structural composite lumber, per NDS-2012

C_v Coefficient of variation, defined as the standard deviation divided by the mean

C_{vx} Vertical distribution factor, based on story weights and heights for the pseudo seismic force

C_Δ Geometry factor for connections, per NDS-2012

D Constant representing the flexibility of a plate

DCR Demand-capacity ratio

$\mathrm{DCR}_{\max}$ Largest demand-capacity ratio for any primary component of a building in the direction under consideration

D_f Depth to the foundation-soil interface

E Young's modulus of elasticity

E_c Modulus of elasticity of concrete

E_m Modulus of elasticity of masonry in compression, per TMS 402-11, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries*, (TMS, 2011)

Modulus of elasticity of masonry

$E_{\min}$ Adjusted modulus of elasticity for beam stability and column stability calculations, per NDS-2012

$E'_{\min}$ Reference modulus of elasticity for beam stability and column stability calculations, per NDS-2012

E_s Modulus of elasticity of steel, per TMS 402-11

F_a Factor to adjust spectral acceleration in the short period range for site class

F_b Reference bending design value, per NDS-2012

F'_b Adjusted bending design value, per NDS-2012

F_b^* Reference bending design value multiplied by all applicable adjustment factors except C_L, per NDS-2012

F_b^{**} Reference bending design value multiplied by all applicable adjustment factors except C_V, per NDS-2012

F_{bE} Critical buckling design value for bending members, per NDS-2012

F_c^* Reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P, per NDS-2012

F_{cE} Critical buckling design value for compression members, per NDS-2012

F_{cr} Critical stress, per AISC 360-10

F_e Elastic buckling stress, per AISC 360-10

F_{EXX} Filler metal classification strength, per AISC 360-10

F_{nv} Nominal shear stress, per AISC 360-10

F_{nw} Nominal stress of the weld metal, per AISC 360-10

F_p Out-of-plane force, per unit area for the analysis of a wall spanning between two out-of-plan supports

F_{pv} Component seismic design force applied vertically at the center of gravity of the component or distributed according to the mass distribution of the component

F_{px} Diaphragm inertial force at floor level x

F_t Reference tension design value parallel to grain, per NDS-2012

F_t' Adjusted tension design value parallel to grain

F_u Specified minimum tensile strength of connecting element, per *Steel Construction Manual* (AISC, 2011)

F_v Factor to adjust spectral acceleration at 1-s for site class

F_v' Adjusted shear design value parallel to grain, per NDS-2012

F_{wx} Force applied to a wall at level x

F_x Pseudo-seismic force applied at floor level x

F_y Specified minimum yield stress for the type of steel being used

G Soil shear modulus

G_0 Initial or maximum soil shear modulus

G_d Shear stiffness of shear wall or diaphragm assembly

G_m Shear modulus of masonry

H_b required shear force on the gusset-to-column connection, per *Steel Construction Manual*

H_c Required axial force on the gusset-to-column connection, per *Steel Construction Manual*

I Moment of inertia

I_g Moment of inertia of gross concrete or masonry section about centroidal axis, neglecting reinforcement

I_p Component performance factor

J Force-delivery reduction factor

K Length factor for brace

K Minimum root diameter of threaded fastener, per *Steel Construction Manual*

K_e Effective stiffness of the building in the direction under consideration, for use with the nonlinear static procedure (NSP)

K_F Format conversion factor, per NDS-2012

K^*_{fixed} Effective fixed-base stiffness of the structure

K_i Elastic stiffness of the building in the direction under consideration, for use with the NSP

K_x Effective translational stiffness of the foundation

K_θ Effective rotational stiffness of the foundation

L Length

L_b Length between points which are either braced against lateral displacement of compression flange or braced against twist of cross section, per AISC 341-10, *Seismic Provisions for Structural Steel Buildings* (AISC, 2010a)

L_c Length of cross wall

Length critical contact area equal to A_c/b

L_e Active bond length of fiber-reinforced polymer (FRP) laminate, per ACI 440-2R.17

L_f Span, in feet, of a flexible diaphragm that provides lateral support for a wall, the span is between vertical primary seismic-force-resisting elements that provide lateral support to the flexible diaphragm in the direction considered

M^* Effective mass for the first mode

M_c Expected ultimate moment capacity of footing

M_{CE} Expected flexural strength of a member or joint

M_{CL} Lower-bound flexural strength of the member

M_{cr} Nominal cracking moment strength, per TMS 402-11

M_n Nominal moment strength at section

M_{OT} Total overturning moment induced on the element by seismic forces applied at and above the level under consideration

M_S Tier 1 system modification factor

M_{ST} Stabilizing moment produced by dead loads acting on the element

M_u Factored moment, per TMS 402-11

M_{UD} Design moment

N Standard Penetration Test (SPT) blow count in soil

N_{60} SPT blow count corrected to an equivalent hammer energy efficiency of 60%

N_{ag} Nominal bond strength in tension of a group of adhesive anchors, per ACI 318-11

N_b Number of bolts or rivets

N_{cbg} Nominal concrete breakout strength in tension of a group of anchors, per ACI 318-11

N_{sa} Nominal strength of a single anchor or individual anchor in a group of anchors in tension as governed by the steel strength, per ACI 318-11

N_u Factored axial load normal to cross-section occurring simultaneously with V_u

P Axial load at failure of a masonry core or prism test sample

P_0 Nominal axial load strength at zero eccentricity

P_{CL} Lower-bound axial strength of a column, wall, or wall pier

P_D Superimposed dead load at the top of the wall or wall pier under consideration

$P_{e\ \mathrm{story}}$ Elastic critical buckling strength for the story in the direction of translation being considered, per AISC 360-10

P_n Nominal axial stress, per AISC 360-10

Nominal axial compressive strength of a concrete section, per ACI 440-2R.17

P_r Required axial strength using LRFP or ASD load combinations, per AISC 360-10

P_u Factored axial load, per TMS 402-11

Required axial strength using LRFD load combinations, per AISC 341-10

P_{UF} Design axial force in a member

P_{uw} Factored weight of wall area tributary to wall section under consideration, per TMS 402-11

P_W Self-weight of wall

Q_{allow} Allowable bearing load specified for the design of deep foundations for gravity loads (dead plus live loads) in the available design documents

Q_c Expected bearing capacity of deep or shallow foundation

Q_{CE} Expected strength of a deformation controlled action of an element at the deformation level under consideration

Q_{CL} Lower-bound estimate of the strength of a force controlled action of an element at the deformation level under consideration

Q_D Action caused by dead load

Q_E Action caused by the response to selected Seismic Hazard Level

Q_G Action caused by gravity loads

Q_L Action caused by live load

Q_S Action caused by snow load

Q_{UD} Deformation-controlled action caused by gravity loads and earthquake forces

Q_{UF} Force-controlled action caused by gravity loads and earthquake forces

R Response modification coefficient, per ASCE 7-10

R_M Coefficient to account for influence of P-δ on P-Δ, per AISC 360-10

R_p Nonstructural component response modification factor

RRS_{bsa} Ratio of response spectra factor for base slab averaging

RRS_e Ratio of response spectra factor for embedment

S Elastic section modulus of a member

S_1 Spectral response acceleration parameter at a 1-second period, obtained from response acceleration contour maps

S_a Spectral response acceleration

S_{DS} Design short-period spectral response acceleration parameter, adjusted for site class, for determining level of seismicity,

S_n Section modulus of the net cross-sectional area of a member, per TMS 402-11

S_S Spectral response acceleration parameter at short periods, obtained from response acceleration contour maps

S_{X1} Spectral response acceleration parameter at a 1-s period for any Seismic Hazard Level and any damping, adjusted for site class

S_{XS} Spectral response acceleration parameter at short periods for the selected Seismic

T Fundamental period of the building in the direction under consideration

$\widetilde{T}$ Fundamental period of the building using a model with a flexible base

T_0 Period at which the constant acceleration region of the design response spectrum begins at a value = $0.2T_S$

T_e Effective fundamental period of the building in the direction under consideration

T_i Elastic fundamental period of the building in the direction under consideration

T_S Characteristic period of the response spectrum at which the constant acceleration segment of the response spectrum transitions to the constant velocity segment

U Shear lag factor, per AISC 360-10

U_{bs} Reduction coefficient, used in calculating block shear rupture strength, per AISC 360-10

V Pseudo-seismic force

Design shear force at section concurrent with moment, M

V_a Shear strength of an unreinforced masonry pier

V_{bjs1} Expected initial shear strength of wall or pier based on bed-joint sliding shear strength

V_{bjs2} Expected final shear strength of wall or pier based on bed-joint sliding shear strength

V_c Nominal shear strength provided by concrete

Required shear force on the gusset-to-column connection, per *Steel Construction Manual*

V_{CE} Expected shear strength of a member

V_{CL} Lower-bound shear strength

V_d Base shear at Δ_d

Diaphragm shear

V_{dt} Lower-bound shear strength based on diagonal tension stress for wall or wall pier

V_f Nominal shear strength provided by FRP stirrups

V_j Story shear force

V_n Nominal shear strength at section

V_o Shear strength of column without modification for flexural ductility

V_p Shear force at the development of the flexural capacity of a concrete element

Shear force on an unreinforced masonry wall pier

V_r Expected shear strength of wall or wall pier based on rocking

V_s Nominal shear strength provided by shear reinforcement

V_{tc} Lower-bound shear strength based on toe crushing for a wall or wall pier

V_u Factored shear force at section

V_{wx} Total shear force resisted by a shear wall at the level under consideration

V_y Effective yield strength of the building in the direction under consideration, for use with the NSP

W Effective seismic weight of a building, including total dead load and applicable portions of other gravity loads

W_d Total dead load tributary to a diaphragm

W_p Weight of the wall, per unit area

W_u Weight of the wall, per unit area

W_{wx} Dead load of an unreinforced masonry wall assigned to level x, taken from mid-story below level x to mid-story above level x

Z Reference lateral design value for a single fastener connection, per NDS-2012

Z' Adjusted lateral design value for a single fastener connection, per NDS-2012

Z'_{NT} Adjusted tension capacity of net section area, per NDS-2012

$Z_{||}$ Reference lateral design value for a single dowel-type fastener connection with all wood members loaded parallel to ground, per NDS-2012

a_p Component amplification factor

b_0 Parameter relating effective foundation area to building period

b_{bf} Flange width of beam, per AISC 341-10

b_e Effective foundation size, ft

b_f Flange width, per AISC 341-10

b_w Web width

c Distance from the fiber of maximum compressive strain to the neutral axis, per TMS 402-11

c_{Na} Projected distance from center of an anchor shaft on one side of the anchor required to develop the full bond strength of a single adhesive anchor, per ACI 318-11

d Distance from extreme compression fiber to centroid of tension reinforcement, per TMS 402-11

d_a Deflection at yield of tie-down anchorage or deflection at load level to anchorage at end of wall determined by anchorage details and dead load, in.

Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, per ACI 318-11

d_b Bolt diameter, per *Steel Construction Manual*

e Foundation embedment depth

Eccentricity of axial load, per TMS 402-11

e_b One-half the depth of the beam, per *Steel Construction Manual*

e_c One half the depth of the column, per *Steel Construction Manual*

e_u Eccentricity of P_{UF}, per TMS 402-11

f Parameter used to measure deformation capacity

f_a Axial compressive stress caused by gravity loads

f_b Actual bending stress, per NDS-2012

f_c Actual compression stress parallel to grain, per NDS-2012

f'_c Compressive strength of concrete

f'_{cc} Compressive strength of confined concrete, per ACI 440-2R.17

f'_{dt} Lower-bound masonry diagonal tension strength

f^*_{fu} Ultimate tensile strength of the FRP material as reported by the manufacturer, per ACI 440-2R.17

f_j^{avg} Average axial stress in diagonal bracing elements at level j

f_l Maximum confining pressure due to FRP jacket

f'_m Lower-bound masonry compressive strength

f_r Modulus of rupture

f_t Actual tension stress parallel to grain, per NDS-2012

f_{uta} Apecified tensile strength of anchor steel, per ACI 318-11

f_y Yield stress of reinforcing steel

f_{ye} Expected yield strength of reinforcing steel

h Distance from inside of compression flange to inside of tension flange

h_{ef} Effective embedment depth of anchor, per ACI 318-11

h_{eff} Effective height of wall or wall pier components under consideration

h_i, h_x Height from the base to floor level i or x

h_n Height above base to roof level

h_p Height of rectangular glass

h_x Height from base to floor level x

k Coefficient used for calculation of column shear strength based on displacement ductility

k_1 Modification factor applied to κ_v to account for concrete strength, per ACI 440-2R.17

k_2 Modification factor applied to κ_v to account for wrapping scheme, per ACI 440-2R.17

k_a Factor to account for diaphragm flexibility

k_c Stiffness of a representative column

k_h Horizontal seismic coefficient in soil acting on retaining wall

k_s Connection stiffness, per FEMA 355D, *State of the Art Report on Connection Performance*, (FEMA, 2000e)

k_{sv} Winkler spring stiffness in vertical direction, expressed as force/unit displacement/unit area

l Clear length of brace

l_e Length of embedment of reinforcement

l_w Length of entire wall or a segment of wall considered in the direction of shear force

m Component demand modification factor to account for expected ductility associated with this action at the selected Structural Performance Level

$m_{\max}$ Largest *m*-factor for all primary elements of the building in the direction under consideration

n Modular ratio, per TMS 402-11

n_i Number of fasteners in a row, per NDS-2012

p Tributary length, per *Steel Construction Manual*

q Vertical bearing pressure

q_{allow} Allowable bearing pressure specified in the available design documents for the design of shallow foundations for gravity loads (dead plus live loads)

q_c Expected bearing capacity of shallow foundation expressed in load, per unit area

r Governing radius of gyration

r_x Equivalent foundation radius for translation

r_θ Equivalent foundation radius for rotation

s Spacing of shear reinforcement

t Effective thickness of wood structural panel or plywood for shear, in.

Thickness of wall

t_{bf} Thickness of beam flange, per AISC 341-10

t_{cf} Minimum required thickness of column flange when no continuity plates are provided, per AISC 341-10

t_f Thickness of flange

Thickness of the flange , per AISC 341-10

Nominal thickness of one ply of FRP reinforcement, per ACI 440-2R.17

$t_{\min}$ Thickness required to eliminate prying action, per *Steel Construction Manual*

t_{sp} Specified thickness of member , per TMS 402-11

t_w Thickness of wall web, in.

Thickness of the web, per AISC 341-10

v Maximum shear in the direction under consideration

v_j^{avg} Average shear stress at level j

v_{me} Expected masonry shear strength,

v_s Effective shear wave velocity for site soil conditions

Effective shear wave velocity for site soil conditions

Shear wave velocity in soil at low strains

v_{te} Mortar shear test value

v_{to} Bed-joint shear stress from single test

w_i Portion of the effective seismic weight located on or assigned to floor level i

Δ_d Lesser of the target displacement or displacement corresponding to the maximum base shear

Δ_y Displacement at effective yield strength

ϕ Strength reduction factor

Ψ_f FRP strength reduction factor, per ACI 440-2R.17

Ω_0 Overstrenth factor, per ASCE 7-10

α_1 Positive post-yield slope ratio equal to the positive post-yield stiffness divided by the effective stiffness

Distance from the face of the column flange or web to the centroid of the gusset-to-beam connection, per *Steel Construction Manual*

α_2 Negative post-yield slope ratio equal to the negative post-yield stiffness divided by the effective stiffness

α_c Coefficient defining the relative contribution of concrete strength to nominal wall shear strength, per ACI 318-11

α_e Effective negative post-yield slope ratio equal to the effective post-yield negative stiffness divided by the effective stiffness

β Factor to adjust empirical fundamental period of the building

Ratio of expected frame strength, to expected infill strength

Distance from the face of the column flange to the centroid of the gusset-to-beam connection, per *Steel Construction Manual*

γ Unit weight, weight/unit volume

δ Moment magnification factor , per TMS 402-11

δ_t Target displacement

δ_u Deflection due to factored loads , per TMS 402-11

ε_c' Compressive strain of unconfined concrete corresponding to f_c' per ACI 440-2R.17

ε_{ccu} Ultimate axial compressive strain of confined concrete corresponding to $0.85 f_{cc}'$ in a lightly confined member, or ultimate axialcompressive strain of confined concrete corresponding to failure in a heavily confined member, per ACI 440-2R.17

ε_{fe} Effective strain in FRP reinforcement attained at failure, per ACI 440-2R.17

ε_{fu} Design rupture strain of FRP reinforcement, per ACI 440-2R.17

ε_{fu}^* Ultimate rupture strain of FRP reinforcement, per ACI 440-2R.17

ε_{mu} Maximum usable compressive strain of masonry , per TMS 402-11

η Displacement multiplier, greater than 1.0, to account for the effects of torsion

θ Generalized deformation

θ_g Maximum rotation, per FEMA 355D

θ_y Yield rotation

κ A knowledge factor used to reduce component strength based on the level of knowledge obtained for individual components during data collection

κ_a Efficiency factor for FRP reinforcement in determination of f'_{cc}, per ACI 440-2R.17

κ_v Bond-dependent coefficient for shear, per ACI 440-2R.17

λ Modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete of the same compressive strength, per ACI 318-11

λ_{hd} Limiting slenderness parameter for highly ductile compression elements, per AISC 341-10

λ_{md} Limiting slenderness parameter for moderately ductile compression elements, per AISC 341-10

λ_p Limiting slenderness parameter for compact element, per AISC 360-10

λ_r Limiting slenderness parameter for compact web, per AISC 360-10

$\mu_{\max}$ Maximum strength ratio

μ_{OT} Response modification factor for overturning moment M_{OT}

μ_{strength} Ratio of the elastic strength demand to yield strength

ρ Ratio of nonprestressed tension reinforcement

ρ_g Ratio of area of longitudinal steel reinforcement to cross-sectional area of a compression member, per ACI 440-2R.17

ρ_h Horizontal reinforcement ratio in a wall or wall pier

ρ_t	Ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, per ACI 318-11
ρ_v	Vertical reinforcement ratio in a wall or wall pier
χ	A factor for calculation of out-of-plane wall anchorage forces

Project Participants

ATC Management and Oversight

Jon A. Heintz (Program Executive, Program Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Ayse Hortacsu (Project Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Veronica Cedillos (Associate Project Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

FEMA Oversight

Mike Mahoney (Project Officer)
Federal Emergency Management Agency
500 C Street, SW
Washington, DC 20472

Andrew Herseth (Project Monitor)
Federal Emergency Management Agency
500 C Street, SW
Washington, DC 20472

William T. Holmes (Subject Matter Expert)
Consulting Structural Engineer
375 Beale St. Suite 310
San Francisco, California 94105

Project Technical Committee

Bret Lizundia (Project Technical Director)
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

Michael Braund
Degenkolb Engineers
225 Broadway, Suite 1325
San Diego, California 92101

Jim Collins
PCS Structural Solutions
1250 Pacific Avenue, Suite 701
Tacoma, Washington 98402

Ronald W. LaPlante
Division of the State Architect
10920 Via Frontera, Suite 300
San Diego, California 92127

Brian McDonald
Exponent, Inc.
149 Commonwealth Drive
Menlo Park, California 94025

Mark Moore
ZFA Structural Engineers
100 Bush Street, Suite 1850
San Francisco, California 94104

Project Review Panel

David Biggs
Biggs Consulting Engineering, PLLC
26F Congress Street, Suite 305
Saratoga Springs, New York 12866

Anthony B. Court
A. B. Court & Associates
4340 Hawk Street
San Diego, California 92103

Roy Lobo
California Office of Statewide Health Planning and Development
400 R Street, Suite 200
Sacramento, California 95811

James Parker
Simpson Gumpertz & Heger
1055 W. 7th Street, Suite 2500
Los Angeles, California 90017

Robert Pekelnicky
Degenkolb Engineers
235 Montgomery Street, Suite 500
San Francisco, California 94104

Peter Somers
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

Williston L. Warren*
SESOL Inc.
1001 W. 17th Street, Suite A
Costa Mesa, California 92627

*ATC Board Representative

Working Group

Ryan Bogart
ZFA Structural Engineers
100 Bush Street, Suite 1850
San Francisco, California 94104

Lawrence Burkett
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

Casey Champion
ZFA Structural Engineers
100 Bush Street, Suite 1850
San Francisco, California 94104

Alex Chu
Degenkolb Engineers
225 Broadway, Suite 1325
San Diego, California 92101

Jie Luo
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

Steve Patton
ZFA Structural Engineers
100 Bush Street, Suite 1850
San Francisco, California 94104

Kylin Vail
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

Illustrator

Chris Tokas
920 El Dorado Way
Sacramento, California 95819

SEAOC Reviewers

Russell Berkowitz (Program Manager)
Forell Elsesser
160 Pine Street
San Francisco, California 94111

Jonathan Buckalew
Nabih Youssef
1 Sansome Street, Suite 3670
San Francisco, California 94104

Lachezar Handzhiyski
Simpson Gumpertz & Heger
100 Pine Street, Suite 1600
San Francisco, California 94111

Kari Klaboe
Wiss, Janney, Elstner Associates, Inc.
2000 Powell Street, Suite 1650
Emeryville, California 94608

Jeremiah Legrue
Hohbach-Lewin
260 Sheridan Avenue
Palo Alto, California 94306

Dion Marriott
Holmes Structures
523 West 6th Street
Suite 1122
Los Angeles, California 90014

Rose McClure
Simpson Gumpertz & Heger
100 Pine Street, Suite 1600
San Francisco, California 94111

Ben Mohr
Mar Structural Design
2629 Seventh Street, Suite C
Berkeley, California 94710

Brian Olson
Earthquake Engineering Solutions
1714 Franklin Street, Suite 100-220
Oakland, California 94607

Jim Reber
Buehler & Buehler Structural Engineers
951-A Industrial Road
San Carlos, California 94070

Andrew Shuck
Wiss, Janney, Elstner Associates, Inc.
2000 Powell Street, Suite 1650
Emeryville, California 94608

Fred Turner
State Seismic Safety Commission
1755 Creekside Oaks Drive, Suite 100
Sacramento, California 95833

Focus Group

Marshall Carman
Steven Schaefer Associates, Inc.
10411 Medallion Drive, Suite 121
Cincinnati, Ohio 45241

Joseph R. Jones
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

Valerie Martin
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

Ryan McDaniel
A2H, Inc.
3009 Davies Plantation Road
Lakeland, Tennessee 38002

Meghann Riedner
Rutherford + Chekene
375 Beale St. Suite 310
San Francisco, California 94105

FEMA

FEMA P-2006
Catalog No. 1879-1