Guidance and Recommendations for the Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories

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Cover images: SWOF building retrofitted with new steel moment frames, credit: Degenkolb Engineers (left); collapsed SWOF buildings from the 1994 Northridge earthquake, credit: J. Dewey, U.S. Geological Survey, NOAA/NCEI (top and bottom right).

Foreword

The Federal Emergency Management Agency (FEMA) strives to reduce the ever-increasing cost that disasters inflict on our country. Preventing losses before they happen by designing and building to withstand anticipated forces from these hazards is one of the key components of mitigation and is the only truly effective way of reducing the cost of disasters.

As part of its responsibilities under the National Earthquake Hazards Reduction Program (NEHRP), and in accordance with the National Earthquake Hazards Reduction Act of 1977 (PL 94-125, as amended), FEMA is charged with supporting activities necessary to improve technical quality in the field of earthquake engineering. The primary method of addressing this charge has been supporting the investigation of seismic technical issues as they are identified by FEMA, the development and publication of technical design and construction guidance products, the dissemination of these products, and support of training and related outreach efforts.

One of the issues of significant concern for the Program continues to be the risk to the nation presented by older, existing buildings that were constructed prior to the development, adoption, and enforcement of modern building codes. Existing buildings built before moder building codes represent a significant percentage of the nation's building stock, and their often poor performance in earthquakes poses a significant risk to the resilience of our nation's communities.

In May 2012, FEMA originally addressed the collapse risk from multi-unit wood-frame buildings with brittle, weak, and torsionally irregular stories by developing and publishing FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories*. Since that time, retrofits of these types of buildings have increased, more municipalities have adopted seismic retrofit ordinances, and more information about the variations in construction has been identified. This supplemental report represents those advancements in understanding and provides recommendations and retrofit design examples, while still supporting the original FEMA P-807 methodology.

FEMA acknowledges the Applied Technology Council, the Project Technical Committee, and their seemingly unending patience and tireless commitment to satisfying "one more question." They went above and beyond in the coordination and thoroughness of this report. All who participated in this project, listed at the end of this report, have moved the needle forward on reducing the risk of SWOF buildings.

FEMA also recognizes Michael Mahoney, who retired from FEMA during this project, for setting this project up for success and his incredible mentoring, as well as Robert D. Hanson for acting as Technical Advisor until the very last period of the very last sentence.

Preface

In 2012, the Federal Emergency Management Agency published FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories*. The report presented a new methodology for evaluating and retrofitting multi-unit wood-frame soft-story buildings and was intended to complement existing codes and standards. FEMA P-807 is simpler and more streamlined to apply to these buildings than ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings*, and, unlike Chapter A4 of the *International Existing Building Code*, FEMA P-807 accounts for the strength and stiffness provided by archaic or non-conforming materials in these buildings.

Over the last decade, an increasing number of jurisdictions in California have enacted mandatory retrofit programs for multi-unit wood-frame soft-story buildings. These programs often allow several analytical and design methods, including FEMA P-807, to determine the strength and stiffness required for new vertical elements used in retrofits. In response to this growing demand for the retrofit of these buildings, in 2020, FEMA awarded the Applied Technology Council the first in a series of task orders under contract HSFE60-17-D-0002 to develop a supplement report to FEMA P-807 that provides guidance and recommendations for the evaluation and retrofit of these buildings. It is hoped that this report will help to improve the performance and reliability of seismic retrofits, as well as inform jurisdictions that are developing retrofit programs.

ATC is indebted to the leadership of David Mar, Project Technical Director, and to the other members of Project Technical Committee, including Kelly Cobeen, Garrett Hagen, and Daniel Zepeda, who managed and performed the technical development effort. Kamiar Kalbasi Anaraki, with support from Sina Basereh, developed the analytical models. Weichiang Pang provided review and guidance for the application of the FEMA P-695 methodology in calculating collapse statistics. Kaat Ceder, Christopher Neumann, Carmen O'Rourke, and Justin Tan helped develop and document the retrofit recommendations and design examples. The Project Review Panel, consisting of Jonathan Buckalew, Kristijan Kolozvari, Jay Kumar, John Wallace, and Cynthia Zabala, provided technical review and advice at key stages of the work.

Several California cities provided inventory data that helped inform the project team and influenced the selection of the archetype buildings for analytical modeling and the design examples. In particular, ATC thanks the cities of Berkeley, Beverly Hills, Oakland, Santa Monica, and West Hollywood for their willingness to contribute to this effort. Ali Vahdani, Jason Park, and Lily Yang at design-build contractor Optimum Seismic were generous with their time and information, providing inventory data and retrofit cost estimates.

ATC also would like to thank Charlie Kircher for his advice in the application of FEMA P-695.

ATC gratefully acknowledges Mike Mahoney (FEMA Project Officer), Christina Aronson (FEMA Task Monitor/Final Project Officer), and Bob Hanson (FEMA Technical Advisor) for their input and guidance in the preparation of this report, and Ginevra Rojahn and Kiran Khan for ATC report production services. The names and affiliations of all who contributed to this report are provided in the list of Project Participants at the end of this report.

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Executive Summary

Older, multi-unit wood-frame buildings with brittle, weak, and torsionally irregular stories have collapsed in past earthquakes. Often designated as soft, weak, or open-front (SWOF) buildings, many were constructed in the 1950s through 1970s and can be found across the United States, most notably along the West Coast.

FEMA originally addressed the risk from SWOF buildings by developing and, in May 2012, publishing FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories*. This guideline introduced a methodology to focus the retrofit on the first story to protect the building from collapse without transmitting excessive additional seismic forces into the upper stories. This approach accounted for the strength provided by the nonstructural walls and resulted in retrofits that balance performance with economics.

Since that time, California municipalities increasingly have enacted mandatory or voluntary seismic retrofit ordinances for these buildings. The ordinances reflect regional differences in their approaches, including the engineering design requirements for retrofits. These ordinances have increased retrofit experience and highlighted regionally based information regarding the configuration and construction materials used in these types of buildings. Many cities in Northern California require that the entire first story be considered and addressed, whereas many cities in Southern California allow retrofits to directly mitigate the open-front (or open-line) vulnerability without considering or strengthening the entire first story.

The purpose of this report is to advance the understanding of the behavior of SWOF buildings and to encourage improved practice in the design of retrofits. The report provides technical information about the expected seismic collapse performance of common SWOF building configurations, both in their unretrofitted (or original) and retrofitted conditions. It also presents retrofit design examples. The report is intended to be used by jurisdictions and their consultants to inform decisions regarding ordinance scope and retrofit methods. Throughout the report, both prevalent methods—full story and open-front retrofits—are analyzed and discussed, and much of the content, in particular the retrofit recommendations, is relevant to all types of SWOF building retrofits.

An evaluation of common SWOF building characteristics was conducted using Northern and Southern California datasets. This evaluation informed the selection of archetype buildings. The selected forms are rectangular in plan with an open front on either a long or short elevation, with either two or three stories. Wall and diaphragm material assemblies reflect the most common construction types identified in the datasets. In addition, a number of archetype variants were developed, including those with wing walls, those without the open-front vulnerability, and those with a range of diaphragm material properties. Three types of retrofits were designed for most archetypes: (1) a retrofit that only addresses the open-front vulnerability (line retrofit), as is common practice in Southern California, (2) a retrofit that only addresses the open-front vulnerability but without deflection limits on the vertical retrofit elements (optimized line retrofit), and (3) a retrofit that addresses the entire first story, as is common practice in Northern California. The story retrofits were designed using FEMA P-807 and the included Weak-Story Tool. Material properties were assigned using default values available within the Weak-Story Tool for the strength calculations of the full-story retrofits. In total, 122 archetypes were developed.

For each archetype, a three-dimensional, nonlinear model was created, incorporating the most up-to-date material properties available from experimental tests. (This means that the properties used for the analytical modeling varied from the default properties within the Weak-Story Tool that were used for the full-story retrofit designs.) The models were analyzed using the procedures of FEMA P-695, *Quantification of Building Seismic Performance Factors*. Incremental dynamic analyses were performed to calculate probabilities of collapse given different levels of response spectral acceleration. The analytical results provide data on the expected collapse performance of unretrofitted and retrofitted SWOF archetypes and thus the relative safety improvements of these retrofits.

The report summarizes key findings from the analytical studies and provides recommendations for seismic retrofit ordinances based on these findings. The recommendations are intended to assist government officials in developing and implementing seismic retrofit ordinances, as well as structural engineers who are advising property owners regarding seismic retrofit of SWOF buildings. The recommendations include but are not limited to:

- Full-story retrofits should be required, where practicable. Where this is not possible, it is recommended that screening occur for all exterior wall lines, including those perpendicular to the evident open-front wall, and retrofits be provided where determined to be necessary. This may result in retrofits being required for open-front wall lines on multiple exterior walls of a building.

- Where prioritization of SWOF building retrofits is desired, SWOF buildings three stories or more should be given higher priority than two-story SWOF buildings.

- Local seismic hazard levels should influence the adoption of a seismic retrofit ordinance. Unretrofitted collapse potential of SWOF buildings varies significantly with seismic hazard, thereby varying the need for and benefit of retrofit.

- SWOF building ordinances should address all relevant SWOF building configurations in jurisdictions and not be limited to buildings with tuck-under parking along one or more sides. These other SWOF building configurations include residential units over commercial space and multi-family dwellings over crawlspaces.

- Where line retrofits are permitted, new vertical steel elements should be designed based on strength only (i.e., drift limits need not be considered).

The report provides a series of engineering design recommendations for retrofit of SWOF buildings. These recommendations include a discussion of common seismic-force-resisting systems used in SWOF retrofits, items to consider when selecting those systems, and strategies for protecting existing structural systems. Design recommendations also are provided related to connections to diaphragms, collectors lengths, placing new vertical elements outside the building footprint, bracing new steel systems, foundations, and quality assurance.

Two SWOF retrofit design examples with conceptual construction details, which implement these recommendations, are presented. One example retrofit uses an optimized line design method and the other uses FEMA P-807.

No change to the FEMA P-807 methodology is deemed necessary. Where evaluation of a building is desired before a retrofit is designed, the FEMA P-807 methodology and accompanying Weak-Story Tool are believed to be the best available tools.

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Chapter 1: Introduction

1.1 Background and Purpose

Older, multi-unit wood-frame buildings with brittle, weak, and torsionally irregular stories have collapsed in past earthquakes. Often designated as soft, weak, or open-front (SWOF) buildings, many were constructed in the 1950s through 1970s (see Figure 1-1). The seismic-force-resisting systems consist of nonengineered sheathing and architectural finish materials, such as diagonal- and straight-lumber sheathing, cement stucco, plaster, and gypsum wallboard. The geometric irregularity due to open fronts and sparsity of walls exacerbate their vulnerability, as does, in some cases, weakness in the second-floor diaphragm. These buildings were seldom engineered for wind or seismic loads. They were built prior to building codes addressing structural irregularities and requiring fully detailed load paths. The use of plywood or oriented-strand board (OSB) structural sheathing for shear walls was uncommon at the time of construction.

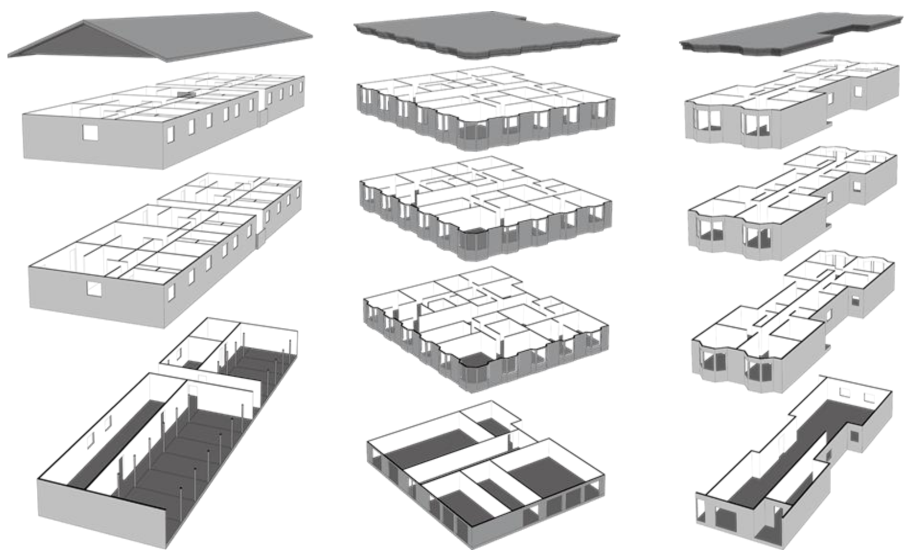


Figure 1-1 Typical configurations of soft, weak, or open-front buildings (image credit: FEMA P-807).

SWOF buildings can exist across the United States but are most prevalent along the West Coast, with tens of thousands of structures housing many more thousands of people. As a result, California municipalities have enacted mandatory or voluntary seismic retrofit ordinances for these buildings. However, the ordinances reflect regional differences in their approaches, including the engineering design requirements for retrofit. Many cities in Northern California, such as San Francisco, require that the entire first (i.e., ground) story be considered and addressed. Whereas many cities in Southern California, such as Los Angeles, allow retrofits to directly mitigate the open-front (or open-line) vulnerability without considering or strengthening the entire first story.

FEMA P-807, *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories* (FEMA, 2012), was published in 2012. The report presented a new methodology for evaluating and retrofitting SWOF buildings and was the first guideline to focus solely on the weak first story, providing enough additional strength to improve seismic performance but not so much as to drive excessive earthquake forces into the upper stories, placing them at higher risk of collapse. In the decade since FEMA P-807 was published, California municipalities increasingly have established retrofit programs for SWOF buildings, driving demand for engineering services to advise property owners and others on SWOF building seismic performance and retrofit needs.

The purpose of this report is to advance the understanding of the behavior of SWOF buildings and to encourage improved practice in the design of retrofits. The report provides technical information about the expected seismic collapse performance of common SWOF building configurations, both in their unretrofitted (or original) and retrofitted conditions. It also presents a series of retrofit design examples. The report is intended to be used by jurisdictions and their consultants to inform decisions regarding ordinance scope and retrofit methods. Throughout the report, both prevalent methods—full story and open-front retrofits—are analyzed and discussed, and much of the content, in particular the retrofit recommendations, is relevant to all types of SWOF building retrofits.

The report documents results of analytical studies that include FEMA P-807 retrofits of SWOF buildings. The report does not include any changes to the FEMA P-807 methodology.

1.2 SWOF Building Vulnerabilities

A general discussion of the vulnerability of SWOF buildings can be found in the introduction to FEMA P-807. The significant vulnerability of SWOF buildings was highlighted by their poor performance in the 1989 Loma Prieta earthquake and the 1994 Northridge earthquake. Images of collapsed and nearly collapsed SWOF buildings from these events have been widely shared and, in some cases, have become iconic examples of the destructive potential of earthquakes.

In the process of developing this report, available images and descriptions of damaged SWOF buildings were revisited to better understand the observed performance, to provide a check on long-held perceptions of performance, and to allow comparisons to performance predictions from the analytical studies. This review was prompted in part by the introduction of SWOF building-type designations by Southern California jurisdictions to aid in the screening of the existing building stock. With the separation of SWOF building types comes the potential for differentiating performance by those building types. Figure 1-2 provides an excerpt from the screening form used by the City of West Hollywood (CWH, 2019). This portion of the form illustrates three of the seven building types used for screening. Building Type A in Figure 1-2 is identified as a long-side-open building in this report, and Type 2 is identified as a short-side-open building. These two building types (with some qualifications, as discussed in Appendix A) together make up the great majority of the building stock in both Southern and Northern California. See Appendix A for more information.

The following sections discuss characteristics of SWOF buildings that inform discussion in later chapters.

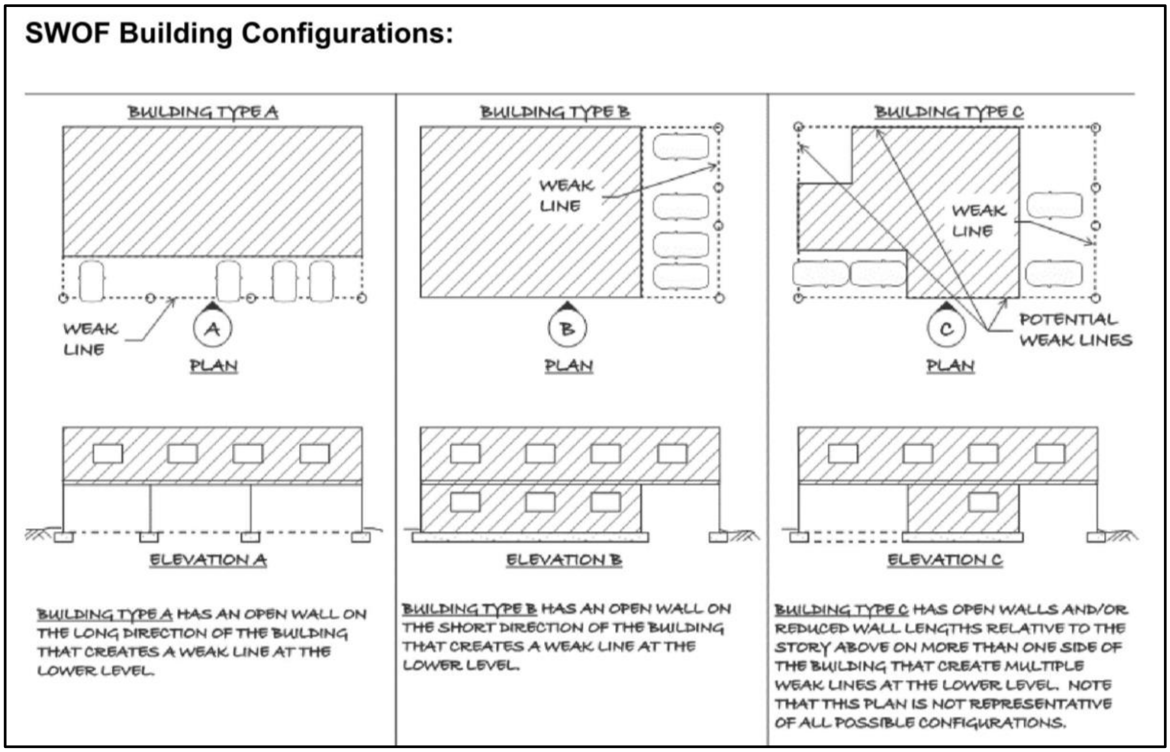


Figure 1-2 Excerpt from West Hollywood SWOF building screening form showing three of seven SWOF building types (image credit: CWH, 2019).

1.2.1 Building Configuration

In images surveyed from the Northridge earthquake, the majority of Los Angeles SWOF buildings with collapse, extensive damage, or shoring suggesting extensive damage appear to have been long-side-open buildings (SEAOC, 1991; EERI, 1994; Hamburger, 1994; NIST, 1994; EERI, 1996; Schierle, 2001; Mosalam et al., 2002; Schierle, 2003; FEMA, 2012). Figure 1-3 is representative. Several images were found of short-side-open buildings, which appeared or were identified to have limited or no damage (Figure 1-4). In the case of San Francisco buildings from the Loma Prieta earthquake, a large portion of the apartment buildings having significant damage were corner buildings that had both long and short sides open (Figure 1-5).

Based on these observations, it is concluded that the majority of collapse and extensive damage was to long-side-open buildings or to buildings with a combination of short and long sides open.

Figure 1-3 The Northridge Meadows Apartment building, representative of a long-side-open SWOF building type with a collapsed weak and soft story (image credit: Robert Reitherman, CUREE).

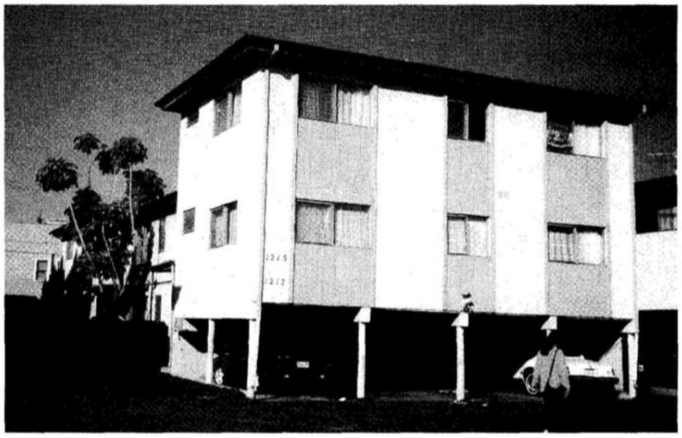


Figure 1-4 Short-side-open SWOF building type without damage (image credit: EERI, 1996).

Figure 1-5 SWOF building with both sides open in the San Francisco Marina District showing weak-and-soft story behavior (image credit: Ron Gallagher).

1.2.2 Collapse Direction

Among the images that were surveyed, collapse or significant residual drift was observed parallel to the open front in some buildings and perpendicular to the open front in others (SEAOC, 1991; EERI, 1994; EERI, 1996; Schierle, 2001; Schierle, 2003). Figure 1-6 and Figure 1-7 show SWOF buildings with collapse or significant residual drift in the direction perpendicular to the open front. In addition, the Earthquake Engineering Research Institute (EERI) preliminary reconnaissance report (EERI, 1994) notes that SWOF buildings collapsed in both directions. Based on these observations it is concluded that instead of collapse being primarily in the direction parallel to the open front, collapses occurred in both orthogonal directions.

Figure 1-6 Apartment building following the Northridge earthquake with a collapsed column showing direction of collapse perpendicular to the open front (image credit: EERI, 1996).

Figure 1-7 Northridge Gardens apartment building following the Northridge earthquake with direction of drift perpendicular to the open front (image credit: Schierle, 2003).

1.2.3 Buildings with and without SWOF Building Conditions

Buildings with a residential-unit layout that is the same for all stories, including the first story, do not have reductions in the length of wall at the first story (both interior and exterior) that characterize SWOF buildings. These buildings provide a point of comparison since performance in past earthquakes has not identified them as particularly vulnerable compared to SWOF buildings. This raises the question of damage experienced by these buildings relative to SWOF buildings. To determine if this can be deemed a reliable conclusion, a literature review indicated the following:

- Apartment buildings in the San Fernando Valley were noted to typically have tuck-under parking (Hamburger, 1994)
- A focused study that included 18 apartment buildings within one mile of Northridge Meadows and constructed between 1941 and 1976 indicated 15, or 83%, had tuck-under parking (Schierle, 2003).
- Data in Appendix A of this report show the great majority of SWOF buildings were built in the 1950s and 1960s. During this time, tuck-under parking was a common building feature.

Based on this information, it is concluded that SWOF buildings with tuck-under parking were likely prevalent in the San Fernando Valley locations where significant damage occurred to SWOF buildings. Multi-unit wood-frame buildings without SWOF conditions do not appear to have made up an appreciable portion of the building stock; therefore, the absence of reported damage to these buildings does not provide sufficient evidence that they would likely perform well in future earthquakes.

1.2.4 Line Versus Story Vulnerability

The language used to describe vulnerable SWOF buildings has included a broad mix of terms, identifying both a vulnerable (i.e., soft, weak, or open-front) wall line and a vulnerable (i.e., soft or weak) story. This includes the language used in reports on earthquake performance, as well as building code provisions, such as those in Chapter A4 of the *International Existing Building Code* (IEBC) (ICC, 2021a). The difference in language between line and story vulnerability communicates a difference in perception of the vulnerability, which can lead to a difference in the retrofit solution.

Although IEBC Chapter A4 requires evaluation and, if needed, retrofit of the soft or weak story in both orthogonal directions, an exception (included through the 2018 edition but eliminated in the 2021 edition) allowed the use of a line retrofit for two-story buildings in which the unoccupied area was 20% or less of the overall building footprint. This shows that the line retrofit concept was present in the IEBC provisions.

Based on this information, it is concluded that the perception of the vulnerability and the retrofit solution has been a mix of line and story concepts since initial descriptions of the damage and remains a mix today.

1.3 Retrofit Ordinances

Various cities have enacted or plan to enact mandatory or voluntary retrofit ordinances related to SWOF buildings (WJE, 2022). Two of the first to adopt mandatory ordinances were the City of San Francisco, in 2013, and the City of Los Angeles, in 2015. The actions in turn influenced cities in their respective regions. Table 1-1 and Table 1-2 include information about SWOF retrofit ordinances in Northern and Southern California, respectively.

These seismic mitigation ordinances have significant differences in the design approaches and retrofit requirements, partially explained by regional architecture, as well as by political and economic considerations. Most cities cite a building’s potential for a soft or weak story based on a screening process that is usually triggered by a visually identified open-front wall line as a perceived vulnerability. Although many cities agree on the problem, there have been various approaches to mitigate the issue, which are discussed in more detail in Section 1.4.

Table 1-1 Northern California SWOF Ordinances

Jurisdiction	Number of Affected Buildings	Scope	Ordinance Type
Alameda	64 reported	Soft story in wood-frame buildings built prior to 1985 with 5 units or more	Mandatory retrofit
Berkeley	327 reported	Soft story in wood-frame buildings built prior to 1978 with 5 units or more	Mandatory retrofit
Fremont	Unknown	Soft story in wood-frame buildings built prior to 1978	Mandatory retrofit
Hayward	Unknown	Soft story in wood-frame buildings built prior to 1979	Mandatory retrofit
Mountain View	Unknown	In development	In development
Oakland	1,380 reported	Soft story in wood-frame buildings built prior to 1991 with 5 units or more	Mandatory retrofit
Palo Alto	294 reported	In development	In development
Richmond	Inventory in progress	In development	In development
San Francisco	4,956 reported	Soft story in wood-frame buildings built prior to 1978 with 5 units or more	Mandatory retrofit
San Jose	Unknown	In development	In development

Table 1-2 Southern California SWOF Ordinances

Jurisdiction	Number of Affected Buildings	Scope	Ordinance Type
Beverly Hills	300 reported	SWOF lines in wood-frame buildings built prior to 1978	Mandatory retrofit
Burbank	Unknown	SWOF lines in wood-frame buildings built prior to 1978	Voluntary retrofit
Carpentaria	Unknown	In development	In development
Culver City	393 reported	SWOF lines in wood-frame buildings built prior to 1978	Mandatory retrofit
Los Angeles	13,500 reported	SWOF lines in wood-frame buildings built prior to 1978 with 4 units or more	Mandatory retrofit
Long Beach	Unknown	Soft story in wood-frame buildings built prior to 1995	Voluntary retrofit
Pasadena	500 reported	SWOF lines in wood-frame buildings built prior to 1976	Mandatory retrofit
Santa Monica	1,573 reported	SWOF lines in wood-frame buildings built prior to 1980	Mandatory retrofit
West Hollywood	738 reported	SWOF lines in wood-frame buildings built prior to 1978	Mandatory retrofit

1.4 Discussion of Retrofit Methods

Retrofit methods for SWOF buildings can be categorized as three types: (1) comprehensive, such as the procedures in ASCE/SEI 41, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2017), (2) story, such as the procedures in FEMA P-807 and IEBC Chapter A4, and (3) line, such as the procedures in the City of Los Angeles SWOF ordinance and associated city guidelines (LAMC, 2015).

ASCE/SEI 41 provides a comprehensive retrofit methodology that is suitable in all conditions, especially when strengthening would need to occur in more than a single story. The performance-based method accounts for the inelastic strength and deformation capacity of materials and incorporates design values for existing lumber sheathing and architectural finish materials.

FEMA P-807 and IEBC Chapter A4 are story retrofit methods. FEMA P-807 accounts for the strength and stiffness contributions of existing sheathing and finish materials and assesses the capacity of buildings in terms of a probability of exceeding drift limits (as a surrogate for probability of collapse).

The retrofits improve performance by strengthening the first story and increasing displacement capacity. The method tries to avoid over strengthening the first story to keep inelastic response within the first story. FEMA P-807 has limits of applicability, and it invokes ASCE/SEI 41 if these are not met.

IEBC Chapter A4 is a force-based method that evaluates both directions of the first story and adds new capacity as required. The method neglects existing lumber sheathing and finish material at the first story and does not require consideration of the capacity or performance limitations associated with the upper structure.

Line retrofit ordinances focus on mitigating the open-front vulnerability of SWOF buildings. The thinking is that correcting the weak open-front condition (or conditions) is the most efficient means to reduce the risk of a partial or total collapse of the first story. This is because the open wall-line condition is believed to be the primary contributor to the collapse potential of these buildings. The design method is prescriptive and primarily force based. It directly addresses the *observed* vulnerability (or vulnerabilities) but does not consider the existing finish materials or the capacity of the building as a whole or by story.

1.4.1 Advantages and Challenges of Each Method

Each method (line to IEBC Chapter A4 to FEMA P-807 to ASCE/SEI 41) is progressively more sophisticated, offering higher performance potential, improved understanding of behavior, and higher confidence in the effectiveness of the retrofit design. However, each method is also progressively more expensive and time consuming to implement.

ASCE/SEI 41 is applicable to all SWOF buildings, even the most complex. However, ASCE/SEI 41 is the most difficult method to apply because it requires a comprehensive building analysis and assessment of the load path even when it is being used for a building without structural drawings, which is common for SWOF buildings. ASCE/SEI 41 retrofits are highly reliable, but the method is relatively expensive and difficult to use compared to other methods for analyzing and retrofitting SWOF buildings.

FEMA P-807 forgoes the complexity of ASCE/SEI 41 by taking advantage of behavioral characteristics of SWOF buildings. The FEMA P-807 Weak-Story Tool, which is a freely available electronic resource, was developed to help users apply the rules and perform the calculations described in FEMA P-807. FEMA P-807 offers the advantages of a comprehensive understanding of behavior with high confidence in the retrofit effectiveness, but at a lower cost to implement than ASCE/SEI 41. However, there is a learning curve for engineers to educate themselves on how to properly apply the methods of FEMA P-807 and the Weak-Story Tool. Relative to IEBC Chapter A4, FEMA P-807 also offers the potential for lower-cost retrofits by accounting for the properties of existing walls.

The procedures of IEBC Chapter A4 are simpler to apply than FEMA P-807 in that only the first story must be considered and existing, noncompliant materials are neglected. Because of its simplicity, IEBC Chapter A4 is widely used by engineers. However, the disadvantage compared to FEMA P-807 is that by neglecting noncompliant material, retrofits can be more expensive. Also, IEBC Chapter A4 does not require consideration of the capacity or performance limitations associated with the upper structure, introducing the possibility of a first-story retrofit causing failure of the second story during strong earthquake shaking. Thus, although the expectation is that retrofits based on IEBC Chapter A4 are effective in most cases, there is lower confidence in their reliability relative to FEMA P-807.

Relative to full-story retrofits, line retrofits can be less expensive to design and construct, more straightforward to implement, and less disruptive to occupants on the first floor. The primary disadvantage is that the method offers the engineer little understanding of its effectiveness, and the safety benefit is limited in many cases, as will be demonstrated in Chapter 2.

1.4.2 Regional Trends

In Northern California, most, perhaps all, of the SWOF building retrofit ordinances require story retrofits, with IEBC Chapter A4 believed to be the most common method used. In Southern California, line retrofits are the most common, although ASCE/SEI 41, FEMA P-807, and IEBC Chapter A4 are allowed. A review of city ordinances (Table 1-1 and Table 1-2) suggests that the San Francisco ordinance influenced the mitigation approach in Northern California, and the Los Angeles ordinance influenced the mitigation approach in Southern California.

1.5 Approach and Scope of Study

Three-dimensional, nonlinear analytical models were developed to investigate the expected seismic collapse performance of SWOF buildings. The models were developed for a suite of archetypes of varying plan layout, location of open-front wall lines, number of stories, and diaphragm and wall materials. The selection of the archetype characteristics was based on a survey of inventory data collected from Northern and Southern California jurisdictions, data from a design-build contractor with significant experience retrofitting SWOF buildings, and the judgment of the project team.

Retrofit schemes using both line and FEMA P-807 methods were developed and modeled. The FEMA P-807 retrofits were designed using the default material property values available within the Weak-Story Tool. However, the models developed for this study and used to evaluate the effectiveness of these retrofits are more sophisticated than those of the original FEMA P-807 analyses. The primary differences are that this study includes the modeling of nonlinear diaphragms, updated material properties, and the explicit modeling of collapse.

Using the procedures of FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009), incremental dynamic analyses were performed to calculate the probabilities of collapse of the archetypes given different levels of response spectral acceleration. The results, which are presented in Chapter 2, provide insights into the expected seismic collapse performance of common SWOF buildings, both in their unretrofitted and retrofitted conditions. The key findings from the analyses, as well as recommendations that emerged from those findings, are presented in

Chapter 3. These analytical insights also informed the retrofit design recommendations and the retrofit design examples, which are presented in Chapter 4 and Chapter 5, respectively.

1.6 Organization and Content

This report describes the results of analytical studies that investigated the seismic collapse performance of SWOF buildings in their unretrofitted and retrofitted conditions. It presents recommendations for jurisdictions and their consultants developing SWOF building ordinances, as well as retrofit design recommendations. The report also includes retrofit design examples using both line and FEMA P-807 methods.

Chapter 2 summarizes the methods used to develop the analytical models and the computed probabilities of collapse based on incremental dynamic analyses for unretrofitted and retrofitted SWOF buildings.

Chapter 3 presents key findings and recommendations regarding the vulnerabilities and seismic collapse performance of SWOF buildings, in addition to a series of recommendations for jurisdictions developing retrofit ordinances and structural engineers who are advising property owners.

Chapter 4 presents in-depth and practical recommendations for retrofit design of SWOF buildings.

Chapter 5 illustrates the recommendations from Chapter 4 in retrofit design examples for the same building using both line and FEMA P-807 methods.

Chapter 6 provides concluding remarks.

Appendix A summarizes inventory data collected from Northern and Southern California about common characteristics of SWOF buildings.

Appendix B includes an overview of the evolution of building code provisions related to SWOF buildings in areas of high seismicity.

Appendix C provides a detailed discussion of the three-dimensional, nonlinear models that were developed and analyzed, including a complete reporting of all results.

Appendix D summarizes information collected related to wood diaphragm strength and hysteretic behavior and documents the considerations included in the selection of diaphragm model properties.

References and a list of project participants are provided at the end of the report.

Chapter 2: Analytical Studies

2.1 Introduction

This chapter describes analytical studies that were conducted to better understand the expected behavior of SWOF buildings subjected to strong earthquake shaking. Archetypes were developed to represent common SWOF building configurations. From these, three-dimensional, nonlinear models were created, incorporating the most up-to-date element properties from experimental tests. Retrofits were designed using line and FEMA P-807 methods, and these retrofit elements were incorporated into the models. Variant archetypes were developed to investigate the impact of different configurations and material properties. The models were analyzed using FEMA P-695 procedures to compute probabilities of collapse given different levels of response spectral acceleration. The analytical results provide data on the expected collapse performance of unretrofitted and retrofitted SWOF conditions and thus the relative safety improvements of these retrofits. Additional modeling information is provided in Appendix C and Appendix D.

2.2 Archetypes

A survey of SWOF buildings commonly constructed on the West Coast during the 1940s–1970s was completed to identify common building layouts and materials. The results of that survey are provided in Appendix A. The basic forms are rectangular in plan with an open front on either a long or short side, with either two or three stories. Four-story buildings were omitted because very few examples were found in the survey. Complex configurations exist, in the form "U", "C", and "E" shaped plans, but these are made up of the basic rectangular elements. The wall materials are typically stucco exterior siding and either gypsum wallboard or lath-and-plaster interior finishes. The diaphragms are either straight or diagonal sheathing.

Informed by the survey, two principal archetype configurations were selected, one with a long elevation open (designated LO) and one with a short elevation open (SO). Both principal archetype configurations have floor-to-floor heights of 9 feet. The long-side-open archetype (Figure 2-1), with plan dimensions 100 feet by 36 feet, uses the building configuration developed for FEMA P-2006, *Example Application Guide for ASCE/SEI 41-13 and Retrofit of Existing Buildings with Additional Commentary for ASC/SEI 41-17* (FEMA, 2018). The configuration with parking along a long side is the most common form of SWOF building surveyed in Southern California (67%) and the second most common form surveyed in Northern California (27%). The two-story archetype is like the three-story archetype but with one upper story omitted.

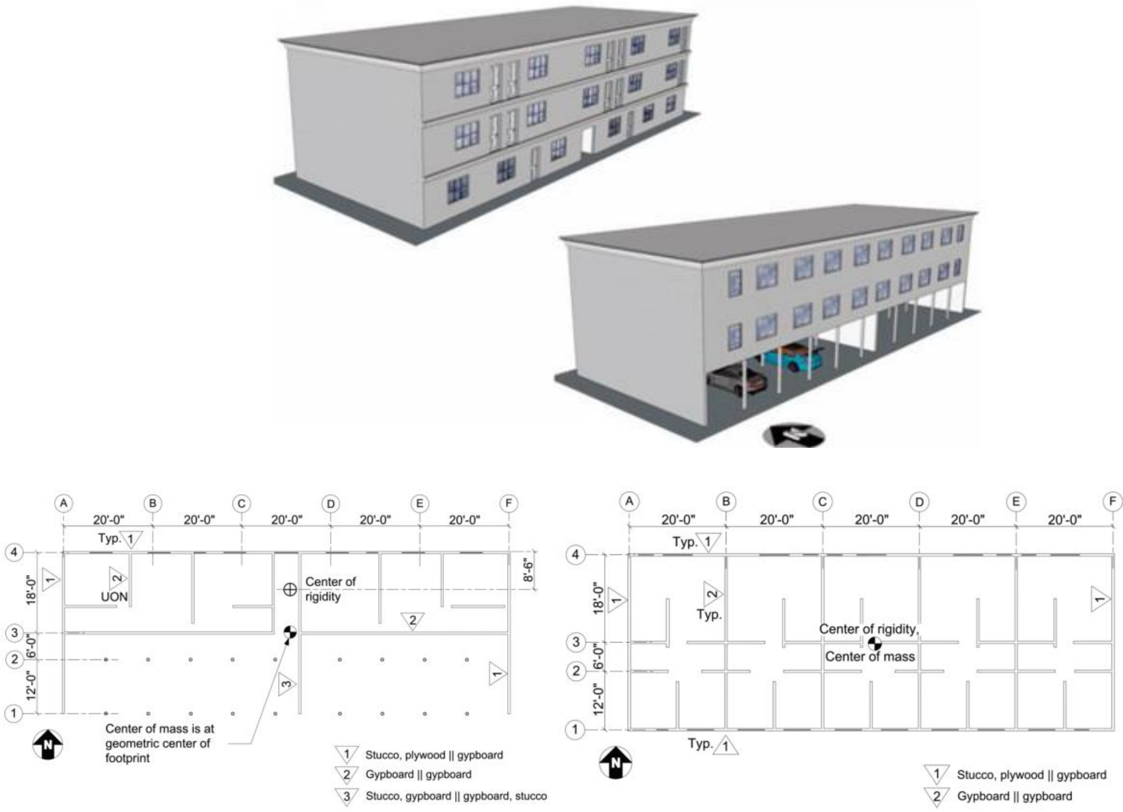


Figure 2-1 The long-side-open archetype is based on the building configuration developed for FEMA P-2006 (image credit: FEMA P-2006).

The short-side-open archetype (Figure 2-2), with plan dimensions 80 feet by 40 feet, was modeled on the archetype developed by Anaraki et al. (2019). The configuration with parking along a short side is the most common form of SWOF building surveyed (47%) in Northern California. In Southern California, this configuration makes up 12% of the surveyed results. The two-story archetype is like the three-story archetype but with one upper story omitted.

The short-side-open archetype has a variation (SOW) with exterior longitudinal walls at the first story that extend the full length of the building (i.e., into the area designated for parking). These are often called wing walls.

Figure 2-2 The short-side-open archetype is based on the archetype developed by Anaraki et al. (2019).

Both the long-side-open and short-side-open forms have variations (LN or SN) with no open-front vulnerability, where the upper story wall configuration extends to the foundation.

The archetypes have two material wall combinations, one for strong walls (SW) and one for weak walls (WW). The strong walls have stucco exterior finishes and plaster interior finishes. The weak walls have stucco exterior finishes and gypsum wallboard interior finishes. Information about the wall model properties is provided in Section 2.3.1.

A review of the evolution of building code provisions related to SWOF buildings in areas of high seismicity was also conducted and influenced the selection of material properties and strengths for the analytical models. The results of that review are provided in Appendix B.

The archetypes include six types of diaphragms: rigid (RD), strong (SD), brittle (BD), weak (WD), very weak (VWD), and lower bound (LBD). The rigid diaphragm constrains all nodes at a floor level to deflect together. The remaining diaphragm model properties are based on experimental tests, which vary significantly. More information is provided in Section 2.3.2 and Appendix D.

The wall type and diaphragm type configurations for the primary study archetypes follow construction age-based trends observed in the survey. The older buildings from the 1940s and 1950s often have plaster interior walls and straight-sheathed diaphragms. These buildings combine strong walls with weak diaphragms. The younger buildings from the 1960s often have gypsum wallboard interior walls and diagonal-sheathed diaphragms. These buildings combine weak walls with strong diaphragms. Buildings from the 1970s often have walls with gypsum wallboard and plywood diaphragms. Other combinations of materials were used less frequently.

Archetypes with three types of retrofits were studied: (1) line (L), (2) optimized line (OL), and (3) FEMA P-807 (P807). The line retrofits comply with the structural design guidelines (LADBS, 2015) developed by the City of Los Angeles Department of Building and Safety to support implementation of the SWOF building retrofit ordinance. The optimized line retrofits follow the analytical studies by Anaraki et al. (2019). Their retrofit optimization was developed to improve and make more efficient the vulnerability-based retrofits of the original Los Angeles ordinance by removing deflection limits on frames at the open front. In doing so, the frames are controlled by strength requirements. The FEMA P-807 retrofits are in accordance with that document. More details about the retrofits are provided in Section 2.5.

2.2.1 Archetype Naming Convention

The naming convention of the archetypes is the following string: form, number of stories, wall type, diaphragm type, and retrofit type. For example, LO3-WW-SD-P807 is a three-story building with the open front on the long elevation, with weak walls, strong diaphragms, and a FEMA P-807 retrofit. For a complete summary of naming abbreviations, see the Modeling Naming Convention Key in Section 3.1.

2.2.2 Primary Study Archetypes and Variants

The primary study archetypes encompass a wide range of building characteristics. They consist of the long-side-open and short-side-open forms, with both the strong-wall/weak-diaphragm (SW-WD) and weak-wall/strong-diaphragm (WW-SD) material configurations, of two and three stories. The primary study archetypes are presented in Table 2-1.

Table 2-1 The Primary Study Archetypes

		Material Types	
		WW-SD	**SW-WD**
Long Side Open	**2 Story**	LO2-WW-SD	LO2-SW-WD
	3 Story	LO3-WW-SD	LO3-SW-WD
Short Side Open	**2 Story**	SO2-WW-SD	SO2-SW-WD
	3 Story	SO3-WW-SD	SO3-SW-WD

Each archetype of the primary study has a version that incorporates each of the three types of retrofits studied—line, optimized line, and FEMA P-807. For example, the two-story, long-side-open, weak wall/strong diaphragm archetype has three different retrofitted versions, designated LO2-WW-SD-L, LO2-WW-SD-OL, and LO2-WW-SD-P807.

Variant archetypes were also studied to investigate the impact of wing walls (Section 2.6.4), no open-front vulnerability (Section 2.6.5), and diaphragm properties (Section 2.6.6) on seismic performance. The specific configurations of the variant archetypes that were studied are presented in Section 2.6. In total, 122 archetypes were developed and analyzed, but this chapter only presents the most representative and relevant subset of those analyzed. The complete set of archetypes is listed in Appendix C.

2.3 Analytical Modeling

This section describes the analytical modeling of the archetypes, including material inputs for walls and diaphragms, weight takeoffs, and model configurations. The intent was to use a limited number of building configurations to understand the seismic performance of a widely varying existing building stock. The archetype wall and diaphragm construction have an important influence on performance. Variations in these properties were selected to represent the range of prevalent buildings without necessarily reflecting the extremes of possible construction.

The archetypes were modeled in three dimensions using the analysis program OpenSees (McKenna et al., 2000). The walls, diaphragms, and retrofit elements were modeled with nonlinear material properties, except when rigid diaphragms were used, and were represented as assemblages of lumped-plasticity nonlinear springs connected to lumped masses. The springs were calibrated to physical tests of the representative wall and diaphragm materials. The wall and diaphragm material properties represent construction materials from the 1930s through the 1970s. The analyses combined state-of-the-art information about SWOF building material properties with advanced methods in nonlinear dynamic analyses. The models were used to run pushover studies and incremental dynamic analyses, per the FEMA P-695 protocol. Collapse was modeled explicitly up to 20% drift. In the few cases where models had capacity at 20% drift, the analyses were terminated to account for non-simulated failure modes of the gravity system. Additional information is provided in Appendix C.

2.3.1 Modeling Inputs for Walls

The strength of the SWOF buildings, built before the application of formal engineering design and the use of plywood wall sheathing in the mid-1970s, is dominated by the architectural finishes—stucco, plaster, and gypsum wallboard sheathing. The survey found two prevalent configurations that are grouped as strong and weak wall material sets. The strong wall material set has stucco exterior finishes and plaster interior finishes. This material set is typically found in construction earlier than the 1960s. The finishes are both stronger and significantly heavier than their weaker counterparts. The weak wall material set has stucco exterior finishes and gypsum wallboard interior finishes. This material set is typically found in construction from the 1960s and later. In both cases, the walls are brittle, with a steep loss of strength post peak.

The modeling inputs for the walls are nonlinear springs with values taken from PEER Report 2020/22, *Technical Background Report for Structural Analysis and Performance Assessment* (Welch and Deierlein, 2020). In all cases, the *best estimate* curves were used.

The material backbone curves and OpenSees *Pinching4* modeling inputs for the strong wall exterior wall assemblies (SLP2) and interior wall assembly (LP2, doubled for double-sided assembly input) are shown in Figure 2-3 and Figure 2-4, respectively. The weak wall modeling inputs are shown in Figure 2-5 for exterior wall assemblies (S2) and Figure 2-6 for interior wall assemblies (G2, doubled for double-sided assembly input).

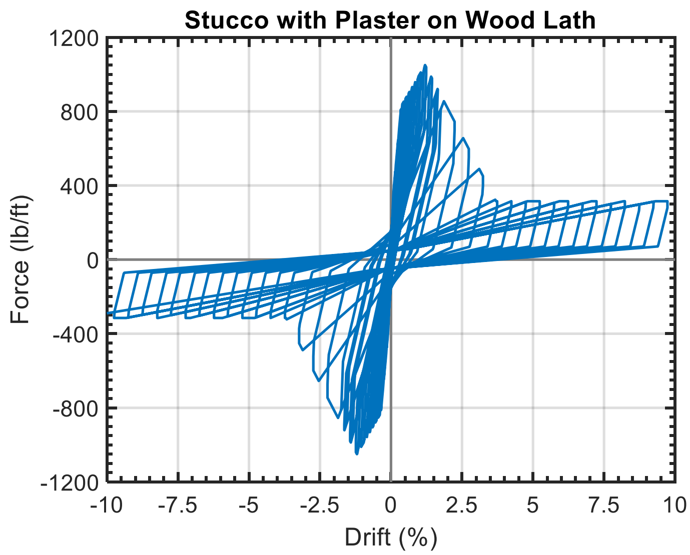


Figure 2-3 **Hysteretic input for exterior strong walls of stucco and plaster on wood lath. Material SLP2 (best estimate) from Welch and Deierlein (2020).**

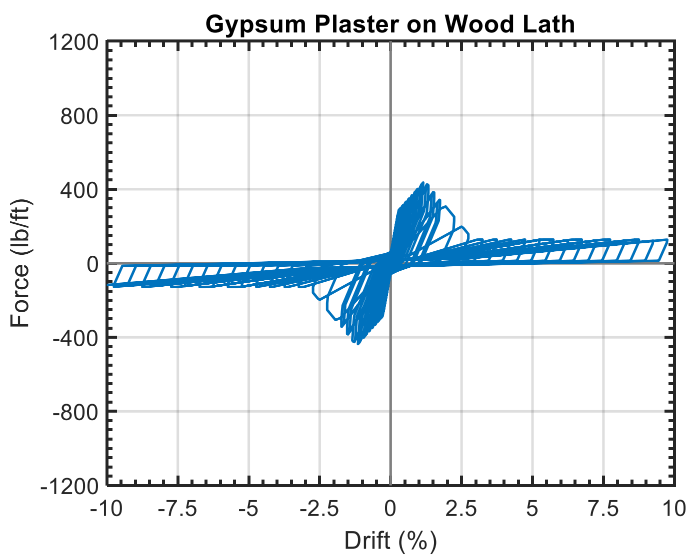


Figure 2-4 **Hysteretic input for interior strong walls of plaster on wood lath. The modeled value was doubled for sheathing on two sides of a wall. Material LP2 (best estimate) from Welch and Deierlein (2020).**

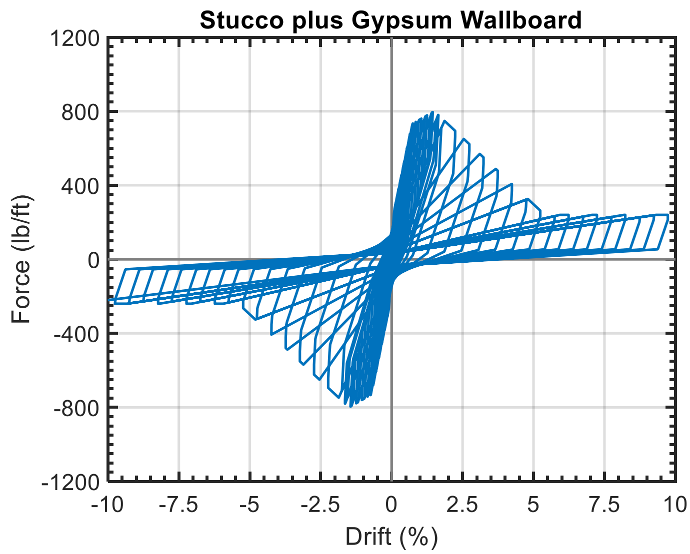


Figure 2-5 **Hysteretic input for exterior weak walls of stucco and gypsum wallboard. Material S2 (best estimate) from Welch and Deierlein (2020).**

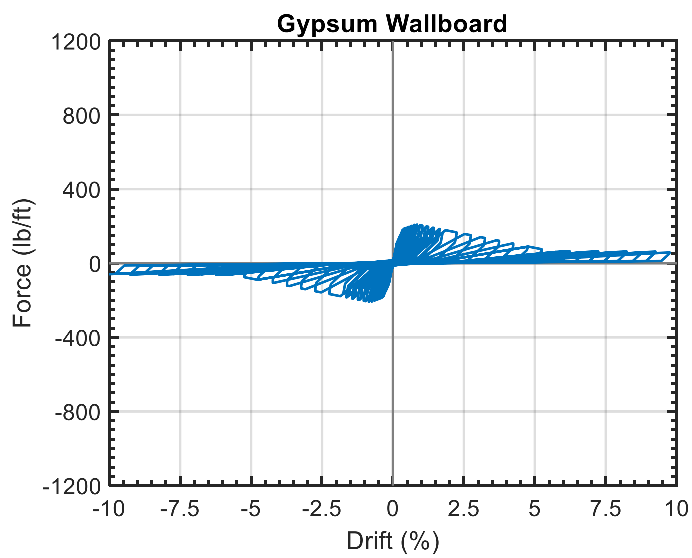


Figure 2-6 **Hysteretic input for interior weak walls of gypsum wallboard. The modeled value was doubled for sheathing on two sides of a wall. Material G2 (best estimate) from Welch and Deierlein (2020).**

2.3.2 Modeling Inputs for Diaphragms

Six types of diaphragms were used in the study: rigid (RD), strong (SD), brittle (BD), weak (WD), very weak (VWD), and lower bound (LBD). Other than the rigid case, all diaphragms were modeled as nonlinear shear springs interconnected between walls. The objective in selecting diaphragm properties was to encompass a range of properties representative of the varying building stock. The strength and stiffness of finish materials, such as hardwood floors and ceiling materials, was neglected, thereby biasing the diaphragm strengths towards lower bounds. The SD and WD diaphragm properties of the primary study archetypes were chosen to broadly represent the prevalent building stock, whereas the BD, VWD, and LBD were used to explore a range of lower bound diaphragm properties. More information about the selection of diaphragm properties, including a summary of diaphragm strengths found in a literature review, is provided in Appendix D.

The modeling inputs for the strong diaphragms comes from cyclic tests of diagonally sheathed walls (Ni and Karacabeyli, 2007). The hysteretic curves have a stronger direction where the boards are in compression, and a weaker direction where the boards are in tension. The post-peak plateaus of the modeling elements (nonlinear springs) extend beyond the tested data. The archetypes with strong diaphragms were not expected to experience significantly large displacements because they were paired with weak walls in the primary study archetypes. Moreover, the brittle diaphragm described below accounts for conditions with steep post-peak degradation. The properties are shown in Figure 2-7.

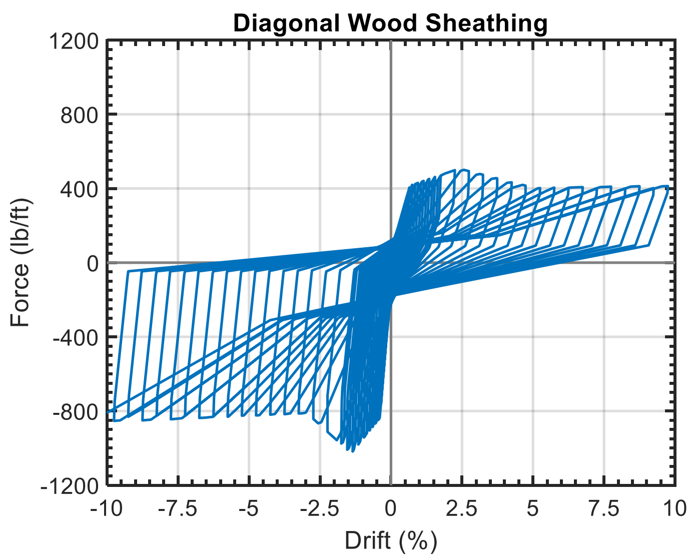


Figure 2-7 Hysteretic input for strong diaphragms derived from tests of walls with diagonal sheathing.

The modeling inputs for the brittle diaphragms were derived from diagonally sheathed diaphragms to study the effects of lower diaphragm strengths and brittle post-peak behavior. The values for this diaphragm were based on engineering judgment, in response to field observations of poor construction practices (e.g., misdriven nailing) and long-term deterioration from cracked wood due to drying. The peak strengths are half of those for the strong diaphragm. Additionally, the post-peak strength drops to zero at 5% drift. The properties are shown in Figure 2-8.

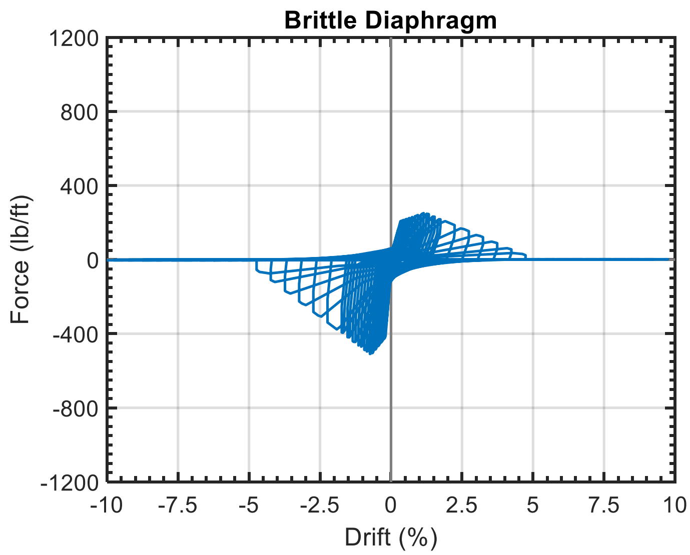


Figure 2-8 **Hysteretic input for brittle diaphragms derived from data from tests of diagonal sheathing but modified to reduce peak strength by half and reduce strength to zero at 5% drift.**

The modeling inputs for the weak diaphragms were taken from Welch and Deierlein (2020) and derived from cripple wall studies with straight sheathing. The loading for the tests was parallel to the boards. These properties were used for the diaphragm in both directions of the model. The post-peak plateau can be sustained to significant deformations in accordance with the cyclic tests. The properties are shown in Figure 2-9.

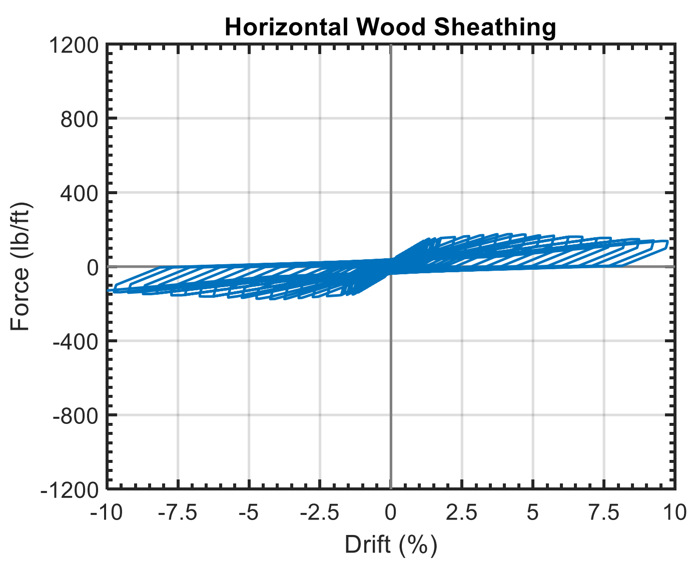


Figure 2-9 **Hysteretic input for weak diaphragms derived from cripple wall studies with straight sheathing. Material CW-HS1 from Welch and Deierlein (2020).**

The very weak diaphragm (Figure 2-10) was based on the weak diaphragm but modified to have a peak strength of 100 plf, as compared to 177 plf for the weak diaphragm.

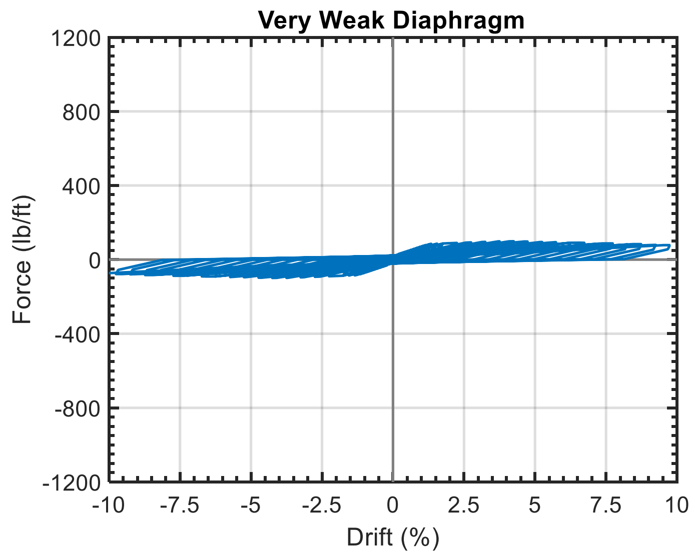


Figure 2-10 Hysteretic input for the very weak diaphragm derived from the weak diaphragm but modified to have a reduced peak strength of 100 plf.

The lower bound diaphragm was derived from the weakest diaphragm test found in the literature review. Its peak strength (60 plf) was taken as two-thirds of the weakest diaphragm test found to account for condition effects, such as poor construction and material degradation.

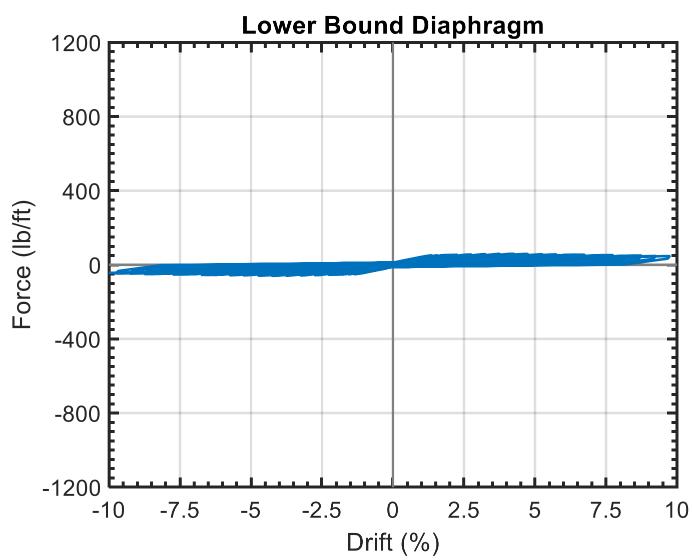


Figure 2-11 Hysteretic input for the lower bound diaphragm derived from the weakest test found and further reduced for condition effects.

2.3.3 Archetype Weight Calculations

The archetype weight calculations are shown in Table 2-2 through Table 2-6. The assemblies are the expected properties of the strong and weak wall material sets, which generally correspond to older and younger vintage SWOF buildings, respectively. The weak materials are lighter due to the gypsum wallboard partitions and ceilings. The strong materials are heavier due to the cement plaster partitions and ceilings. The change from plaster to gypsum wallboard finishes occurred in the mid-to-late 1950s (see Appendix A). Each floor assembly weight was determined from typical construction plus a 5 psf allowance for unaccounted for elements, such as the effects of contents. Windows were assumed to be 8 psf. The window areas and interior partition layout are explicit in the long-side-open archetype. For the short-side-open archetype, from the perspective of wall strength, 50% of the wall was assumed to be solid and 50% was assumed to be window. The window weight was assumed to be 85% of that for a solid wall. This high percentage accounts for the solid head and sill sections above and below the glass. The weight and strength distribution of interior partitions for the short-side-open archetype was based on wall lines that are around 50% solid, placed every 10 feet. See Figure 3-1 and Figure 3-2 for the locations and sizes of the wall segments.

Table 2-2 Floor Weights

Floor: Older Vintage (SW-WD)		Floor: Younger Vintage (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
Floor finish (7/8" hardwood)	3.6	Floor finish (carpet and pad)	1.4
Diaphragm (1" horiz. Sheathing)	2.3	Diaphragm (1" diag. sheathing)	2.3
Insulation	0.5	Insulation	0.5
M.E.P.	0.5	M.E.P.	0.5
1" plaster / wood lath ceiling	8.0	½" gypsum wall board ceiling	2.5
Joists (2×8 @ 16")	2.1	Joists (2×8 @ 16")	2.1
Tile	2.0	Tile	1.0
Miscellaneous	0.8	Miscellaneous	0.9
Added weight for contents, etc.	5.0	Added weight for contents, etc.	5.0
Total	**24.8**	**Total**	**16.2**

Table 2-3 Roof Weights

Roof: Older Vintage (SW-WD)		Roof: Younger Vintage (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
Roofing (asphalt shingles – 2 layers)	4.0	Roofing (asphalt shingles – 2 layers)	4.0
1× skip sheathing	2.0	1× skip sheathing	2.0
Insulation	0.5	Insulation	0.5

Table 2-3 Roof Weights (continued)

Roof: Older Vintage (SW-WD)		Roof: Younger Vintage (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
M.E.P.	0.5	M.E.P.	0.5
1" plaster / wood lath ceiling	8.0	½" gypsum wall board ceiling	2.5
Roof rafters (2×8 @ 24")	1.3	Roof rafters (2×8 @ 24")	1.3
Ceiling joists (2×6 @ 24")	1.0	Ceiling joists (2×6 @ 24")	1.0
Miscellaneous	0.4	Miscellaneous	0.4
Total	**17.7**	**Total**	**12.2**

Table 2-4 Interior Wall Weights

Interior Walls: Older Vintage, Heavy (SW-WD)		Interior Walls: Younger Vintage, Lighter (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
1" gypsum plaster / wood lath (2 sides)	16.0	½" gypsum wallboard (2 sides)	5.0
2×4 @ 16"	1.0	2×4 @ 16"	1.0
M.E.P.	0.5	M.E.P.	0.5
Miscellaneous	0.5	Miscellaneous	0.5
Total	**18.0**	**Total**	**7.0**

Table 2-5 Exterior Wall Weights

Exterior Walls: Older Vintage, Heavy (SW-WD)		Exterior Walls: Younger Vintage, Lighter (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
7/8" cement Stucco (1 side)	10.0	7/8" cement Stucco (1 side)	10.0
1" lumber siding and waterproofing	2.7	-	
2×4 @ 16"	1.0	2×4 @ 16"	1.0
Insulation	0.5	Insulation	0.5

Table 2-5 Exterior Wall Weights (continued)

Exterior Walls: Older Vintage, Heavy (SW-WD)		Exterior Walls: Younger Vintage, Lighter (WW-SD)	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
1" gypsum plaster / wood lath (1 side)	8.0	1/2" gypsum wall board (1 side)	2.5
Miscellaneous	0.8	Miscellaneous	0.5
Total	**23.0**	**Total**	**14.5**

Table 2-6 Archetype Weight Summary

Archetype Form		Strong Wall (kips)	Weak Wall (kips)	Archetype Form		Strong Wall (kips)	Weak Wall (kips)
LO3	Roof	120.1	72.6	SO3	Roof	121.8	74.2
	3rd flr.	202.1	115.6		3rd flr.	164.3	96.7
	2nd flr.	181.9	115.2		2nd flr.	158.9	98.9
	Total	504.1	303.4		Total	445.0	269.8
LO2	Roof	120.1	72.6	SO2	Roof	121.8	74.2
	2nd flr.	181.9	115.2		2nd flr.	158.9	98.9
	Total	302.0	187.8		Total	280.7	173.1

Note: All archetype weights can be determined from this table. For example, LO3-SW-WD and other variations of LO3-SW have total weights of 504.1 kips. LO3-WW-SD and other variations of LO3-WW have total weights of 303.4 kips. LN and SN archetype weights can be calculated by replacing the 2nd floor weights with the 3rd floor weights. For example, LN3-SW-WD has a total weight of 524.3 kips.

2.3.4 Model Configurations

The buildings were modeled with OpenSees using an assemblage of nonlinear shear springs to represent the walls, diaphragms, and retrofit frames. The retrofit frames were modeled as point springs at the second floor, centered in the open front. Figure 2-12 shows the model for the three-story, short-side-open archetype with wing walls. The X direction is parallel to the open side and the Y direction is perpendicular. The walls are shown in blue (internal) or red (exterior), depending on the type, and the diaphragm elements are shown in orange.

The tributary seismic masses were applied directly to the nodes of the models at each level (Figure 2-13). The masses act independently in the X and Y directions. For example, the tributary mass assigned to the second-floor node at C9 only acts in the X direction. The tributary mass for this node comes from the walls and floors between line A and line E, and line 8.5 and line 9. It is connected to the rest of the structure by the diaphragm link at line C, between line 8 and line 9. All the X-direction tributary masses are on the diaphragm spine at line C. The diaphragm spine responds only with shear modes. Similarly, the Y-direction tributary masses are on the diaphragm spine at line 5.

The sum of the lengths of each shear wall at a line (or elevation) is concentrated at the wall nodes. For example, the sum of shear panels below the second floor on line 9 is captured by a first-story shear spring acting in the X direction at C9. The line 9 wall is connected to the rest of the structure by a second-floor diaphragm shear spring at line C between line 9 and line 8.

Figure 2-14 shows the OpenSees assemblages of nonlinear shear springs representing walls and diaphragms for the short-side-open archetypes, and Figure 2-15 shows the OpenSees assemblages for the long-side-open archetypes.

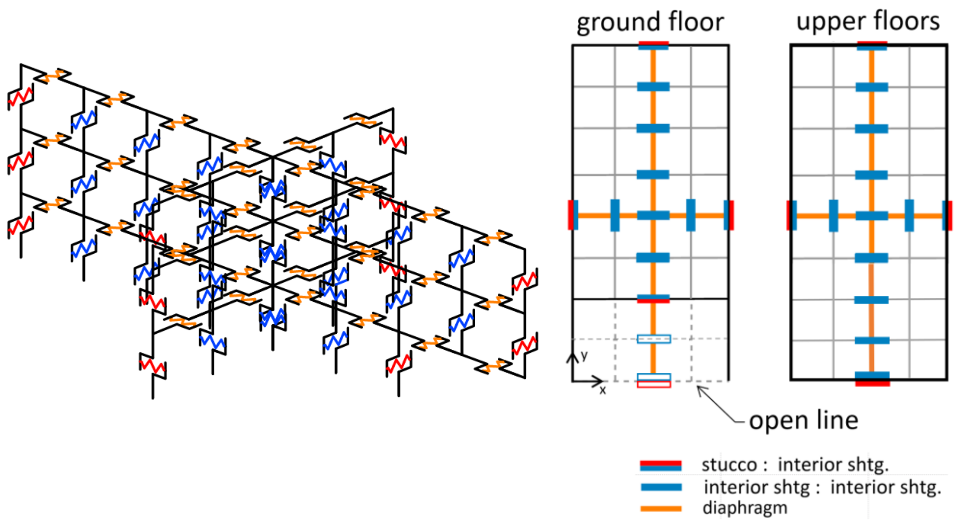


Figure 2-12 The diagram on the left is the OpenSees assemblage of nonlinear shear springs representing walls and diaphragms for the short-side-open archetype with wing walls. The plans on the right are a diagram of the walls.

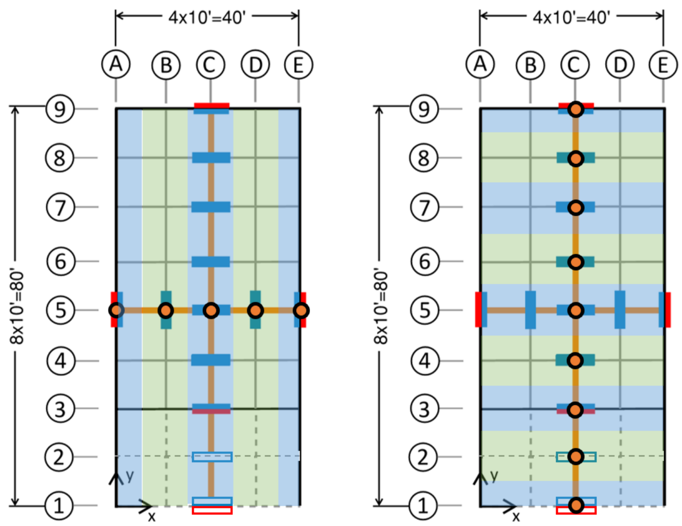


Figure 2-13 **Diagrams of tributary masses and nodes for the short-side-open archetypes. The Y-direction masses are shown in the left figure, and the X-direction masses are shown in the right figure.**

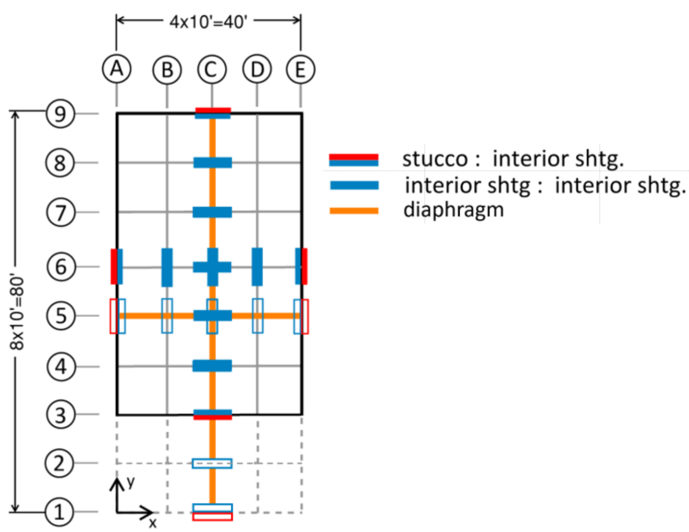


Figure 2-14 **Diagram of the OpenSees assemblages of nonlinear shear springs representing walls and diaphragms for the short-side-open archetypes.**

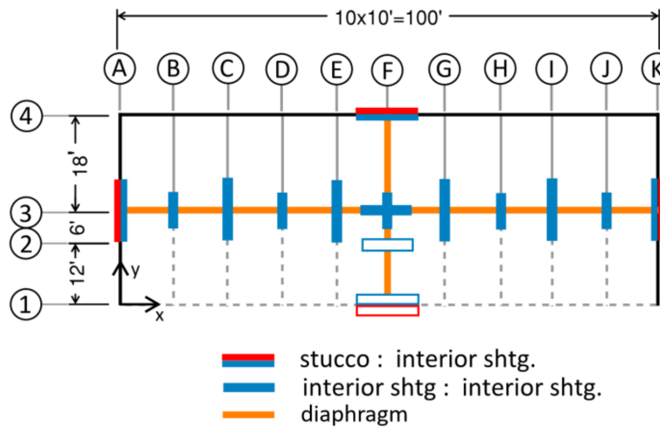


Figure 2-15 Diagram of the OpenSees assemblages of nonlinear shear springs representing walls and diaphragms for the long-side-open archetypes.

Using only shear springs for walls and diaphragms is a reasonable modeling assumption because these elements primarily respond in shear. Some of the flexural and axial effects of the elements that do occur are indirectly accounted for with tested values of their nonlinear responses. While it was not common to design for continuity in residential construction practices prior to the mid-1970s, this simplification is deemed adequate considering that the shear modes dominate distortion patterns and failure mechanisms of weak-story buildings. The rooms in multi-unit apartment buildings of the relevant vintage tend to be small, and there are numerous interconnected walls. This geometric structure, along with low aspect ratios, the stabilizing effects of gravity loads, and the relatively weak and brittle shear values, make neglecting flexural distortions of the walls reasonable for the archetypes used. Moreover, an analytical study as part of FEMA P-807 (Appendix E, Section E.3.6) isolated the impacts of neglecting flexural wall modes and found them to be modest, especially at the lower stories, which are more critical to the response. It is also reasonable to neglect flexural and axial distortions of the diaphragms due to their aspect ratios.

P-delta effects for the buildings were accounted for in the wall material models, by modifying the material backbone curves. This strategy efficiently distributes the P-delta effects at every node and minimizes numerical convergence problems at high drift levels. $V_{with\ P\text{-}\Delta,j}$ is the implemented material backbone of story j in the numerical model. $V_{initial,j}$ is the original material backbone. W_i, δ_i, and h_i are weight, lateral displacement, and height from the base of story i, respectively. The resulting equation is $V_{with\ P\text{-}\Delta,j} = V_{initial,\ j} - \sum_{i=j}^{n} W_i(\delta_i/h_i)$. A different approach was required at the open front of the archetypes, where there are no walls at the first story. In these areas, zero-lateral-stiffness leaning columns and tributary weights were added to capture the P-delta effects near the open front. The nodes are connected by the diaphragm links. For example, as the first-story wall at C9 displaces in the X direction, it directly experiences the P-delta force in proportion to its tributary weight and displacement. Static pushover tests were conducted to isolate the P-delta effect, as shown in Figure 2-16.

The retrofits were modeled with nonlinear lateral springs at the second floor of the open front (line 1). The springs represent a frame in the first story that is adequately anchored to the ground and connected to the building at the second floor. The springs, which have capacity in the X direction but no capacity in the Y direction, represent the flexural properties of two columns of an inverted moment frame. (The open front is always the X direction.) The columns are compact steel wide-flange sections embedded in a reinforced concrete grade beam. The grade beam can develop the flexural capacity of the columns, and the effects of foundation flexural stiffness were deemed to be negligible and ignored. In addition to the steel columns, the FEMA P-807 retrofits include new plywood shear walls in both X and Y directions that were modeled as nonlinear springs in the first story. See Section 2.5 for more information about how the retrofits were modeled.

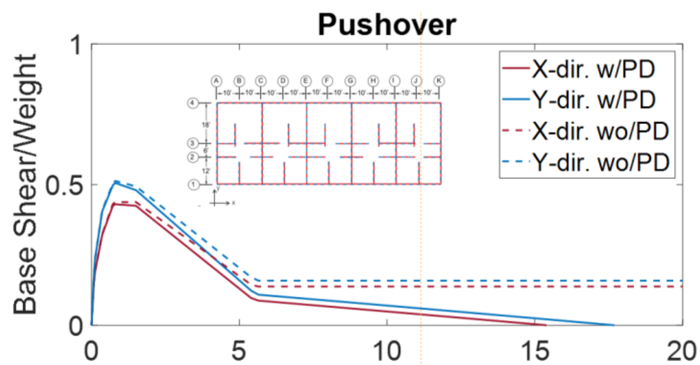


Figure 2-16 The pushover curves of a long-side-open archetype showing the P-delta effect incorporated in the material backbones. The dashed line plots do not include the P-delta effects.

2.4 Unretrofitted Archetype Capacities and Vulnerabilities

The seismic resistance of the archetype buildings is limited by multiple vulnerabilities. These are the lateral strengths in each direction, the diaphragm strength, and the torsional imbalance of the structure. A useful analogy is that of a chain with several potential weak links, where the resistance to collapse is controlled by the weakest link (Figure 2-17). The buildings were found to have multiple vulnerabilities with similar capacities. As such, mitigating one, does little to improve the building's safety. A seismic retrofit usually needs to address several or all the vulnerabilities to substantially improve safety.

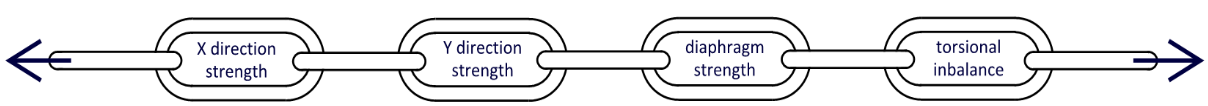
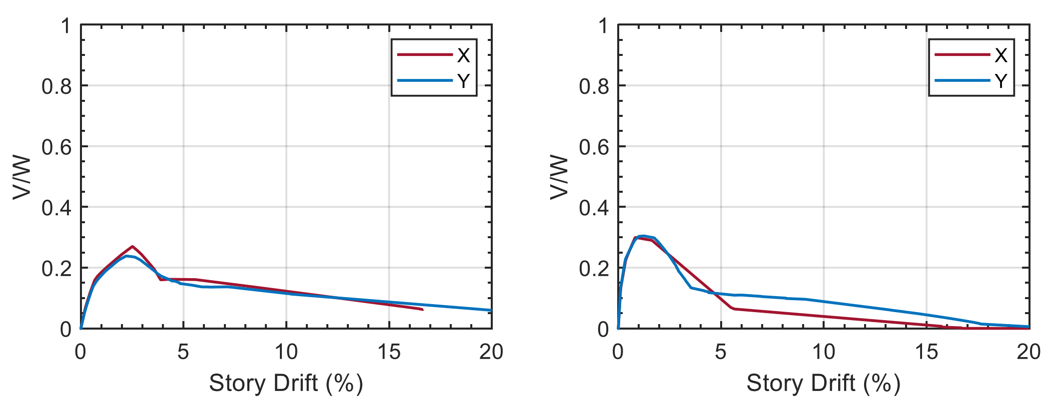


Figure 2-17 Like links in a chain, a building's lateral capacity is controlled by the weakest of several potential vulnerabilities.

2.4.1 Static Pushovers of Unretrofitted Archetypes

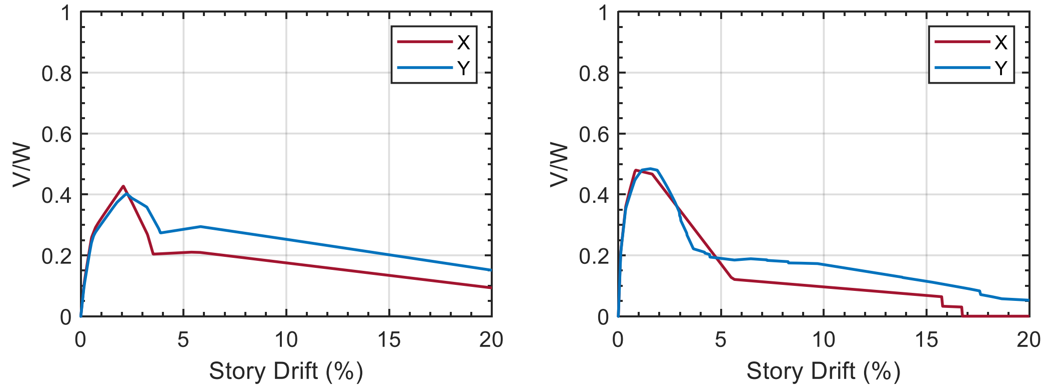
Static pushovers of the primary study archetypes in their unretrofitted condition are presented below. The most direct way to assess the building's lateral capacity is to examine the first-story strength-to-weight ratio (Figure 2-18 and Figure 2-19). Pushover studies were made of the primary archetype set. Important results to consider are as follows:

- Archetype buildings with weak walls (WW) have slightly higher maximum strength-to-weight ratios than buildings with strong walls (SW). This is because the buildings with strong walls are also heavier.
- The archetypes are brittle, with very limited ductility. All pushover curves have a steep strength loss after reaching the peak strength.
- The long-side-open (LO) archetypes have similar strength-to-weight ratios in the X and Y directions for both the strong- and weak-wall conditions. The presence of an open side does not lead to appreciable weakness in the open direction. This is because the walls adjacent to the tuck-under parking are solid, without windows, unlike the typical exterior elevations.
- The short-side-open (SO) archetypes are weaker parallel to the open front (X direction) for both wall types. The strength-to-weight ratio difference is greater with the strong walls.
- The maximum strength-to-weight ratios of the two-story buildings are significantly greater than their three-story counterparts. This is because the ground floor wall layout is the same, but the two-story building is significantly lighter because there is one fewer floor. For the long-side-open models, the two-story archetype is stronger with around 0.5g base shear-strength capacity compared to the three-story model with around 0.3g base shear-strength capacity**.**



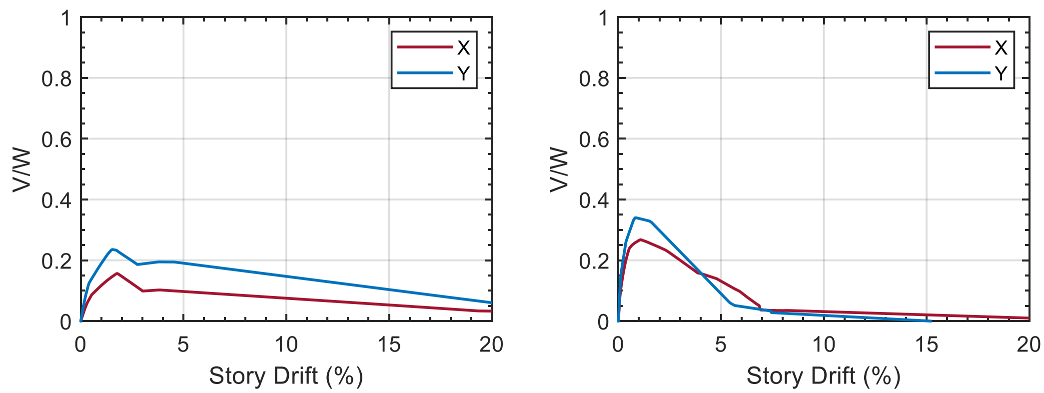
(a) LO3-SW-WD **(b) LO3-WW-SD**

Figure 2-18 Pushover curves of the unretrofitted long-side-open archetypes: (a) LO3-SW-WD, (b) LO3-WW-SD, (c) LO2-SW-WD, and (d) LO2-WW-SD.

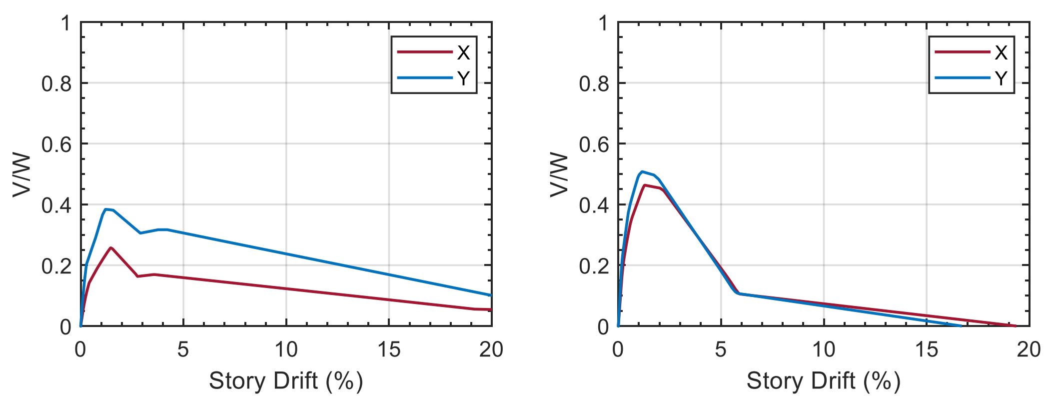


(c) LO2-SW-WD **(d) LO2-WW-SD**

Figure 2-18 Pushover curves of the unretrofitted long-side-open archetypes: (a) LO3-SW-WD, (b) LO3-WW-SD, (c) LO2-SW-WD, and (d) LO2-WW-SD. (continued)



(a) SO3-SW-WD **(b) SO3-WW-SD**



(c) SO2-SW-WD **(d) SO2-WW-SD**

Figure 2-19 Pushover curves of the unretrofitted short-side-open archetypes: (a) SO3-SW-WD, (b) SO3-WW-SD, (c) SO2-SW-WD, and (d) SO2-WW-SD.

2.4.2 Analysis of the Unretrofitted Conditions: Vulnerabilities and Failure Modes

The archetypes were subjected to seismic shaking in accordance with the FEMA P-695 protocol. A set of 22 bi-directional, far-field records were used as the inputs for incremental dynamic analysis (IDA). Each set of records was rotated 90 degrees to expand the set to 44 inputs. A common period, *T*, for evaluation was chosen for all archetypes. The value of *T*=0.25s was selected as being reasonably close for both the two-story and three-story archetypes.

The records were scaled with increasing intensities until the models were identified to have collapsed. The peak inputs usually corresponded to walls reaching 5%–10% drift.

Collapse in these analyses usually corresponded to an explicitly modeled P-delta collapse, primarily driven by the P-delta effects in the hysteretic wall spring models along with P-delta columns at the open front. In the IDA data, collapse is seen as infinite increase in drift without increase in spectral acceleration (Figure 2-20). In a few instances, models had slight gains in strength out to 20% drift, the point where the analysis was discontinued. This point was deemed to be the collapse limit, due to non-simulated limitations of the gravity system. Additional information about the application of FEMA P-695 is provided in Appendix C.

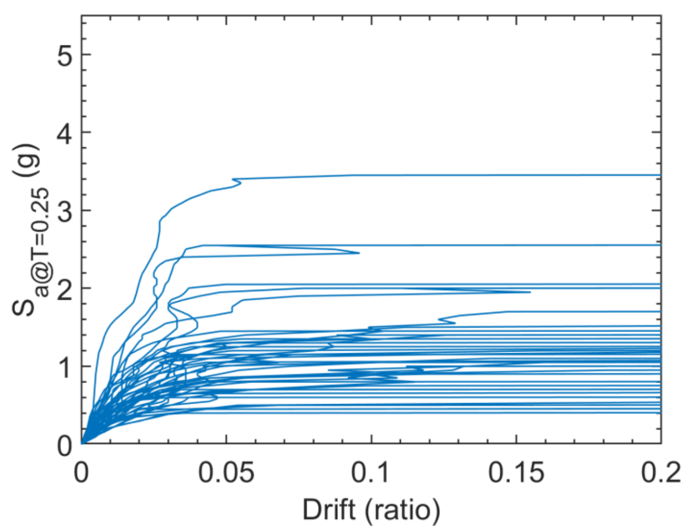


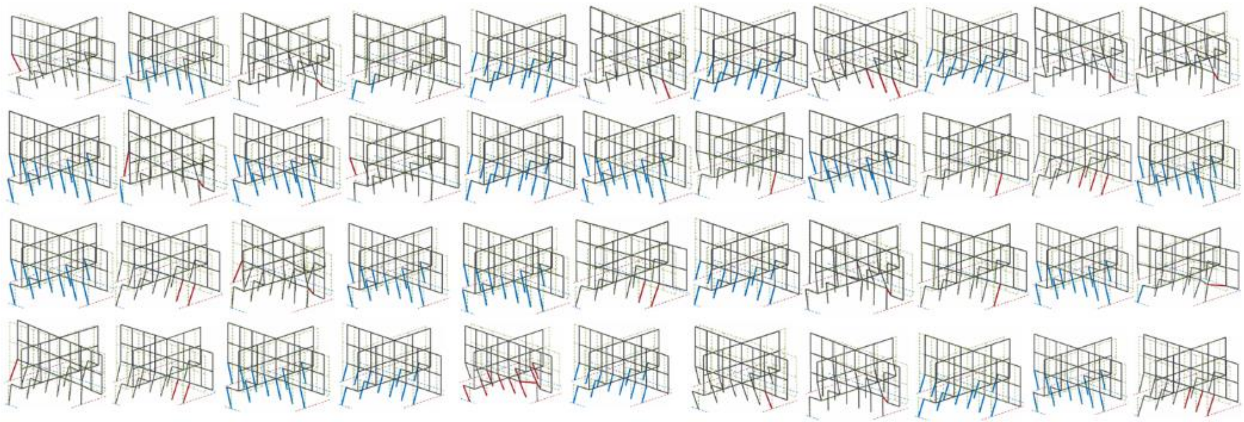
Figure 2-20 IDA data for the three-story, short-side-open, weak wall/brittle diaphragm archetype (SO3-WW-BD).

Each of the primary archetypes failed in one of multiple modes depending on the seismic record. Figure 2-21 is an array of the three-story, short-side-open, strong wall/brittle diaphragm archetype (SO3-SW-BD) at the point of near-collapse from each of the 44 earthquake records. The X-direction walls and Y-direction walls turn either red or blue when the drift reaches 20%, respectively. The images are a snapshot at the instant before collapse. The open front is shown in the lower right region of each model. This archetype experienced six distinct modes of failure. The Y-direction modes (i.e., Y and Y torsion), which are perpendicular to the open front, make up 53% of the failures.

The Y failure mode indicates that the model collapsed in a positive or negative Y direction. The Y (torsion) failure mode has significant torsion. The X-direction modes (i.e., X, X open, X non-V), which are parallel to the open front, make up 37% of the failures. The X failure mode is essentially pure translation. The X (open) failure mode has torsion that causes the walls near the open elevation to fail first. The X (non-V) failure mode also has torsion, but the walls opposite the open front (i.e., walls without open-front vulnerabilities) fail first. The D failure mode indicates diaphragm failure near the open front, which accounted for 11% of the failures.

The table also shows the distribution of failure modes for the three-story, long-side-open, strong wall/brittle diaphragm archetype (LO3-SW-BD) at the point of near-collapse. The Y-direction modes (i.e., Y and Y torsion), which are perpendicular to the open front, make up 64% of the failures. The Y (torsion) failure mode has significant torsion, and it has the highest failure rate at 52%. The X-direction mode, which is parallel to the open front, makes up the remaining 36% of the failures. Unlike the more complex behavior of the short-side open archetype, there are no failures in the X(open), X(non-V), or D modes. The percentage totals in Figure 2-21 are less than 100% due to rounding.

The diversity of failure modes is common to all the archetypes studied. The archetypes do not have a single obvious vulnerability due to the open front (a single weak link). The buildings were found to have a series of weak links (multiple vulnerabilities) with similar capacities. Consequently, a vulnerability-based retrofit program would not catch less obvious deficiencies, such as the lack of strength and brittleness in the Y direction.



Failure modes	Y	Y (torsion)	X	X (open)	X (non-V)	D
SO3-WW-BD	21	2	6	6	4	5
	48%	5%	14%	14%	9%	11%
LO3-WW-BD	5	23	16	0	0	0
	11%	52%	36%	0%	0%	0%

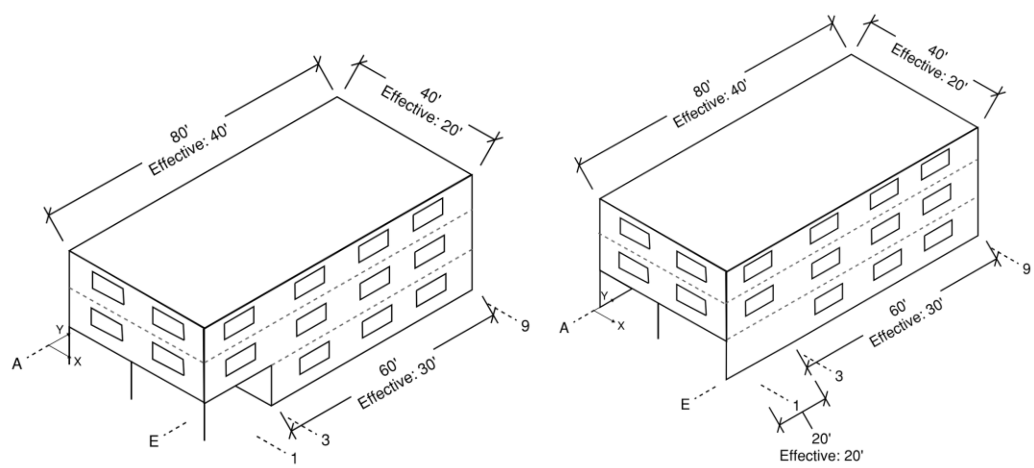
Figure 2-21 Failure modes and their distribution for the three-story, short-side-open, weak wall/brittle diaphragm archetype (SO3-WW-BD) and long-side-open, weak wall/brittle diaphragm archetype (LO3-WW-BD).

2.5 Retrofitted Archetypes

Three types of seismic retrofits were created for the archetypes: line, optimized line, and FEMA P-807. The seismic design parameters were based on ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2016), for a site near Los Angeles City Hall at 200 North Spring Street. Assuming Site Class D, the maximum considered earthquake short-period spectral acceleration, S_{MS}, is 1.979g, which corresponds to a design short-period spectral response acceleration, S_{DS}, of 1.32g. All retrofit designs follow the requirements of the Los Angeles SWOF ordinance and associated city guidelines ("Los Angeles SWOF ordinance"). The line and optimized line retrofits were designed for 75% of the design spectral acceleration, corresponding to 1.0g. The FEMA P-807 retrofits were designed for $0.5S_{MS}$, which also corresponds to 1.0g. The Los Angeles SWOF ordinance specifies that acceptable performance for FEMA P-807 retrofits is based on drifts corresponding to onset of strength loss and that the maximum drift limit probability of exceedance is 20% at the specified hazard.

The line retrofits use cantilever column moment frames along the open front. The frames are cast into reinforced concrete grade beams that are strong and stiff enough to develop the capacities of the columns in flexure. It is assumed that the tops of the frames connect to the second-floor diaphragms with collectors that are capable of developing the frames. The behavior of the inverted moment frames is idealized as fixed-base columns with the strengths and stiffnesses of the retrofit columns. The fixed-base columns are further idealized as nonlinear springs connected to the open front at the second floor. The nonlinear springs are modeled with recommended parameters for steel moment frames from Section 9.4, Figure 9-2 of ASCE/SEI 41-17. The optimized line retrofits are similar to the line retrofits except that the deflection limits are not required in their design, resulting in retrofits with lighter and more flexible frames since they are controlled by strength requirements.

Implementation of the Los Angeles SWOF ordinance has varied, due to changing interpretations over time. The line retrofits included in the primary study only provide new vertical elements along the single line that is the most obvious open front (X direction in Figure 2-22a). This extent of retrofit was used for long-side-open and short-side-open archetypes, where the latter have portions of the perimeter walls orthogonal to the open front removed (Y direction in Figure 2-22a). This is consistent with the implementation of the ordinance at certain points in time when the walls orthogonal to the obvious open front were deemed by the building department to not require retrofit. At other times, the missing orthogonal walls could have triggered the need for new vertical elements in that direction. The wing wall sensitivity study addresses the performance when the orthogonal walls are present (Figure 2-22b) or are added as part of a retrofit.



(a) Short-side-open archetype **(b) Short-side-open archetype with wing walls**

Figure 2-22 The three-story, short-side-open archetype used in the primary study (Figure a), and the three-story, short-side-open archetype with wing walls used in the wing wall sensitivity study (Figure b).

The FEMA P-807 retrofits, which were designed using the default material property values in the Weak-Story Tool, also use cantilever column moment frames along the open front. Similar to the line retrofits, the FEMA P-807 retrofits were modeled using nonlinear springs to emulate the seismic response. Unlike the line retrofits, the FEMA P-807 retrofits include plywood shear walls. The new walls were modeled in parallel with the existing walls using hysteretic properties for plywood based on Welch and Deierlein (2020). Figure 2-23 shows the first-story floor plan of a long-side-open archetype with the location of the FEMA P-807 retrofit columns and plywood shear walls. Similar to the line retrofits, secondary failure modes, such as foundation failures or collector failures, were deemed to be precluded. See Table 2-7 for more details about the modeled retrofit designs. The initial elastic periods for the retrofitted archetypes are given in Table C-18.

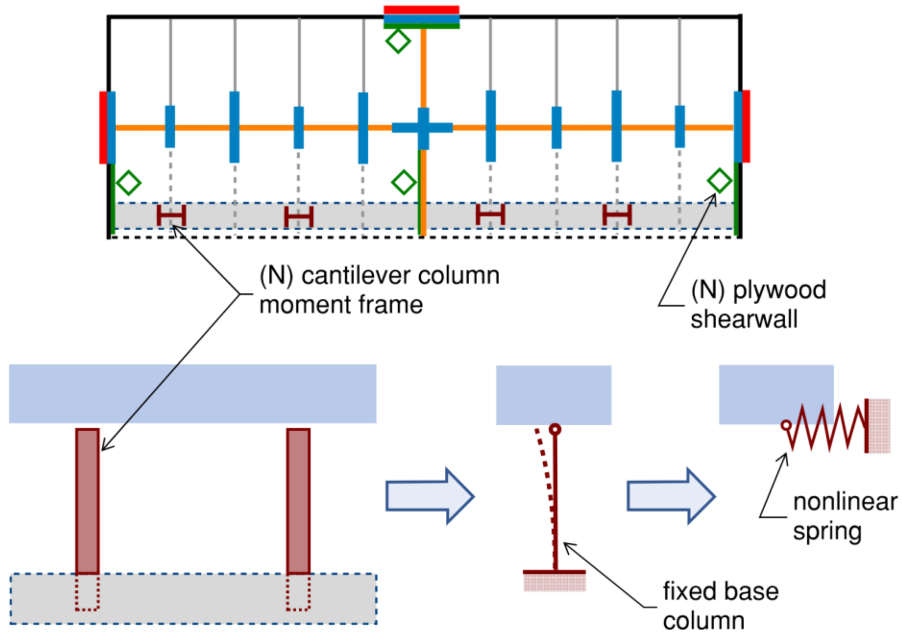


Figure 2-23 The first-story elements of the FEMA P-807 retrofit for a long-side-open archetype. The cantilever column moment frames at the open front are idealized as fixed-base columns and nonlinear springs.

Table 2-7 Selected Seismic Retrofit Parameters

Archetype	Building Seismic Weight, *W (kips)*	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
LO3-WW-SD-L	303.4	3.5	0.376	3	(4) W12x26	NA	NA
LO3-WW-SD-OL	303.4	3.5	0.376	1	(4) W10x22	NA	NA
LO3-WW-SD-P807	303.4	NA	NA	NA	(4) W10x22	20	72

Table 2-7 Selected Seismic Retrofit Parameters (continued)

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
S03-WW-SD-L	269.9	3.5	0.376	3	(2) W14x38	NA	NA
S03-WW-SD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
S03-WW-SD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
L02-WW-SD-L	187.8	3.5	0.376	3	(4) W12x19	NA	NA
L02-WW-SD-OL	187.8	3.5	0.376	1	(4) W8x18	NA	NA
L02-WW-SD-P807	187.8	NA	NA	NA	(4) W8x18	20	53
S02-WW-SD-L	173.2	3.5	0.376	3	(2) W12x19	NA	NA
S02-WW-SD-OL	173.2	3.5	0.376	1	(2) W12x16	NA	NA
S02-WW-SD-P807	173.2	NA	NA	NA	(2) W12x16	15	30

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

2.6 Analysis Results

This section presents the results of the analyses in terms of archetype probability of collapse (POC) at a spectral response acceleration, S_a, of 1.0g, which corresponds to the seismic demand for retrofits at the selected site described in Section 2.5. Pushover strength, *V*, in both orthogonal directions, normalized by weight, *W*, is provided for selected archetypes. The results are organized into groups and presented in tables. Table 2-8 and Table 2-9 present the primary study archetypes in unretroffitted and retrofitted conditions. Table 2-10 includes the three-story archetypes with no first-story open front for both long-side-open and short-side-open forms. Table 2-11 and Table 2-12 show the results of the diaphragm sensitivity study for three-story, long-side-open and three-story, short-side-open archetypes, respectively. Table 2-13 presents the results of the wing walls sensitivity study for three-story, short-side-open archetypes. The complete set of analytical results is in Appendix C.

Table 2-8 Primary Study Results for Three-Story Archetypes

Archetype	POC (%) @ S_a = 1.0g	V_x/W	V_y/W	Wall	Diaphragms	Retrofit
Long-Side-Open Archetypes						
LO3-SW-WD	22	0.29	0.24	strong	weak	-
LO3-SW-WD-L	18	0.43	0.24	strong	weak	line
LO3-SW-WD-OL	17	0.42	0.24	strong	weak	opt. line
LO3-SW-WD-P807	14	0.45	0.36	strong	weak	P807
LO3-WW-SD	27	0.30	0.30	weak	strong	-
LO3-WW-SD-L	18	0.49	0.30	weak	strong	line
LO3-WW-SD-OL	19	0.44	0.30	weak	strong	opt. line
LO3-WW-SD-P807	8	0.52	0.52	weak	strong	P807
Short-Side-Open Archetypes						
SO3-SW-WD	38	0.15	0.30	strong	weak	-
SO3-SW-WD-L	37	0.25	0.30	strong	weak	line
SO3-SW-WD-OL	38	0.25	0.30	strong	weak	opt. line
SO3-SW-WD-P807	12	0.25	0.33	strong	weak	P807
SO3-WW-SD	27	0.26	0.34	weak	strong	-
SO3-WW-SD-L	24	0.33	0.34	weak	strong	line
SO3-WW-SD-OL	24	0.33	0.34	weak	strong	opt. line
SO3-WW-SD-P807	13	0.35	0.42	weak	strong	P807

Table 2-9 Primary Study Results for Two-Story Archetypes

Archetype	POC (%) @ S_a = 1.0g	V_x/W	V_y/W	Wall	Diaphragms	Retrofit
Long-Side-Open Archetypes						
LO2-SW-WD	10	0.43	0.40	strong	weak	-
LO2-SW-WD-L	8	0.69	0.40	strong	weak	line
LO2-SW-WD-OL	8	0.58	0.40	strong	weak	opt. line
LO2-SW-WD-P807	6	0.72	0.56	strong	weak	P807
LO2-WW-SD	12	0.49	0.49	weak	strong	-
LO2-WW-SD-L	8	0.78	0.49	weak	strong	line
LO2-WW-SD-OL	8	0.64	0.49	weak	strong	opt. line
LO2-WW-SD-P807	4	0.84	0.83	weak	strong	P807
Long-Side-Open Archetypes						
SO2-SW-WD	24	0.25	0.39	strong	weak	-
SO2-SW-WD-L	20	0.42	0.39	strong	weak	line
SO2-SW-WD-OL	20	0.40	0.53	strong	weak	opt. line
SO2-SW-WD-P807	8	0.39	0.53	strong	weak	P807
SO2-WW-SD	18	0.45	0.52	weak	strong	-
SO2-WW-SD-L	15	0.58	0.52	weak	strong	line
SO2-WW-SD-OL	15	0.56	0.52	weak	strong	opt. line
SO2-WW-SD-P807	9	0.56	0.66	weak	strong	P807

Table 2-10 Results for Three-Story Archetypes with No First-Story Open Front

Archetype	POC (%) @ S_a = 1.0g	V_x/W	V_y/W	Wall	Diaphragms	Retrofit
LN3-SW-RD	29	0.46	0.58	strong	rigid	-
LN3-SW-WD	36	0.27	0.21	strong	weak	-
LN3-WW-RD	19	0.46	0.53	weak	rigid	-
LN3-WW-SD	19	0.32	0.33	weak	strong	-
SN3-SW-RD	38	0.36	0.42	strong	rigid	-
SN3-SW-WD	42	0.23	0.30	strong	weak	-
SN3-WW-RD	28	0.33	0.41	weak	rigid	-
SN3-WW-SD	28	0.26	0.33	weak	strong	-

Table 2-11 Diaphragm Sensitivity Study Results for Three-Story, Long-Side-Open Archetypes with Weak Walls

Archetype	POC (%) @ S_a = 1.0g	Wall	Diaphragms	Retrofit
LO3-WW-RD	27	weak	rigid	-
LO3-WW-BD	28	weak	brittle	-
LO3-WW-VWD	22	weak	very weak	-
LO3-WW-BD-L	21	weak	brittle	line
LO3-WW-VWD-L	13	weak	very weak	line
LO3-WW-BD-P807	19	weak	brittle	P807
LO3-WW-VWD-P807	9	weak	very weak	P807

Table 2-12 Diaphragm Sensitivity Study Results for Three-story, Short-Side-Open Archetypes with Weak Walls

Archetype	POC (%) @ S_a = 1.0g	Wall	Diaphragms	Retrofit
SO3-WW-RD	29	weak	rigid	-
SO3-WW-SD	27	weak	strong	-
SO3-WW-BD	28	weak	brittle	-
SO3-WW-WD	29	weak	weak	
SO3-WW-VWD	33	weak	very weak	-
SO3-WW-LBD	41	weak	lower bound	-
SO3-WW-BD-L	25	weak	brittle	line
SO3-WW-BD-OL	25	weak	brittle	opt. line
SO3-WW-BD-P807	17	weak	brittle	P807
SO3-WW-WD-L	25	weak	weak	line
SO3-WW-WD-OL	25	weak	weak	opt. line
SO3-WW-WD-P807	12	weak	weak	P807
SO3-WW-VWD-L	25	weak	very weak	line
SO3-WW-VWD-OL	24	weak	very weak	opt. line
SO3-WW-VWD-P807	14	weak	very weak	P807
SO3-WW-LBD-L	22	weak	lower bound	line
SO3-WW-LBD-OL	21	weak	lower bound	opt. line
SO3-WW-LBD-P807	18	weak	lower bound	P807

Figure 2-13 Wing Walls Sensitivity Study Results for Three-Story, Short-Side-Open Archetypes with Weak Walls

Archetype	POC (%) @ S_a = 1.0g	Wall	Diaphragms	Retrofit
SOW3-WW-SD	21	weak	strong	-
SOW3-WW-BD	22	weak	brittle	-
SOW3-WW-SD-L	16	weak	strong	line

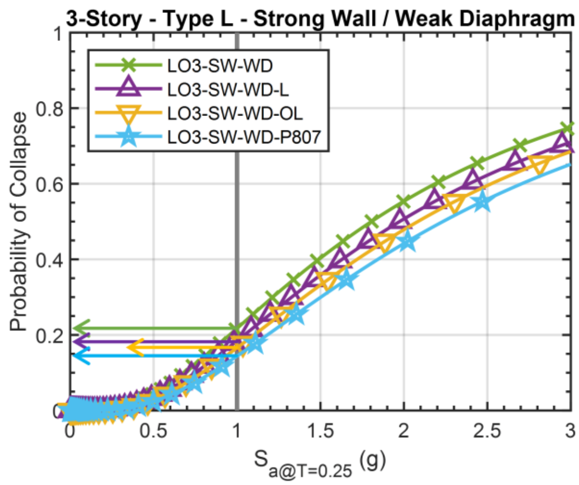
Figure 2-13 Wing Walls Sensitivity Study Results for Three-Story, Short-Side-Open Archetypes with Weak Walls (continued)

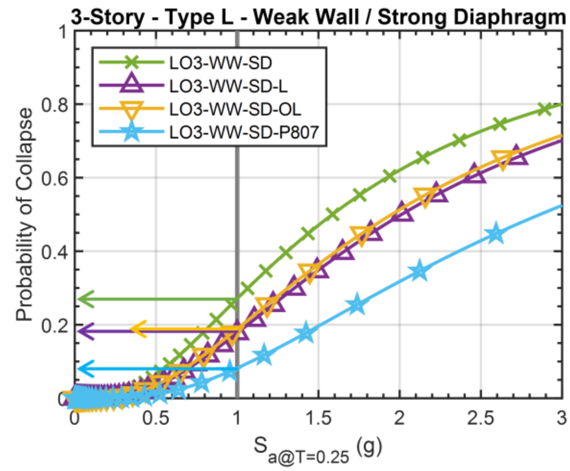
Archetype	POC (%) @ S_a = 1.0g	Wall	Diaphragms	Retrofit
SOW3-WW-BD-L	19	weak	brittle	line
SOW3-WW-SD-OL	16	weak	strong	opt. line
SOW3-WW-SD-P807	15	weak	strong	P807

2.6.1 Retrofit Effectiveness of Long-Side-Open Archetypes: Primary Study

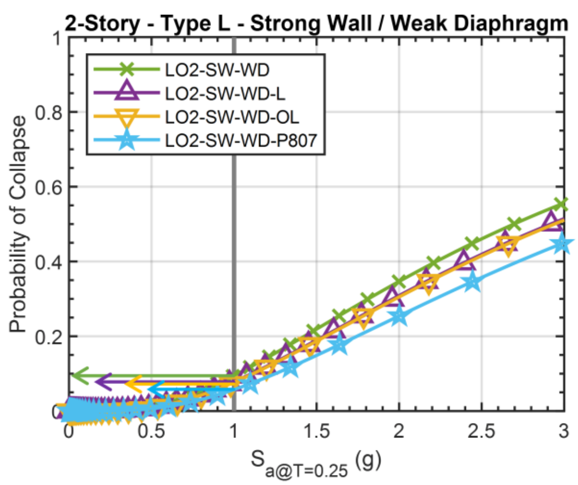
Fragility function plots of probability of collapse versus mean spectral response acceleration for the long-side-open archetypes of the primary study are provided in Figure 2-24. The following trends are noted:

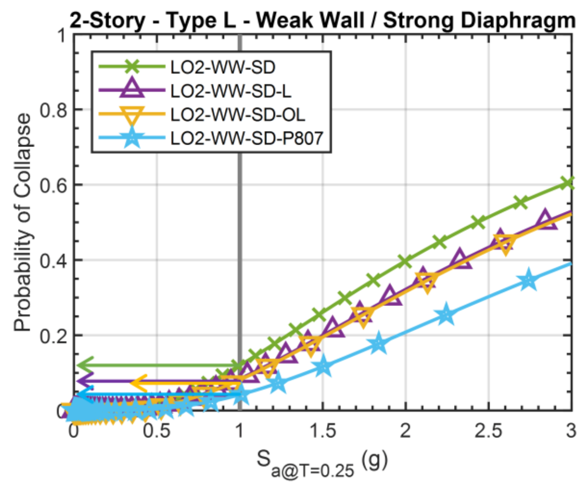
- The FEMA P-807 retrofits, which were designed with a 20% probability of exceedance target (20% POE), had probabilities of collapse (POC) of less than 20%.
- The optimized line retrofit achieved the same benefit as its line retrofit counterpart, in all configurations.
- For the weak-wall archetype (LO3-WW-SD) the probability of collapse of the unretrofitted archetype was 27%. For the line retrofit, the probability of collapse was reduced to 18%. For the FEMA P-807 retrofit, the probability of collapse was reduced to 8%.
- For the strong-wall archetype (LO3-SW-WD) the probability of collapse of the unretrofitted archetype was 22%. For the line retrofit, the probability of collapse was reduced to 18%. For the FEMA P-807 retrofit, the probability of collapse was reduced to 14%. The two-story archetypes were significantly less vulnerable and safer in the unretrofitted conditions, as compared to their three-story counterparts with similar configurations. This difference is explained by the lighter two-story archetypes having higher strength-to-weight ratios, and thus higher base shear capacities, than their three-story counterparts.
- For the weak-wall archetype (LO2-WW-SD) the probability of collapse of the unretrofitted archetype was 12%. For the line retrofits, the probabilities of collapse were reduced to 8% for the line retrofit. For the FEMA P-807 retrofit, the probability of collapse was reduced to 4%.
- For the strong-wall archetype (LO2-SW-WD) the probability of collapse of the unretrofitted archetype was 10%. For the line retrofits, the probabilities of collapse were reduced to 8% for the line retrofit. For the FEMA P-807 retrofit, the probability of collapse was reduced to 6%.





(a) LO3-SW-WD **(b) LO3-WW-SD**





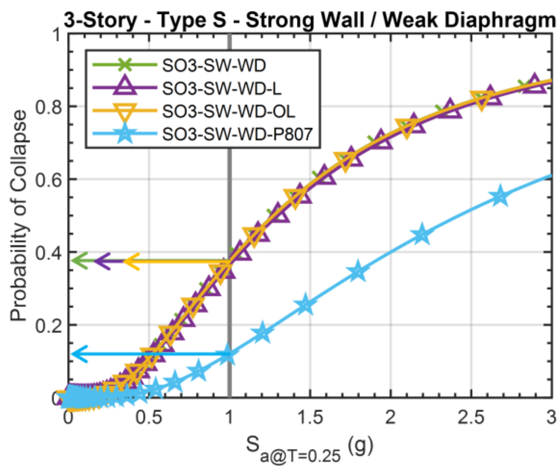
(c) LO2-SW-WD **(d) LO2-WW-SD**

Figure 2-24 Probability of Collapse plots for the (a) LO3-SW-WD, (b) LO3-WW-SD, (c) LO2-SW-WD, and (d) LO2-WW-SD archetypes in their unretrofitted conditions and with line, optimized line, and FEMA P-807 seismic retrofits.

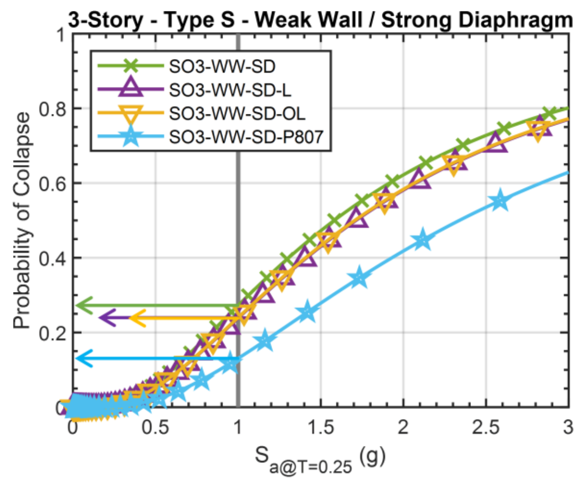
2.6.2 Retrofit Effectiveness of Short-Side-Open Archetypes: Primary Study

Fragility function plots of probability of collapse versus mean spectral response acceleration for the short-side-open archetypes of the primary study are provided in Figure 2-25. The following trends are noted:

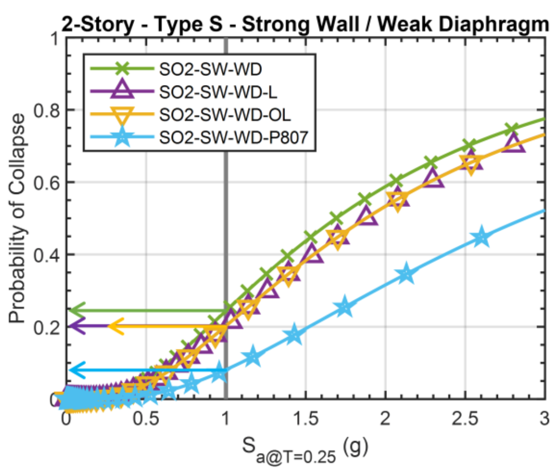
- The FEMA P-807 retrofits, which were designed with a 20% probability of exceedance target (20% POE), had probabilities of collapse of less than 20%.
- The optimized line retrofit achieved the same benefit as its line retrofit counterpart, in all configurations.
- For the weak-wall archetype (SO3-WW-SD) the probability of collapse of the unretrofitted archetype was 27%. For the line retrofits, the probabilities of collapse were slightly reduced to 24%. For the FEMA P-807 retrofit, the probability of collapse was reduced to 13%.
- For the strong-wall archetype (SO3-SW-WD) the probability of collapse of the unretrofitted archetype was 38%. For the line retrofit, the probability of collapse was slightly reduced to 37%. For the optimized line retrofit, the probability of collapse was unchanged from the unretroffitted condition. For the FEMA P-807 retrofit, the probability of collapse was reduced to 8%.
- The two-story archetypes were significantly less vulnerable and safer in the unretrofitted conditions, as compared to their three-story counterparts with similar configurations, due to the higher strength-to-weight ratios (higher base shear capacities).
- For the weak-wall archetype (SO2-WW-SD) the probability of collapse of the unretrofitted archetype was 18%. The probabilities of collapse were reduced to 15% for the line retrofit. For the FEMA P-807 retrofit, the probability of collapse was reduced to 9%. For the strong-wall archetype (SO2-SW-WD) the probability of collapse of the unretrofitted archetype was 24%. The probabilities of collapse were reduced to 20% for the line retrofit. For the FEMA P-807 retrofit, the probability of collapse was reduced to 12%.



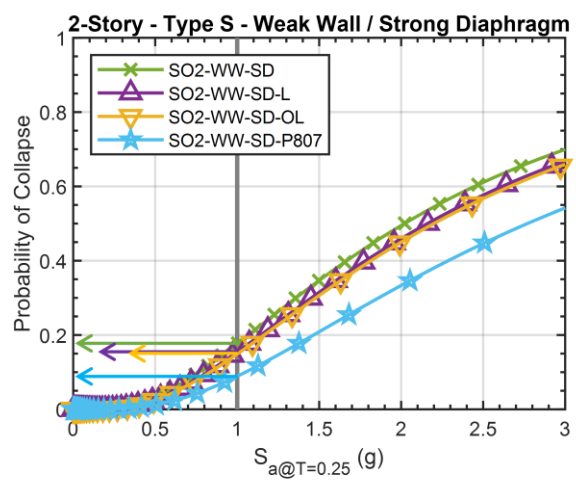
(a) SO3-SW-WD



(b) SO3-WW-SD



(c) SO2-SW-WD



(d) SO2-WW-SD

Figure 2-25 Probability of collapse plots for the (a) SO3-SW-WD, (b) SO3-WW-SD, (c) SO2-SW-WD, and (d) SO2-WW-SD archetypes in their unretrofitted conditions and with line, optimized line, and FEMA P-807 seismic retrofits.

2.6.3 Primary Study Performance Summary

This section summarizes the analytical results of the primary study in tabular (Table 2-14) and graphical (Figure 2-26) forms. The following trends are noted:

- FEMA P-807 retrofits are effective with results better than 20% POC.
- Line and optimized line retrofits do not consistently improve safety. Three-story, long-side-open archetypes show moderate improvements. Short-side-open archetypes show limited improvements.

- Line and optimized line retrofits provided similar results for a given archetype.
- Three-story archetypes are more vulnerable than their two-story counterparts.
- Short-side-open archetypes are usually more vulnerable than their long-side-open counterparts.

Table 2-14 Probabilities of Collapse (%) at S_a = 1.0g for the Primary Study Archetypes

Archetype	Stories	Unretrofitted	Line	Optimized Line	P807
LO3-SW-WD	3	22	18	17	14
LO3-WW-SD	3	27	18	19	8
SO3-SW-WD	3	38	37	38	12
SO3-WW-SD	3	27	24	24	13
LO2-SW-WD	2	10	8	8	6
LO2-WW-SD	2	12	8	8	4
SO2-SW-WD	2	24	20	20	8
SO2-WW-SD	2	18	15	15	9

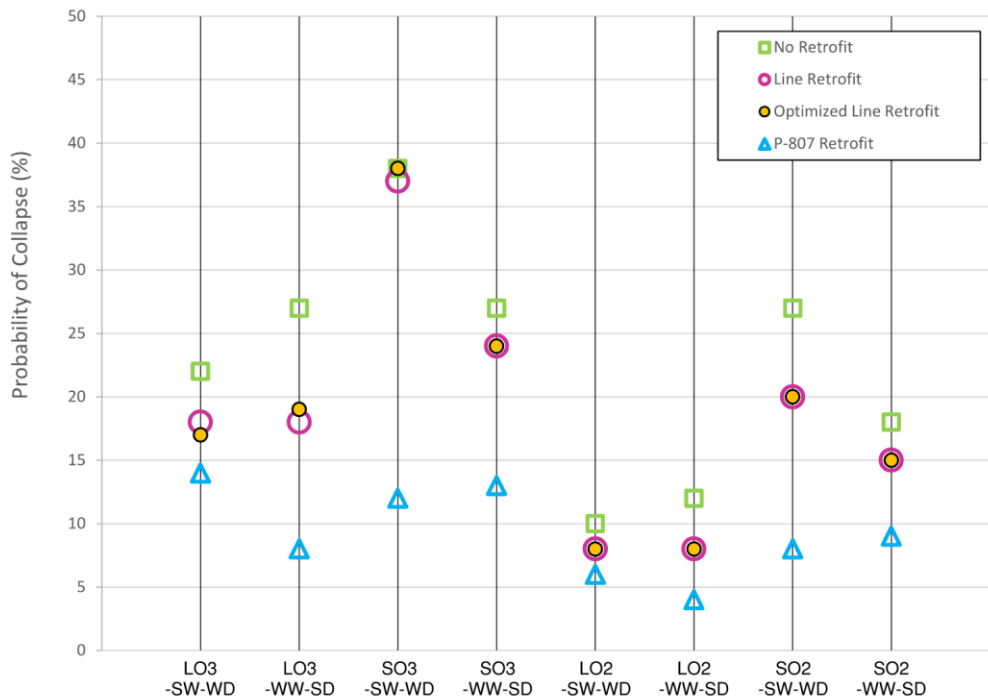


Figure 2-26 Probabilities of collapse (%) at S_a = 1.0g for the primary study archetypes.

2.6.4 Isolating the Effects of Wing Walls on Short-Side-Open Archetypes

This section summarizes the analytical results of the wing walls sensitivity study in tabular (Table 2-15) and graphical (Figure 2-27) forms. The short-side-open archetypes with wing walls (SOW) are stronger in the Y direction (orthogonal to the open front) than the typical short-side-open archetypes because of the longer first-floor walls. The following trends were noted:

- The unretrofitted archetypes with wing walls had lower probabilities of collapse than their counterparts without wing walls. This was likely due to reducing the collapse potential of the wing-wall archetypes in the Y direction.
- The line and optimized line retrofits were more effective with the wing-wall archetypes as compared to their counterparts without wing walls. This was likely due to wing-wall archetypes having a dominant X-direction failure mode that was mitigated with the line or optimized line retrofits.
- For the weak-wall wing wall archetype (SOW3-WW-SD) the probability of collapse of the unretrofitted archetype was 21%. For the line retrofits, the probabilities of collapse were reduced to 16%. For the FEMA P-807 retrofit, the probability of collapse was reduced to 15%. The retrofits using FEMA P-807 were effective for both of the wing-wall and non-wing-wall archetypes, yielding similar improvements. The non-wing-wall archetype performance was slightly better (13% to 15%). This was likely due to the non-wing-wall archetype having more added strengthening in the Y direction. The added strengthening was with plywood sheathing, and this is more ductile and less brittle than the stucco finish of the wing walls.

Table 2-15 Probabilities of Collapse (%) at S_a = 1.0g for Short-Side-Open Archetypes with and without Wing Walls

Archetypes with Wing Walls		Archetypes without Wing Walls	
Archetype	**POC (%) @ S_a = 1.0g**	**Archetype**	**POC (%) @ S_a = 1.0g**
SOW3-WW-SD	21	SO3-WW-SD	27
SOW3-WW-BD	22	SO3-WW-BD	28
SOW3-WW-SD-L	16	SO3-WW-SD-L	24
SOW3-WW-BD-L	19	SO3-WW-BD-L	25
SOW3-WW-SD-OL	16	SO3-WW-SD-OL	24
SOW3-WW-SD-P807	15	SO3-WW-SD-P807	13

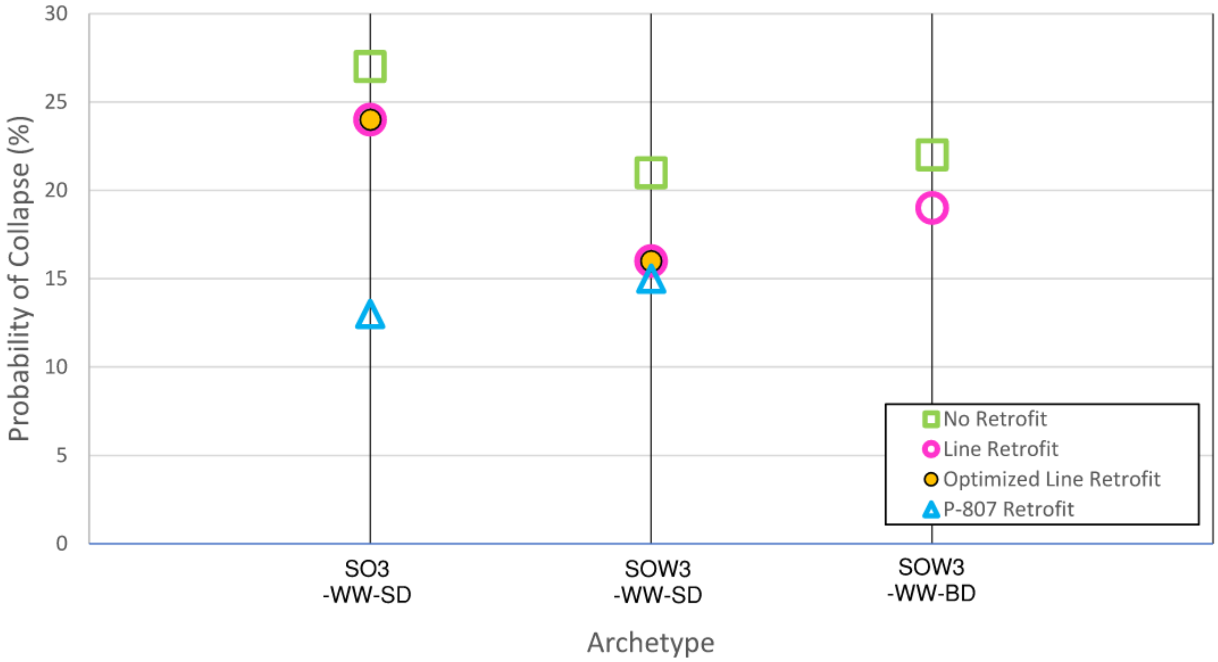


Figure 2-27 Probabilities of collapse (%) at S_a = 1.0g for short-side-open archetypes with and without wing walls.

2.6.5 Assessing the Performance of Archetypes without Open-Front Vulnerabilities

This section summarizes the results of the sensitivity study that investigated the performance of three-story archetypes without open-front vulnerabilities (LN or SN) compared to their SWOF counterparts (Table 2-16). The archetypes without open-front vulnerabilities tend to have high probabilities of collapse due to weak and brittle walls. SWOF archetypes are missing the first-floor walls at the open-front elevation, but the missing walls (or strength) are offset by solid walls, with no windows, adjacent to the parking. Examining the total length of first-floor walls of both forms of archetypes shows that the collapse capacities follow the trend of strength-to-weight ratios, shown in Table 2-16 as V_x/W and V_y/W for the X direction and Y direction, respectively.

The LN3 archetypes generally have lower probabilities of collapse than their SWOF counterparts, whereas the SN3 archetypes have collapse capacities that are nearly equivalent to their SWOF counterparts. The trends between the long-side-open and short-side-open archetypes tend to align with the first-floor strength-to-weight ratios.

The trends of collapse rates tracking first-floor strength-to-weight ratios are well aligned in the cases with rigid diaphragms (RD) and strong diaphragms (SD), where diaphragm failure modes are limited. In the weak diaphragm cases, diaphragm failure modes lead to more complex responses, and the trends are less clear.

Table 2-16 Probabilities of Collapse (%) of Three-Story Archetypes with No First-Story Open-Front Vulnerabilities and their SWOF Counterparts

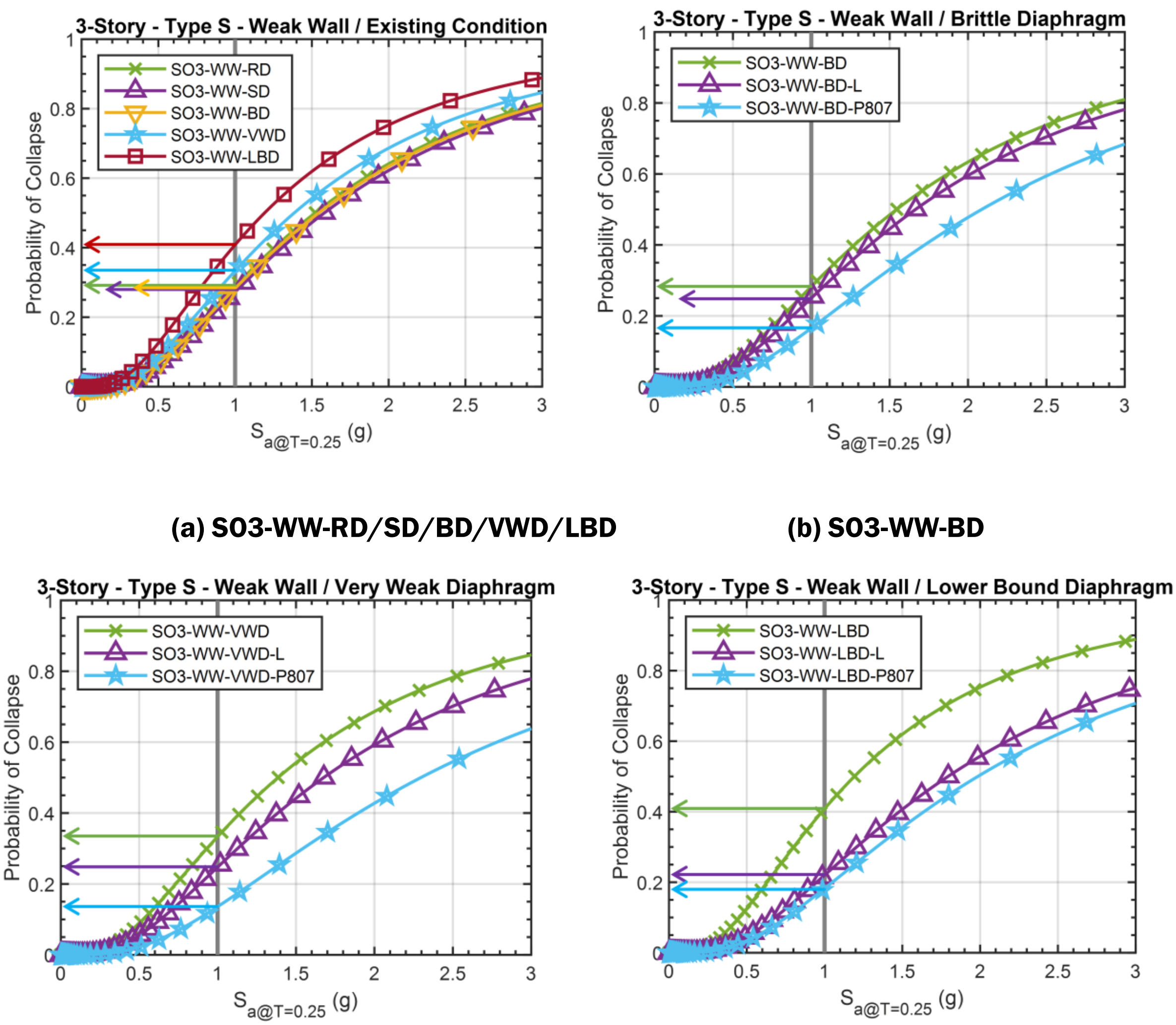
No First-Story Open-Front Vulnerability				SWOF			
Archetype	V_x/W	V_y/W	POC (%) @ S_a = 1.0g	Archetype	V_x/W	V_y/W	POC (%) @ S_a = 1.0g
LN3-SW-RD	0.46	0.58	29	LO3-SW-RD	0.34	0.35	36
LN3-SW-WD	0.27	0.21	36	LO3-SW-WD	0.29	0.24	22
LN3-WW-RD	0.46	0.53	19	LO3-WW-RD	0.34	0.32	27
LN3-WW-SD	0.32	0.33	19	LO3-WW-SD	0.30	0.30	27
SN3-SW-RD	0.36	0.42	38	SO3-SW-RD	0.33	0.32	39
SN3-SW-WD	0.23	0.30	42	SO3-SW-WD	0.15	0.25	38
SN3-WW-RD	0.33	0.41	28	SO3-WW-RD	0.33	0.31	29
SN3-WW-SD	0.26	0.33	28	SO3-WW-SD	0.26	0.34	27

2.6.6 Isolating the Effects of Diaphragms

This section summarizes the results of the sensitivity study that investigated variations of weak diaphragms to better understand their potential impacts on seismic performance. A common concern is that the diaphragm adjacent to the open front could be a critical weak link in the flow of seismic forces. The brittle diaphragm (BD) can be conceived of as a lower-bound form of the diagonally sheathed strong diaphragm (SD). The brittle diaphragm has half of the strength of the strong diaphragm, and the strength drops to zero at 5% drift. The very weak diaphragm (VWD) has a peak strength of 100 plf and is a weaker form of the straight-sheathed weak diaphragm (WD). The lower-bound diaphragm (LBD) has a peak strength of 60 plf, is based on the weakest tested strength data, and is further reduced for condition effects.

Additional variations of the weak diaphragms (WD-s), very weak diaphragms (VWD-s), and lower - bound diaphragms (LBD-s) were studied based on shifting interior wall positions of the three-story, short-side-open forms. The upper-story walls at line 3 (Figure 2-13) are offset towards the open front at line 1. This shift increases critical diaphragm demands as it accounts for potential detrimental effects of internal wall locations. The archetypes studied are SO3-WW-WD-s, SO3-WW-VWD-s, SO3-WW-LBD-s, SO3-SW-WD-s, and SO3-SW-VWD-s. The effect of wall position was minor. The archetypes with shifted walls had slightly higher rates of collapse compared to their non-shifted counterparts.

The three-story, short-side-open condition with weak walls (SO3-WW) was studied. Figure 2-28(a) shows that the archetype is only moderately sensitive to diaphragm capacities, from rigid (RD) to very weak (VWD). The very weak diaphragm has a somewhat higher rate of collapse, at 33%, as compared to the forms with either the rigid diaphragm, the strong diaphragm, or the brittle diaphragm, all at around 28%. The lower-bound diaphragm (LBD) has a significant effect, with a probability of collapse of 41%. The type of the diaphragm had a modest effect on the FEMA P-807 retrofitted results. The line retrofits improve somewhat with the weaker diaphragms. Figures 2-28(b–d) show the performance of the SO3-WW archetypes in their unretrofitted and retrofitted conditions.



(a) SO3-WW-RD/SD/BD/VWD/LBD **(b) SO3-WW-BD**

(c) SO3-WW-VWD **(d) SO3-WW-LBD**

Figure 2-28 Probability of collapse plots for (a) unretrofitted SO3-WW archetypes with all diaphragm variations, (b) retrofitted SO3-WW archetypes with brittle diaphragms, (c) retrofitted SO3-WW archetypes with very weak diaphragms, and (d) retrofitted SO3-WW archetypes with lower-bound diaphragms.

Results are summarized for the diaphragm sensitivity study in Table 2-17 and Figure 2-29. The following trends are noted:

- The unretrofitted archetypes have progressively higher probabilities of collapse with weaker diaphragms. This is especially true for models with lower-bound diaphragms. In these cases, the extreme diaphragm weakness compromized the load paths between the upper and lower walls.
- The FEMA P-807 retrofits are generally effective with the weak diaphragms, and the results are somewhat insensitive to the diaphragm types. Eighteen of nineteen archeteypes performed well.
- The probabilities of collapse of line and optimized line retrofits are somewhat insensitive to the diaphragm types, up to and including the very weak diaphragm. The retrofits are more effective with the lower-bound diaphragm.
- The effects of shifting walls positions on the three-story, short-side-open archetypes have a small-to-negligiable detrimental effect on collapse capacities, both in the unretrofitted and retrofitted states.
- The line and optimized line retrofits are somewhat ineffective in improving performance of the three-story, short-side-open archetypes up to and including the weak diaphragm. The improvement is moderate for the very weak diaphragm but significant for the lower-bound diaphragm.
- The line and optimized line retrofit results are nearly equivalent for each archetype.

Table 2-17 Probabilities of Collapse (%) at S_a = 1.0g for the Diaphragm Sensitivity Study Archetypes

Archetype	Stories	Unretrofitted	Line	Optimized Line	P807
SO3-WW-RD	3	29	-	-	-
SO3-WW-SD	3	27	24	24	13
SO3-WW-BD	3	28	25	25	17
SO3-WW-WD	3	29	25	25	12
SO3-WW-WD-s	3	30	26	26	13
SO3-WW-VWD	3	33	25	24	14
SO3-WW-VWD-s	3	34	27	25	15
SO3-WW-LBD	3	41	22	21	18
SO3-WW-LBD-s	3	42	22	21	18
SO3-SW-WD	3	38	37	38	12

Table 2-17 Probabilities of Collapse (%) at S_a = 1.0g for the Diaphragm Sensitivity Study Archetypes (continued)

Archetype	Stories	Unretrofitted	Line	Optimized Line	P807
SO3-SW-WD-s	3	39	37	38	13
SO3-SW-VWD	3	43	35	36	15
SO3-SW-VWD-s	3	45	36	37	16
SO3-SW-LBD	3	56	36	36	20
LO3-WW-RD	3	27	-	-	-
LO3-WW-SD	3	27	18	19	8
LO3-WW-BD	3	28	21	-	19
LO3-WW-VWD	3	22	13	-	9
LO3-SW-WD	3	22	18	17	14
LO3-SW-VWD	3	34	21	19	20
LO3-SW-LBD	3	57	26	21	25

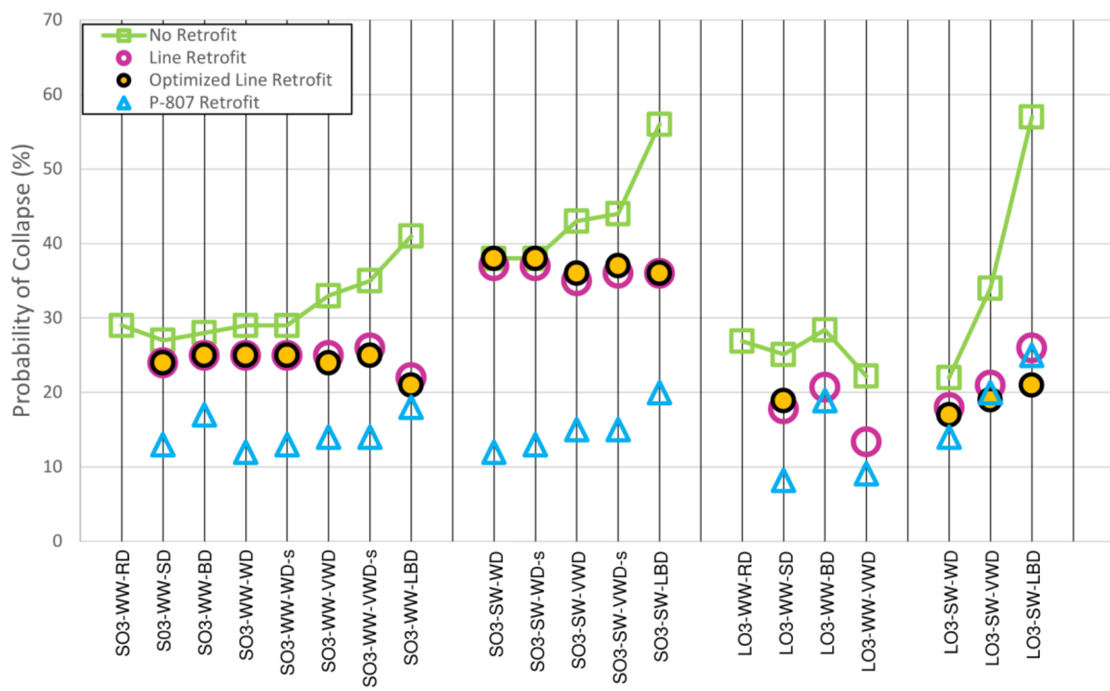


Figure 2-29 Probabilities of collapse (%) at S_a = 1.0g for the diaphragm sensitivity study archetypes.

Chapter 3: Key Findings and Recommendations for Seismic Retrofit Ordinances

3.1 Introduction and Purpose

This chapter presents key findings and recommendations drawn from the analytical studies presented in Chapter 2. The primary objective of this chapter is to assist government officials in developing and implementing seismic retrofit ordinances for SWOF buildings, as well as structural engineers who are advising property owners regarding seismic retrofits.

The key findings and recommendations of this chapter are based on analytical studies that have used building archetypes to represent a portion of the widely varying SWOF building stock. In particular, the studies are based on long-side-open and short-side-open archetypes with both parking and occupied residential units at the first story, as seen in Figure 3-1 and Figure 3-2. The overall SWOF building stock includes other configurations, materials, combinations of materials, and material strengths that vary from those studied. The archetypes were selected in part because they have configurations for which line retrofits would potentially be used. They are representative of SWOF buildings constructed from the 1950s on and that are more prevalent in Southern California than Northern California. While the behavioral trends apply to other SWOF buildings, careful consideration should be given before extrapolating the findings and recommendations of this chapter beyond the archetypes studied. In particular, buildings with fewer or no occupied residential units at the first story would likely have higher probabilities of collapse.

The analytical studies documented in this report provide an approximation of anticipated seismic performance. The approximation is believed to be biased towards overprediction of collapse (i.e., the reported probabilities of collapse are higher than what is expected among SWOF buildings following actual earthquakes). Further, the analytical models were found to be somewhat sensitive, with notable changes in performance sometimes occurring with small changes in modeling properties. For these reasons, the emphasis of this chapter is on general trends observed in the data.

With the exception of diaphragm properties in the diaphragm sensitivity study, the effects of poor element conditions (e.g., poor initial construction or deterioration) were not explicitly included in the analytical studies. Where widespread condition issues are present, the unretrofitted probabilities of collapse would be expected to be higher.

MODELING NAMING CONVENTION KEY [AA#-WW-DD-RR]

AA: Building Archetype (Section 2.2)
LO = Long Side Open
SO = Short Side Open
SOW = Short Side Open with Wing Walls
LN = LO with No First-Story Open Front
SN = SO with No First-Story Open Front

#: No. of Stories
2 = Two Stories
3 = Three Stories

WW: Wall Type (Section 2.3.1)
WW = Weak Wall (stucco plus gypboard)
SW = Strong Wall (stucco plus plaster)

DD: Diaphragm Type (Section 2.3.2)
RD = Rigid Diaphragm
WD = Weak Diaphragm (175 plf)
SD = Strong Diaphragm (505/1024 plf)
BD = Brittle Diaphragm (252/524 plf)
VWD = Very Weak Diaphragm (100 plf)
LBD = Lower-Bound Diaphragm (60 plf)
-s = indicates that selected upper-story walls are shifted away from the first-story walls

RR: Retrofit Type (Section 2.5)
Blank = No Retrofit
L = Line Retrofit
OL = Optimized Line Retrofit
P807 = FEMA P-807 Retrofit

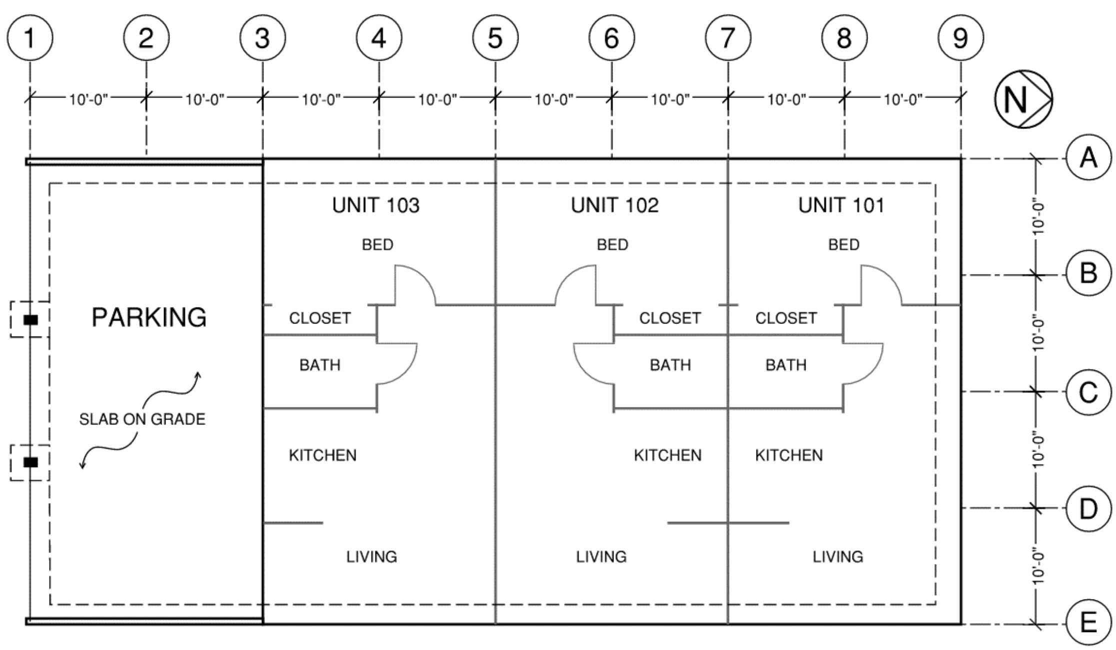


Figure 3-1 First story of the short-side-open archetype with wing walls. A large portion of the first story is occupied.

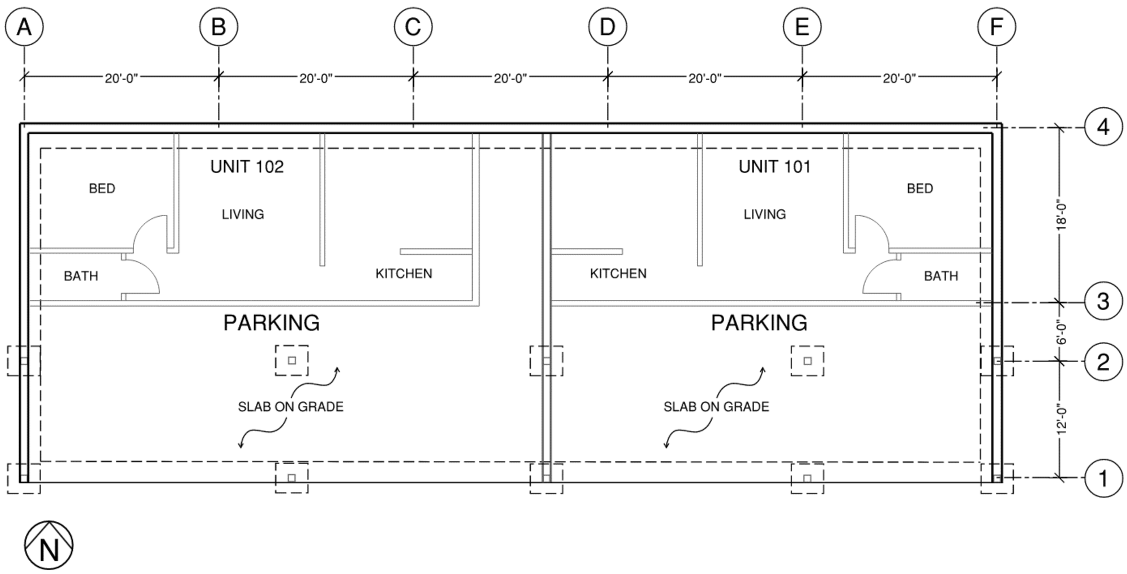


Figure 3-2 **First story of the long-side-open archetype floor. A large portion of the first story is occupied.**

3.2 Key Findings

3.2.1 Data for Key Findings

The primary method used to express resistance to collapse is fragility curves developed from the analytical studies. These are presented as fragility function plots of probability of collapse (POC) versus spectral response acceleration, S_a, as shown in Figure 3-3. POC was determined using incremental dynamic analysis and FEMA P-695 methods, where collapse was directly modeled in the analytical studies.

For purposes of comparison across archetypes, the POC at a spectral response acceleration of 1.0g has been determined from the fragility functions and is provided in tables that follow. The value of 1.0g represents the seismicity at a site in downtown Los Angeles and corresponds to approximately 75% of the seismic demand from the design-basis earthquake specified for new buildings (i.e., 75% of 2/3 S_{MS}) at that site. This value is consistent with the demand that would be used for a line or FEMA P-807 retrofit in accordance with the City of Los Angeles SWOF ordinance and is consistent with the seismic demands used for the retrofits included in the analytical studies (see Section 2.5). The tabulated POC data at 1.0g illustrates data trends for regions of high-seismic hazard. Discussion is provided in Section 3.2.2 and Section 3.2.5 regarding locations where other spectral response accelerations are of interest.

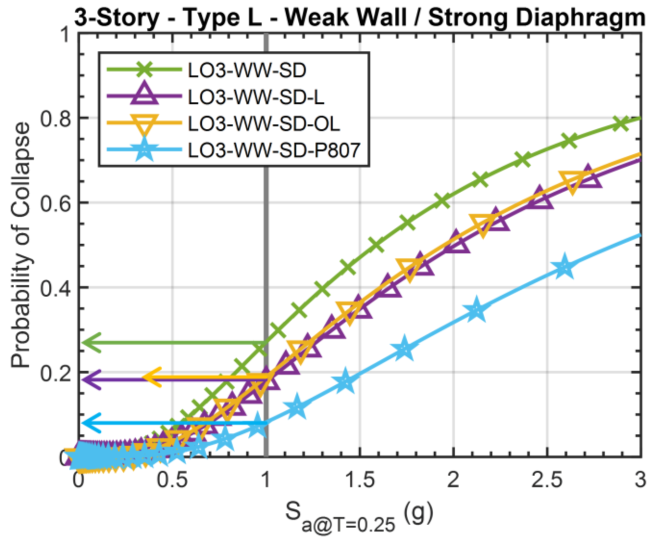


Figure 3-3 Example fragility function results from analytical studies.

Table 3-1 through Table 3-6 provide results from the analytical studies. Data are provided from the primary study, including for varying seismic hazard, and from the diaphragm sensitivity and wing walls sensitivity studies. See Chapter 2 for description of each study.

Included in Figure 3-4 and other figures that follow are lines demarking 10% POC and 20% POC at the 0.5S_{MS} spectral acceleration of 1.0g, provided as suggested lines of reference for retrofit performance. For retrofits provided under the Los Angeles SWOF ordinance and designed using the FEMA P-807 methodology, the Los Angeles retrofit design criterion used with the FEMA P-807 methodology is 20% probability of exceedance (POE) of a drift associated with onset of strength loss at 0.5S_{MS} (equal to 1.0g for the downtown Los Angeles location selected for this project). The 20% POE was viewed in the original FEMA P-807 study as a surrogate for 20% POC. For this reason, 20% POC is a suggested line of reference for data at a spectral acceleration of 0.5S_{MS}. The POCs for FEMA P-807 retrofits designed based on 20% POE were found to, on average, be more consistent with 10% POC than 20% POC. For this reason, 10% POC is a second suggested line of reference for data at a spectral acceleration of 0.5S_{MS}. The reference lines of 10% and 20% POC at 0.5S_{MS} are also loosely associated with SWOF retrofit design under IEBC Appendix A4, for which design to 75% of 2/3S_{MS} (= 0.5S_{MS}) is specified. These reference lines are noted in Figure 3-4 and Figure 3-6 through Figure 3-14.

A more stringent retrofit design criterion (e.g., lower POE, higher response spectral acceleration, or both) could have been selected for this project. FEMA P-807 shows, however, that there are limitations to performance of first-story retrofits based on the capacity of the second story. In addition to being more costly and invasive, higher levels of retrofit of the first story can result in diminishing improvement in performance because additional failure modes, including second-story collapses, limit overall building performance. Diminishing improvement is anticipated to be most prevalent in areas of very high seismic hazard with greater improvement in areas of high and moderate seismic hazard.

Both 10% and 20% POC at $0.5S_{MS}$ are less stringent than the ASCE/SEI 7 target criterion for design of new buildings; the ASCE/SEI 7 criterion is 10% POC at $1.0S_{MS}$ (equal to 2.0g for the downtown Los Angeles location). Figure 3-5 provides side-by side plots of the primary study POCs at $0.5S_{MS}$ (1.0g) and $1.0S_{MS}$ (2.0g), with the 10% and 20% POC lines marked as lines of reference. Here it can be seen that the FEMA P-807 retrofit POCs ranging from 21% to 44%, (for retrofits designed using 20% POE at $0.5S_{MS}$ and evaluated at $1.0S_{MS}$) are higher than the 10% POC ASCE/SEI 7 target for new buildings, consistent with the current existing building retrofit philosophy.

Table 3-1 Probabilities of Collapse (%) at Spectral Response Acceleration of 1.0g for the Primary Study

Archetype	Stories	Unretrofitted	Line	Optimized Line	FEMA P-807
L03-SW-WD	3	22	18	17	14
L03-WW-SD	3	27	18	19	8
S03-SW-WD	3	38	37	38	12
S03-WW-SD	3	27	24	24	13
L02-SW-WD	2	10	8	8	6
L02-WW-SD	2	12	8	8	4
S02-SW-WD	2	27	20	20	8
S02-WW-SD	2	18	15	15	9

Table 3-2 Probabilities of Collapse (%) at Spectral Response Acceleration of 2.0g for the Primary Study

Archetype	Stories	Unretrofitted	Line	Optimized Line	FEMA P-807
L03-SW-WD	3	55	50	48	44
L03-WW-SD	3	60	50	51	32
S03-SW-WD	3	73	71	72	40
S03-WW-SD	3	62	58	58	42
L02-SW-WD	2	35	30	30	25
L02-WW-SD	2	40	32	32	21
S02-SW-WD	2	59	54	54	32
S02-WW-SD	2	50	45	45	34

Table 3-3 Probabilities of Collapse (%) for Unretrofitted Archetypes at Varying Spectral Response Accelerations for the Primary Study

Archetype	0.25g	0.50g	0.75g	1.0g	1.25g
LO3-SW-WD	1	4	13	22	31
LO3-WW-SD	1	5	15	27	35
SO3-SW-WD	2	10	25	38	49
SO3-WW-SD	1	6	16	27	37
LO2-SW-WD	0	2	5	10	16
LO2-WW-SD	0	2	6	12	19
SO2-SW-WD	1	5	15	27	34
SO2-WW-SD	0	3	10	18	26

Table 3-4 Probabilities of Collapse at Spectral Response Acceleration of 1.0g for the Diaphragm Sensitivity Study, Short-Side-Open Archetypes

Archetype	Stories	Unretrofitted	Line	Optimized Line	FEMA P-807
SO3-WW-RD	3	29	-	-	-
SO3-WW-SD	3	27	24	24	13
SO3-WW-BD	3	28	25	25	17
SO3-WW-WD	3	29	25	25	12
SO3-WW-WD-s	3	29	25	25	13
SO3-WW-VWD	3	33	25	24	14
SO3-WW-VWD-s	3	35	26	25	14
SO3-WW-LBD	3	41	22	21	18
SO3-WW-LBD-s	3	42	22	21	18
SO3-SW-WD	3	38	37	38	12
SO3-SW-WD-s	3	38	37	38	13
SO3-SW-VWD	3	43	35	36	15
SO3-SW-VWD-s	3	44	36	37	15
SO3-SW-LBD	3	56	36	36	20

Table 3-5 Probabilities of Collapse at Spectral Response Acceleration of 1.0g for the Diaphragm Sensitivity Study, Long-Side-Open Archetypes

Archetypes	Stories	Unretrofitted	Line	Optimized Line	FEMA P-807
LO3-SW-WD	3	22	18	17	14
LO3-SW-VWD	3	34	21	19	20
LO3-SW-LBD	3	57	26	21	25

Table 3-6 Probabilities of Collapse at Spectral Response Acceleration of 1.0g for the Wing Walls Sensitivity Study

Archetype	Stories	Unretrofitted	Line	Optimized Line	FEMA P-807
SO3-WW-SD	3	27	24	24	13
SOW3-WW-SD	3	21	16	16	15

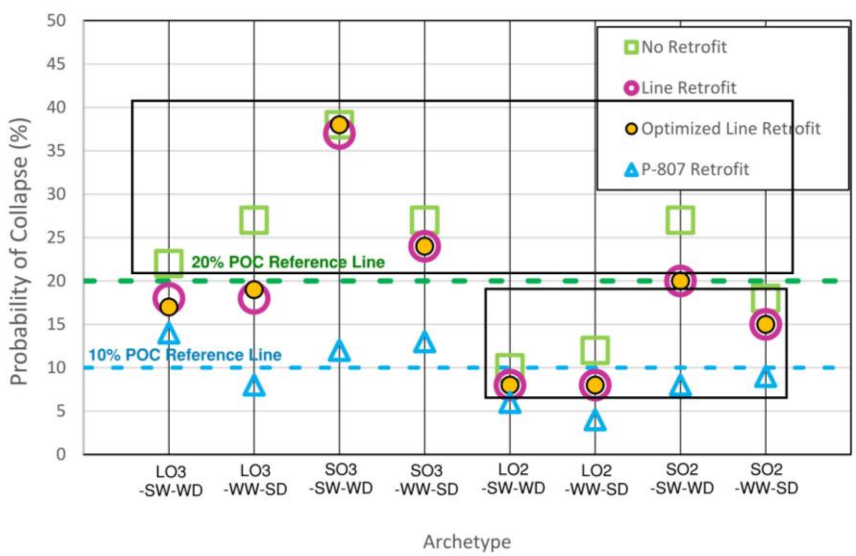


Figure 3-4 Primary study POC data at spectral response acceleration of 1.0g, grouped by unretrofitted archetype POC.

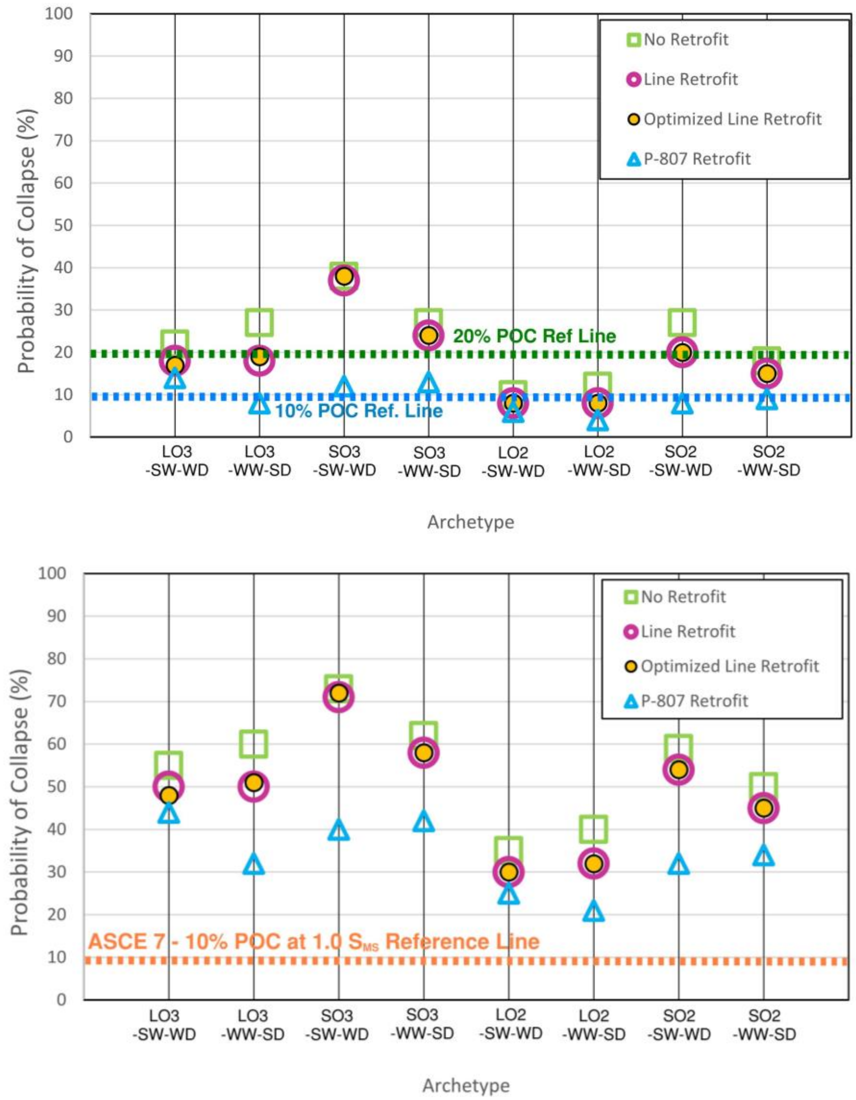


Figure 3-5 Primary study POC data (top) at a spectral response acceleration of 1.0g with 10% and 20% POC reference lines noted and (bottom) at a spectral response acceleration of 2.0g with the ASCE/SEI 7 10% POC reference line noted.

3.2.2 Vulnerability of Unretrofitted SWOF Buildings

Many in the engineering community anticipate that SWOF buildings are significantly more vulnerable to collapse in the direction parallel to the open front relative to other potential failure modes, with most collapses assumed to initiate at the open front line, as discussed in Section 1.2. The analytical studies show that existing SWOF buildings are instead very weak at the entire first story, susceptible to multiple failure modes, and exhibit behavior more complex than widely assumed. As a result, multiple vulnerabilities (e.g., X direction, Y direction, torsion, diaphragm) can initiate story collapse. Varying numbers of stories and building configurations add further complexity to the resulting data. The following discusses observations drawn from the analytical study POC data, followed by findings derived from the observations.

Capacity Parallel and Perpendicular to the Open Front. Pushover curves developed for the archetype buildings show that the pre-retrofit SWOF buildings are similarly brittle (i.e., peak capacity at low drift level, significant capacity drop post peak) and weak (i.e., ratio of peak capacity to weight is low) in both orthogonal directions. This behavior can be seen in Figures 2-18 and 2-19, which show very similar load-deflection behavior in the x and y directions. This is contrary to the common assumption that SWOF buildings are notably weaker in the direction parallel to the open front. The closely matched peak strengths in both directions are believed to occur in part because the wall parallel to the open front at the back of the parking area has few or no openings, compensating for the strength not present in the open-front wall line. This helps to make the peak strength parallel to the open front similar to the peak strength perpendicular.

Given that peak ground motions can occur in any horizontal direction, the most effective retrofit designs will address vulnerability to collapse in both orthogonal directions. Another implication of the observed behavior is that the vulnerable building stock is likely larger than the group of buildings that can be visually identified to have an open-front wall line.

Finding #1: For the unretrofitted SWOF archetypes studied, the pushover curves illustrate similar peak strengths and brittleness in both orthogonal directions, rather than illustrating reduced strength in the direction parallel to the open front.

Complexity of Collapse Modes. The analyses for the unretrofitted archetypes show numerous controlling failure modes. As an example, Table 3-7 summarizes failure modes for archetype SO3-WW-BD (see Figure 2-2). The most common mode of failure is in the Y direction, perpendicular to the open front (48%). X-direction failures initiating at the open front make up 14% of the collapses, while all X-direction collapses make up 37%. Torsion and diaphragm-driven collapses make up the balance of collapse modes (16%). While the failure modes were not identified for all archetype buildings, this range of failure modes was common across the archetypes for which failure modes were identified. One of the implications of this observed behavior is that the vulnerable building stock is likely larger than the group of buildings that can be visually identified to have an open-front wall line.

Table 3-7 Distribution of Failure Modes for Archetype SO3-WW-BD

Failure Mode	Number of Occurrences (Out of 44 Total)	Percent Occurrence
X-Direction (no torsion, failure parallel to open front)	6	14%
X-Direction (with torsion, failure originating at open front)	6	14%
X-Direction (with torsion, failure originating at back)	4	9%
Y-Direction (no torsion, failure perpendicular to open front)	21	48%
Y-Direction with Torsion Failure	2	5%
Diaphragm Failure	5	11%

Finding #2: For the unretrofitted SWOF archetypes studied, rather than collapse occurring primarily in the direction parallel to the open front and initiating at the open front, the modes of collapse were varied.

Finding #3: Although the primary vulnerability is often perceived to be the open front, there are other significant vulnerabilities in the SWOF building stock that may be equally or more prevalent.

Vulnerable Characteristics. The primary study POC data in Table 3-1 and Figure 3-4 can be separated into two groups with respect to unretrofitted building collapse vulnerability. Group 1, shown in the upper box, includes the three-story and the SO2-SW-WD unretrofitted archetypes that are highly vulnerable to collapse, with POC values ranging from 22% to 38%. Group 2, shown in the lower box, includes the two-story, long-side-open (LO2) archetypes and SO2-WW-SD, for which POCs prior to retrofit are 10% to 18%. These fall between the 10% and 20% POC reference lines and are moderately vulnerable to collapse.

Finding #4: The unretrofitted LO3, SO3, and SO2-SW-WD primary study archetypes have high POCs at S_a=1.0g.

Finding #5: The unretrofitted LO2 and SO2-WW-SD primary study archetypes have lower POCs at S_a=1.0g than other archetypes.

Effect of Variation in Seismic Hazard. The great majority of the SWOF building stock is believed to not have been engineered for seismic forces at the time of original design and construction. Instead, these buildings were constructed using conventional materials and building practices. As a result, the construction of SWOF buildings and therefore their strengths remain relatively constant, irrespective of the seismic hazard varying greatly by building location.

Although most of the data presented in Chapter 3 is specific to a spectral response acceleration of 1.0g, representative of the Los Angeles site of interest and the basis of the retrofit design, the fragility functions resulting from the analytical studies provide POC data for unretrofitted buildings for a range of spectral accelerations. Consideration should be given to location and spectral accelerations of interest when developing retrofit requirements.

Table 3-2 and Figure 3-6 provide data for the unretrofitted archetypes of the primary study at selected spectral response acceleration intervals to show the variation in POC. From these data, it can be seen that in locations where spectral response accelerations up to 0.5g are of interest for retrofit design, all of the studied unretrofitted archetypes have POCs below 10%. In addition, most of the unretrofitted archetypes in locations where spectral response accelerations are up to 0.75g have POCs below or slightly higher than 20%. Archetype retrofits designed for varying seismic hazard were not studied and thus only unretrofitted archetype data are included.

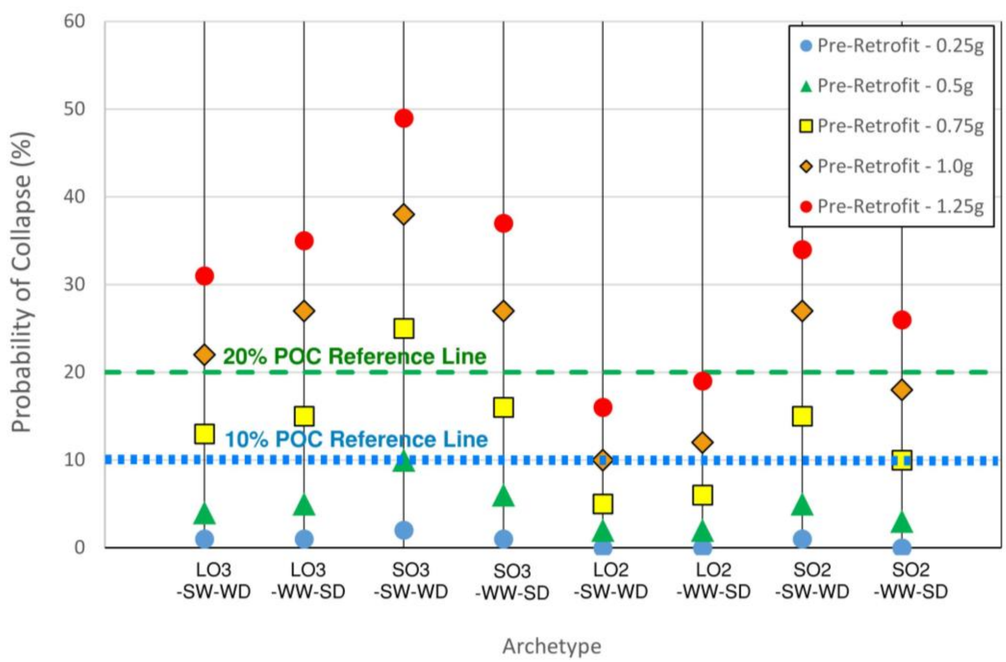


Figure 3-6 Primary study POC results with varying seismic hazard.

Finding #6: The unretrofitted primary study archetypes have POCs that vary significantly with spectral response acceleration. In regions of lower seismicity, unretrofitted archetype POCs are often less than the 10% or 20% reference lines, indicating lower vulnerability and reduced priority for retrofit.

Effects of Wall and Diaphragm Strength. In order to capture variations in SWOF building construction materials in the existing building stock, the primary study included strong walls in combination with weak diaphragms (SW-WD) and weak walls in combination with strong diaphragms (WW-SD). See Chapter 2 for discussion of these variations. Table 3-1 and Figure 3-7 summarize the resulting unretrofitted archetype POC data. For each archetype form and height (LO3, SO3, LO2, and SO2), SW-WD and WW-SD variations are shown side by side and enclosed in boxes. Moderate differences are seen between the unretrofitted POCs (2% to 11%) in each of the pairs, suggesting that, within the range studied, the primary study materials of construction have a moderate effect on the unretrofitted POC performance. The POCs for the paired archetypes and the range between the pair are believed to best represent the majority of the existing SWOF building stock, understanding that some portion of the building stock have building materials with properties that are outside these bounds and thus would have POCs outside these ranges. The strong (plaster) wall and ceiling materials added both strength and weight to the archetype buildings, resulting in story strength-to-weight (V/W) ratios that were fairly constant between strong and weak wall materials (see Figures 2-18 and 2-19).

Finding #7: For the archetypes and the range of materials studied in the primary study, the materials of construction had a moderate effect on the unretrofitted POC performance. Other aspects, such as the number of stories and open-front side, have more significant effects.

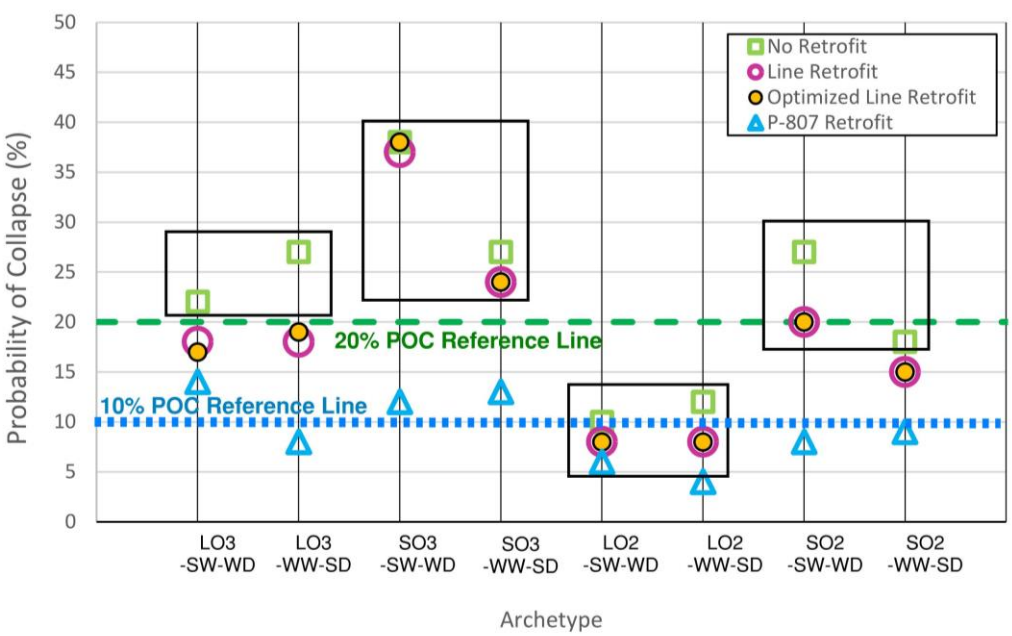


Figure 3-7 Primary study POC results at spectral response acceleration of 1.0g showing paired archetypes with varied wall and diaphragm properties.

Effects of Further Reduced Diaphragm Strength. The diaphragm sensitivity study was used to further investigate changes in POC due to variations in diaphragm strength. This diaphragm study was in part in response to the very limited benefit computed for line and optimized line retrofits in the SO3-SW-WD, SO3-WW-SD, and SO2-WW-SD archetypes, as seen in the yellow boxed data in Figure 3-8. It was postulated that if the diaphragm were to be weaker, the benefit of line and optimized line retrofits would increase. This led to a series of archetypes that incrementally reduced diaphragm capacity and concentrated demand until the diaphragm condition notably affected the archetype performance.

Starting with the SO3-WW-SD archetype, diaphragm properties were varied to represent BD, WD, WD-s, VWD, VWD-s, LBD, and LBD-s variations, as seen in Table 3-4 and Figure 3-9. For the WD-s, VWD-s, and LBD-s archetypes, not only was the diaphragm strength reduced, but in addition the upper-story walls were shifted off of line 3 and towards the open front at line 1 (see Figure 3-1). This was done to increase the second-floor diaphragm demand in the vicinity of line 3 while simultaneously decreasing capacity, resulting in the diaphragm being more likely to control archetype performance. Diaphragm properties also were varied for the SO3-SW archetypes. Finally, starting with the LO3-SW-WD archetype, VWD and LBD variations were studied as shown in Table 3-5 and Figure 3-9; walls were not shifted in the LO3 archetypes. The diaphragm studies were only conducted for three-story buildings. As illustrated in Figure 3-8, the two-story archetypes either had unretrofitted POCs near 10% or the line and optimized line retrofits provided at least a moderate reduction in POC. For this reason, they were of less interest.

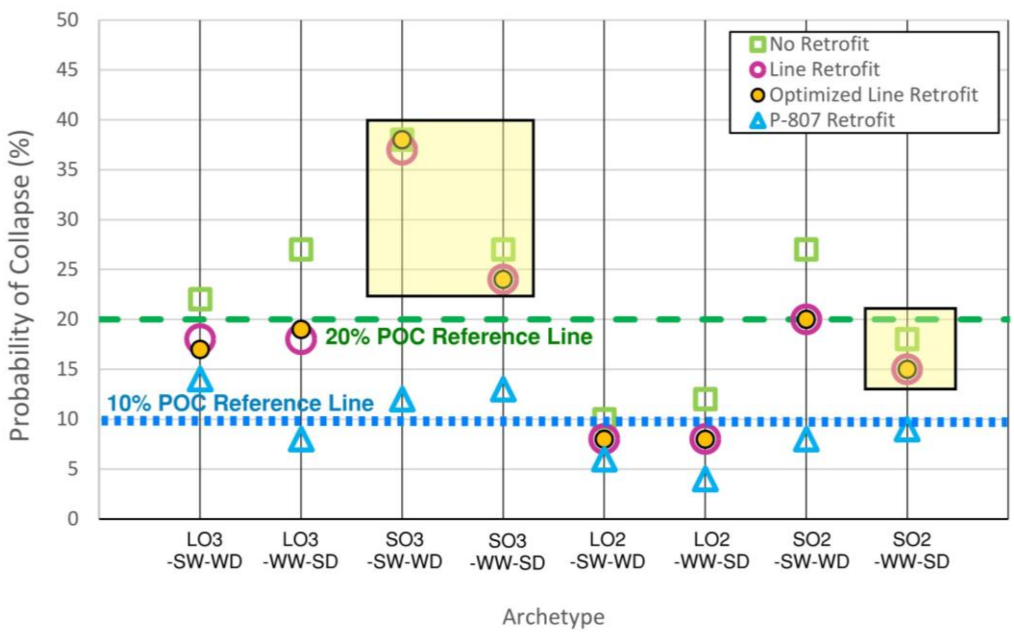


Figure 3-8 **Primary study POC results at spectral response acceleration of 1.0g highlighting three archetypes with very limited benefit from line and optimized line retrofits.**

Figure 3-9 groupings from left to right are associated with the SO3-WW-SD, SO3-SW-WD, and LO3-SW-WD primary study archetypes. Unretrofitted data for SO3 VWD, VWD-s, and LBD archetypes show a distinct pattern of increased POC with weaker diaphragms. The same pattern is seen with the LO3 VWD and LBD variants, with the increase in POC being more dramatic.

The SO3-WW POC for line and optimized line retrofits is seen to remain fairly steady across the range of diaphragm properties, with the mean value of the line retrofits at approximately 23%, and similarly a mean value of approximately 37% POC for SO3-SW. The POC for FEMA P-807 retrofit is also fairly steady across the range of SO3 archetypes, with a mean value of 16% POC. With limited exceptions the FEMA P-807 retrofit POCs fall below the 20% POC reference line, although as a group they are slightly increased from the average of 10% POC in the primary study.

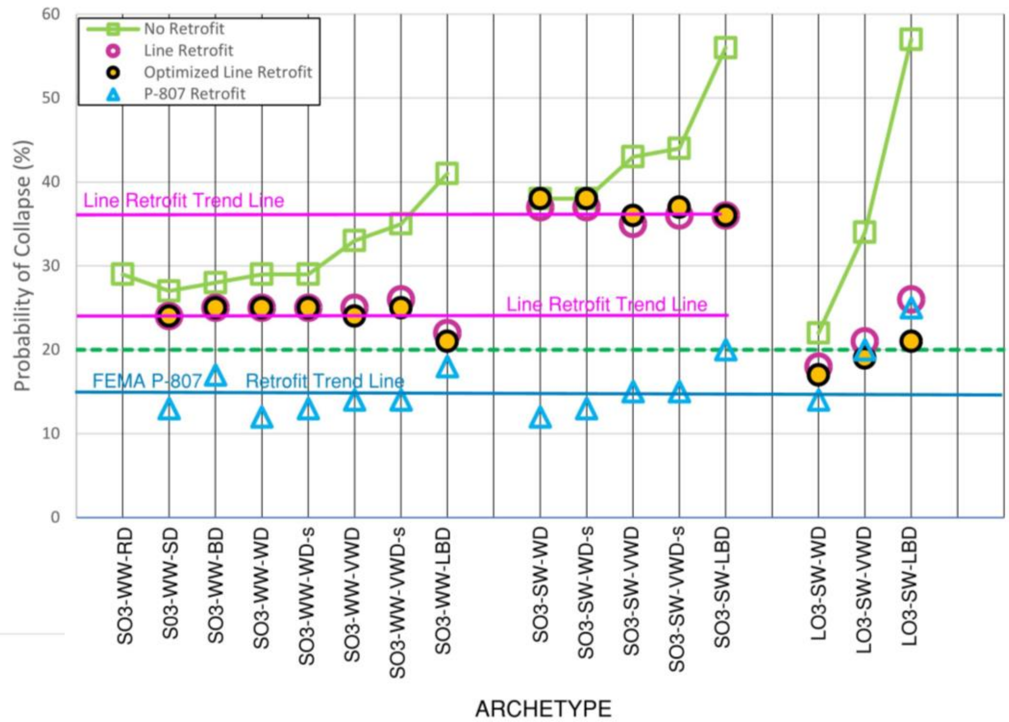


Figure 3-9 Diaphragm study results at spectral response acceleration of 1.0g showing trends of unretrofitted and retrofitted archetype POC with varying diaphragm model properties.

Figure 3-9 illustrates that for the SO3-WW-LBD archetype, the performance of all three retrofit methods is similar, with a range of 18% to 22% POC. This reflects modestly improved performance of the line and optimized line retrofits and modestly reduced performance of the FEMA P-807 retrofit. For this archetype, the analytical studies have identified that the deformation of the diaphragm at the building interior is leading to load concentrations in the interior walls. Because the deformation is at the building interior, this behavior is not necessarily related to the open-front condition and shows that the overall performance of the building is controlled by low diaphragm strength. Measures that can be taken to reduce the retrofit POC include strengthening walls at the interior portions of the first story to reduce demands on the diaphragm or strengthening the diaphragm. Strengthening the diaphragm often involves installing wood-structural-panel sheathing either on the second-floor diaphragm over existing lumber sheathing or overhead in the first story as a ceiling soffit.

This study did not investigate how prevalent VWD and LBD configurations are in the existing building stock. The VWD and LBD properties are intended to represent: (1) the lowest-strength, straight-lumber-sheathed diaphragm (2) further weakened by condition issues (e.g., poor construction or deterioration) (3) in combination with carpet rather than hardwood flooring, and (4) a ceiling or soffit that is fastened to the structure in a manner that it is not able to provide any strength contribution. A change in any of these conditions would likely provide capacity equal to or greater than the WD model properties. The VWD and LBD properties were investigated as a supplemental study rather than being included in the primary study based on the belief that these properties do not broadly occur in the existing building stock (see Appendix D). For engineers who are considering retrofit of an individual building, the occurrence of conditions consistent with the VWD or LBD could be assessed by opening finish materials to observe existing systems. Jurisdictions interested in potential occurrence of VWD or LBD conditions may already have knowledge of the conditions in their building stock and be able to judge the prevalence of these diaphragm types; otherwise, some investigation of the building stock may be needed to determine prevalent floor diaphragm conditions.

Finding #8: For the diaphragm sensitivity study unretrofitted SO3 and LO3-SW archetypes with S_a=1.0g, the VWD, VWD-s, and LBD diaphragm variants resulted in POCs increased above those seen in the primary study.

Finding #9: For the diaphragm sensitivity study SO3 archetypes with S_a=1.0g, the POCs for line and optimized line retrofits remained fairly constant across the varying diaphragm conditions, with mean POC values of approximately 23% for the SO3-WW archetypes and 37% for the SO3-SW archetypes. In general the strength of the diaphragm does not impact the effectiveness of the retrofit.

Finding #10: For the diaphragm sensitivity study SO3 archetypes with S_a=1.0g, the POCs for the FEMA P-807 retrofits remained fairly constant across the varying diaphragm conditions, with a mean of 16% POC.

Finding #11: For the diaphragm sensitivity study unretrofitted SO3-WW-LBD archetype with S_a=1.0g, the performance of all three retrofit methods was similar, with a range of 18% to 22% POC. For this archetype, the analytical studies have identified that the deformation of the diaphragm at the building interior is leading to load concentrations in the interior walls. To counter the modest increase in POC for FEMA P-807 retrofits, measures that can be taken to reduce the retrofit POC include strengthening walls at interior portions of the first story to reduce demands on the diaphragm and strengthening the diaphragm.

Effects of Modified Strong Diaphragm Properties. The diaphragm sensitivity study investigated unretrofitted SWOF archetypes with the brittle diaphragm combination of lower strength and lower deformation capacity relative to the strong diaphragm. This represents a lower-quality version of the diagonal-lumber-sheathed strong diaphragm. Results comparing the SO3-WW-BD archetypes to the paired strong-diaphragm archetypes showed negligible increases in the POC, as seen in Table 3-4 with an increase in POC of 2% to 4% for each archetype with the brittle diaphragm.

Finding #12: For studied unretrofitted SWOF archetypes with S_a=1.0g, the brittle diaphragm combination of lower strength and reduced deformation capacity had a negligible effect on the POC.

Effects of Wing Walls. The wing walls sensitivity study of the short-side-open, unretrofitted archetypes evaluated the addition of “wing walls.” These are stucco-finished walls without door or window openings oriented perpendicular to the open front in the parking area (seen in Figure 3-1 on lines A and E between lines 1 and 3). The POC data are shown in Table 3-6 and Figure 3-10. Wing walls were seen to reduce the unretrofitted POC from 27% to 21% for the strong-diaphragm archetype. In addition, the effectiveness of line and optimized line retrofits is notably increased in wing-wall building configurations.

In instances where line-based screening methods trigger retrofit in both orthogonal wall line directions, Figure 3-10 (SOW-WW-SD) shows the retrofit performance with the line and optimized line retrofits to be equal to the FEMA P-807 retrofit.

Finding #13: For the wing walls sensitivity study and S_a=1.0g, SWOF archetypes with a short-side-open configuration and originally constructed with wing walls were seen to have a moderately lower unretrofitted POC and increased effectiveness of the line and optimized line retrofits relative to those without wing walls. This is believed to result from the reduction in collapses in the direction perpendicular to the open front. A similar benefit is anticipated to be achieved by adding wing walls as part of a retrofit.

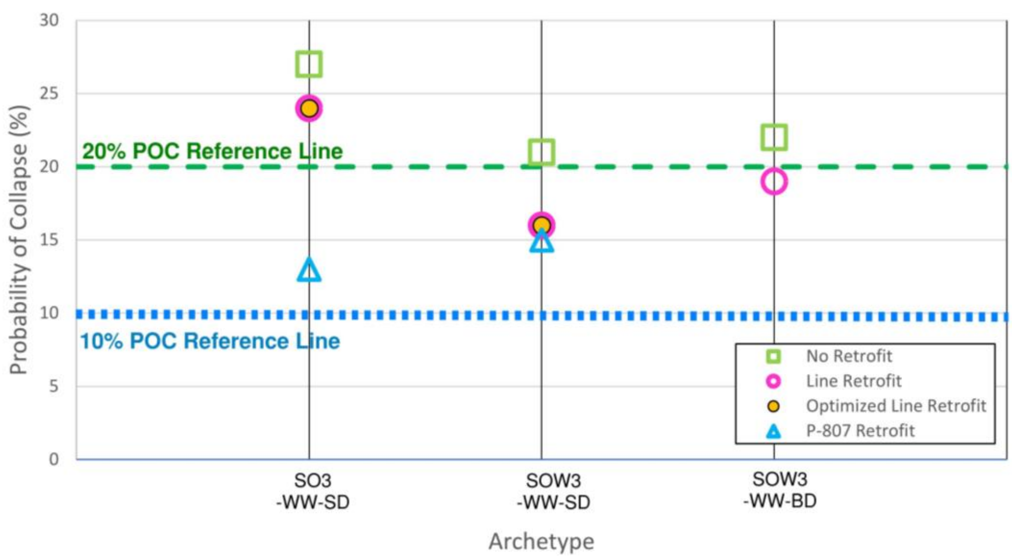


Figure 3-10 Wing walls sensitivity study POC data at spectral response acceleration of 1.0g showing decreased unretrofitted POC and increased line and optimized line retrofit effectiveness with wing walls present. No optimized line or FEMA P-807 retrofits were analyzed for the SOW3-WW-BD archetype.

3.2.3 Benefits of SWOF Building Retrofit

Benefits of Retrofit. Figure 3-11 provides an illustration of the POC of the primary study archetypes prior to retrofit, and with line, optimized line (i.e., without drift limits), and FEMA P-807 retrofits. Some retrofits are seen to provide a significant change in POC, while others do not. For all the archetypes, retrofits are shown to lower the POC, although the amount the POC is lowered varies considerably.

Finding #14: For the studied archetypes and S_a=1.0g, SWOF buildings benefit from retrofit.

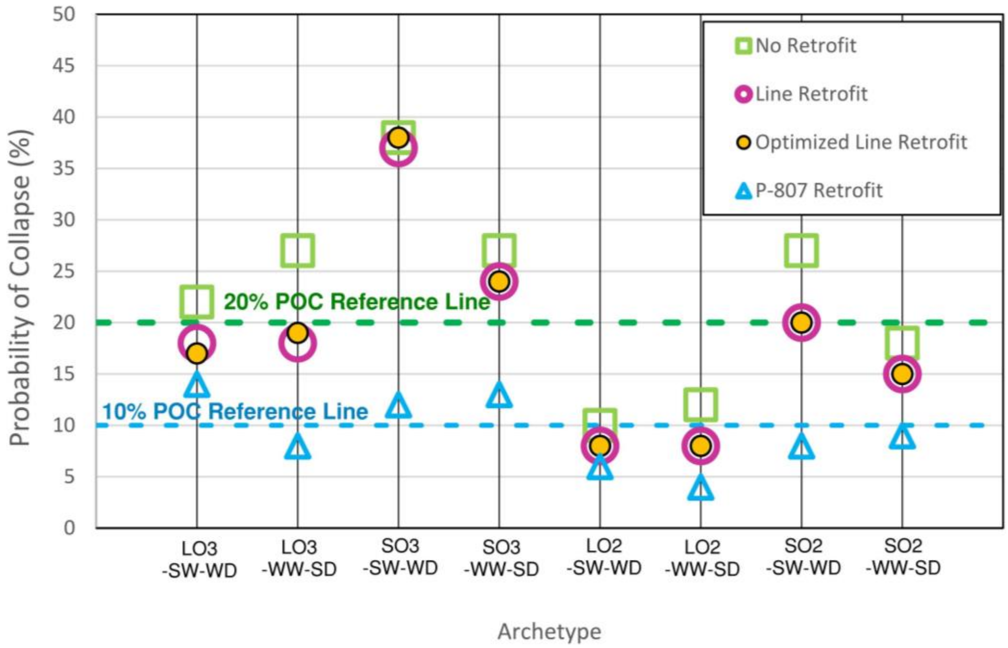


Figure 3-11 Primary study POC data at spectral response acceleration of 1.0g.

Improvement in Performance—FEMA P-807 Retrofits. From the primary study, Table 3-1 and Figure 3-11 illustrate that FEMA P-807 story retrofits are effective for all the archetypes considered. Overall, for the primary study FEMA P-807 retrofits, POC values were reduced from 10% to 38% unretrofitted to 4% to 14% with FEMA P-807 retrofits. All of the retrofitted archetypes had POCs that were below 20%, with an average of approximately 10%. The POCs resulting from FEMA P-807 retrofits are much more consistent than those resulting from line and optimized line retrofits.

The results in Figure 3-11 provide a limited benchmarking of the FEMA P-807 methodology and Weak-Story Tool (WST). The FEMA P-807 retrofits studied were designed using the WST (and its built-in analytical simplifications and material assumptions) and targeting a criterion of 20% probability of exceedance (POE) of drift limits associated with onset of strength loss at a spectral acceleration of 1.0g. The resulting FEMA P-807 retrofits were then incorporated into numerical models of the SWOF archetypes using the most current nonlinear analysis tools and material data to independently calculate the probability of collapse (POC). In doing so it was found that using 20% POE and 1.0g in the different analysis environment of the WST resulted in all POCs being below 20% and averaging approximately 10% at 1.0g using the new nonlinear analysis. This suggests the pattern that when using the FEMA P-807 method and WST, the POC will be moderately lower than the chosen POE.

The selected FEMA P-807 retrofit design criterion of 20% POE at $0.5S_{MS}$ resulted in moderate extents of retrofit, in costs consistent with expectations (see Section 3.2.5), and in reasonably constructable retrofits. While retrofit performance would ideally match the new building target, as previously discussed in Section 3.2.1, higher levels of retrofit of the first story can result in more costly and invasive retrofit work that provides diminishing improvement because additional failure modes, such as second-story collapses, can limit overall building performance. Systematic investigation of incremental performance improvement with increasing seismic retrofit was outside of the scope of this study; as a result, the discussion on this point is qualitative rather than quantitative.

Finding #15: For the primary study archetypes and S_a=1.0g, story retrofits using FEMA P-807 had significant benefit, with all retrofit POCs being below 20% and having an average of approximately 10%. The FEMA P-807 retrofits resulted in moderate and reasonably constructable extents of retrofit while achieving the economies associated with first-story-only retrofit. For this reason, this criterion appears to be a reasonable minimum level of retrofit to target in areas of very high-seismic hazard, such as downtown Los Angeles. A higher retrofit criterion may be of benefit, especially in areas of lesser seismic hazard, and may be achievable with first-story-only retrofit.

Improvement in Performance—Line and Optimized Line Retrofits. The POC data from the primary study can be organized into three groups for discussion of line and optimized line retrofits, as seen in Figure 3-12. The POCs resulting from line and optimized line retrofits are less consistent than those for FEMA P-807 retrofits.

The first group, shown in the green boxes, includes the LO3, and SO2-SW-WD archetypes. For each of these archetypes, the line and optimized line retrofits provide an incremental reduction in POC, with the reduction in POC being on the order of one-third to one-half that of the FEMA P-807 retrofit.

Finding #16: For the primary study archetypes and S_a=1.0g, line and optimized line retrofits provide moderate reduction in POC in LO3, and SO2-SW-WD archetypes.

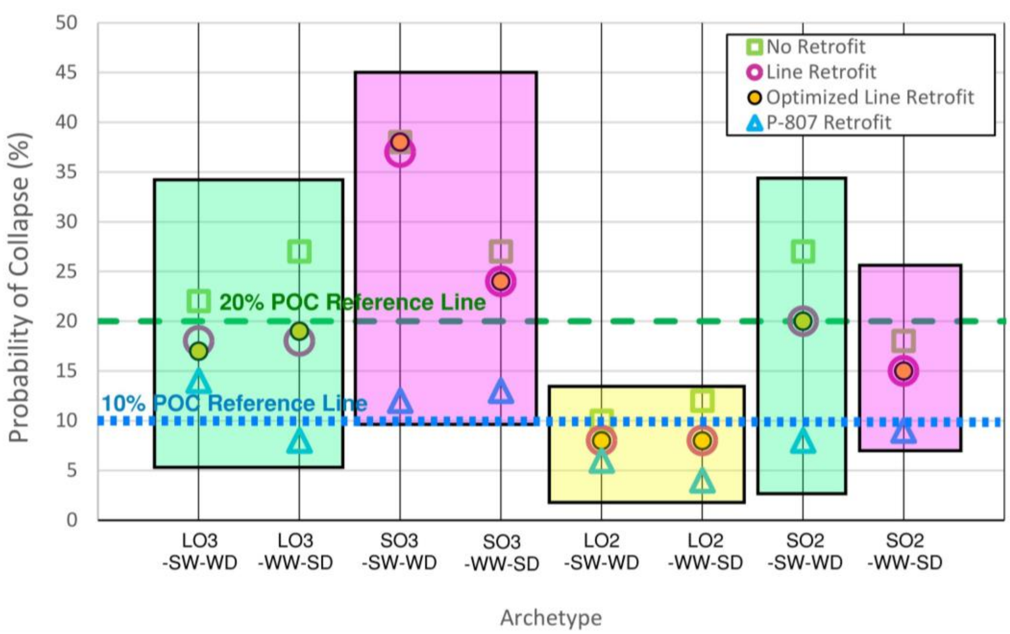


Figure 3-12 Primary study POC data at spectral response acceleration of 1.0g grouped by effectiveness of line and optimized line retrofits.

The second group, shown in the yellow box, includes the LO2-SW-WD and LO2-WW-SD archetypes. These archetypes have POCs in the unretrofitted condition that are close to the average POC of 10% for FEMA P-807 retrofits. In addition, all three retrofits for both archetypes result in similar POCs and show negligible improvements relative to the already very low unretrofitted POC. Overall, there is a reduced need for retrofit, and retrofit is anticipated to provide little benefit.

Finding #17: For the LO2 archetype and S_a=1.0g, the unretrofitted POC was low enough that there is a reduced need for retrofit. None of the three retrofit methods significantly lowered the already low POC.

The third group is composed of the SO3-SW-WD, SO3-WW-SD, and SO2-WW-SD archetypes, seen in magenta boxes in Figure 3-12. For all three of these archetypes, the primary study line and optimized line retrofits provided negligible reduction in POC, while the FEMA P-807 story retrofits provided a significant reduction.

Finding #18: For the SO3-SW-WD, SO3-WW-SD, and SO2-WW-SD archetypes and S_a=1.0g, POC data from the primary study show negligible improvement with the line and optimized line retrofits.

This result was, however, further modified in both the diaphragm and wing walls studies. In Figure 3-9, from the diaphragm study, when the SO3-WW-SD, SO3-SW-WD, and LO3-SW-WD archetypes are changed to LBD, the unretrofitted POCs increase, and the relative improvements of the line and optimized line retrofits increase. For the SO3-WW-LBD and LO3-SW-LBD archetypes, the line and optimized line retrofits are nearly as effective as the FEMA P-807 retrofit.

Finding #19: Further evaluation as part of the diaphragm study showed that at S_a=1.0g, with LBD properties, the line and optimized line retrofits exhibited greater relative improvements and, for SO3-WW-LBD and LO3-SW-LBD archetypes, were approximately as effective as the FEMA P-807 retrofit.

From the wing walls sensitivity study, it was found that line retrofits provide more benefit for short-side-open archetypes with wing walls than those without wing walls, as seen in Table 3-6 and Figure 3-10.

Finding #20: For the studied archetypes and S_a=1.0g, line and optimized line retrofits provide a moderate benefit in short-side-open archetypes with wing walls.

Improvement in Performance—Number of Stories. From the primary study, Table 3-1 and Figure 3-11 illustrate that three-story archetypes benefit most from retrofit, having the greatest unretrofitted POCs (27% to 38%) and a significant reduction in POC with retrofit. Following the three-story archetypes, the two-story, short-side-open archetypes have the next best benefit, followed by the two-story, long-side-open archetypes. The greater benefit from retrofit of three-story archetypes is in large part due to the lower strength-to-seismic weight (V/W) ratios of the unretrofitted three-story archetypes, as discussed in Chapter 2.

Finding #21: For the studied archetypes and S_a=1.0g, three-story SWOF archetypes as a group benefit most from retrofit, with two-story SWOF archetypes as a group having lower unretrofitted POCs and a lower benefit from retrofit.

Improvement in Performance—Line vs Optimized Line Retrofits. From the primary study, the POC for a SWOF archetype with a line retrofit is essentially the same as the POC with an optimized line retrofit. The line retrofit design includes a drift limit that has the effect of increasing the size and weight of steel moment frames and cantilevered columns. For the optimized line retrofit, steel moment frame and cantilevered column sections are smaller and more cost effective.

Finding #22: For the archetypes studied and S_a=1.0g, line retrofits (drift limits imposed) and optimized line retrofits (no drift limits imposed) result in POCs that are essentially identical, providing the same benefit but with the optimized line retrofit costing less.

3.2.4 Other Key Findings

Comparison to Archetypes with No First-Story Open Fronts. Table 3-8 and Figure 3-13 provided POC data for three-story archetypes with no first-story open-front vulnerabilities (designated in Chapter 2 as the LN3 and SN3 archetypes). As introduced in Chapter 2, these buildings have residential units for 100% of the first-story area, with a floor plan at the first story identical to the floor plans of the second and third stories. The LN3 and SN3 archetypes, like their LO3 and SO3 counterparts, represent existing buildings constructed using conventional non-engineered building practices and with light-frame walls braced with stucco, gypboard, and plaster finishes. (The POC results are anticipated to be different for buildings with engineered seismic designs and with wood-structural-panel shear wall systems.)

The archetypes represented in Table 3-8 and Figure 3-13 data were modeled using rigid diaphragms as part of a sensitivity study. Because of the rigid diaphragms and other evolutions in the analytical models, the results in Table 3-8 and Figure 3-13 are not directly comparable to data in the primary study or other sensitivity studies. As seen in Figure 3-13, the LN3 archetypes have notably lower POCs than the associated LO3 archetypes, while the SN3 and SO3 archetypes have very similar POCs. This trend is reflected in the ratios in the right-hand column of Table 3-8. The ratio of the peak first-story shear capacity (i.e., strength) from a static pushover analysis to the seismic weight being supported (V/W) is a strong indicator of POC performance. Table 3-9 tabulates these ratios for the LO3, SO3, LN3, and SN3 archetypes. The ratios of POC (Table 3-8) and the ratios of V/W (Table 3-9) exhibit the same pattern and the same general magnitudes, particularly in the parallel to open front (X) direction.

The trend of collapse rates tracking first-story strength-to-weight ratios is generally aligned in the archetypes with stronger diaphragms, where diaphragm failure modes are limited. However, for archetypes with weaker diaphragms, diaphragm failure modes lead to more complex responses, and the trends are less clear, as discussed in Chapter 2.

Table 3-8 Probabilities of Collapse at Spectral Response Acceleration of 1.0g for Archetypes with No First-Story Open-Front Vulnerabilities and their Unretrofitted Counterparts

Archetype	LN3 or SN3 POC (%)	Unretrofitted POC (%)	POC Ratios (LN3/LO3 or SN3/SO3)
LO3-SW-RD	29	36	0.81
LO3-WW-RD	19	27	0.70
SO3-SW-RD	38	39	0.97
SO3-WW-RD	28	29	0.97

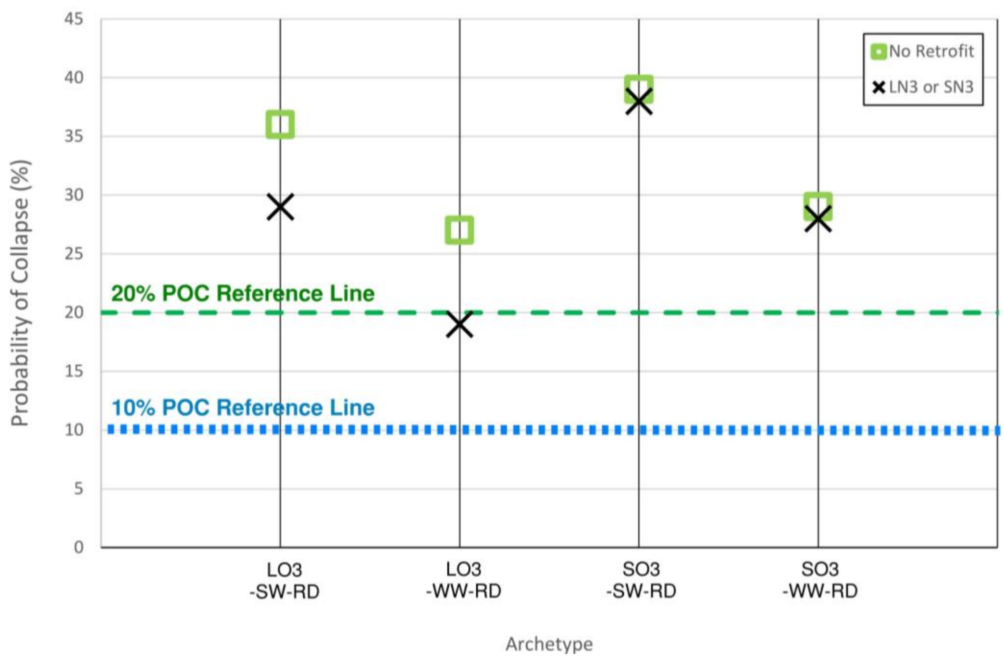


Figure 3-13 POC results at spectral response acceleration of 1.0g comparing unretrofitted, three-story archetypes to three-story archetypes with no first-story open fronts.

Table 3-9 Archetype Strength-to-Weight Ratios

Archetype	X Direction Parallel to Open Front			Y Direction Perpendicular to Open Front			V_x LO3/ V_x LN3 or V_x SO3/ V_x SN3	V_y LO3/ V_y LN3 or V_y SO3/ V_y SN3
	V_x (kips)	W (kips)	V_x/W	V_y (kips)	W (kips)	V_y/W		
LO3-WW-RD	97	303	0.32	103	303	0.34	0.70	0.64
LN3-WW-RD	139	303	0.46	160	303	0.53		
LO3-SW-RD	169	504	0.33	175	504	0.35	0.72	0.59
LN3-SW-RD	231	504	0.46	295	504	0.59		
SO3-WW-RD	88	270	0.33	84	270	0.31	1.00	0.75
SN3-WW-RD	88	270	0.33	112	270	0.41		
SO3-SW-RD	148	445	0.33	140	445	0.31	0.92	0.74
SN3-SW-RD	162	445	0.36	189	445	0.42		

Finding #23: For the paired LO3, LN3, SO3, and SN3 rigid-diaphragm archetypes studied and S_a=1.0g, the data support the notion that V/W is a reasonable general predicter of POC performance. This trend is less clear for archetypes with weaker diaphragms, in which diaphragm failure modes lead to more complex responses.

Finding #24: For the LO3, LN3, SO3, and SN3 rigid-diaphragm archetypes studied and S_a=1.0g, the relatively similar POCs between archetypes with and without open fronts confirms the general seismic vulnerability of three-story wood-frame buildings braced with stucco, gypboard, and plaster. The data support the notion that the vulnerability of SWOF buildings is more extensive than visibly prominent open-front wall lines.

FEMA P-807 Performance Retrospective. The analytical studies shed light on the effectiveness of FEMA P-807 as a retrofit method applied to the archetypes with various configurations and combinations of materials for the walls and diaphragms. The models used for this study are more sophisticated than those used for the original FEMA P-807 analyses. The primary differences are the nonlinear modeling of diaphragms, the use of updated material properties, and the explicit modeling of collapse. The design intent in the creation of the FEMA P-807 method was to create a relatively straightforward design process that would yield reliably safe and cost-effective retrofits. Overall, retrofits with FEMA P-807 achieved a probability of collapse well below 20% and averaging approximately 10% at the 1.0g seismic demand for the primary study archetypes.

Finding #25: Overall, retrofits with FEMA P-807 achieved a probability of collapse (POC) averaging 10%, and falling below the POE design target of 20% at S_a=1.0g. This study found no major

shortcomings with the FEMA P-807 method, and it continues to be a reliable seismic retrofitting option for SWOF buildings.

Finding # 26: While FEMA P-807 continues to provide an acceptable level of safety for SWOF buildings seismically retrofitted using the method, the study did identify two possible areas of refinements that could be considered with a future update. The first is updating the material values for the finish materials that are providing bracing. Additional material testing has occurred in the decade since FEMA P-807 was developed and new data are now available. The second improvement could be the tightening of the criteria for retrofit element placement (see FEMA P-807 Section 6.3). These criteria minimize the diaphragm demands and deformations in the retrofitted structure. This study showed that diaphragm displacements play a larger role in the response than was recognized when the method was developed.

3.2.5 Additional Data

Variation in Seismic Hazard—Diaphragm and Wing Walls Sensitivity Studies. Figure 3-6 illustrates the variation of POCs for the primary study archetypes as a function of varying spectral response accelerations. The data show that the likely performance and need for retrofit can change significantly from location to location based on changing seismic hazard. Figure 3-14 provides similar data for the unretrofitted archetypes used in the diaphragm and wing walls sensitivity studies.

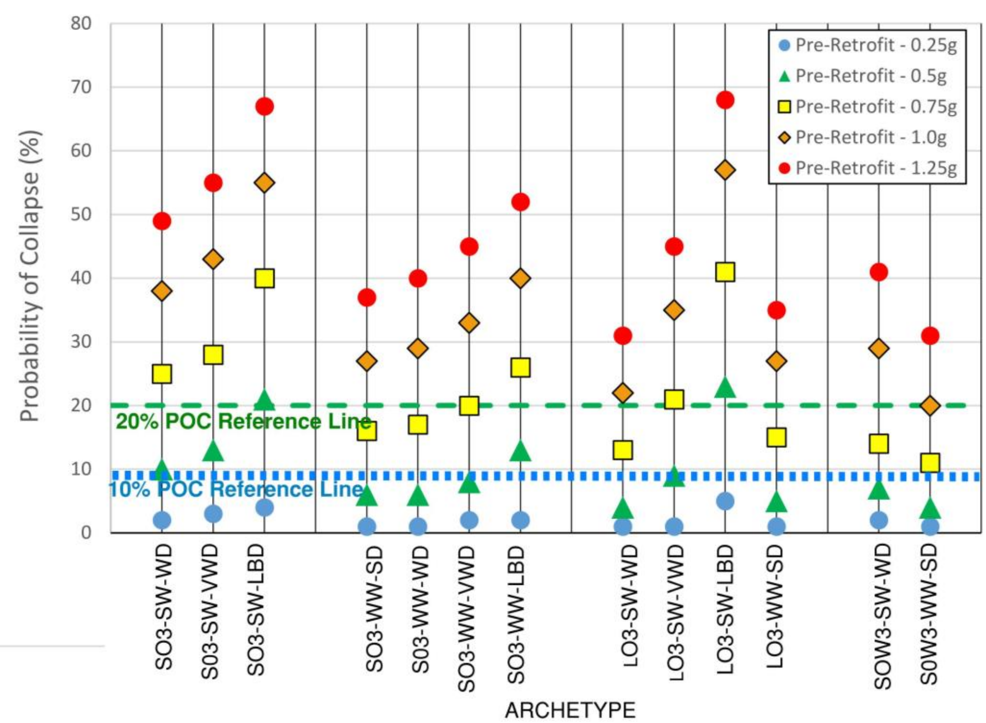


Figure 3-14 POC results from the diaphragm and wing walls studies for unretrofitted archetypes with varying seismic hazard, expressed as spectral response acceleration. For example, the POCs for unretrofitted archetypes at a spectral response acceleration of 1.0g are shown with orange diamonds.

Cost of Retrofit: An estimate of the construction costs for example optimized line and FEMA P-807 retrofits was prepared by design-build firm Optimum Seismic. The example retrofits were for a three-story, long-side-open building with weak walls and strong diaphragms (i.e., the L03-WW-SD archetype). The building is assumed to have 12 units with plan dimensions of 36 feet by 100 feet and story-to-story heights of 9 feet. The retrofits consist of special cantilever columns set in grade beams in the direction parallel to the open front and, for the FEMA P-807 retrofit, wood-structural-panel shear walls in the direction perpendicular to the open front (the retrofit designs duplicate those described in more detail in Chapter 5). The estimated construction cost for the optimized line retrofit ($65,000) is 48% of the cost estimated for the FEMA P-807 retrofit ($135,000).

The cost of retrofit construction can vary dramatically based on many factors, including location, ease of access, size of building, and date of construction. More details about the cost estimates are provided in Chapter 5. Importantly, the reported construction costs do not include costs for engineering design, inspection during construction, or permitting fees. The estimates also assume that retrofit work does not occur in occupied areas of the building. Where retrofits require work in occupied areas, there are additional costs, such as tenant relocation and lost tenant revenue, that can be significantly greater than the cost of physical construction.

3.3 Recommendations

The following recommendations are based on the key findings of Section 3.2 and are provided to assist government officials developing and implementing seismic retrofit ordinances, as well as structural engineers who are advising property owners regarding seismic retrofit of SWOF buildings. The recommendations are primarily oriented towards big-picture issues that might be considered in the development of seismic retrofit ordinances, including the scope of buildings to be included in screening and retrofit ordinances and the scope of retrofit work. Additional recommendations regarding details of retrofit design are provided in Chapter 4; these should be considered for inclusion in retrofit ordinances or supporting documentation.

- **Recommendation A—Importance of Retrofit:** In high-seismic-hazard regions, it is recommended that seismic retrofit ordinances be considered for SWOF buildings as part of a program to identify and address seismically vulnerable buildings.

  Discussion: Based on the archetypes studied and S_a=1.0g, high POCs were identified for unretrofitted SWOF buildings. This is consistent with observed collapses and near collapses of SWOF buildings in the 1989 Loma Prieta and 1994 Northridge earthquakes. The POCs can be reduced through seismic retrofit (see Finding #14). It is recognized that there might be other building types in a given building stock with similar or greater vulnerability or with other characteristics that make them a higher priority for retrofit for a particular community. Consideration of the complete building stock and occupancy types is encouraged when developing retrofit programs.

- **Recommendation B, Part 1—Type of Retrofit:** It is recommended that full-story retrofits be required, where practicable.

Discussion: For the studied archetypes and S_a=1.0g, FEMA P-807 full-story retrofits consistently provided notably better performance than line or optimized line retrofits, with POCs falling below the 20% POC target and averaging approximately 10% POC (see Findings #10 and #15). Although not modeled for this study, published data suggest that, in general, story retrofits in accordance with IEBC Chapter A4 or ASCE/SEI 41 will provide similar or improved performance of the first story relative to the requirements of FEMA P-807.

Published reports include the Buckalew et al. (2015) comparison of FEMA P-807 with 30% probability of exceedance at 0.5 S_{MS} (per City of San Francisco SWOF ordinance) with IEBC Appendix A4 and ASCE/SEI 41-13 retrofits, as well as the Burton et al. (2019) comparison of FEMA P-807 with 20% probability of exceedance at 0.5 S_{MS} (per City of Los Angeles SWOF ordinance) with IEBC Chapter A4 and ASCE/SEI 41-13 retrofits. A provision permitting capping of retrofit element strength, added in the 2021 edition of the IEBC, mirrors the FEMA P-807 retrofit strength capping provision, intending to mitigate against first story retrofit causing a second story collapse. The extent and cost of IEBC Chapter A4 and ASCE/SEI 41 retrofits are anticipated to be higher than FEMA P-807 retrofits because they require more retrofit elements.

- **Recommendation B, Part 2—Type of Retrofit:** Where it is not possible to require a FEMA P-807 or full-story retrofit, it is recommended that screening occur for open-front wall lines on all exterior walls of the building, including those perpendicular to the evident open-front wall. Where suggested by screening criteria, retrofits should be provided for all applicable exterior walls, including those perpendicular to the evident open front.

  Discussion: Where screening involves identification of open-front lines at exterior walls, in a SWOF building without wing walls (see Figure 2-2), a check for an open-front condition should occur at the evident open short side, and in addition at the two long sides, based on the wall opening where wing walls would otherwise be. The analytical studies have shown that the presence of wing walls in the garage area can notably decrease the POC. See Finding #13.

- **Recommendation B, Part 3—Type of Retrofit:** It is recommended that the predicted modestly higher POC and lower performance associated with FEMA P-807 retrofits and lower-bound diaphragms (LBD) in three-story SWOF buildings be recognized in retrofit ordinance documents, and suggestions be provided for increasing the retrofit performance.

  Discussion: Where conditions are consistent with the LBD, the analytical studies indicate increased POC of the unretrofitted building archetypes. For the SO3-WW-LBD archetype, line, optimized line, and FEMA P-807 story retrofits reduce the POCs to a similar extent. A similar pattern occurs for the LO3-SW-LBD archetypes.

  Lower-bound diaphragms are extremely weak and flexible, significantly impacting the performance of SWOF building. The lower-bound diaphragm of concern was identified to be a straight-lumber-sheathed diaphragm combined with a floor finish of carpet only (no hardwood floor), which is additionally of lower-quality construction or highly deteriorated and lacking strength contribution from the ceiling. Where this lower-bound diaphragm occurs, the POC of the unretrofitted condition can be notably higher than other SWOF buildings with stronger and stiffer diaphragms. Where these conditions occur, design professionals advising building owners are encouraged to recommend retrofit. Jurisdictions developing and implementing retrofit ordinances are also encouraged to recommend voluntary retrofit (see Finding #8).

For these archetypes, diaphragm deformation and resulting concentrations of seismic forces in the interior walls are a significant contributor to collapse. It is noted that modeling of lower-bound strength (and associated implied deterioration) was included for diaphragms, without similar deterioration of the walls; this was done based on concern regarding the influence of diaphragm strength. It is not known how prevalent the LBD condition is in the existing building stock, nor is it known to what extent deterioration of floors might occur without similar or more extensive deterioration to walls.

Recommendations for improved performance include retrofit of additional interior shear walls to reduce demands on the diaphragms or strengthening of diaphragms. While diaphragm strengthening can be performed by adding new sheathing on the top of floors, it is more common to add wood-structural-panel sheathing to the underside of the floor framing as a ceiling soffit (see Findings #8 and #11).

- **Recommendation C—Building Prioritization:** Where prioritization of SWOF building retrofits is desired, it is recommended that SWOF buildings three stories or more be given higher priority than two-story SWOF buildings.

  Discussion: Three-story archetypes generally have higher unretrofitted POCs and greater benefit of retrofit reduction in POC than two-story archetypes. As such they are recommended as the highest priority for this building type (see Finding #21).

- **Recommendation D, Part 1—Local Seismic Hazard:** When considering adoption of a seismic retrofit ordinance, it is recommended that local seismic hazard levels be taken into consideration. Unretrofitted collapse potential of SWOF buildings varies significantly with seismic hazard, thereby varying the need for and benefit of retrofit.

  Discussion: The collapse risks and benefits of retrofit for SWOF buildings can differ significantly between locations because of varying seismic demands at those locations. This can be seen in Figure 3-6, where use of a reduced spectral response acceleration significantly reduces the POC (see Finding #6).

- **Recommendation D, Part 2—Scope of Retrofit:** It is recommended that municipalities consider the local seismic hazard and the need for retrofit of two-story, long-side-open (LO2) SWOF buildings separately from the rest of the SWOF building group, because the unretrofitted POCs of these archetypes are significantly lower than other archetypes.

  Discussion: The unretrofitted, two-story, long-side-open archetypes were found to have significantly lower POCs than the other studied archetypes, indicating lower need for and potential benefit from retrofitting. This observation is limited to SWOF buildings with a substantial portion of the first story occupied with residential or similar uses and with closely spaced interior walls (see Figure 3-2), as assumed for this study. Based on these data, if included, it is recommended that these buildings be considered as a lower priority for retrofit (see Finding #5 and #17).

- **Recommendation E—Level of Retrofit:** When considering adoption of a SWOF building seismic retrofit ordinance, it is recommended that the FEMA P-807 retrofit criterion meet or exceed the 20% POE at $0.5S_{MS}$ criterion that was studied in this project. A more stringent retrofit criterion with a lower POE or higher spectral response acceleration might be considered. The more stringent criterion may lead to better performance, especially in areas of moderate- and high-seismic hazard, but may be of limited performance benefit in regions with very high-seismic hazard due to the capacity of the second story.

  Discussion: The FEMA P-807 retrofit criterion of 20% POE at $0.5S_{MS}$ is believed to result in retrofit designs that are reasonably constructable and notably reduce POC. The lower cost of a first-story-only retrofit is an important policy consideration when establishing a retrofit criterion. The FEMA P-807 methodology and Weak-Story Tool (WST) can identify, on a building-by-building basis, where second-story capacity controls and constrains the performance that can be achieved with reasonable economy. A more stringent FEMA P-807 criterion in regions of very high-seismic hazard may result in the need for second-story retrofits; this pragmatic constraint in very high-seismic regions should be recognized. Regions of moderate-to-high seismic hazard are more likely to achieve higher performance with first-story-only retrofits. (see Finding #15).

- **Recommendation F—Other Vulnerable Conditions:** It is recommended that SWOF building retrofit ordinances consider addressing all SWOF building configurations.

  Discussion: While this study has focused on the archetypes representing the long-side-open and short-side-open building types seen in Figures 3-1 and 3-2, other SWOF building configurations are thought to have significant vulnerability. A more complete list of configurations includes:

  - Long-side-open configurations with first-story residential units (West Hollywood Building Type A),
  - Short-side-open configurations with first-story residential units (West Hollywood Building Type B),
  - Partial open-front wall lines on several sides of the building with first-story residential units (West Hollywood Building Type C),
  - Tuck-under parking configurations on hillsides (West Hollywood Building Types D and E),
  - Tuck-under parking configurations with no first-story residential units (West Hollywood Building Types F and G),
  - Residential units over commercial space, and

  - Multi-family dwellings over crawlspaces on flat or hillside sites.

  Further discussion of building configurations is provided in Appendix A.

  Multi-family dwellings over crawlspaces are important to specifically identify because they have vulnerability consistent with other SWOF buildings. Most crawlspaces, however, do not have an open-front wall line, so they do not look like common SWOF buildings. The vulnerability of cripple-wall buildings comes from a lack of wall and wall strength, both interior and exterior, at the cripple-wall level.

- **Recommendation G, Part 1—Screening:** It is recommended that screening consider the overall configuration of the first-story walls relative to upper stories when assessing inclusion in the scope of a retrofit ordinance. This comparison could be made with the FEMA P-807 method, or with alternate methods including summed wall length in each story in each direction or an ASCE/SEI 41 quick strength check.

  Discussion: The collapse risk of SWOF buildings and the benefit from retrofit are not well correlated to visual characteristics, such as an open front. A more meaningful screening approach would look at the total length of exterior and interior walls at the first story relative to the second story in each orthogonal direction. The FEMA P-807 Weak-Story Tool provides a simple method to compare story-shear capacities of the first and second floors. Alternate methods could be used to make this comparison, including summed wall length in each story in each direction or an ASCE/SEI 41 quick strength check (see Findings #1 and #2).

- **Recommendation G, Part 2—Screening:** Where it is not possible to require a FEMA P-807 or full-story screening, it is recommended that screening for open-front wall lines occur on all exterior walls of the building, including those perpendicular to the evident open-front wall.

  Discussion: Where screening involves identification of open-front lines at exterior walls, in a SWOF building without wing walls (see Figure 2-2), a check for an open-front condition should occur at the obviously open short side, and in addition at the two long sides, based on the wall opening where wing walls would otherwise be. The analytical studies have shown that the lack of wing walls in the garage area can increase the POC. See Finding #18.

Screening

Screening is generally conducted by an engineer or architect using forms developed by a building department having authority in the relevant jurisdiction. Screening generally involves use of qualitative information to determine whether an individual building falls within the scope of the retrofit ordinance, although simple quantitative measures are sometimes used. Should owners believe that the building does not fall within the ordinance scope, evidence of this can be provided to the building department for their consideration.

- **Recommendation H—Evaluation:** Where evaluation is provided, a full-story evaluation is recommended. The FEMA P-807 methodology and FEMA P-807 Weak-Story Tool are believed to be the best available tools for evaluation.

  Discussion: The FEMA P-807 evaluation method and Weak-Story Tool are well suited for evaluation of SWOF buildings and are the best tools available at the time of writing this document. IEBC Appendix Chapter A4 is generally not helpful for evaluation, as it does not consider the strength contribution of bracing elements that are prevalent in these buildings. ASCE 41 quick stress checks can be used but tend to be conservative relative to FEMA P-807.

  Although many engineers will proceed directly to retrofit design, there are circumstances where it may be appropriate to conduct an evaluation to determine if retrofit is necessary. This might be particularly appropriate in seismic hazard regions where spectral accelerations less than 1.0g are being used for evaluation and design.

Evaluation

Evaluation is generally conducted by an engineer and provides a quantitative check of estimated seismic demand against building strength using a method specified by the retrofit ordinance. Evaluation is used to determine whether a building that falls within the scope of the ordinance requires retrofit.

- **Recommendation I—Retrofit Design, Line versus Optimized Line Retrofits:** Where line retrofits are permitted, it is recommended that new vertical steel elements (cantilever columns, special moment frames, or ordinary moment frames) be designed based on strength only. Consistent with the optimized line retrofits included in the studied archetypes, drift limits need not be considered.

  Discussion: The analytical studies have repeatedly shown that optimized line retrofits (i.e., line retrofits designed based on strength while omitting drift limitations) have performance substantially the same as line retrofits (i.e., including drift limitations). As a result, it is recommended that deflection criteria not be required for vertical elements of retrofits (see Finding #22). This is applicable to the steel vertical elements listed, and designed to applicable standards; this should not be extended to other vertical systems without further study.

Chapter 4: Recommendations for Retrofit Design

4.1 Purpose

Seismic retrofits of SWOF buildings vary in scope depending on the risk-reduction goals of the project or retrofit program. Some SWOF building retrofit projects or programs focus on strengthening the entire first story (story retrofits) whereas others only strengthen exterior open lines (line retrofits). Figure 4-1 presents examples of different potential scopes of work for line retrofits.

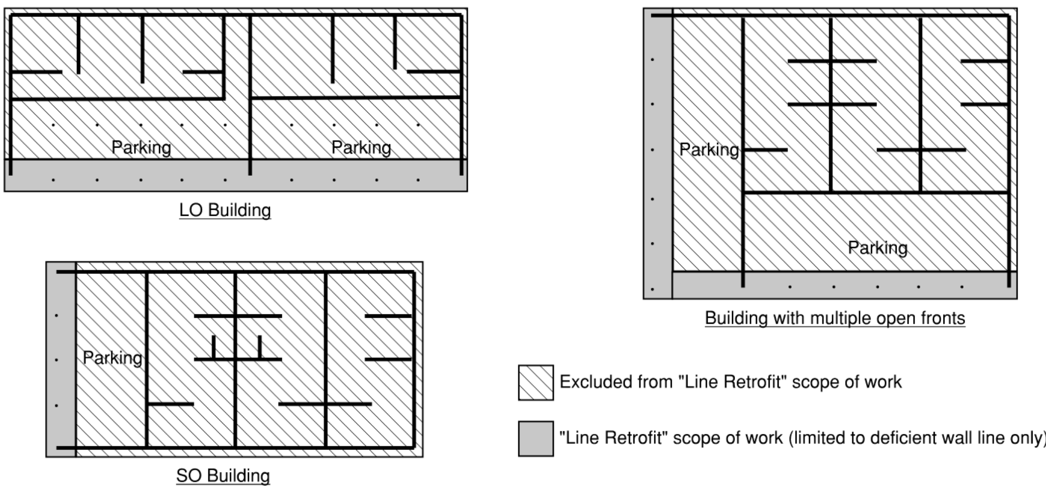


Figure 4-1 Line retrofit scopes of work for various conditions.

Story and line retrofits typically target pre-1980s buildings. These buildings do not have engineered seismic-force-resisting systems (SFRS) and are constructed using nonductile materials that are nonconforming under current design standards. As such, it is essential that retrofit designs do not hinder the performance of the existing building. This chapter highlights some of the issues that may be encountered while implementing SWOF retrofits (story and line) and provides recommendations for designing and constructing these types of strengthening projects. Selected recommendations are highlighted within blue boxes throughout the chapter.

Many of the recommendations provided in this chapter have been highlighted by the Structural Engineers Association of Southern California and the Structural Engineers Association of Northern California (Zepeda et al., 2019) and have been further refined here using the judgement and experience of the project team.

4.2 Seismic-Force-Resisting System Elements in Retrofit Design

4.2.1 Existing Seismic-Force-Resisting System Elements

The existing SFRS of a building can be organized into three main elements: diaphragms, vertical seismic-force-resisting systems, and foundations. The following provides an overview of these elements, with more in-depth discussion where appropriate in other sections of the chapter.

Existing Diaphragm. A diaphragm is the element within a structure that transfers applied out-of-plane building lateral loads, such as wind or seismic, to the in-plane vertical SFRS. Diaphragms also serve to support gravity loads as a floor or roof and are typically horizontal. Modern buildings typically use wood-structural-panel sheathing for their diaphragms. But the existing diaphragms of buildings targeted by SWOF retrofit programs typically are made of straight- or diagonal-lumber sheathing that is nailed to roof or floor joists at 12 inches-to-24 inches on center. Existing diaphragms may be weakened by condition issues, such as poor or missing nailing or deterioration.

Existing Seismic-Force-Resisting System. The vertical elements of an SFRS transfer in-plane loads (i.e., wind or seismic forces) to the foundation. Modern buildings can include a variety of ductile lateral systems, such as moment frames, braced frames, and shear walls. But the buildings targeted by SWOF retrofit programs usually rely on nonductile materials like gypsum wallboard, plaster, and stucco walls for lateral resistance. If existing walls are anchored to the foundation, it is typically with ½-inch anchor bolts at 6-feet on center, and the walls usually do not contain tie-downs at their ends.

Existing Foundations. Foundations in light structures are typically made of concrete footings or thickened portions of the slab-on-grade. These foundations support the building's loads (gravity and lateral) and transfer them to the supporting soil below. The buildings targeted by SWOF retrofit programs typically have strip footings along their exterior wall lines, spread footings below columns, and either strip footings or thickened slabs at the interior walls. The reinforcing is often light and can sometimes lack top reinforcement (i.e., minimal uplift capacity).

4.2.2 New Seismic-Force-Resisting Systems

There are several options when considering retrofits of SWOF buildings including: steel ordinary moment frames (OMF), steel special moment frames (SMF), steel cantilever systems using either a single cantilever column or an inverted frame system (i.e., multiple columns tied together with a grade beam), wood-structural-panel shear walls, and proprietary systems. The following provides a discussion of each common system.

Frame Systems (OMF and SMF). Steel OMF systems have low ductility and are only expected to resist a limited amount of inelastic deformation. These frames do not require use of American Institute of Steel Construction (AISC) prequalified connections and are typically used in low-seismic regions.

Steel SMF systems, on the other hand, are highly ductile and expected to resist a significant amount of inelastic deformation. These frames require use of AISC prequalified connections or connections verified by testing. According to AISC 341-16, *Seismic Provisions for Structural Steel Buildings* (AISC, 2016a), Section E3.6b, a steel SMF connection should be capable of sustaining an interstory drift angle of at least 0.04 radians while still sustaining 80% of the connected nominal plastic flexural strength. As a result, steel SMF systems enjoy a higher response modification coefficient, *R*, when used in new design.

While steel SMF systems are typically used to achieve higher ductility performance, there may not be as much value when performing a line retrofit. This is because other parts of the building are still brittle by comparison, and a line retrofit does not address those deficiencies. Buildings will typically fail at other locations prior to reaching the full ductility of the SMF, and as such, OMF systems are used more often than SMF systems for line retrofits. OMF systems typically are more cost effective than SMF systems because the former require fewer critical welds and the need for quality control is less stringent.

However, whenever possible, it is encouraged that SMF still be used for SWOF line retrofits. When performing a story retrofit, providing an SMF system (or other special system) is likely more advantageous because the added ductility is consistent with the higher seismic performance anticipated to be provided with a story retrofit. This is recognized in both IEBC Appendix 4 and FEMA P-807. 2021 IEBC Appendix 4 Section A403.3 encourages special seismic-force-resisting systems when retrofitting a full story. FEMA P-807 Section 6.5 requires that an SMF system be used when using moment frames to retrofit a soft story.

Recommendation Note

Use special seismic-force-resisting systems whenever possible. Special systems may not be as advantageous for SWOF line retrofits but will be valuable if the retrofit is ever extended to a full story. When using FEMA P-807, special systems are a requirement.

Cantilever Systems with Pole Foundations or Grade Beams. A single steel cantilever column can be constructed with an embedded pole foundation. Where this is the case, it resists seismic demands by transferring the moment from applied forces into the soil through passive pressure. Pole footings traditionally are idealized as being perfectly fixed at their bases. However, this does not align with the actual response of the system because the soil is not perfectly rigid. When applying high-seismic loads at the top of a column, the soil will likely flex and yield increasing the overall deflection at the top of the column. Also, the ductility of the system is highly dependent on the quality of the soil and the embedment. For this reason, unless there is a soils investigation and soil-structure-interaction is considered, the behavior of pole footings under seismic loads is difficult to predict.

An inverted frame is a similar yet more reliable system. It consists of two or more steel cantilever columns used in conjunction with a reinforced-concrete grade beam that connects the columns at their bases. There is more confidence in the predictability of this type of system because yielding can be better controlled, especially when they are detailed as a special cantilever column system (SCCS).

The inverted frame system can be very practical in retrofits. Contrary to single pole systems, inverted frame systems do not require large soil drilling equipment, which can be problematic in low overhead applications. In addition, inverted frames are made with less critical components and do not have critical welds like SMF systems. As such, inverted frames are often more economical and less prone to mistakes than other steel systems. SCCS should be designed per the requirements of AISC 341-16 Section E6 with additional requirements from ASCE/SEI 7. AISC 341-16 requires that SCCS columns be designed for overstrength seismic loads. However, meeting this requirement would mean that the columns would yield at higher forces. The analytical studies documented in Chapter 2 showed that increasing the design forces of the columns does not improve the performance of the retrofit. Designing the columns without overstrength provides better confidence in the yielding mechanism of the new system, as long as the connections to it and the structural foundation are sized for overstrength loads. As such, it is recommended that when using cantilever columns, all the AISC 341-16 requirements for SCCS be met with the exception of the overstrength loads for the columns.

Recommendation Note

Steel special cantilever column systems should meet the requirements of AISC 341 except that columns should not be designed for overstrength load cases.

Wood-Structural-Panel Shear Wall Systems. Wood-structural-panel shear wall systems typically use oriented-strand board (OSB) or plywood sheathing and have special boundary and field nailing to achieve the design strength and ductility. These systems are very ductile and are the most popular SFRS in new residential wood light-frame buildings. For retrofits, wood shear wall systems use economical materials and often do not require new foundations, and construction does not require highly skilled labor or special construction machinery. In line retrofits, new elements are typically located at the open-front line. This is an impractical location for shear walls due to interference with parking or other uses that occur at the open front. In story retrofits, it is common practice to add wood shear walls away from the open front. This generally involves adding new sheathing to existing framed walls, along with shear clips, anchor bolts, and tie-down bolts to complete the load path.

Proprietary Systems. Proprietary systems include steel moment frame systems and pre-engineered high-aspect shear walls that are developed by vendors. These systems offer a combination of economic advantages (e.g., constructability, schedule, materials) and higher structural performance, but they may require fabrication from preapproved fabricators.

As SWOF ordinances have become more common, new systems have come to market to target this specific need. Development of these systems has typically been focused on solving constructability problems, achieving higher structural performance, or both. The design professional is encouraged to research these systems when designing a SWOF retrofit.

4.2.3 Retrofit System Considerations

The following is a discussion of some of the most critical items to consider during design of a seismic-force-resisting system for a SWOF retrofit.

Redundancy Considerations. Although redundancy provisions for new buildings are not commonly applied to existing buildings, it is recommended that redundancy be considered in the design of SWOF retrofits whenever possible. This is especially important when designing line retrofits because the remainder of the story is composed of brittle materials. Single steel cantilever-column solutions should be avoided because of several negative attributes, including:

- Concentrated loads, nonredundant load paths, and potentially significant collector deformations,
- Long collectors with large collector forces,
- Significant out-of-plane column stiffness that can be noncompatible with the existing structure, and
- The need for larger foundations with concentrated loads and increased likelihood of interference with existing foundations.

In some cases, the geometry or access to the building may not allow multiple vertical elements in a line of resistance, and a single steel cantilever column must be used. In those situations, it is recommended that design and plan review ensure each of the four items previously highlighted are addressed by calculations and detailing, and that careful attention be paid to structural observations and inspection in the field.

Recommendation Note

Use steel cantilever columns in groups of two or more that are connected by a common grade beam (i.e., inverted frame). The use of single cantilever columns is discouraged. When unavoidable, ensure that high concentrated loads are addressed by calculations, detailing, and careful field inspection.

Compatibility Considerations. The use of compatible retrofit systems (i.e., similar stiffnesses) is recommended unless it can be shown that the retrofit is not causing local or global problems. This is especially important when performing a line retrofit since placing a stiff element, like a braced frame or concrete shear wall, can have unintended consequences. A stiff element can redistribute forces and cause large concentrations of forces in existing brittle materials. Many mandatory SWOF retrofit ordinances prohibit using stiff systems. In addition, noncompatible retrofit systems have the potential to introduce torsional problems. If placing a noncompatible system is the only option to retrofit a building, it is recommended that a full building evaluation be conducted to ensure that the retrofit will not cause unintended negative consequences.

Recommendation Note

New, stiff seismic-force-resisting systems, such as braced frames and concrete shears walls, should be avoided. When unavoidable, the entire story should be evaluated to ensure that the retrofit will not cause unintended negative consequences.

Seismic-Force-Resisting System Size Considerations. The sizes of new members should be considered as part of the selection of the new SFRS. For example, if a moment frame or cantilever system is going to be placed in a tuck-under parking area, minimum parking clearances should be maintained. Beam and column sizes ranging from 8 inches to 12 inches in depth work in most SWOF retrofit conditions. The sizes of the beams and columns can also dictate the number of SFRS elements that must be used. And when selecting to use an existing wall as a new plywood shear wall, the existing finishes in the selected wall should be considered. For example, adding plywood to an existing wall can introduce inconsistencies in finish thickness.

Location Considerations. Retrofitting a SWOF building requires placing new vertical elements that, if not properly located, can interrupt livable space, utilities, or both, and can introduce unwanted structural behavior. Thus, the potential impacts of the location of new elements should be thoroughly considered. For example, placing all new vertical seismic-force-resisting elements towards the center of a building footprint or towards one side of a building may cause a torsional response during a seismic event. 2021 IEBC Section 503 states that building alterations shall not cause or make existing irregularities worse.

When placing new seismic-force-resisting elements, it is recommended to eliminate cantilever-diaphragm conditions whenever possible. Cantilever diaphragms in SWOF buildings are particularly concerning because they are constructed with nonductile materials, lack modern chord and shear connections, and may have experienced deterioration. When performing a line retrofit, the cantilever diaphragm will likely be mitigated and therefore the concerns outlined above will not exist. However, there can be more variability in the placement of new elements for full story retrofits. Whenever possible, new seismic-force-resisting elements should be placed along open fronts as part of story retrofits.

Recommendation Note

In selecting the locations of new seismic-force-resisting elements:

- Minimize interruptions,
- Do not make existing irregularities worse or introduce new irregularities, and
- Mitigate cantilever diaphragms in both line and story retrofits.

4.3 Protection of Existing Structural Systems

Line and story retrofits can be classified as vulnerability-based (or deficiency-only) retrofits because they address a particular line or story and do not include an evaluation and retrofit of the entire building. With a vulnerability-based retrofit, the overall seismic performance of the building likely will be controlled by existing elements—not the new retrofit elements—whether they are in the first story away from the open front or in a story above. As such, any retrofit work that reduces the capacity of the existing building can reduce the seismic performance of the retrofitted building. The following sections provide a discussion on protecting the existing gravity and seismic-force-resisting systems of a building during a seismic retrofit.

4.3.1 Protecting the Existing Seismic-Force-Resisting System

It's important to take time to understand the existing seismic-force-resisting system in a building to avoid reducing its capacity when designing a retrofit. This is especially important when the retrofit is intended to mitigate a localized deficiency.

Local demolition of stucco at the second-floor line is often necessary to install new retrofit elements, such as collectors, that tie the existing structure directly to new vertical elements (Figure 4-2a and Figure 4-2b). But exterior stucco walls in SWOF buildings often are a major contributor to lateral strength, particularly above the first story. It is recommended that demolition details are provided to the contractor to avoid removal of critical sections of stucco that act as the existing seismic-force-resisting system.

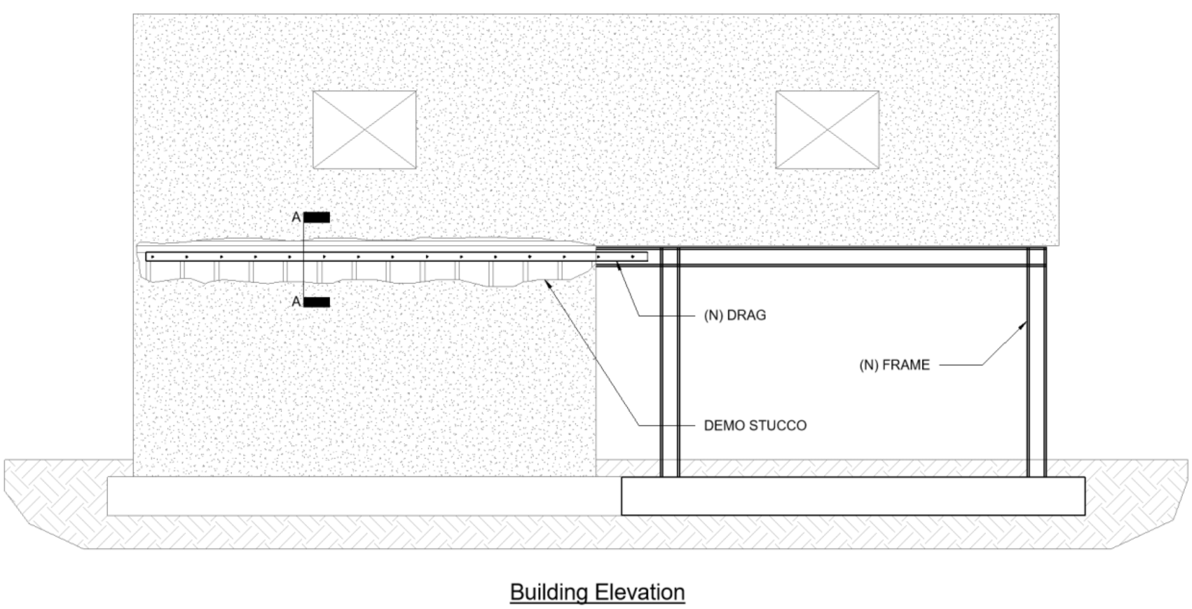


Figure 4-2a Damaged load path when installing new drag.

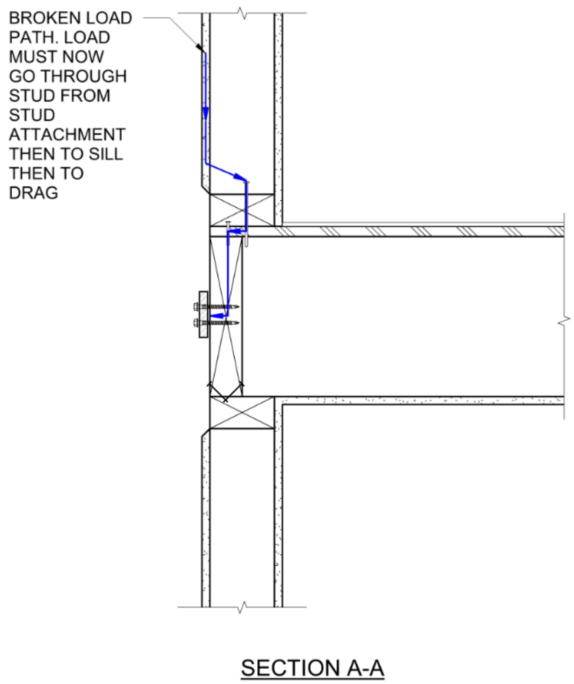


Figure 4-2b Damaged load path when installing new drag.

As seen in Figure 4-2a and Figure 4-2b, demolition has disconnected the second-story stucco from the bottom plate of the second-story wall, interrupting the load path from the second-story walls to the second floor. The lath in the wall is commonly exposed and lapped with new lath for patching of stucco walls. However, this may not be enough for the purposes of transferring shear, and there are no reliable methods for restoring the shear capacity of a stucco wall when it has been cut in this way. The stucco may need to be removed and reapplied along the entire story of the wall line, which is often impractical. Rather than demolishing the existing stucco, alternative methods are recommended for transferring the required lateral load into the new vertical elements. Two alternatives are illustrated in Figure 4-3 and Figure 4-4.

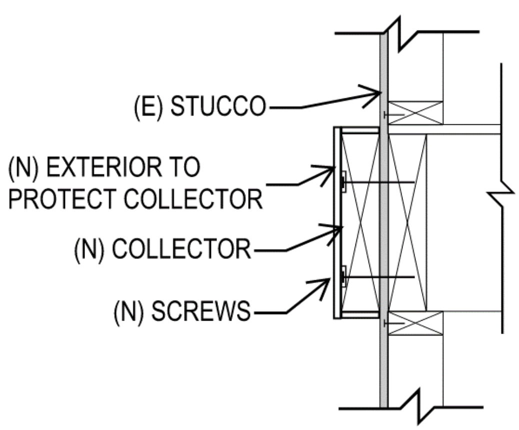


Figure 4-3 An alternative detail to stucco demolition that uses a new collector.

In Figure 4-3, a new collector is installed over the existing stucco, leaving said stucco largely intact. With this approach, new waterproofing detailing is required since the new screws will likely damage the existing waterproofing membrane. Figure 4-4 provides a detail of an overhang in which the stucco is left intact and new plywood is added that connects the existing rim joist to a new lateral element (not shown) inside the building. If the new element is placed directly under an overhang, then cutting the existing stucco may be necessary to attach the new system to the rim joist. In such cases, the amount of stucco removed from the rim joist should be minimized. As good practice, cutting no more than 3 inches of the stucco generally is sufficient to make the attachment, leaving the remainder of the stucco over the rim joist intact.

Recommendation Note

Existing exterior stucco walls often are part of a SWOF building's seismic-force-resisting system. Take time to understand the existing SFRS to avoid damaging it or its load path in the design of the retrofit. Clearly indicate on details the extent of demolition and load path between the existing SFRS and new elements.

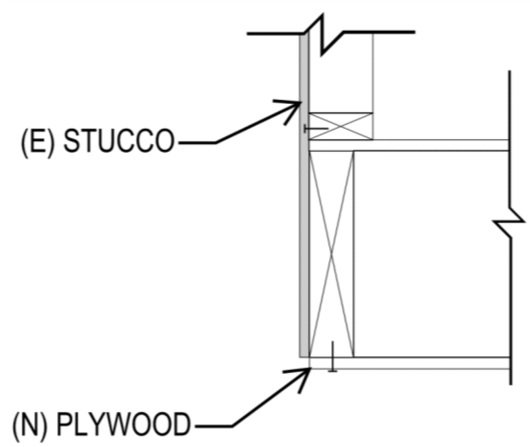


Figure 4-4 An alternative detail to stucco demolition that uses new plywood.

4.3.2 Protecting the Existing Gravity System

It is important to take time to understand the existing gravity system. Seismic retrofits should not reduce the capacity or compromise the stability of the existing gravity system. Figure 4-5 highlights a case where a new cantilever column imposes a torsional demand on the existing gravity beam, likely causing instability in the existing gravity system. In this detail, a new steel cantilever column is extended up to the underside of an existing steel wide-flange beam. The seismic load path between the structure above and the new vertical element causes torsion in the existing beam. This condition can cause the gravity girder to "roll" as the shear moves from the building to the new seismic-force-resisting system. The existing steel beam will not be adequate to transfer this torsional demand and simultaneously resist gravity demands. The full load path from the story above should be followed through to the new vertical element, and proper stiffeners, bracing, or other means should be provided to accommodate related local demands. Figure 4-6 and Figure 4-7 illustrate ways to address torsion on an existing beam when it is required to attach a new seismic-force-resisting system to it.

Recommendation Note

Take time to understand the existing gravity system to avoid damaging it or its load path. If the new SFRS requires attachment to the existing gravity system, clearly indicate on details how seismic forces will be transferred through the existing gravity system without causing damage or instability.

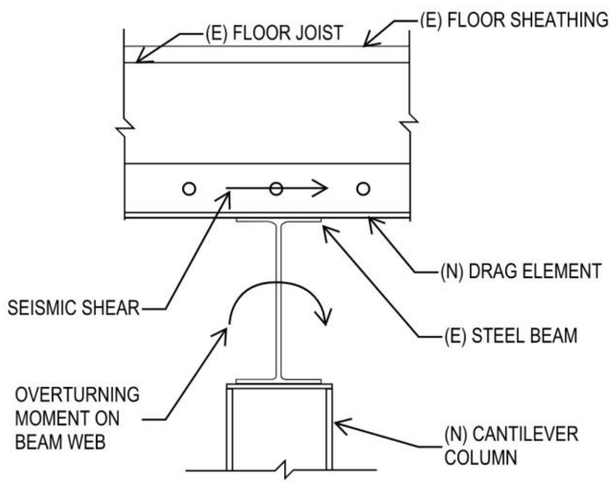


Figure 4-5 Undesirable lateral forces on existing gravity systems.

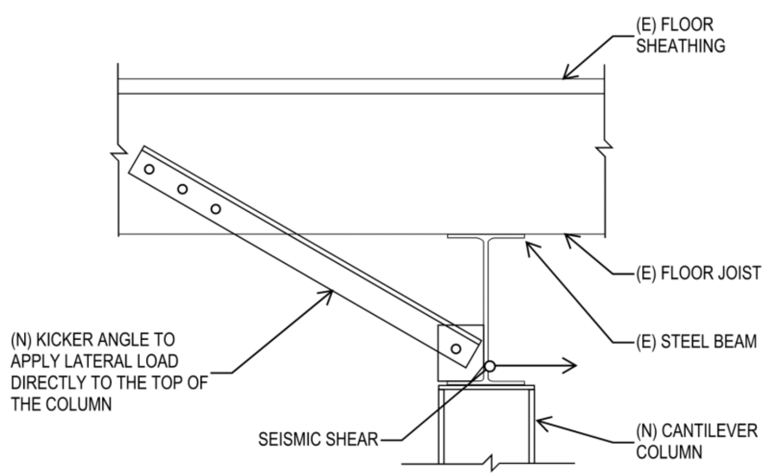


Figure 4-6 Detail illustrating how torsional forces on an existing beam can be addressed with the use of a new kicker angle.

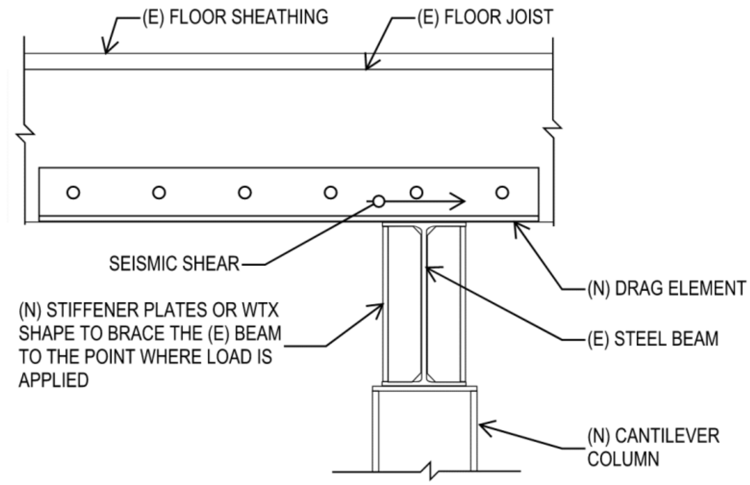


Figure 4-7 Detail illustrating how torsional forces on an existing beam can be addressed with the use of new stiffener plates.

4.4 Load Path to New Retrofit Elements

Typically, an overall evaluation of an existing diaphragm is outside the scope of a SWOF retrofit. The designer only needs to demonstrate that the load from the diaphragm can transfer into the new SFRS. This is true for both diagonal- and straight-lumber sheathed diaphragms, as well as plywood or OSB diaphragms. Besides having an adequate fastening scheme from the new lateral system to the diaphragm, there is often the bottom plate of a wall above, which is nailed through at the original diaphragm boundary, that helps distribute the load into the diaphragm. However, it is still important to understand the diaphragm construction to properly design the fasteners. Below is a discussion of important considerations when designing the load path between the existing diaphragm and the new SFRS.

4.4.1 Connections to Diaphragms with Diagonal or Straight Sheathing

Straight- and diagonal-lumber sheathing are common types of diaphragms for SWOF buildings. It is recommended that the design professional conduct a pre-design investigation of the structure to understand the diaphragm material and existing lateral-load path. In some instances, the design professional might be able to verify materials through existing openings in the ceiling or walls. In other instances, it might be necessary to remove finishes to identify the diaphragm construction. If this is done, care should be taken to avoid damaging the existing gravity and lateral systems. If the diaphragm material cannot be confirmed, it is recommended to assume the worst-case condition when designing the retrofit, to verify the as-built conditions when finishes are removed during construction, and to adjust details as necessary.

Recommendation Note

Existing diaphragms can vary in detailing and quality. Conduct a pre-design investigation to better understand the existing conditions. If this isn't possible, worst-case conditions should be assumed for design, assumptions verified during construction, and adjustments made if necessary.

Special care must be taken when new elements are attached to existing diagonal- or straight-lumber-sheathed diaphragms. For example, Figure 4-8 demonstrates a case where a new collector (or drag) was added to the underside of a straight-sheathed diaphragm. The seismic load that is being transferred to the collector comes from the second-story wall to the left of the collector, from additional second-story walls to the right of the collector, and from forces generated by mass tributary to the floor system. At a minimum, the collector should be extended to a length adequate for the unit-shear transfer from the collector to the diaphragm to be less than the diaphragm capacity. This can be the summed capacity on both sides of the collector, recognizing that the loads are being transferred from both sides. Where this is not possible, it is recommended to also provide a plywood soffit to transfer shear loads from the collector to the wall above and to the existing second-floor diaphragm.

In addition, the straight sheathing must transfer the load horizontally from the exterior wall to the drag. If built as shown on Figure 4-8, the direct load path to the new drag beam from the exterior wall will be weakened. The existing load path goes from the exterior stucco wall to the horizontal diaphragm, which consists of straight sheathing and stucco plaster. Once the horizontal stucco is broken during retrofit to place the new element, the entire load now must go through the straight-sheathed diaphragm. The capacity of the straight sheathing relies on two nails applied to each wood plank at each floor joist. Although it may be possible to justify the diaphragm capacity through calculations, it is not a desirable load path since all the load that was in the vertical stucco now must redirect itself through the bottom wall sill and into the diaphragm. This transfer diaphragm is critical to the performance of the retrofit. This situation can be enhanced by placing new plywood on the underside of the framing, between the existing ring beam and new drag as shown in Figure 4-9.

Adding the new plywood provides a direct load path between the exterior stucco wall and the new drag. If new plywood is added to the underside of framing, it is good practice to extend the plywood on both sides of the new drag so that the load transfer from each side of the diaphragm is more reliable. How much to extend the plywood on the right side of the drag (i.e., interior side) is based on judgement and accessibility, but whenever possible a 4-foot minimum dimension is recommended.

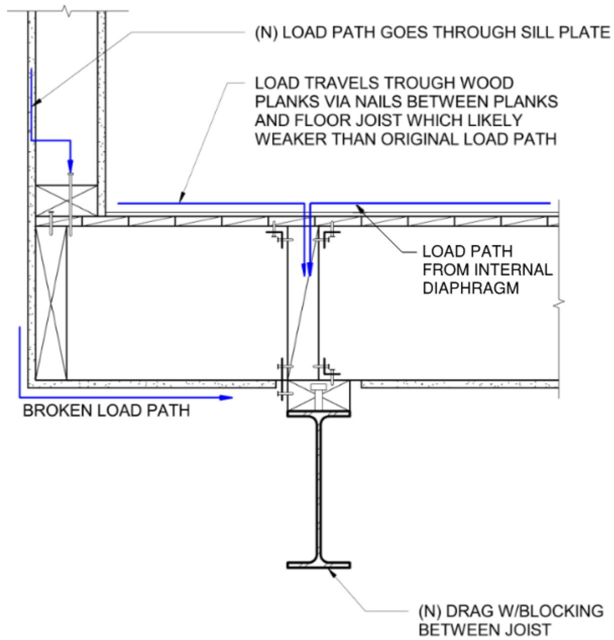


Figure 4-8 Force transfer through straight sheathing to a new collector (or drag).

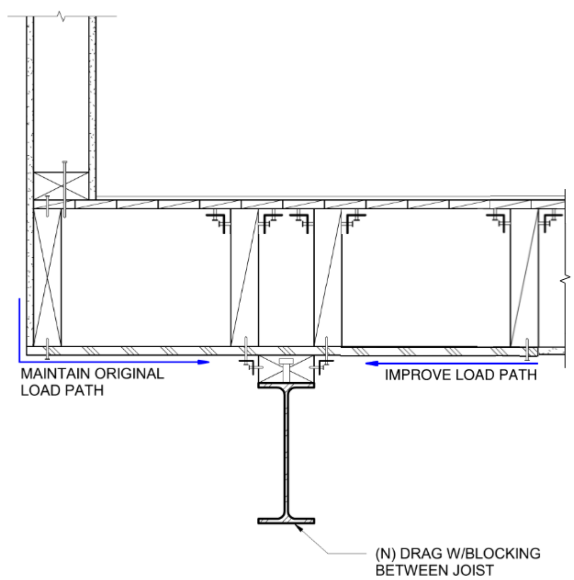


Figure 4-9 Force transfer through straight sheathing to a new collector (or drag), with the addition of plywood to the underside of existing framing.

4.5 Collectors, Moment Frame Beams, and Columns

4.5.1 Collector Length Limitations

For new buildings, ASCE/SEI 7 allows the design professional to determine the minimum length of a collector using code forces, without applying the overstrength factor. Many SWOF retrofit ordinances permit this approach when determining the minimum length of a new collector, and as a result, the collector often does not extend the full length of the building. However, this is of concern in a SWOF retrofit that relies heavily on existing nonductile materials to transfer the load, and the existing diaphragm strength may not be consistent between every location in the building. In addition, SWOF buildings typically do not have well-defined load paths, such that designing collectors based on assumed diaphragm capacities may yield undesirable behavior in seismic events. Figure 4-10 illustrates the load path of a new seismic-force-resisting system along the open front of a SWOF building, including a long collector.

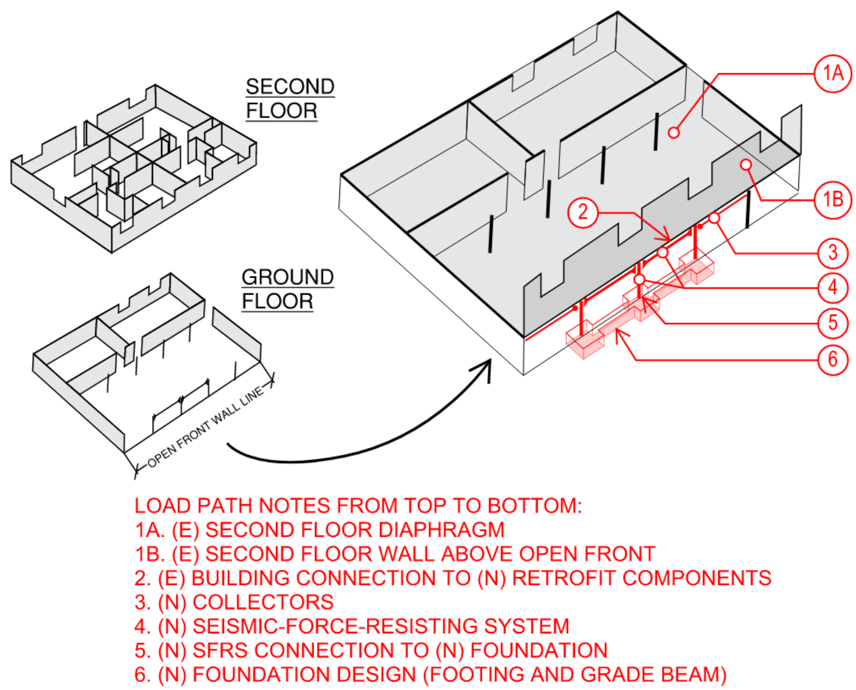


Figure 4-10 Typical load path of a new seismic-force-resisting system along the open front of a SWOF building.

For this reason, it is recommended to determine the minimum length of the new drag based on the capacity of the new vertical seismic-force-resisting system. Although this does not necessarily address the ambiguity related to the capacity of the diaphragm, sizing the collector using this approach will yield a larger drag length and increase the probability that the yield mechanism occurs in a more predictable manner. As an alternative, and where practicable, the design professional may choose to design the collector to extend the full length of the diaphragm.

However, placing a long collector should not be used as a reason to minimize the number of vertical resisting elements. If a retrofit collector becomes too long before reaching the next vertical resisting element, it may have undesirable behavior due to differences in deformation capacities between the nonconforming diaphragm materials and the new collector. For this reason, it is good practice to avoid having new vertical elements spaced more than 60 feet apart or 30 feet from the ends of buildings. Where this is not possible, it is recommended that collector designs account for expected deformations between the diaphragm and the collector itself.

Recommendation Note

Use an increased number of smaller vertical elements distributed along the length of the line as an effective way to reduce the load carried by collectors and their associated deformations. Consider placing new vertical seismic-force-resisting elements no more than 60 feet from each other or 30 feet from building ends to avoid collector lengths that may not be compatible with existing nonconforming materials. Minimum collector lengths should be determined using the capacity of the new vertical seismic-force-resisting elements, and where practicable should extend the entire length of the diaphragm.

4.5.2 Vertical Elements Located Outside of the Building Footprint

In some jurisdictions, modifications to foundations below occupied stories requires shoring to support the second story, which then requires a separate permit. To avoid this requirement, new retrofit columns are located outside of the building footprint, resulting in eccentricities between the vertical element and the drag line. This is often done to minimize interaction of the new grade beam foundation with the existing gravity foundations. The new foundation is poured adjacent to an existing gravity column footing. The second-floor collector is often a girder that supports gravity load, which is now eccentric to the column, as shown in Figure 4-11. The eccentricity between the drag line and cantilever column creates a torsion, or moment, that must be resisted by the existing structural system.

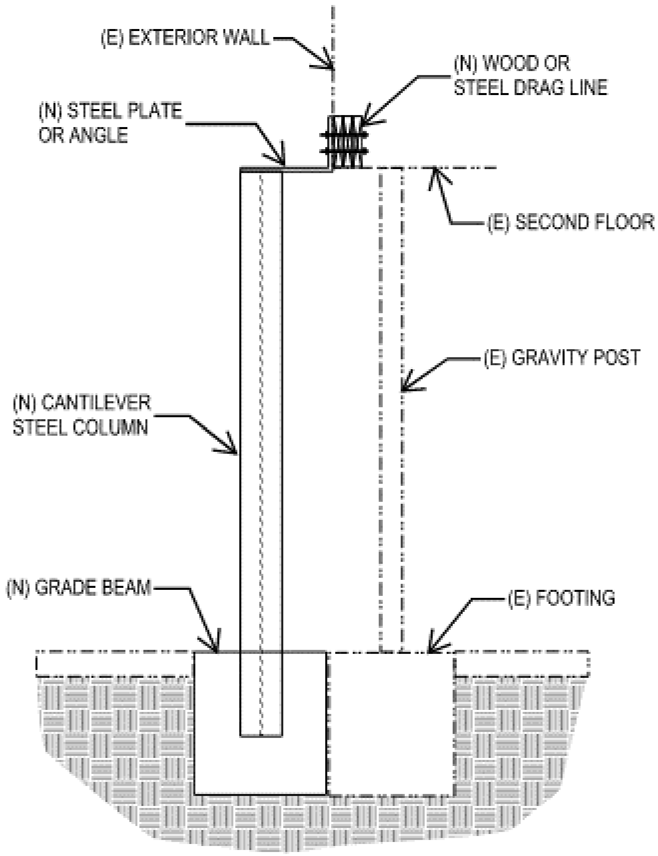


Figure 4-11 New cantilever column located outside the building footprint, introducing an eccentricity.

If geometric constraints of the existing building elements require offsets of the retrofit elements from the existing building diaphragm, it is critical that the load path including the torsion or moment be adequately designed for. There are two strategies for resolving this torsion.

One strategy is to resolve the torsion in the new SFRS; however, this is not a recommended approach. Some designers try to design the steel cantilever columns or moment frames for the resulting torsion, but this generates significant design issues. Currently, AISC does not have provisions for steel inelastic behavior with a torsion-flexure or torsion-shear failure mode. It is not clear how stable the column will be when resisting torsion while a hinge forms at the base of the column. The best resource for torsion on steel sections is AISC's Design Guide 9, *Torsional Analysis of Structural Steel Members* (AISC, 1997). This document is limited to the elastic design of steel elements. The design professional can choose to design a cantilever column to remain elastic using AISC Design Guide 9. However, this option would require significantly bigger sections or more columns.

In the cantilever column condition, warping stresses normal to the cross section of the flanges at the base of the column are high when considering even small eccentricities, such as those within the flange width of the column. Figure 4-12 illustrates the forces that are formed in an eccentric column. For this reason, when torsional moments are present, the use of wide-flange cantilever columns, which have limited torsional capacity, is strongly discouraged.

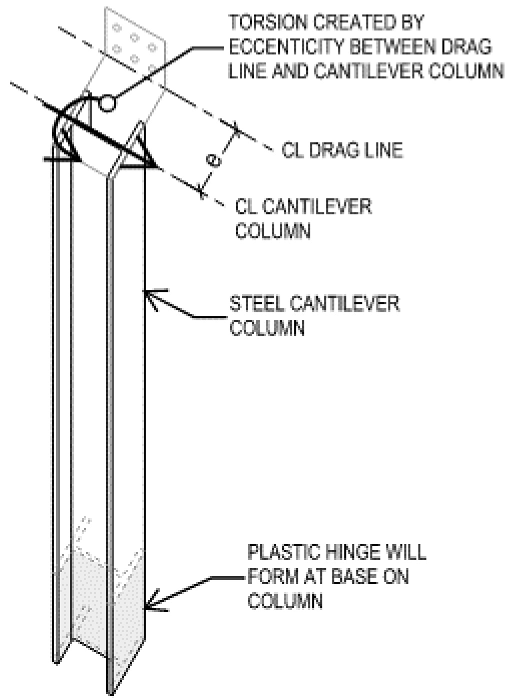


Figure 4-12 Offset cantilever column showing torsion created by the eccentricity with the drag line.

Recommendation Note

Columns are difficult to justify for torsion induced by seismic loads. Care should be taken to properly design the connections to resist the torsional force back into the diaphragm, rather than trying to design the column for torsion.

While closed shapes, such as a hollow structural sections (HSS), are more efficient at resisting torsion, they still do not provide a reliable torsional yield mechanism. It is therefore strongly discouraged to rely on an HSS section to resolve the torsion. Instead, it is recommended that the moment due to the eccentricity be carried into the existing diaphragm, with an adequate load path designed for shear and moment. As shown in figure 4-13, a wide connection plate, or horizontal angle, can be designed to transfer the moment due to the eccentric connection and resolve it into a tension-compression couple. For this configuration to work, drag lines need to be designed to resist the tension-compression forces back into the diaphragm. A question could rise as to whether the connection needs to be designed for 100% in-plane load plus 30% out-of-plane load per ASCE/SEI 7-16 Section 12.5.4 because the connection will experience both in-plane forces, as illustrated in Figure 4-13, and out-of-plane forces, as described in Section 4.6.3. However, if the retrofit is in a primary building line and not part of an intersecting seismic-force-resisting system, then the connection design may be based on 100% in-plane load with appropriate eccentric tension-compression forces, and on 100% out-of-plane loads, independently.

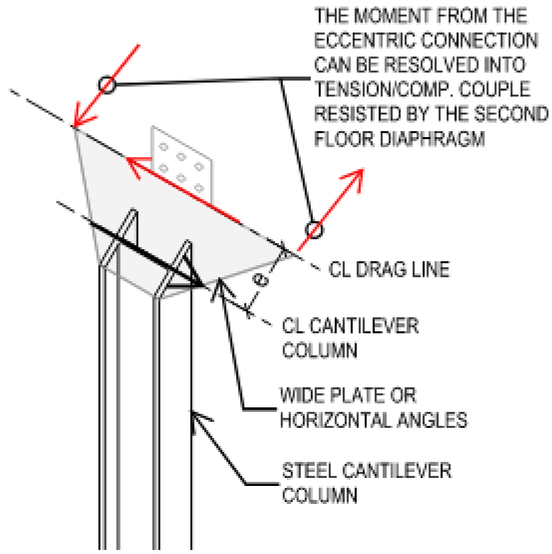


Figure 4-13 Offset cantilever column with wide plate used to resolve the tension-compression couple into the diaphragm.

4.5.3 Bracing Requirements of New Steel Systems

Moment Frames. AISC 341-16 Section E3.4b requires that beams in special moment frames be "braced to satisfy the requirements for highly ductile members" in accordance with Section D1.2b (stability of highly ductile beams). This is different than beams in ordinary moment frames, for which AISC 341-16 Section E1.5a does not require stability bracing. For line retrofits, the expected ductility demand on a new moment frame is not large because the unretrofitted portion of the building contains low ductility. In addition, line retrofits are typically designed with a low *R* value, such as a maximum *R*=3.5 for the City of Los Angeles (LADBS, 2015). As such, even if the design professional chooses to place a special moment frame in a line retrofit, the moment frame bracing requirements are not necessary. However, if special moment frames are placed in a line retrofit, the design professional is encouraged to design the beam bracing in case the owner decides in the future to extend the retrofit to an entire story. Beam bracing may require the addition of supplemental steel beams, columns, or both to meet AISC strength and stiffness requirements, and it's therefore advantageous to place them while new frames are being added.

Recommendation Note

AISC 341 requires that beams in special moment frames (SMF) be braced. This requirement can be applicable for story retrofits, where the use of SMFs is common, but bracing generally is not required for line retrofits because of the low ductility expectations and the use of low *R* values.

However, as good practice, SMF beams should be braced in case a line retrofit is expanded in the future to a story retrofit.

Cantilever Column System. Section E6.4b of AISC 341-10 and AISC 341-16 requires that special cantilever columns be "braced to satisfy the requirements applicable to beams classified as moderately ductile members" in accordance with Section D1.2a (stability of moderate ductile beams). The two versions of AISC 341 have slightly different equations but yield similar results. However, the lateral-bracing equations are intended for a different system configuration (moment frames versus cantilever columns). As such, the 2021 IEBC, as well as some jurisdictions, like the City of San Francisco, have clarified that for cantilever column systems, the AISC results (i.e., the unbraced length) are for columns that are twice their actual height (DBI, 2017). If these equations are applied to commonly used W-sections for cantilever columns in SWOF retrofits, they produce maximum unbraced lengths between 16 feet and 18 feet. It then follows that when using these criteria, and depending on column size, lateral bracing often will not be required for cantilever columns that are less than 8 feet to 9 feet in height.

New criteria for unbraced length and bracing requirements were under consideration by AISC at the time of preparing this report. The new requirements, if approved, would decrease the permitted unbraced length but would provide the following three exceptions:

- The first exception would allow bracing to be omitted for round and square HSS sections.
- The second exception would allow bracing to be omitted for weak-axis bending.
- The third exception mirrors the San Francisco approach and would allow bracing to be omitted when the required bracing distance is greater than or equal to twice the column height.

Although the first and third proposed exceptions are aligned with current SWOF retrofit approaches, the new requirements reducing the unbraced length would make commonly used wide-flange sections no longer adequate without bracing at their tops. Bracing the top of the column could add significant cost and complications and may discourage design professionals from using wide-flange sections for cantilever columns. Since the new unbraced length equation is not yet approved, and its applicability to SWOF retrofits has not been clarified, it is recommended that AISC 341-16, Section E6.4b be used for calculating unbraced length.

Recommendation Note

Bracing at the top of a cantilever column that uses a wide-flange section may be omitted if the required maximum unbraced length calculated per AISC 341-16 Section E6.4b is greater than or equal to twice the column height. Bracing is also recommended to be omitted if round or square HSS sections columns are used.

4.6 Foundations

Design of foundations for new seismic-force-resisting systems can be challenging because they often must be integrated with the existing foundation system. The following sections provide a discussion of important aspects to consider when designing new foundations for SWOF retrofits.

4.6.1 Sliding, Uplift, Overturning, and Soil Bearing Considerations

Chapter 12 of ASCE/SEI 7-16 requires foundations to be checked for sliding for new construction. The resistance to sliding is a combination of friction between the soil and the footing, and passive pressure at the end of the footing. The amount of passive pressure achieved at the end of a footing is relatively minor, so the primary resistance to sliding is the shear friction between the soil and the footing. Typically, the friction coefficient between the soil and the footing is on the order of 0.3. This can make it difficult to justify sliding numerically because the seismic base shear coefficients used to design SWOF retrofits are often much greater than 0.3 for high-seismic zones.

Fortunately, sliding typically is not a detriment to building performance if the sliding does not break other parts of the load path. When sliding is not able to be justified numerically, it is recommended that new footings be tied to the existing foundation system. This ensures that the entire foundation system moves as one unit. Where new foundations are cast alongside existing foundations, it is recommended that the new foundation is doweled to the existing every few feet using adhesive or mechanical anchors. Where new foundations are not cast alongside an existing foundation, it is recommended that the new foundation be extended to intersect with perpendicular existing foundations and doweled with adhesive or mechanical anchors.

Recommendation Note

The foundation system of a new seismic-force-resisting system should be tied to the existing foundation system to minimize possible negative effects of sliding, uplift, and overturning.

Uplift and overturning stability can be difficult to resolve when the new seismic-force-resisting system is not carrying the gravity weight of the structure. If a design cannot meet the uplift and overturning stability checks required by the code, it will likely rock during a seismic event. Global rocking of foundations often is thought of as providing beneficial energy dissipation if it does not damage adjacent foundation and framing elements. For example, a frame that rocks will rotate as a rigid body and can possibly damage the overhead frame-to-diaphragm connections. To avoid this, it is recommended that new foundations be connected to existing foundations (preferably perpendicular walls) so that the extra building weight that is engaged can help resist overturning.

Soil bearing should be checked to meet ASCE/SEI 7 with relevant material requirements. When designing moment frame or shear wall retrofits, both the structural foundations design and soil bearing checks are performed to code-level forces (i.e., no overstrength forces). However, when designing cantilever columns, the structural foundation is designed for overstrength forces per ASCE/SEI 7-16 Section 12.2.5.2.

As an alternative to overstrength forces, ASCE/SEI 7-16 Section 12.14.3.2 permits the use of capacity-limited horizontal seismic load. This becomes important when designing elements using FEMA P-807, which is based on the capacity of the existing building. If capacity-based forces are used to check the new seismic-force-resisting elements, the same loading can be carried down to check the soil. In such cases, the soil demands need to be checked against the expected soil capacities, which are larger than the presumptive load-bearing values of soils provided in Section 1806 of the 2021 *International Building Code* (IBC) (ICC, 2021b). To convert the 2021 IBC Section 1806 presumptive values to expected soil capacities, it is recommended that they be multiplied by three unless otherwise noted by a geotechnical report.

Recommendation Note

Check soil pressures against presumptive values provided in the IBC or those given by a geotechnical report. When checking soil pressures against capacity-based loads, it is recommended that the presumptive values be converted to expected soil capacities by multiplying IBC presumptive values by a factor of three unless otherwise noted by a geotechnical report.

4.6.2 Recommended Detailing for Fixed-Base Retrofits

Where columns are embedded in the grade beam to create fixity, the design should be based on overstrength or capacity-based forces per AISC 341-16 Section D2.6. In addition, 2021 IBC Section 1905.1.5 indicates that ductile detailing requirements specified in Section 18.13 of ACI 318-19, *Building Code Requirements for Structural Concrete* (ACI, 2019), should be met for shallow-foundation grade beams. The current code does not have an exception for omitting the ductile detailing when designing for overstrength or capacity-based forces on shallow-foundation grade beams. However, at the time of preparing this report, there is a code-change proposal for the next version of IBC that, if passed, would add a new exception to Chapter 18 for shallow-foundation grade beams when the expected differential settlement is small. Given that expected ductility demands for new SWOF retrofit foundations that are sized for overstrength or capacity-based forces are small, it is considered acceptable to omit the ductile detailing requirements specified in ACI 318 Chapter 18. However, there are still important detailing considerations described in this section when designing fixed-base connections.

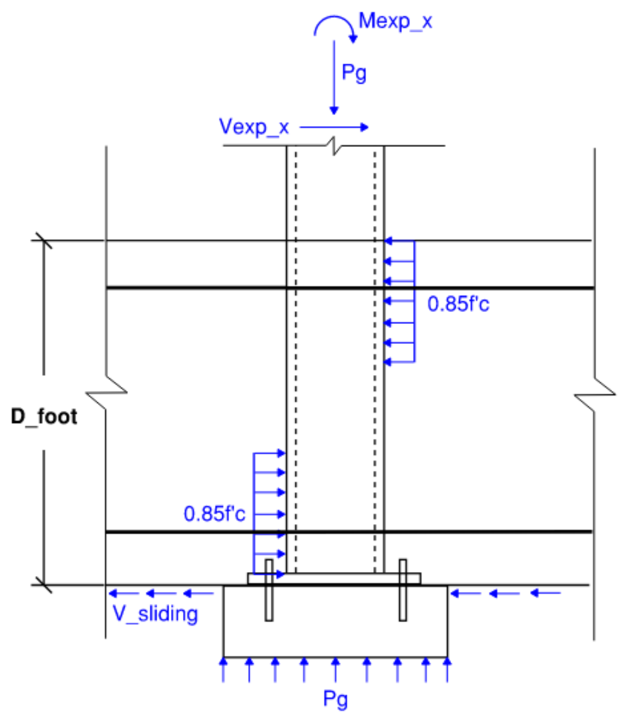


Figure 4-14 Forces at embedded column that is not near the end of a grade beam.

The foundation design for the embedded column can be simplified by checking the stresses on opposite sides of the column, as illustrated Figure 4-14, when the column is not near the end of the grade beam. When the column is located near the end of the grade beam, it is important to recognize that there is a possible breakout zone that may form when the column is pushing against the edge. When the column reverses direction, the breakout zone will form at the bottom of the grade beam. For this reason, it is important to place U-shaped bars or welded bars that can transfer the load back into the center of the grade beam, as illustrated in Figure 4-15.

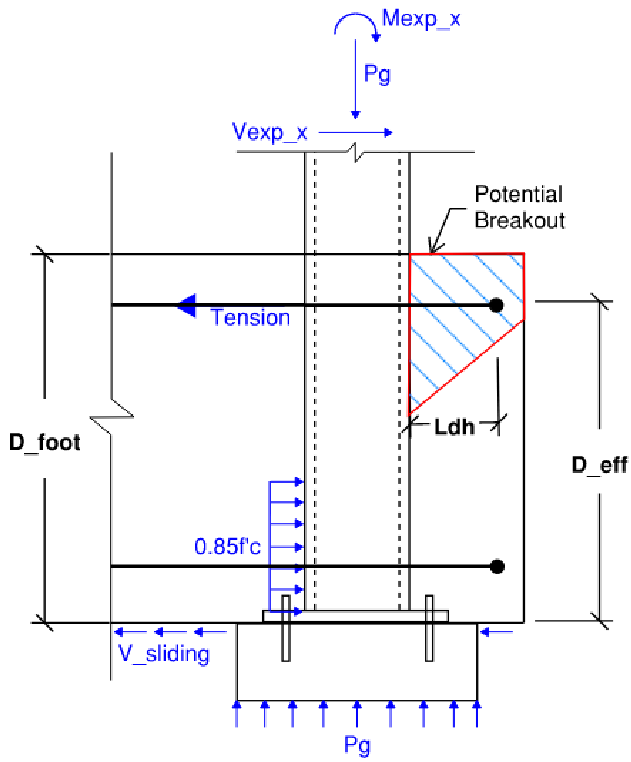


Figure 4-15 Possible embedded column breakout zone at edge condition.

In addition, there is another concern where the applied loads at the base are high. The oscillation of the column may cause degradation of the concrete at the face of the column. For this reason, unless it is justified otherwise, it is good practice to place closely spaced stirrups that will extend a horizontal distance that is equal to at least half the grade beam depth. It is recommended that #4 diameter stirrups be placed no farther than 4-inches on center within the critical connection zone.

Recommendation Note

For embedded columns, ductile detailing is not necessary if the design is based on overstrength or capacity-based forces. However, closely placed stirrups should be added in the connection zone, and where closed sections (e.g., HSS) are to be used, casting concrete or grout inside of the closed sections at the column-to-foundation interface is recommended.

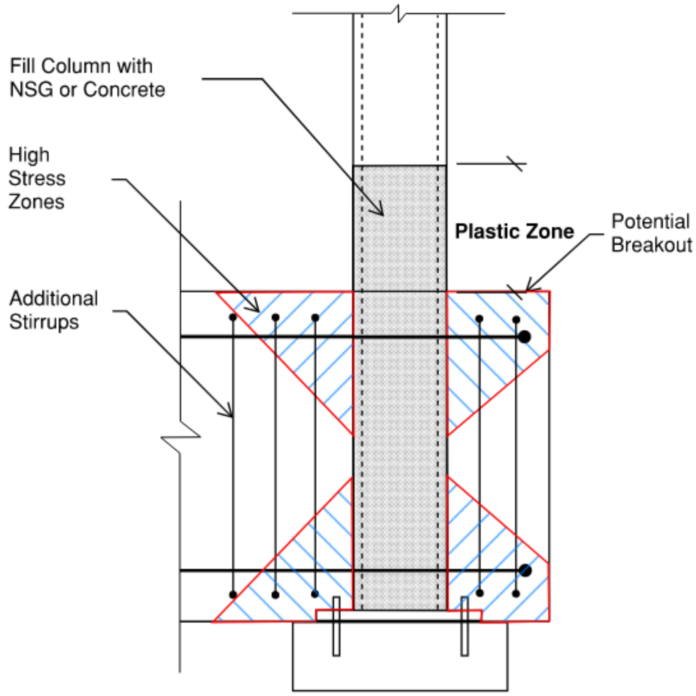


Figure 4-16 Recommended embedded-column details.

The use of HSS sections in cantilever column systems may pose concerns that the steel-to-concrete interface will not be ductile due to local buckling behavior. Similar concerns with HSS sections have risen in the past for concentrically braced frames. In those cases, casting concrete or grout inside of the HSS section has been one approach to mitigate local buckling. The design professional is cautioned against the use of HSS sections where ductility is desired. But if they are to be used, casting concrete or grout inside of the HSS section at the column-to-foundation interface is recommended. Although round shapes behave better than rectangular shapes, this recommendation is extended to all closed-shaped sections. The grout should extend above the top of the footing at least 12 inches in the area where the plastic hinge is expected to form (Figure 4-16).

4.6.3 Weak-Axis Implications for Fixed-Base Retrofits

The design professional should consider drift of the existing building in all directions when detailing a new fixed-based retrofit systems to avoid deformation incompatibility. Many SWOF retrofit designs are controlled by drift requirements. This is especially true in Southern California, where some ordinances require the drift limit to be 2% when there is no plywood in the walls above the retrofitted wall line. To meet the drift requirement, design professionals commonly use a fixed connection at the base of the steel moment frame columns. The fixity is typically achieved by embedding the steel column into a new footing or grade beam. The same is also done for cantilever columns.

Embedding columns will also create fixity at the base of the column section in the direction perpendicular to the principal direction of the retrofitted system. This means that large out-of-plane forces can be generated in the new retrofit elements, and as a result, in the connection between the top of the column and the second-floor diaphragm. These weak-axis forces can fail the connection of the cantilever column to the diaphragm if not considered in the design and can compromise the benefits of the retrofit. Figure 4-17 and Figure 4-18 illustrate how the building movement will tend to pull the new system in multiple directions, which can cause a compatibility issue.

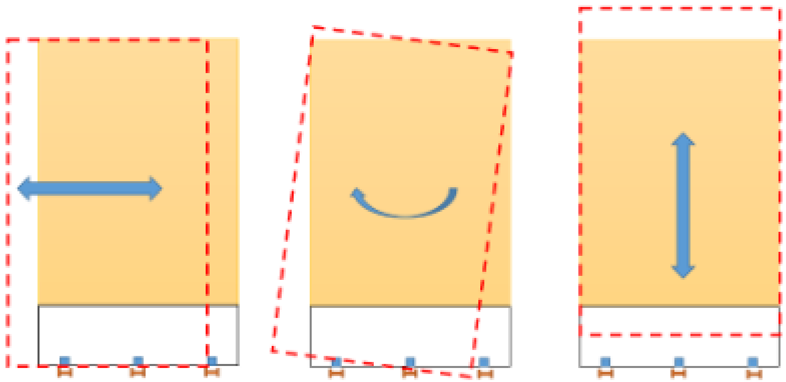


Figure 4-17 Building plan illustrating building drifts.

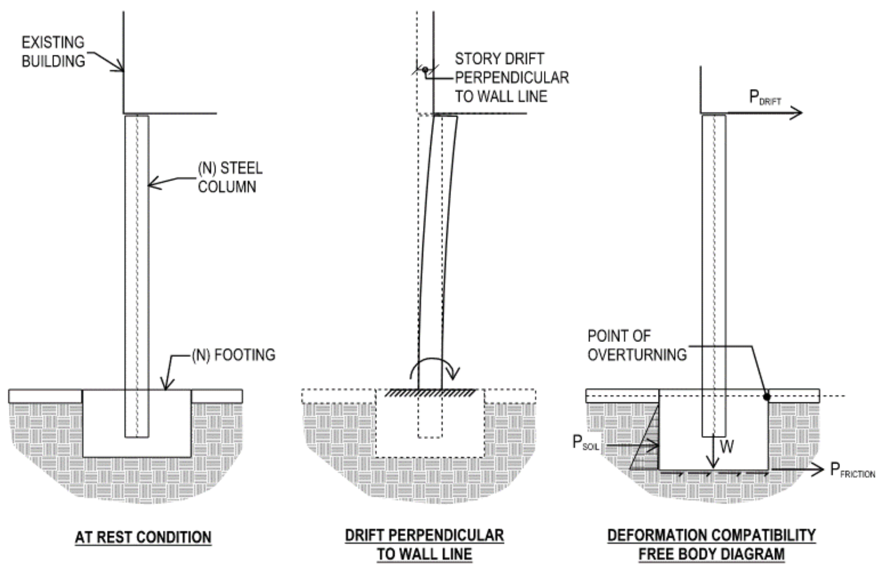


Figure 4-18 Deformation compatibility perpendicular to the open front.

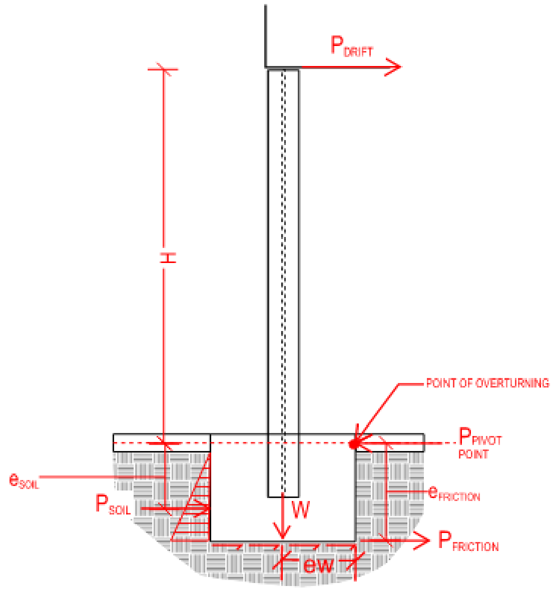


Figure 4-19 Fixed-base overturning forces.

For moment frames and cantilever columns with full column fixity at their bases, some rotation will occur at the column base under weak-axis loading. Unless detailed evaluation of this behavior is conducted, it is recommended that potential rotation be neglected for the purposes of the column-top connection design. In addition, to have weak-axis fixed-base behavior, moment capacity must be provided by the foundation and soil bearing system. Several mechanisms developing this moment capacity are shown in Figure 4-18. Although it may be tempting to calculate the moment capacity and use this to calculate the connection load at the second-floor diaphragm, the variability of the soil capacity makes it difficult to predict the column weak-axis moment that can be developed. For this reason, when using a fixed-column retrofit, the design professional is recommended to design the top-of-column connection for weak-axis loading to the diaphragm for one of the following:

- **Capacity Design**: Designing for the force required to yield the steel column, including the effects of strain hardening (i.e., AISC expected strength).
- **Drift Demand**: Determine and design for forces associated with pushing the fixed column to a 5% drift at the top of the column. The limit of 5% was chosen as the minimum inelastic drift that is expected based on the analysis described in Chapter 2.
- **Deflection Allowance**: Provide a connection at the top of the vertical element to greatly reduce or eliminate transfer of the weak axis reaction. In some cases, this can be achieved by using slotted connections or similar approaches. However, this option may not work where a connection needs to transfer other perpendicular loads, as described in Section 4.5.2.

It is noted that a moment frame with a near-surface base plate designed as a pinned connection to the footing may be assumed also to be pinned in the perpendicular direction, eliminating the need for a compatibility check in the perpendicular direction.

Recommendation Note

The top connection of a fixed-base column should be designed to resist weak-axis (or perpendicular) loading based on:

- Capacity of the column,
- Demand generated by 5% drift, or
- Provide a connection that accommodates 5% drift.

4.6.4 Protecting Existing Foundations

Where new foundations are placed immediately alongside or impinge on existing foundations (as occurs when existing footings need to be cut back to allow for new footings), there are several important considerations that affect detailing. First, the new foundation work should not undermine the existing foundation. It is recommended to keep the bottom elevation of the new foundation within several inches of the existing foundation. Where possible, this is achieved by using a new foundation with depth to match the existing. Where this is not possible, it is recommended to fully remove the existing foundation and cast a new deeper foundation, monolithic with the new retrofit foundation. Second, if the existing foundation is being cut back or undermined, it is recommended to shore the existing beams that are supported by the foundation during the foundation work. Third, where existing foundations are cut back, it is recommended to provide dowelling between the existing foundation and new foundation. The loads that need to be considered when designing the doweling include:

- Vertical reaction to regain gravity load capacity that was provided by portion of existing foundation that has been removed,
- Vertical reaction from seismic loading to new foundation that might be resisted by the existing foundation,
- Horizontal reaction due to the existing foundation acting as a key to resist horizontal sliding, and
- Moment between new foundation and existing due to the existing foundation acting as a key to resist horizontal sliding.

In addition, the existing foundation reinforcing should be maintained and cast into the new foundation so as to not weaken the existing foundation. If it is not retained, the flexural capacity of the existing foundation could be reduced.

Recommendation Note

Care should be taken when constructing a new footing adjacent to an existing footing. In particular:

- Avoid undermining the footing whenever possible, or
- Demo and cast monolithic footing while providing proper shoring for existing building during construction.

The connection between the new and existing footings should be properly designed to account for the loads of both the existing structure and new elements.

4.6.5 Protecting Existing Underground Utilities

It has been reported that design professionals do not always specify what to do when a contractor encounters an underground utility line during a SWOF retrofit. Structural engineers might believe that underground utilities should be addressed by the mechanical engineer or architect, wheras they might think this is the responsibility of the structural engineer. This poses a problem when a contractor encounters underground utilities that can break, such as gas lines. If the line is encased in concrete, there is a potential for it to rupture during a seismic event due to pinching when the footing slides, rocks, or uplifts. This document is not providing guidance on the responsibility for these issues or how to coordinate detailing around utilities. This section is flagging this as a potential issue that should be resolved between the designers, contractors, and authority having jurisdiction. This issue can be mitigated by rerouting utility lines or by sleeving the foundation to allow movement between the existing utility and the footing.

Recommendation Note

Design teams should coordinate during design and construction of SWOF retrofits regarding how to handle existing underground utilities that may be encountered.

4.7 Quality Assurance Recommendations

Seismic retrofit projects can be especially challenging when there are no as-built construction drawings, no certainty on the existing framing, or limited information about the materials used in the construction of the building. As a result, structural observations should be done for all SWOF retrofit projects. (FEMA P-807 has specific requirements addressing surveying the existing building both to determine materials of construction and load-path detailing.)

It is recommended that the owner employ the engineer of record responsible for the structural design or another registered engineer designated by the engineer of record to perform structural observations as defined in the applicable code or standard. It is recommended that the designated design professional visit the site to verify applicable existing materials and framing details in the location of the new work. Where the condition of the materials is observed to be deteriorated or structurally compromised, the design professional should work with a testing lab and contractor to address the situation. It is important that construction drawings clearly identify milestones for when the contractor should notify the design professional to visit the site. Special Inspections should be provided as required by the applicable code or standard. Additional inspections should be noted on drawings as required by the authority having jurisdiction.

Recommendation Note

Owners should employ design professionals to perform site observations during design and construction of SWOF retrofits to ensure that the designs align with as-built conditions and construction is in general conformance with approved documents.

Chapter 5: Retrofit Design Examples

5.1 Introduction

Chapter 5 illustrates designs of vulnerability-based seismic retrofits for SWOF buildings using line and FEMA P-807 methods. The examples are intended to provide end-to-end examples of retrofit designs and to illustrate implementation of the recommendations described in Chapter 4. The intended audience is practicing engineers, including engineers new to SWOF building retrofits as well as those with retrofit experience.

Section 5.2 introduces the example buildings used in the retrofit examples. Section 5.3 provides an end-to-end retrofit design example using an optimized line retrofit design method, followed by related topical discussions. Section 5.4 provides an end-to-end retrofit design example using a FEMA P-807 design method and topical discussions. The calculations in this chapter are excerpted from more complete sets of calculations for both retrofit designs that can be found at https://femap8071.atcouncil.org/.

5.2 Example Buildings

This section describes the example buildings used as the basis for the Chapter 5 design examples. The buildings are identified as long side open (LO) and short side open (SO). These were identified as Type A and Type B buildings, respectively, in soft-story screening forms developed by the City of Los Angeles and other Southern California jurisdictions. The building inventory research described in Appendix A identified that these two building types are prevalent in the existing building stock in California. Another important characteristic of the example buildings is that a significant portion of the first story includes occupied residential units and the interior framed walls that occur with these units. The details of the Chapter 5 example buildings are similar to but may not be exactly the same as the buildings used for the analytical studies discussed in Chapter 2 and Chapter 3.

5.2.1 Long-Side-Open Building

The primary building type used for the design examples is long side open. The building occurs in both two-story and three-story versions and is roughly rectangular in plan. The building has first-story parking located under a portion of the building footprint on a long side, as seen in Figure 5-1. The portion of the first story (about 50%) that is not parking contains residential units. The following provides a general description of the example LO building.

Figure 5-1 Example of a long-side-open building with parking along one long side.

Table 5-1 Characteristics of the Long-Side-Open Example Building

Item	Description
Overall Plan Dimensions	36 ft × 100 ft
Occupied Dimensions at First Story	36 ft × 80 ft
Assumed Date of Original Construction	1950–1960
Number of Units	7 for two story 12 for three story
Floor-to-Floor Height	9 ft
Story Clear Height	8 ft
Exterior Wall Finish	Stucco
Interior Wall Finish	Gypsum Wallboard (1)
Floor Sheathing	Diagonal Lumber Sheathing (2)

(1) Consistent with the Chapter 2 analytical study weak-wall combination.

(2) Consistent with the Chapter 2 analytical study strong-diaphragm combination.

5.2.1.1 BUILDING PLANS AND ELEVATIONS

Figure 5-2 and Figure 5-3 provide a schematic illustration of the building plans at each floor level, and Figure 5-4 provides elevations of the three-story building. This building is based on the example building from FEMA P-2006, where it was used as an example building for illustration of a weak-story retrofit using ASCE/SEI 41 procedures.

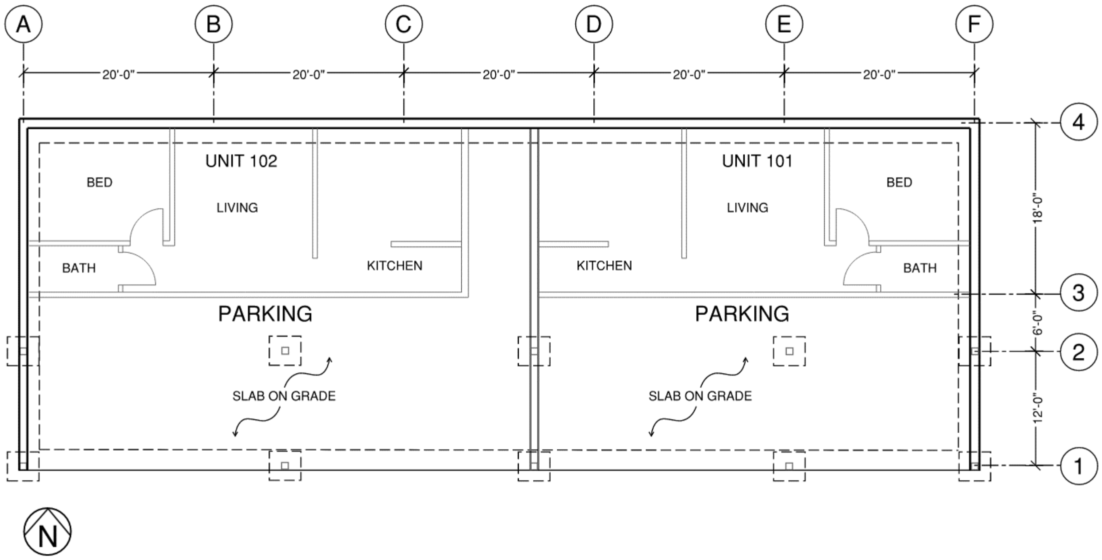


Figure 5-2 First-story plan of the design example long-side-open building.

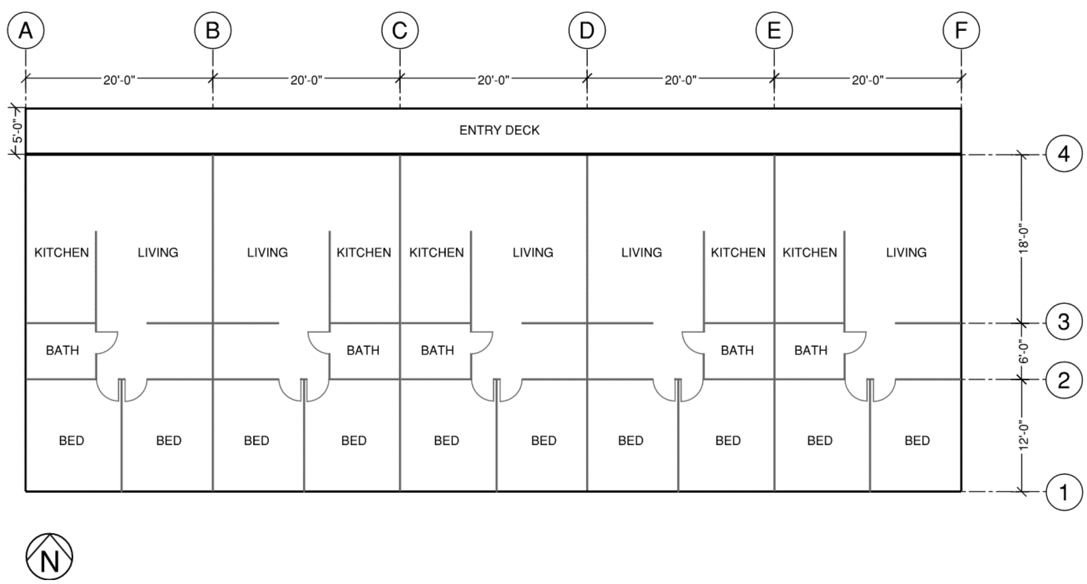


Figure 5-3 Second- and third-story plan of the design example long-side-open building.

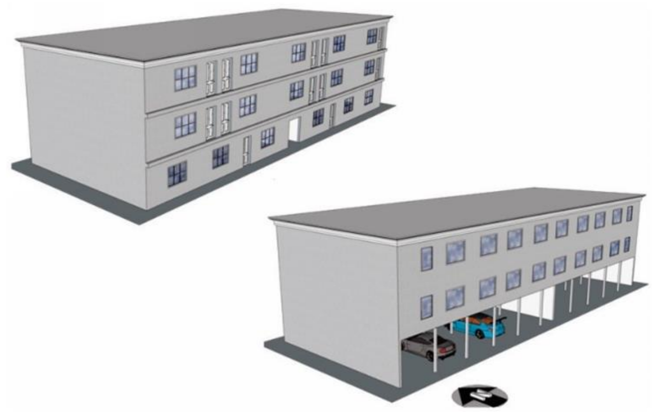


Figure 5-4 Elevations of the design example long-side-open building (credit: FEMA P-2006).

5.2.1.2 MATERIALS OF CONSTRUCTION AND WEIGHTS USED FOR SEISMIC MASS

In order to design a SWOF building seismic retrofit, it is necessary to understand the materials used to construct the existing building. Differences in existing finish materials can make a significant difference in the weight and calculated seismic demands, as well as the seismic capacity. For this reason, it is recommended to determine what finish materials are in place. While this applies equally to interior and exterior finish materials, most significantly it is important to know whether the interior finish on existing walls and ceilings are primarily constructed with plaster on wood lath or gypsum wallboard. This can be determined by accessing representative areas in the occupied building.

Recommendation Note

The weight, seismic mass, and seismic demand in a SWOF building can vary significantly with the materials of construction, as can the seismic capacity. For this reason, the materials of construction should be identified at the start of retrofit design. This should include both interior and exterior finish materials for walls, floors, and ceilings. It is also recommended that interior wall layouts be determined.

The most accurate determination of building weight is made by both identifying the finish materials and determining the plan layout of the apartment interior walls. Where unit layouts are similar it is generally adequate to determine interior wall layouts for a limited number of representative units. Using this information, the total weight of the interior walls at each floor of the building can be summed; often this is divided by the floor square footage to determine the average weight of interior walls per square foot of floor area. The FEMA P-807 design method requires that plans be developed identifying interior and exterior walls at each story level. When using the FEMA P-807 method, a detailed weight take-off for the interior walls should be provided. In order to facilitate this, Chapter 4 of FEMA P-807 specifically requires a building survey.

The line retrofit design method does not require that plans including interior walls be developed. While it is recommended to develop such plans, it is also possible to use an assumed weight for the interior walls. In preparing this report, a limited study was conducted to determine the weight of interior walls in common SWOF building configurations. It was determined that the gypsum wallboard interior wall weight was on average 7 pounds per square foot of floor area at each story, and a maximum of 9 pounds per square foot of floor area. This includes walls interior to the unit and between units but does not include building exterior walls. Similarly, plaster-on-wood lath walls were determined to be an average of 18 pounds per square foot of floor area, with a maximum of 23 pounds per square foot. In addition, plaster ceilings increase the weight at each floor level by approximately 6 pounds per square foot. For purposes of this design example, interior wall layouts are shown, and calculation of the weight based on interior wall plans is demonstrated.

Based on the collected building inventory information (Appendix A) and the age of construction, the typical wall and ceiling finish materials for these examples are assumed to be stucco on the exterior and gypsum wallboard on the interior. This combination is identified as the weak-wall combination in the Chapter 2 analytical studies, whereas the combination of stucco and plaster on wood lath is identified as strong wall combination. These designations highlight differences in the wall in-plane shear strength in addition to the weight.

For purposes of the design examples, diagonal lumber sheathing was selected as the example building floor and roof sheathing. Building inventory research indicated that lumber sheathed diaphragms were still common in Southern California at the time of original construction of the example buildings (1950s to 1960s). Both diagonal- and straight-lumber-sheathed diaphragms are thought to be present in the building stock. The diagonal-lumber-sheathed diaphragm in this example is stronger than corresponding straight-lumber-sheathed diaphragms.

Table 5-2 through Table 5-6 provide the detailed weight take-offs used to establish the seismic mass for the seismic retrofit designs.

Table 5-2 Floor Assembly Detailed Weight Take-off

Typical Floor		Floor Over Parking	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
Floor finish (carpet and pad)	1.4	Floor finish (carpet and pad)	1.4
Tile at entry area (average over full unit)	1.0	Tile at entry area (average over full unit)	1.0
1" horiz. lumber sheathing	2.3	1" horiz. lumber sheathing	2.3
Insulation	0.5	Insulation	0.5
M.E.P.	0.5	M.E.P.	0.5
1/2" Gypsum ceiling	2.5	Plaster ceiling	8.0

Table 5-2 Floor Assembly Detailed Weight Take-off (continued)

Typical Floor		Floor Over Parking	
Material	**Weight (psf)**	**Material**	**Weight (psf)**
Joists (2×8 @ 16")	2.1	Joists (2×8 @ 16")	2.1
Beams	0.0	Steel Beams	4.0
Misc.	0.9	Misc.	0.9
TOTAL:	11.2	TOTAL:	20.7

Table 5-3 Roof Assembly Detailed Weight Take-off

Material	Weight (psf)
Roofing (3-ply felt with one reroof)	4.0
1× lumber sheathing	2.0
Insulation	0.5
M.E.P.	0.5
1/2" gypsum ceiling	2.5
Ceiling joists (2×6 @ 24")	1.0
Roof rafters (2×8 @ 24")	1.3
Beams	0.0
Misc.	0.4
TOTAL:	12.2

Table 5-4 Interior Wall Assembly Detailed Weight Take-off

Material	Weight (psf)
1/2" gyp. wall board (2 sides)	5.0
2×4 @ 16" o.c.	1.0
Insulation	0.0
M.E.P.	0.5
Misc.	0.5
TOTAL:	7.0

Table 5-5 Exterior Wall Assembly Detailed Weight Take-off

Material	Weight (psf)
Stucco (7/8" thick one side)	10.0
2×4 @ 16" o.c.	1.0
Insulation	0.5
1/2" gyp. wall board (1 side)	2.5
Misc.	0.5
TOTAL:	14.5

Table 5-6 Entry Deck Assembly Detailed Weight Take-off

Material	Weight (psf)
Wood Decking	8.0
2×8 @ 16" o.c.	2.1
Railing	1.0
Misc.	0.5
TOTAL:	11.6

Based on these unit weights and assumed configurations of interior and exterior walls, the masses used for seismic design of the SWOF retrofits are calculated. Table 5-7 provides a summary of the weights contributing to the seismic mass at the second floor for loading in the transverse direction.

For purposes of estimating the weight of the exterior wall, it was assumed that 15% of the area of the exterior wall was windows and doors, resulting in a unit weight of 8 psf.

For purposes of determining the weight of interior walls, a detailed weight take-off is performed. A typical unit plan with the lengths of full-height walls is shown in Figure 5-5. The total length of interior walls in the second story is 55 feet interior to the unit times 5 units plus 36 feet between units times 4 walls for a total length of 419 feet. When multiplied by the 8-foot clear height and divided by 2, this results in the 1676 square feet shown in Table 5-7.

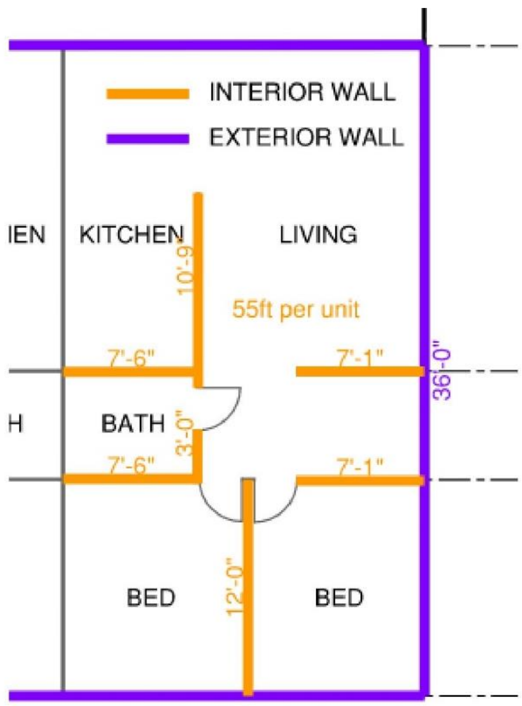


Figure 5-5 Plan of representative unit used to establish length of interior walls for purposes of weight take-off.

Table 5-7 Weight Acting at the Second Floor for Seismic Loading in the Longitudinal Direction

Item	Weight (psf)	Area (sf)	Total Weight (lb.)
Floor	11.2	1800	20200
Second Floor Over Parking	20.7	1800	37300
Entry Deck	11.6	500	5800
Interior Walls Above	7.0	1676	11730
Interior Walls Below	7.0	352	2460
Exterior Walls Above	13.5[1]	1088	14715
Exterior Walls Below	13.5[1]	1272	17200
	Total	109,500	

[1] Exterior wall weight has been reduced to account for 15% window area at 8 psf (0.85*14.5 psf + 0.15*8 psf = 13.5 psf)

The resulting weights acting at each floor can be similarly summed and are summarized in Table 5-8.

Table 5-8 Weights Acting at Each Floor

Item	For Two-Story Building (lb.)	For Three-Story Building (lb.)
Weight at Roof	70,700	70,700
Weight at Third Floor	0	99,200
Weight at Second Floor	109,500	109,500
TOTAL	180,200	279,400

The above tabulated weights are used to determine seismic demands in the design examples that follow. It is noted that although the two design examples that follow use the same building configurations, material assumptions, and associated weights, the total seismic weights for each example vary slightly due to differences in dimension measurement assumptions.

5.2.2 Short-Side-Open Building

The short-side-open building is addressed in the topical discussions that follow the design examples. The building occurs in both two-story and three-story versions and is roughly rectangular in plan. The short-side-open building contains first-story parking located in a portion of the building footprint on a short side of the building, as seen in Figure 5-6. The short-side-open building also has residential units occupying 75% of the first story area. In Figure 5-6, garage doors enclose the parking area. This configuration often occurs with no garage doors. Table 5-9 summarizes some basic information about the short-side-open example building, and further description follows.

Figure 5-6 Example of the short-side-open building with parking areas on a short side of the building. The parking area shown is enclosed by garage doors.

Table 5-9 Characteristics of the Short-Side-Open Example Building

Item	Description
Overall Plan Dimensions	40 ft × 80 ft
Occupied Dimensions at First story	40 ft × 60 ft
Number of Units	7 for Two Story 11 for Three Story
Assumed Era of Original Construction	1950-1960
Floor-to-Floor Height	9 ft
Story Clear Height	8 ft
Exterior Wall Finish	Stucco
Interior Wall and Ceiling Finish	Gypsum Wallboard
Floor Sheathing	Diagonal Lumber Sheathing

5.2.2.1 BUILDING PLANS AND ELEVATIONS

Figure 5-7 and Figure 5-8 provide schematic illustrations of the building plans at each story. Figure 5-9 and Figure 5-10 provide elevations of the three-story building. The two-story building elevations are similar. In the Chapter 2 descriptions of the analytical studies, this variation of a short-side-open building is described as having wing walls. In Figure 5-7, the wing walls occur on Line A and Line E between Line 1 and Line 3.

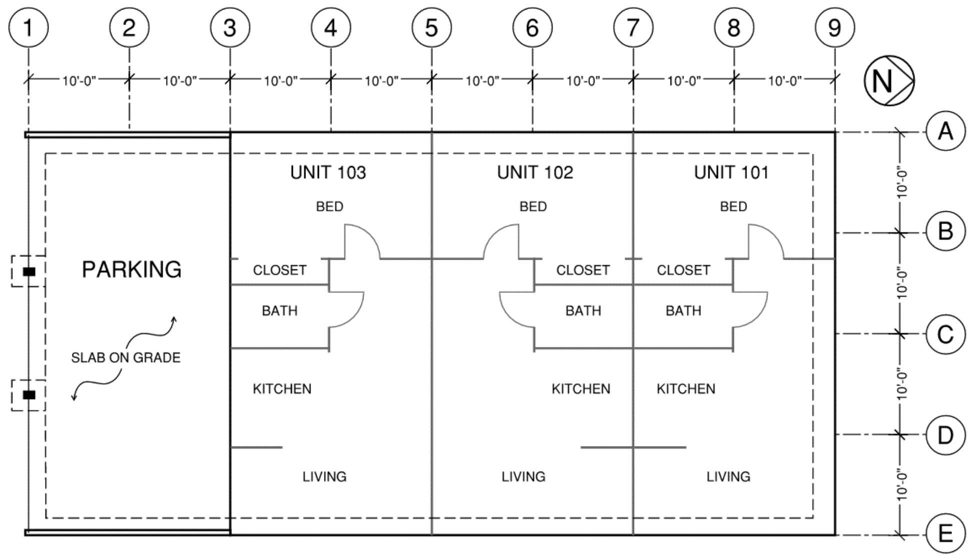


Figure 5-7 First-story plan of the design example short-side-open building.

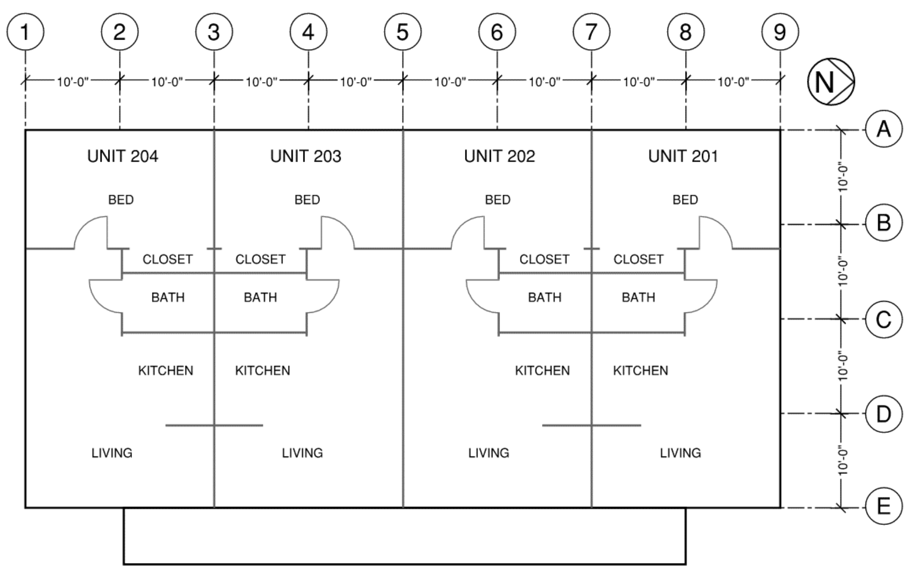


Figure 5-8 Second-story plan of the design example short-side-open building, where the third story is similar.

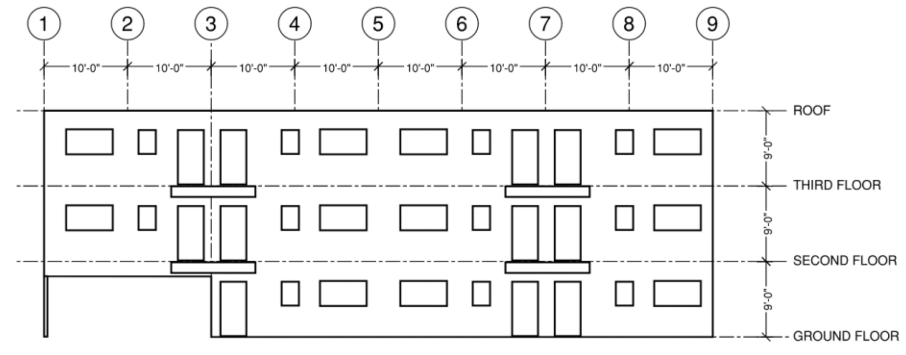


Figure 5-9 **Long-side elevation of the design example three-story, short-side-open building, where wing walls are now shown.**

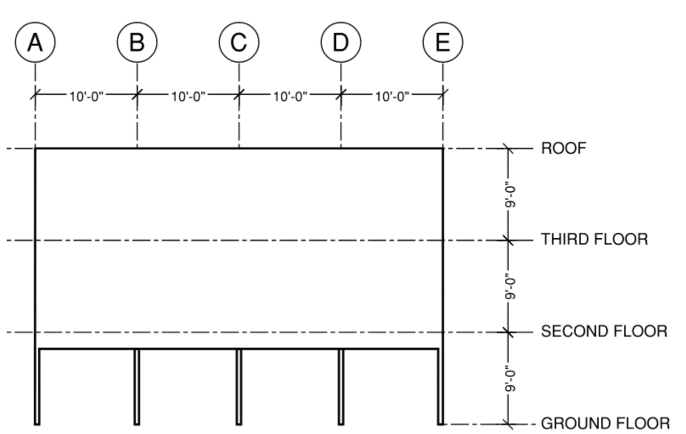


Figure 5-10 **Short-side elevation of the design example three-story, short-side-open building.**

5.2.2.2 MATERIALS OF CONSTRUCTION AND WEIGHTS USED FOR SEISMIC MASS

Similar to the long-side-open example building, the finish materials for the short-side-open example building are stucco on the exterior and gypsum wallboard on the interior.

Table 5-2 through Table 5-6 provide the detailed weight take-offs that are used to establish the seismic mass for purposes of seismic retrofit design.

Based on these unit loads and assumed configurations of interior and exterior walls, the masses used for seismic design of the retrofit are calculated. Table 5-10 provides a summary of the weights contributing to the seismic mass at the second floor for loading in the transverse direction. For purposes of estimating the weight of the exterior walls, it was assumed that 15% of the area of the exterior walls was windows and doors, for which a unit weight of 8 psf was used. For purposes of determining the weight of interior walls, a detailed take-off of the length and weight of the interior walls was provided.

Table 5-10 Weight Acting at the Second Floor for Seismic Loading in the Longitudinal Direction

Item	Weight (psf)	Area (sf)	Total Weight (lb.)
Floor	11.2	2400	26900
Second Floor Over Parking	16.7(1)	800	13400
Entry Deck	11.6	400	4700
Interior Walls Above	7.0	1040	7280
Interior Walls Below	7.0	880	6160
Exterior Walls Above	13.5(2)	960	12980
Exterior Walls Below	13.5(2)	800	10820
Windows and Doors (15% area)	8.0	288	2400
		Total	82400

(1) The floor over parking weight differs between the long-side-open and short-side-open buildings because the long-side-open garage requires steel beams to span the parking spaces at the open front.

(2) Exterior wall weight has been reduced to account for 15% window area at 8psf (0.85*14.5psf + 0.15*8psf = 13.5psf)

The resulting weights acting at each story can be similarly summed and are summarized in Table 5-11.

Table 5-11 Weights Acting at Each Floor

Item	For Two-Story Building (lb.)	For Three-Story Buildings (lb.)
Weight at Roof	59,400	59,400
Weight at Third Floor	0	81,200
Weight at Second Floor	82,400	82,400
TOTAL	141,800	223,000

The above tabulated weights are used to determine seismic demands in the design examples that follow.

5.2.3 Vertical Elements of the Seismic-Force-Resisting System

Cantilever steel columns were selected to be the new vertical elements of the seismic-force-resisting system, added at the open fronts for the end-to-end seismic retrofit design examples. For FEMA P-807 retrofits, the cantilever columns are commonly used in combination with wood-structural-panel (plywood or OSB) shear walls that occur away from the open fronts. Following the end-to-end examples, discussion is provided addressing design differences when instead using moment frames. Cantilever columns were selected because they are becoming increasingly common, whereas moment frames have been used over a longer period of time and design is already illustrated in other publications.

Where FEMA P-807 retrofits require retrofit elements away from the open fronts, it is most common to provide wood-structural-panel sheathing on existing stud walls, thus minimizing the impact of the retrofit on the building floor plan. Along with the added sheathing, detailing for shear and overturning load paths is required.

5.2.4 Seismic Demand Parameters for Retrofit Design

Seismic design parameters were selected based on spectral accelerations from ASCE/SEI 7-16 and 2018 IBC seismic hazard maps. The site selected was Los Angeles City Hall at 200 North Spring Street, a site reasonably representative of the seismic hazard in Southern California. Using Site Class D, the maximum considered earthquake short-period spectral response acceleration, S_{MS}, was identified to be 1.979g.

For design of the line retrofits, the short-period spectral response acceleration, S_{DS}, is 1.32g. When multiplied by 75%, as permitted by the Los Angeles SWOF ordinance, the design spectral acceleration is 1.0g.

The FEMA P-807 performance criteria were selected to be 20% probability of exceeding the FEMA P-807 specified drift corresponding to onset of strength loss with demands based on 0.50 times S_{MS} (0.5*1.979 = 0.989g). These criteria are consistent with the Los Angeles SWOF ordinance.

Both retrofit methods effectively use a demand of $0.5S_{MS}$ (= 1.0g). This can be compared to the slightly higher spectral acceleration of $2/3S_{MS}$ (= 1.32g) that would be applicable to the site for design of new short-period buildings.

5.2.5 Basis of Design Statement

It is common practice to provide a statement of the design basis at the beginning of structural calculation practices, identifying the scope and design criteria used. A sample basis of retrofit design follows. Whether design using retrofit ordinances or FEMA P-807, it is important to identify that the retrofit design is governed by a combination of the ordinance or FEMA P-807 supplemented by provision of the currently adopted building code and associated standards. Where direction beyond that in the ordinance or FEMA P-807 is needed, it should come from the building code. Where the ordinance or FEMA P-807 includes provisions that deviate from the building code, the deviations are permitted.

BASIS OF DESIGN

Description: *The building used for this example is a three-story residential structure with tuck-under parking and an open front along Line 1 at the long end of the building, creating a potential weak story. A schematic plan of the first story of the building is shown. The first story includes a garage and two dwelling units. The second and third stories each contain five dwelling units, which are accessed by an elevated entrance deck. The exterior of the structure has a stucco finish, and the interior finishes are typically gypsum board. The floor finishes are carpet over lumber sheathing with some areas of tile. Foundations are continuous perimeter footings with isolated spread footings at the locations of columns in the garage area.*

Purpose: *The purpose of this partial, vulnerability-based, seismic retrofit is to promote public welfare and safety by reducing the risk of death or injury as a result of the effects of earthquakes on existing wood-frame, multi-unit residential buildings. The ground motions of past earthquakes have caused the loss of human life, personal injury, and property damage in these types of buildings. The retrofit is in accordance with the minimum standards noted below to strengthen the more vulnerable portions of these structures.*

Scope of Retrofit (Line): *The seismic retrofit scope involves strengthening of the one identified open front. This includes addition of new vertical elements of the seismic force-resisting system along the open front, as well as new collectors and foundations for the vertical element load path. A complete load path is provided for the new retrofit elements from the second-floor diaphragm to the supporting soils.*

Scope of Retrofit (FEMA P-807): *The seismic retrofit scope involves strengthening of the entire first story in both orthogonal directions. This includes the addition of new vertical elements of the seismic-force-resisting system, as well as new collectors and foundations for the vertical element load path. A complete load path is provided for the new retrofit elements from the second-floor diaphragm to the supporting soils. Load path connections for existing first-story walls that contribute to seismic resistance will be verified during construction of the retrofit.*

GOVERNING CODES AND STANDARDS

Primary Governing Standard (Line): *The Los Angeles ordinance provisions and supplementary guidance issued by the building department in administrative bulletins or similar documents provide the primary basis of this seismic retrofit design.*

Primary Governing Standard (FEMA P-807): *The model code language in Appendix B.3 of FEMA P-807 provides the primary basis of this seismic retrofit design.*

Supplementary Standards (Line and FEMA P-807):

- *2018 International Building Code* (ICC, 2018a)
- *2018 International Existing Building Code* (ICC, 2018b)
- *ACI 318-14, Building Code Requirements for Structural Concrete* (ACI, 2014)
- *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*
- *AISC 360-16, Specification for Structural Steel Buildings* (AISC, 2016b)
- *AISC Steel Construction Manual, 15th Edition* (AISC, 2017)
- *AISC 341-16, Seismic Provisions for Structural Steel Buildings*
- *AISC Seismic Design Manual, 3rd Edition* (AISC, 2018)
- *AWC NDS-2018, National Design Specification for Wood Construction* (AWC, 2018)
- *AWC SDPWS-2015, Special Design Provisions for Wind & Seismic* (AWC, 2015)

5.3 Line Retrofit Design

5.3.1 Information Summary for Retrofit Design

This example addresses design of a line retrofit to a three-story long-side-open building. The open front is retrofitted using cantilever column retrofit elements, located in the vicinity of the open front but falling outside the building footprint. The retrofit design uses linear-static design methods consistent with the primary methods outlined in Los Angeles retrofit ordinance and associated city guidelines. Although the example uses the line approach and generally follows the Los Angeles ordinance and associated city guidelines, the example does not enforce drift limits. This is described in Chapter 2 as an optimized line retrofit and is recommended in Chapter 3 where line retrofits are being pursued. As noted in Section 5.2.3, the vertical elements are special cantilever steel columns. Consistent with ASCE/SEI 7-16, the columns are designed using a response modification coefficient, R, of 2.5 and an overstrength factor, Ω_O, of 1.25. As noted in Section 5.2.4, the seismic demand for design of the retrofit is calculated using an S_{DS} = 1.32g.

Example Calculations

A complete set of calculations for the optimized line retrofit design is documented in Calculation Package 1, which can be found at https://femap8071.atcouncil.org/. Calculations illustrated in Section 5.3 are excerpted from this calculation set.

Provided in Table 5-12 are the values used for the detailed weight take-offs, with the calculated weight at each story provided in Table 5-13. The building base shear is then calculated using ASCE/SEI 7-16 (note that the Los Angles ordinance 0.75 factor will be applied in Section 5.3.3, after the calculation of story forces). These values are aligned with those from Table 5-2 through Table 5-6. As in Section 5.2, an entry deck measuring 5-feet wide and spanning the length of the building along Line 4, as shown in the calculation below, is included for the second and third floors. The assigned value for “Floor 2 (Stucco)” is only used for the tributary area over the parking garage. The weight labeled “Floor 3 (No Stucco)” is used for the remainder of the second floor.

Table 5-12 Detailed Weight Take-offs

Item	Weight (psf)
Roof	12.2
Floor 3 (No Stucco)	11.2
Floor 2 (Stucco)	20.7
Entry Deck	11.6
Interior Walls	7.0
Exterior Walls	13.5

Table 5-13 Story Plan Dimensions and Seismic Weight

Story	*L*: Length (ft)	*W*: Width (ft)	Footprint: Story Area (s.f.)	w_x: Seismic Weight assigned to Level x (kips)	h_x: Height from base to Level x (ft)
Roof	100.0	36.0	3600.0	71.2	27.0
2	100.0	36.0	3600.0	100.8	18.0
1	100.0	36.0	3600.0	117.9	9.0
			Total Weight:	289.8	

Building Seismic Weight:

W = 289.8 kips

Controlling Seismic Response Coefficient:

C_s = 0.527 ASCE/SEI 7-16 Section 12.8.1.1

Base Shear:

$V = W \times C_s = 153$ kips   ASCE/SEI 7-16 Eq. 12.8-1

5.3.2 Selecting Location and Number of Vertical Retrofit Elements

Figure 5-11 shows a plan of the first-story floor with exterior walls, interior walls, and proposed location of retrofit elements. The number and placement of vertical retrofit elements have been selected in accordance with the recommendations of Chapter 4. The placement of columns should consider vehicle access and avoid impedances to parking and passengers exiting vehicles after being parked. When using steel special cantilever columns, it is recommended to provide pairs of columns with a substantial foundation connecting the pair and to engage existing foundations to mobilize uplift resistance. Engaging existing footings under posts provides dead load to assist with the restoring moment for stability under seismic overturning demands and helps avoid issues related to differential movement.

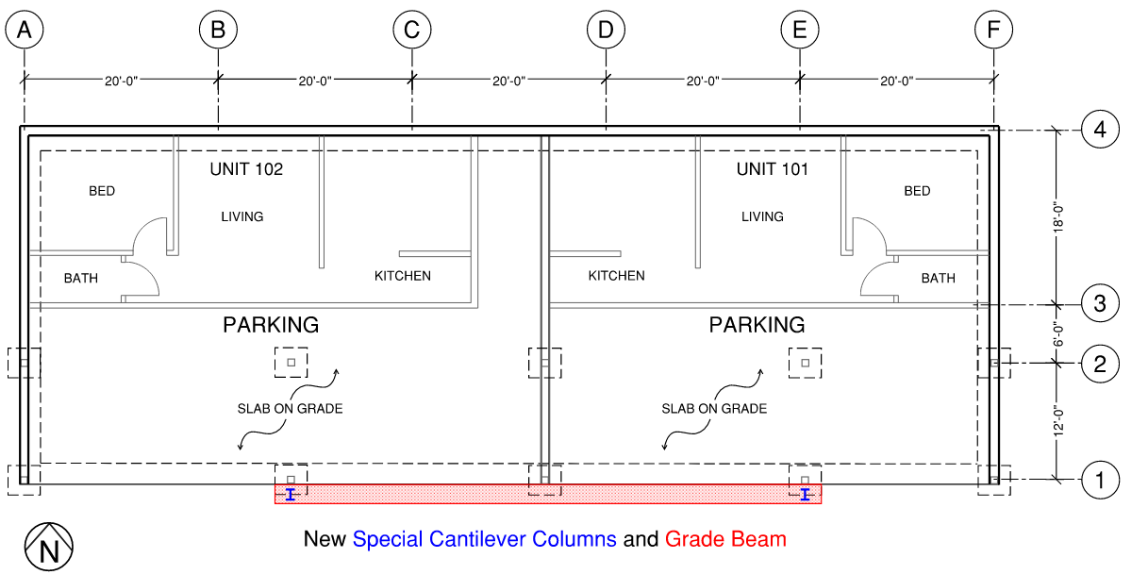


Figure 5-11 First-story plan of the long-side-open building with proposed retrofit elements.

5.3.3 Seismic Forces Tributary to the Retrofit Wall Line

The seismic force tributary to the retrofit wall line is established by evaluating the load path above and assumes a flexible diaphragm idealization. Figure 5-12 show a plan view of the applicable tributary area for the long-side-open building, where the width is equivalent to half the depth of the parking area between Line 1 and Line 3 (or 9 feet), and the length is equal to the full dimension of the open front (or 100 feet).

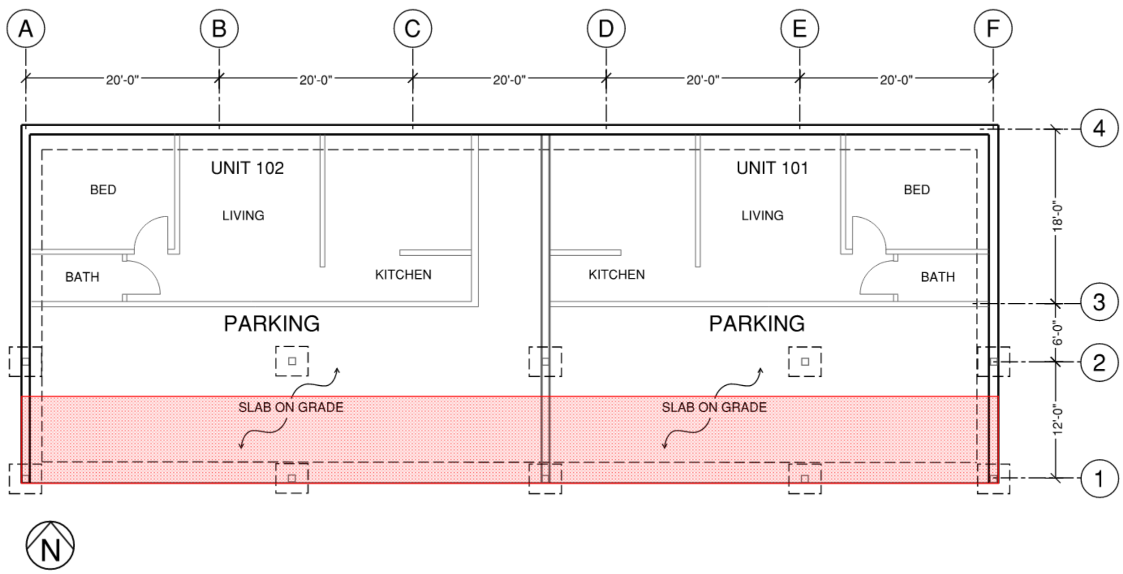


Figure 5-12 Tributary shear area.

For flexible diaphragm buildings, such as in this example, it is common practice to identify lines of seismic resistance. A tributary force is then assigned to each line when analyzing or designing the wall or retrofit. The tributary load assigned to the lines is independent of whether the line has the capacity to resist the seismic loading. For this example, Line 3 is identified as a line of resistance. Therefore, the tributary area assigned to Line 1 is half the total distance between Line 1 and Line 3.

Judgement must be used to define what is a line of resistance. In new buildings, this decision is clear because seismic-force-resisting systems are defined by ASCE/SEI 7. However, in existing buildings that use nonconforming materials and systems, the engineer must use judgement to define what constitutes a seismic-force-resisting system. For line retrofits, ordinances typically allow a designer to consider a stucco wall as a line of resistance, whereas gypsum walls are typically not allowed to be classified as such. Most buildings having the configuration shown in this example will have stucco walls along Line 3.

Initial calculations are performed using detailed weight take-offs and story plan dimensions to find the base shear of the structure followed by the story forces.

Table Definitions:

$k = 1.0$	Vertical force distribution exponent; ASCE/SEI 7-16 Section 12.8.3
$C_{vx} = (w_x \times h_x^k)/\Sigma(w_i \times h_i^k)$	Vertical distribution factor: ASCE/SEI 7-16

Eq. 12.8-12

$V_x = \Sigma F_i$ — Seismic design story shear at level x; ASCE/SEI 7-16 Eq. 12.8-13

$F_x = C_{vx} \times V$ — Seismic story force at level x; ASCE/SEI 7-16 Eq. 12.8-11

Table 5-14 Story Forces

Story	$W_x \times h_x^k$	C_{vx}	F_x (kips)	V_x (kips)
Roof	1923	0.4	61.2	61.2
2	1814	0.4	57.7	119
1	1061	0.2	33.7	153
			Total Shear:	153

$0.75 \times V = 114.7$ kips — Los Angeles Ordinance 183893 Section 91.9309.2

The shear force tributary to the open front is 25% of this total, or 28.6 kips.

5.3.4 Modeling of the Vertical Elements

Where steel cantilever columns are used for a retrofit design, they are modeled in pairs with fully fixed moment connections at the base and pinned connections at the top. At a minimum, a pair of columns is used to ensure redundancy of the retrofit system and assist with overturning, but additional columns may be required based on demands. Also, by designing the retrofit with pairs of cantilever columns, smaller-sized members can be used versus using a single column.

A concrete grade beam is used to provide fixity parallel to the open front, with an assumed cracked stiffness modifier of 0.3 from ASCE/SEI 41 Table 10-5. Ultimately, this creates an inverted moment frame system. Based on the geometry and relative seismic demands of this long-side-open building, two special cantilever columns were sufficient to meet the retrofit design criteria.

5.3.5 Design of the Vertical Elements

There are several important steps in the design of the vertical elements. This section steps through those that are considered most critical. Designing the components for strength, checking detailing requirements in accordance with AISC, and confirming lateral bracing requirements are all included in this process. It was found from the analytical work documented in Chapter 2 that an optimized line retrofit (i.e., no drift limits; strength only) provides similar benefits as a line retrofit, and a resulting recommendation in Chapter 3 is that, where line retrofits are permitted, they be designed for strength only. Thus, drift limits are not considered for this example. The following seismic design parameters are used: $R = 2.5$ (special cantilever column); $\Omega_0 = 1.25$; importance factor, $I_e = 1.0$; and redundancy factor, $\rho = 1.0$.

Although the term "inverted moment frame" is used in Section 5.3.4, the Los Angeles ordinance guidelines define that a special cantilever column system must have an R = 2.5. An official "moment frame," with an R = 3.5, must meet AISC definitions according to the ordinance guidelines.

The material properties, geometry, and demands are used to choose a preliminary size for the pair of steel special cantilever columns, as shown in Table 5-15 and the accompanying calculations. The values presented for V_e and V_{ue} are for the open front only, not the entire story. It then follows that each new column takes half of this load. Also, having a redundancy factor of 1.0 leads to the factored ($\rho \times V_e$) and unfactored (V_e) story shears being the same.

Table 5-15 Material Properties

Property	Value	Definition
F_y	50 ksi	Steel Yield Strength
E	29000 ksi	Elastic Modulus
h	8 ft	Story Height
H	27 ft	Building Height
L	50 ft	Distance Between Columns
C_d	2.5	Deflection Amplification Factor (ASCE/SEI 7-16 Table 12.2-1)

Seismic Demands:

$\rho = 1.0$ — Redundancy factor

$V_e = 28.6$ kips — Unfactored story shear tributary to the new vertical elements

$V_{ue} = \rho \times V_e = 28.6$ kips — Factored story shear; ASCE/SEI 7-16 Chapter 2

Select a W8×40 column.

(It is noted that the retrofit for this same archetype in the analytical studies of Chapter 2 used four W10×22 columns. That retrofit was intended to reflect the average strength and stiffness of a typical optimized line retrofit. The retrofit used here was selected to adhere more closely to actual conditions in the field. In practice, it is common for engineers to seek to minimize the number of retrofit columns and their depth. The retrofit selected in this chapter uses only two columns, but they are heavier than those of Chapter 2.)

The columns then need to be checked to ensure they meet the compactness requirements of AISC 341. The appropriate equations assume the new retrofit columns do not support any gravity load from the structure, meaning a value of zero for C_a, as shown in the calculations below. Even if the preliminary selection of a column is acceptable for strength requirements, it must be upsized if it does not conform to the compactness limits for highly ductile members.

$R_y = 1.1$	Expected yield strength factor; AISC 341-16 Table A3.1
$F_y = 50$ ksi	Yield Strength
$E = 29000$ ksi	Modulus of Elasticity
$b/t_f = 7.21$	AISC Steel Manual Table 1-1
$\lambda_{hd\_flange} = 0.32(E/(R_y \times F_y))^{1/2} = 7.35$	AISC 341-16 Table D1.1
	Compactness < Required; OK
$h/t_w = 17.6$	AISC Steel Manual Table 1-1
$\lambda_{hd\_web} = 2.57(E/(R_y \times F_y))^{1/2} = 59.01$	AISC 341-16 Table D1.1
	Compactness < Required; OK

As shown in Table 5-16 and the accompanying calculations, column capacity values for strength are evaluated. The results of the demand-to-capacity ratio for this example are shown in Table 5-17.

Table 5-16 Resistance Factors

Factor	Value	Definition
ϕ_c	0.9	Compression Resistance Factor (AISC 360-16 Chapter E)
ϕ_b	0.9	Moment Resistance Factor (AISC 360-16 Chapter F)
ϕ_t	0.9	Tension Resistance Factor (AISC 360-16 Chapter D)
ϕ_v	1.0	Shear Resistance Factor (AISC 360-16 Chapter G)

$V_{ue} = 28.6$ kips	Tributary force on retrofit line
$V_u = V_{ue} / 2 = 14.3$ kips	Shear demand per column

h_{wall} = 8 ft — Column height

$M_u = V_u \times h_{wall}$ = 115 kip-ft — Moment demand per column

Column Capacities:

L_b = 8 ft — Unbraced length

K = 1.0 — Effective length factor

KL/r = 47.1

$$4.71\sqrt{\frac{E}{F_y}} = 113$$

$F_e = \pi^2 * E / (KL/r)^2$ = 129 ksi — Elastic critical buckling stress; AISC 360-16 Eq. E3-4

$F_{cr} = (0.658^{F_y / F_e}) \times F_y$ = 42.5 ksi — Flexural buckling stress; AISC 360-16 Eq. E3-2)

$\phi_b M_n = 0.9 \times F_y \times Z_x$ = 149 kip-ft — Available flexural strength

C_v = 1.0 — AISC 360-16 Chapter G

$\phi_v V_n = 1.0 \times 0.6 \times F_y \times A_w \times C_v$ = 89.1 kips — Available shear strength

Table 5-17 Column DCRs

DCR	Value
Flexure (M_u / $\phi_b M_n$)	0.77
Shear (V_u / $\phi_v V_n$)	0.16

Following that step is the check for stability bracing. Member requirements for special cantilever column systems are specified in AISC 341. In this case, these elements are classified as moderately ductile members, with a required bracing spacing that equals a value greater than twice their height. Therefore, designing for stability bracing is not required.

5.3.6 Collectors and Shear Transfer into the Vertical Elements

Design and detailing to ensure that seismic forces can be transferred into the top of new vertical elements is critical, especially when the retrofit columns lie outside the building. It should be shown that the current collector, whether it be wood or steel, is able to meet the requirements to adequately transfer the shear to the new retrofit elements. This includes the seismic forces from the seismic event at overstrength design levels. If the existing member does not have sufficient strength, a new collector, or drag, installed along the full length of the open front, or strengthening of the existing member, is recommended. The collector calculation for the long-side-open example building is shown in Figure 5-13 and its accompanying calculations, with the collector loads calculated as the capacity of the special cantilever columns. The intent is to check for the smallest adequate member for the applied forces and compare the existing collector against this size. The demand-to-capacity ratios for the collector are presented in Table 5-18.

Figure 5-13 Preliminary collector design.

$V_{ue'} = V_{ue} / L = 286$ plf

$P_{uL} = V_{ue'} \times 25$ ft $= 7.15$ kips

$V_{exp\_x} = 20.5$ kips

$V_u = 14.3$ kips

$\Omega = V_{exp\_x} \times R_y / V_u = 1.58$

$P_{uL} = \Omega \times P_{uL} = 11.3$ kips

Try a W8×31 beam

$L_b = 30$ ft

$K = 1.0$

$KL/r = 178$

$$4.71\sqrt{\frac{E}{F_y}} = 113$$

$F_e = \pi^2 \times E / (KL/r)^2 = 9.01$ ksi — Elastic critical buckling stress; AISC 360-16 Eq. E3-4

$F_{cr} = 0.877 \times F_e = 7.9$ ksi — Flexural buckling stress; AISC 360-16 Eq. E3-3

$\phi_c P_n = 0.9 \times F_{cr} \times A_g = 64.9$ kips — Available compression strength

$\phi_t P_n = 0.9 \times F_y \times A_g = 411$ kips — Available tension strength

Table 5-18 Collector DCRs

DCR	Value
Compression (P_{uL} / $\Phi_c P_n$)	0.17
Tension (P_{uL} / $\Phi_t P_n$)	0.03

Following this step is a check to ensure the shear is transferred from the existing diaphragm to the collector through an adequate load path. For this example, it is assumed that the existing load path from the second-story exterior walls to the diaphragm is adequate, however it is recommended to verify the existing connections in field to confirm adequacy against the calculated retrofit demands. A calculation to determine the load entering the diaphragm is shown below:

$V_{ue} = 28.6$ kips — Tributary force on retrofit line

$L_{collector} = 100$ ft — Collector length

$v = V_{ue} / L_{collector} = 286$ plf — Collector load entering diaphragm

The nailer, nailer connection to steel, and collector splices, where necessary, are not included in this example, but it is recommended to confirm field conditions relative to the required transfer forces to the strengthening elements. Strengthening the load path and tying the vertical elements to the collector must be approached with care. As discussed in Chapter 4, the stucco on the exterior walls helps to provide a load path to the existing collector. Cutting into and removing part of this material during the retrofit to provide a connection from the frame's beam to the collector should be avoided. The proposed approach in this example preserves the stucco load path by avoiding damage to the stucco, as detailed in Figure 5-14.

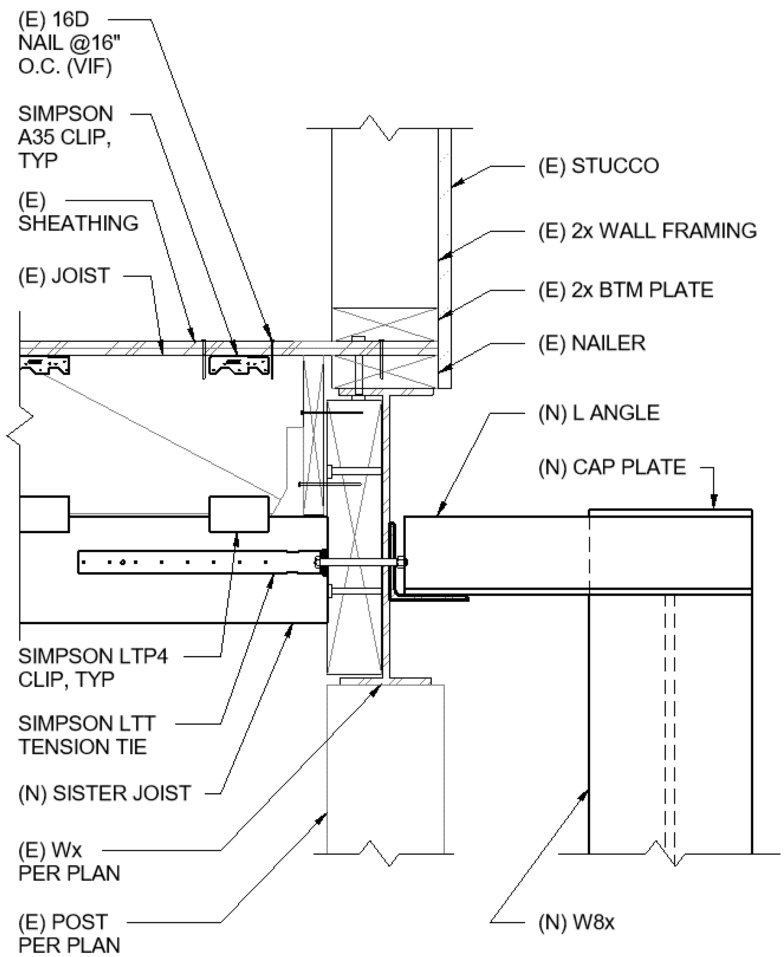


Figure 5-14 Elevation of new column to existing structure connection.

As previously mentioned, the new retrofit columns exist outside the building footprint. This eccentricity causes a torsional force at the top of the column due to in-plane forces. The torsion is resolved into a moment couple that is transferred through new steel angles configured as a truss to spread out the lateral force. The load then moves through Simpson LTTP2 holddowns attached through the collector to new wood sister joist members provided under the existing joists. These new sister joists transfer the out-of-plane shear through new Simpson LTP4 clips to the existing floor joists. Finally, new Simpson A35 clips are used to transfer the load from the existing floor joists to the diaphragm. Diagrams for this load path are shown in Figure 5-15 and Figure 5-16 for clarity.

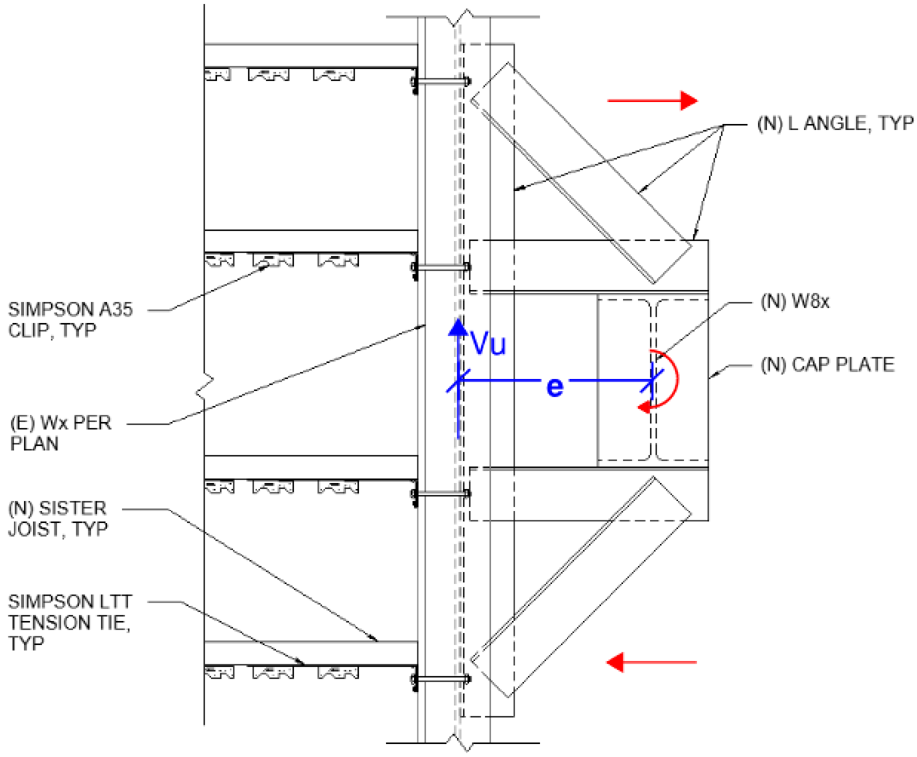


Figure 5-15 Plan view of in-plane shear and resultant moment couple.

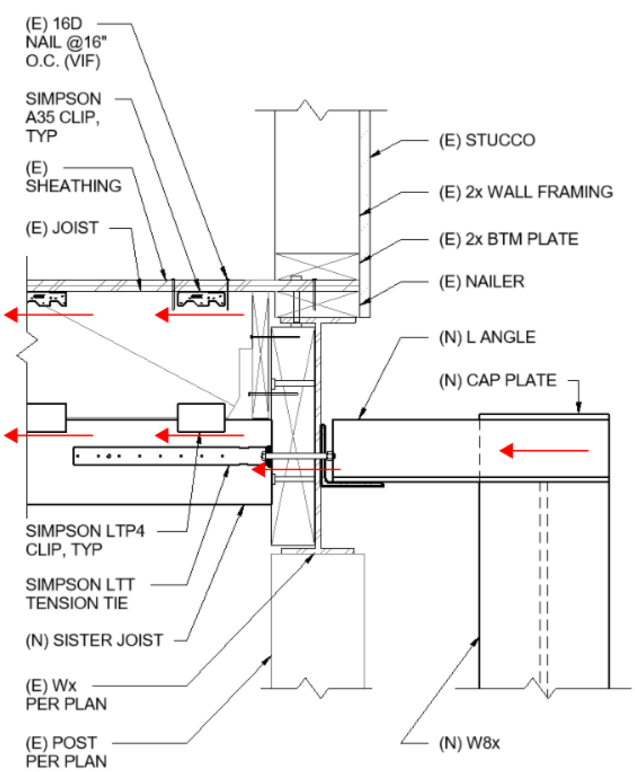


Figure 5-16 Elevation view of in-plane load path.

5.3.7 Deformation Compatibility Considerations

Deformation demands between the new retrofit elements and the existing components should be considered in the design, and connections between deforming elements must be designed to accommodate expected deflections to avoid detachment prior to building collapse. Based on the analysis from Chapter 2, the existing buildings, along with retrofit components, generally display global instability on the order of 3%–5% drift. As such, it is recommended to design for a potential drift of up to 5% or the maximum force that can be delivered based on the capacity of the retrofit vertical element designed to yield (in this case, the cantilever column).

As discussed in Chapter 4, critical considerations for compatibility requirements include orthogonal displacements from the retrofit column direction and interaction effects between new and existing foundation elements. For drift compatibility in the orthogonal direction, a capacity-based design approach is demonstrated whereby the weak axis moment strength of the cantilever column is used to estimate the maximum shear that may be required to be developed into the existing diaphragm perpendicular to the collector element, as shown in the following equations:

$R_y = 1.1$

$F_y = 50$ ksi

$Z_y = 18.5$ in^3

$M_{expWEAK} = R_y \times F_y \times Z_y = 84.8$ kip-ft

$h_{wall} = 8$ ft

$V_{WEAK} = M_{expWEAK} / h_{wall} = 10.6$ kip

For this example, the expected shear force is then transferred through the new steel angle truss. A diagram of this load path is shown in Figure 5-17 for clarity. The length of transfer required is determined by the minimum value between the capacity of the clips over their spacing, or the diaphragm shear capacity. This occurs in combination with the strong-direction load shear and torsion load path demands; however, because capacity-based demands are, designing for each orthogonal case independently is considered sufficient without considering bidirectional effects. Using this approach, it was found that the force required to resolve the eccentricity of in-plane forces governed over the out-of-plane deflection compatibility demands. As an alternative, the designer may conservatively choose to consider a 100%–30% combination of orthogonal forces, as demonstrated in Section 5.4.

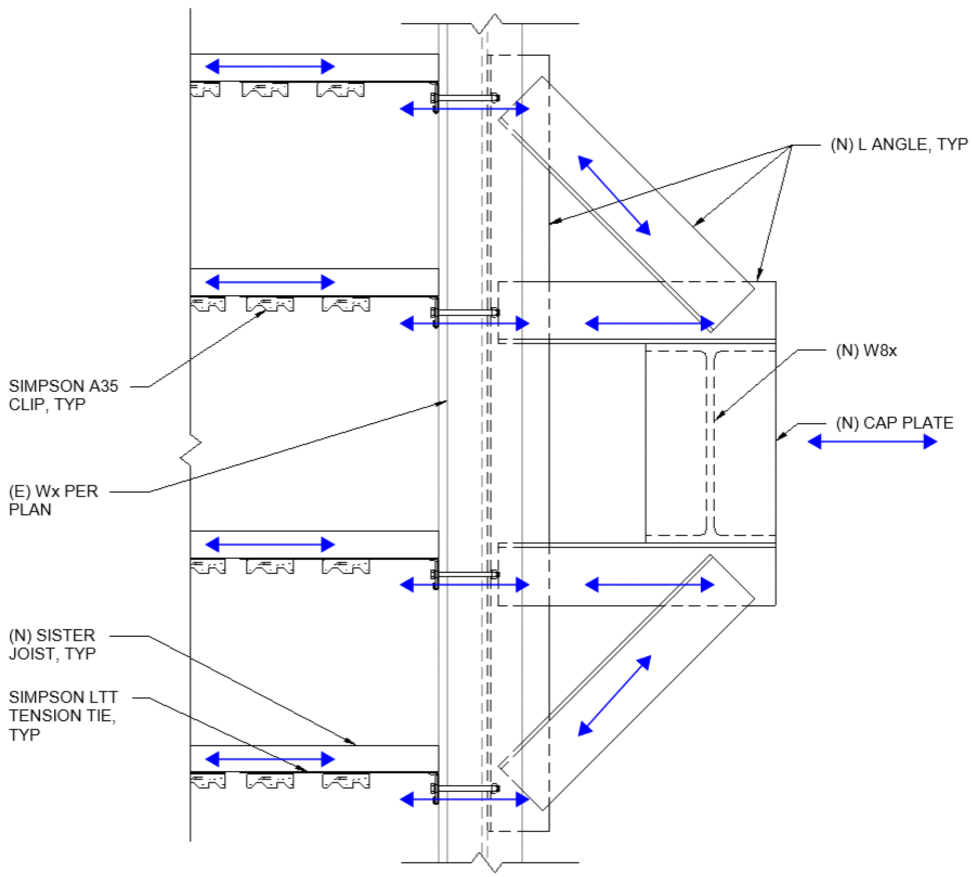


Figure 5-17 Plan view of out-of-plane shear load path.

5.3.8 Foundations and Force Transfer from Vertical Elements

Several concepts should be considered when designing the retrofit foundation and evaluating how to transfer forces out of the vertical elements. These include, but are not limited to, overturning, sliding, and uplift.

To help resist overturning demands, the new grade beam should be tied into the existing foundations along its length. This allows the grade beam to pick up gravity load from the structure, in addition to its self-weight and the weight of the columns. Tying the new footing to the existing foundation also provides resistance to sliding. Figure 5-18 shows a free body diagram of this loading condition, where the value of P_g includes the tributary weight of the roof, the third floor, the second floor, the new retrofit columns, and the concrete footing. In this example where the layout is symmetric, and given that the new grade beam is tied continuously to the existing foundation, the dead load in Figure 5-18 simplifies P_g as one downward resultant force at the center of geometry of the foundation system.

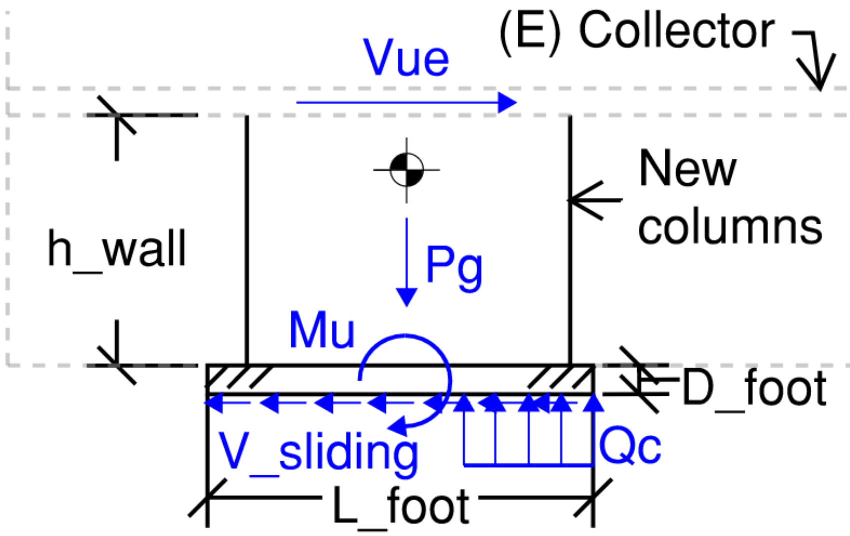


Figure 5-18 Free body diagram of new concrete footing.

Once the shape of the pressure distribution, Q_c, is evaluated, the maximum applied pressure should be checked against the allowable bearing pressure, whether using presumptive building code values or from a geotechnical report for the site.

Reinforcement for the grade beam is then designed such that it can adequately develop the full moment demand (capacity) of the special steel cantilever columns, as shown in Figure 5-19. An alternative method of resolving the moment into the footing is highlighted in Section 5.4.13.2, where capacity is relied upon by compression blocks of only the concrete itself.

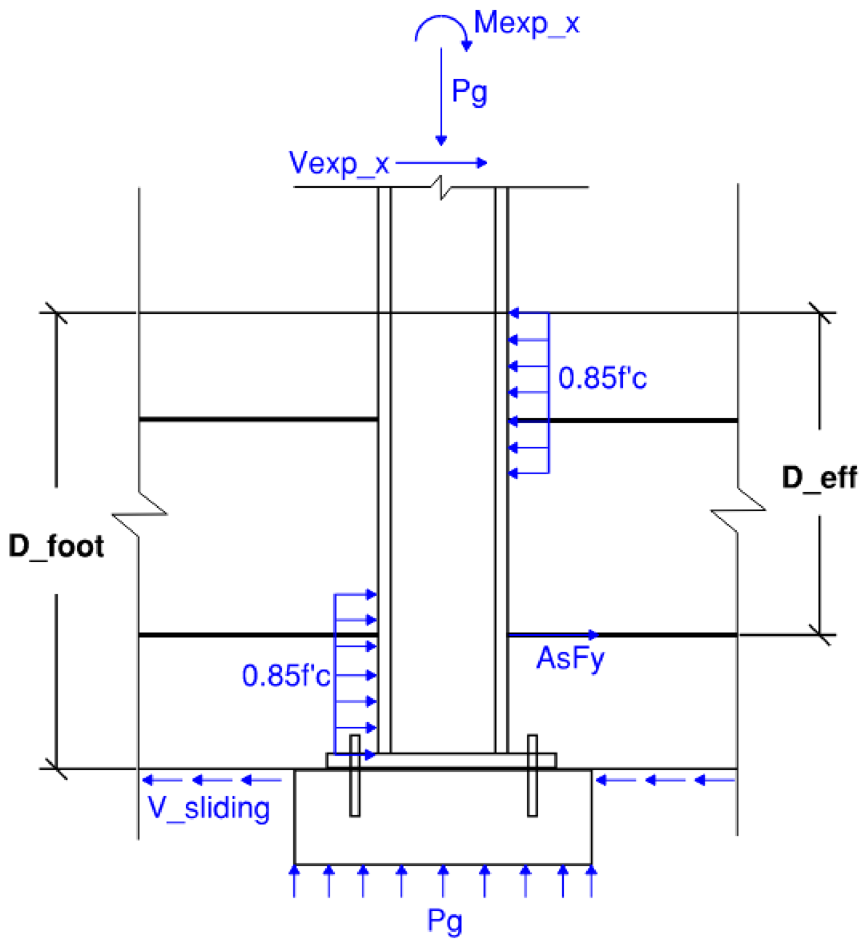


Figure 5-19 Free body diagram for footing reinforcement.

The values of M_{exp_x} and V_{exp_x} are calculated below for reference.

$R_y = 1.1$

$F_y = 50$ ksi

$Z_x = 39.8$ in^3

$M_{exp\_x} = R_y \times F_y \times Z_y = 164$ kip-ft

$h_{wall} = 8$ ft

$V_{exp\_x} = M_{exp\_x} / h_{wall} = 20.5$ kips

Next, the moment capacity of the footing overhang (the longitudinal section of the grade beam that extends past the columns) should also be checked against bending from soil bearing pressure (see

Figure 5-20). Comparison of the shear capacity against demand should follow to ensure the footing is deep enough.

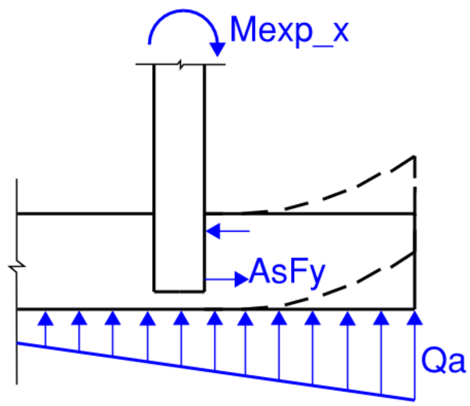


Figure 5-20 Free body diagram for footing overhang.

Additional evaluation of internal force interaction between the new and existing foundations should also be considered, depending on uplift and eccentricities for the specific existing structure layout.

5.3.9 Implications of Design Using Steel Ordinary Moment Frames

An alternative option for the vertical retrofit elements is a traditional steel, pin-based, moment frame. This frame can either be ordinary or special based on IEBC requirements and several California ordinances; however, special frames are required in applying the FEMA P-807 methodology.

5.3.10 Implications of Design Using Vertical Elements Located Inside the Building Footprint

This example demonstrated the use of new vertical elements located outside of the building footprint, thereby creating an eccentricity between the existing load path and new strengthening elements. Designing a retrofit with the new elements inside the building footprint tends to allow for a more direct diaphragm shear transfer and avoid such an eccentricity. This approach is ideal when geometry and parking clearances allow for it. An inboard design for the line retrofit would be similar to that of a FEMA P-807 retrofit, and as such Section 5.4 should be used as a reference for this condition.

5.3.11 Implications for Retrofit of a Short-Side-Open Building

For a short-side-open building, the open front will be shorter and likely only two columns would be required, similar to the demonstrated long-side-open condition shown in this example. It is more likely that both the collector and new foundation elements would need to extend the full length of the open front for strength and stability requirements. Connecting to the orthogonal foundation elements should consider restoring gravity load requirements and dowel strength requirements for the relative shear transfer, as well as internal forces imposed on the existing foundations.

5.4 FEMA P-807 Retrofit Design

5.4.1 Information Summary for Retrofit Design

This example presents a full-story retrofit design using the FEMA P-807 methodology for the three-story long-side-open building introduced in Section 5.2.1. The FEMA P-807 report describes the methodology and the Weak-Story Tool developed to be used with the methodology. The Weak-Story Tool allows the input of a building plan and identification of the wall sheathing and finish materials. The tool is used to determine whether the building meets the performance objective, which is specified in terms of a probability of exceeding drift levels given a seismic hazard (characterized by a spectral response acceleration). In addition to other output, the Weak-Story Tool indicates whether the existing building configuration requires retrofit, and whether a modeled retrofit is adequate (i.e., meets the performance criteria). A user's guide to the Weak-Story Tool is provided in Appendix A of FEMA P-807. Notes on updates to the tool since the original release, as well as installation instructions, can be found at https://www.atcouncil.org/fema-p-807-product-support.

Example Calculations:

A complete set of calculations for the FEMA P-807 retrofit design is documented in Calculation Package 2, which can be found at https://femap8071.atcouncil.org/. Calculations illustrated in Section 5.4 are excerpted from this calculation set.

FEMA P-807 is a full-story methodology that checks the conformance of each story to the specified criteria considering both orthogonal directions and torsion. When using FEMA P-807, however, retrofit is only required in the first story. Any additional retrofit is optional.

The benefit of using the FEMA P-807 methodology is that the seismic resistance provided by existing wall finish materials, including stucco, gypsum wallboard, and plaster, is included in the evaluation. Under other retrofit methodologies (such as IEBC Appendix A4), the resistance of these materials tends to be ignored. The FEMA P-807 method can result in a reduced extent of retrofit relative to other full-story retrofit methodologies that neglect the bracing capacity provided by these materials.

The retrofit design process using FEMA P-807 is different from that used for line retrofits. With FEMA P-807, retrofit design starts with the development of a computer model in the FEMA P-807 Weak-Story Tool. This involves input in a graphical interface of: the existing building plan, wall layout, identification of wall sheathing and finish materials, and information on the story weights.

While there are descriptions of interest throughout the FEMA P-807 document, the following retrofit design example will rely on the "Model Provisions for Mitigation Programs" located in Appendix B.3 of FEMA P-807. This provides the most succinct summary of implementation.

As discussed in Section 5.2.5, use of FEMA P-807 for retrofit design will in some instances also require use of the building code and associated standards. FEMA P-807 Appendix B.3 Section 7.4 sets design criteria for retrofit elements. Item 5 of this section requires that materials and systems for all retrofit elements be consistent with provisions of the building code for new construction. In order to meet this requirement, this example uses the 2018 IBC and its referenced standards. These companion codes and standards will be followed except where otherwise directed by FEMA P-807 or this guideline.

5.4.2 Retrofit Scope

FEMA P-807 Appendix B.3 Section 7.5 indicates: "The retrofit design shall confirm or provide a load path from the second-floor diaphragm through the first-story seismic-force-resisting elements and their foundations to the supporting soils." Items anticipated to be included in the retrofit include:

- New vertical elements in the first story,
- Collectors at the second floor to transmit loads to new vertical elements,
- Foundations where required to support new elements,
- Load path connections for new vertical elements, and
- Load path connections for existing vertical elements to remain.

5.4.3 Performance Objective

As discussed in Section 5.2.4, for purposes of this design example, 20% probability of exceedance of drift limits at a spectral acceleration level of 0.50 times S_{MS} will be used. For the selected site, $0.5S_{MS}$ equals 0.989g.

5.4.4 Eligibility Requirements

It is necessary to verify that the eligibility requirements of FEMA P-807 Appendix B.3 Section 3 are met. There are a range of eligibility requirements, many of them reflecting the building configurations that were considered when developing the Weak-Story Tool. Examples include the number of stories above grade, the height of the stories, the existence of significant torsion, the primary materials of construction, and diaphragm aspect ratios. The long-side-open building used in this example conforms to all of the eligibility requirements.

5.4.5 Building Survey

FEMA P-807 Appendix B.3 Section 4 discusses the detailed building survey that is required when using the FEMA P-807 provisions. The purpose of the survey is to collect adequate information for input into the Weak-Story Tool. It also includes information on the load path connections for existing vertical elements that will be relied upon. Where available, existing drawings should be used to supplement the survey.

5.4.6 Determining Seismic Forces Tributary to the Retrofit Wall Line

For the line retrofit addressed in Section 5.3, it is necessary to determine the seismic base shear tributary to the retrofit elements. This step is not necessary when using the FEMA P-807 Weak-Story Tool. The tool will distribute forces based on vertical element stiffness (using the hysteretic spring properties). This results in the equivalent of a rigid diaphragm analysis based on the stiffness of the vertical elements. For this reason, tributary seismic forces are not applicable for the FEMA P-807 methodology.

5.4.7 Creating the Weak-Story Tool Model

The weak story model is created by:

- Layout of the building footprint of each level,
- Layout of the walls,
- Using pull-down menus to select the appropriate sheathing and finish materials for each wall line,
- Modeling new vertical elements (cantilever columns, shear walls),
- Providing the mass to be distributed over each floor and roof footprint, and
- Defining perforations (door and window openings) and overturning resistance for each wall element.

Table 5-19 summarizes Weak-Story Tool general input information.

Table 5-19 Weak-Story Tool General Inputs

General	Number of stories	3
	Spectral demand, S_d	0.989g

Assemblies are defined using combinations of pre-defined sheathing layers or by defining custom backbone curves. For this example, three as-built assemblies and two retrofit assemblies are defined. Table 5-20 summarizes these assemblies, where “standard layers” refers to the use of default material properties that are built into the Weak-Story Tool.

Table 5-20 Weak-Story Tool Assembly Inputs

	Assembly Type	Layers	Description
Assemblies	As built using standard layers	Stucco + Gypsum wall board	Exterior walls
	As built using standard layers	Gypsum wall board + Gypsum wall board	Interior walls
	As built using standard layers	Stucco + Stucco	Wing walls at garage
	Retrofit with custom backbone (lbs)	Steel cantilever column pair	Retrofit column pair
	Retrofit using standard layers	Wood structural panel (8d@4") + Stucco + Stucco	Retrofit wing walls with wood-structural-panel sheathing

The custom backbone curve for the cantilever columns can be defined using a bilinear curve such as the one shown in Figure 5-21. Inclusion of foundation flexibility affects may be appropriate in some circumstances and will be discussed later in this example.

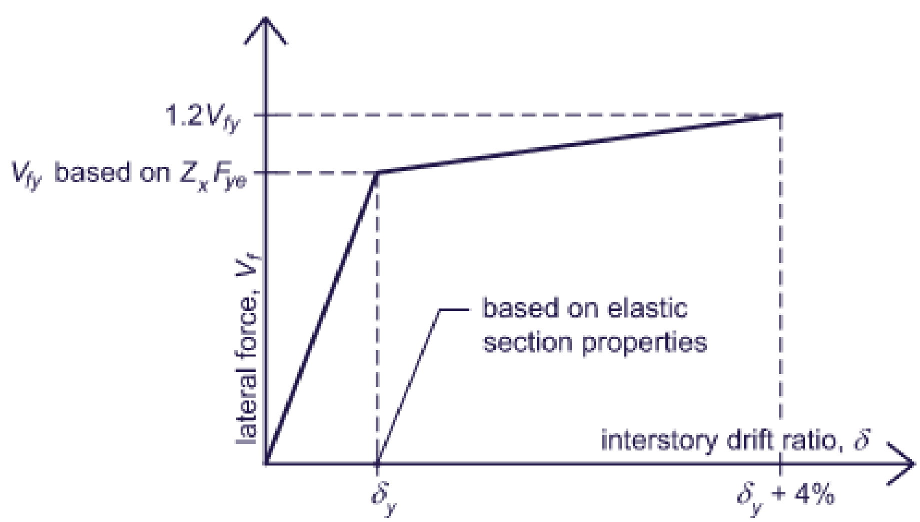


Figure 5-21 Simplified load-drift curve for steel special moment frame or special cantilever column retrofit elements. Z_x and F_{ye} are properties of the yielding member (credit: FEMA P-807).

Mass is assigned to each level by taking the total weight tributary to each level (including exterior and interior walls) and spreading it over the area of the level. Table 5-21 shows the masses assigned to each level for the example building. The total building weight can be confirmed in the Summary tab and compared to the building weight calculated from the building survey and weight take-off to ensure the input weights have been correctly calculated.

Table 5-21 Weak-Story Tool Level Unit Weights

Level	Weight (kips)	Unit Weight (psf)
Roof	70.7	19.5
Third Floor	99.0	27.5
Second Floor	109.5	30.4

Perforations (door and window openings) and overturning resistance are input in the pulldown menu for each individual wall element.

See FEMA P-807-1 Calculation Package 2 for illustrations of the graphical interface and details of the input.

5.4.8 Weak-Story Tool Evaluation of Existing Building

Once the building configuration, wall configurations and materials, and applicable performance criteria are input into the Weak-Story Tool, the tool will report key information about the story strength required, the story strength provided and the need for retrofit. Figure 5-22 illustrates the Weak-Story Tool graphical interface output for the first story. In this case the output identifies that retrofit is required in order to meet the specified performance criteria. The first-story strength that is targeted is 113 kips and the available strength is 94.7 kips.

```
Executive Summary----------------------------------
X-Direction: Existing performance is NOT ADEQUATE;
Y-Direction: Existing performance is NOT ADEQUATE;
```

Detailed output for X-Direction (parallel to open front):

```
X-Direction----------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         99.7 kips
Upper-story Strength Ratio (Au)   0.358
Upper-story Strength Ratio (Cu)   0.589

Ground-Story Strength Limits
Target (Vrl)                      113.0 kips
Estimated Minimum (Vr,min)        230.2 kips
Maximum (Vr,max)                  125.6 kips
0.9 Vr,max                        113.0 kips

Est. of dV req'd (Vrl - V1)       18.3 kips
Added Retrofit Strength, (dV1)    0.0 kips

                                  Current Condition
Ground-story Strength (V1)        94.7 kips
Base Shear Ratio (C1)             0.340
Degradation Ratio (Cd)            0.323
Weak-story Strength Ratio (Aw)    0.950
Spectral Capacity (Sc: P20, OSL) 0.426


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.426) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is (0.9 Vr,max) 113.0 to (1.1 Vr,max) 138.1 kips.
```

Figure 5-22 Weak-Story Tool output for evaluation of the existing building.

Recommendation Note

The following are recommended when selecting the number and location of vertical retrofit elements:

- Locate vertical elements to reduce torsion,
- Locate vertical elements to reduce floor diaphragm spans,
- Distribute vertical elements along the line of resistance so as to minimize collector length and collector forces,
- Include not less than two cantilever steel columns or one moment frame on any one line of resistance, and
- Use a shared grade beam foundation for pairs of cantilever columns.

5.4.9 Selecting Location and Number of Vertical Retrofit Elements

The Weak-Story Tool allows easy exploration of the adequacy of both the unretrofitted configuration and a series of retrofit solutions, encouraging the designer to explore multiple solutions. Section 7.3.2 of FEMA P-807 Appendix B.3 requires placement of retrofit elements along perimeter wall lines, except where otherwise permitted by the building official. In addition, Section 6.3 suggests that where possible these elements be located so as to minimize first story torsion. Section 6.3 also notes that retrofit elements can be located to reduce diaphragm aspect ratios and can be used to make buildings eligible for use of FEMA P-807 where they might not otherwise be.

For this example, the minimum retrofit that might be anticipated is the addition of steel cantilever column elements (as discussed in Section 5.2.3) at the open front, as occurred with the line retrofit in Section 5.3. Figure 5-23 shows a plan of the first story with the steel cantilevered column elements located immediately north of the Line 1 open front and a concrete grade beam between the cantilever columns and connecting to the existing foundation at Lines A, C.7, and F.

In this example, the new cantilever columns are located a few feet inside of the exterior walls at Line 1.3. This allows the columns and grade beams to be constructed without interrupting the existing Line 1 columns and foundations. This placement is judged to meet the intent of the requirement that new elements be located at perimeter walls.

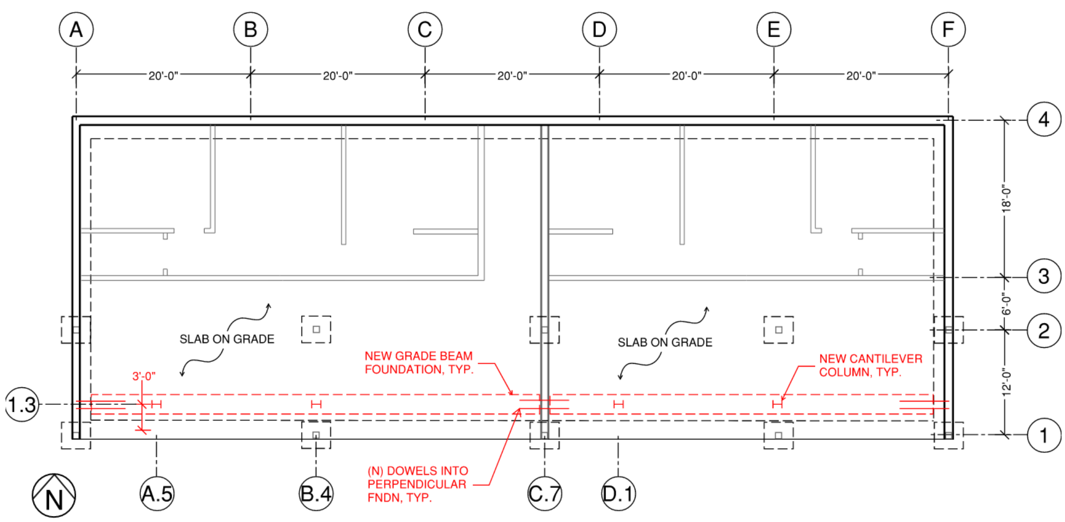


Figure 5-23 First-story plan showing steel cantilever column and grade beam retrofit location.

The number and placement of vertical retrofit elements are selected considering the recommendations of Chapter 4. The minimum number of cantilever columns provided in a line of resistance is two, oriented as a pair on a shared grade beam foundation. For the retrofit example building plan, it is convenient to include four column elements, one column pair for each section of parking (either side of the walkway between Line C and Line D). One of each pair of columns is aligned with an existing column, and the spacing within each pair is consistent with the spacing designated for two parking spots (approx. 20 ft). The extension of the new grade beams to intersect the existing perimeter foundation provides a method to mobilize the existing dead load of the building to resist uplift loads generated by seismic overturning of the cantilever columns and helps to mobilize sliding resistance. This extension of the new grade beams and doweling to the existing foundation is also a recommendation from Chapter 4.

Recommendation Note

The retrofit steel columns for this design example are located in the immediate vicinity of the open front. This provides additional story capacity while at the same time reducing the building torsion. This also reduces demands imposed on the cantilevered diaphragm.

For the selected number and location of cantilever columns shown in Figure 5-23, the total of four columns along the length of Line 1.3 helps to keep the force in each column low, to reduce demands on the collectors and the foundations, and to provide redundancy. Should fewer columns be provided, the forces and the size of the foundation and collector elements would increase.

At this stage in the retrofit design, it has not yet been determined whether new vertical elements will be required, in addition to the cantilever steel columns in order for the retrofit to meet the FEMA P-807 criteria. Based on the Weak-Story Tool report that retrofit is also required in the Y-direction, it is proposed to add wood-structural-panel sheathing to the existing transverse direction stud walls (Figure 5-24). This new sheathing is installed on the interior face of the garage walls (Lines A, C.7, and F). Sheathing on the wall interior face is selected because this face has less weather exposure than at the building exterior and is less critical from an appearance standpoint. These locations also provide the greatest torsional resistance, as Lines A and F are located far from the center of rigidity of the vertical elements. The Weak-Story Tool will be used in steps that follow to determine whether the retrofit plan shown is adequate to meet the retrofit performance objective.

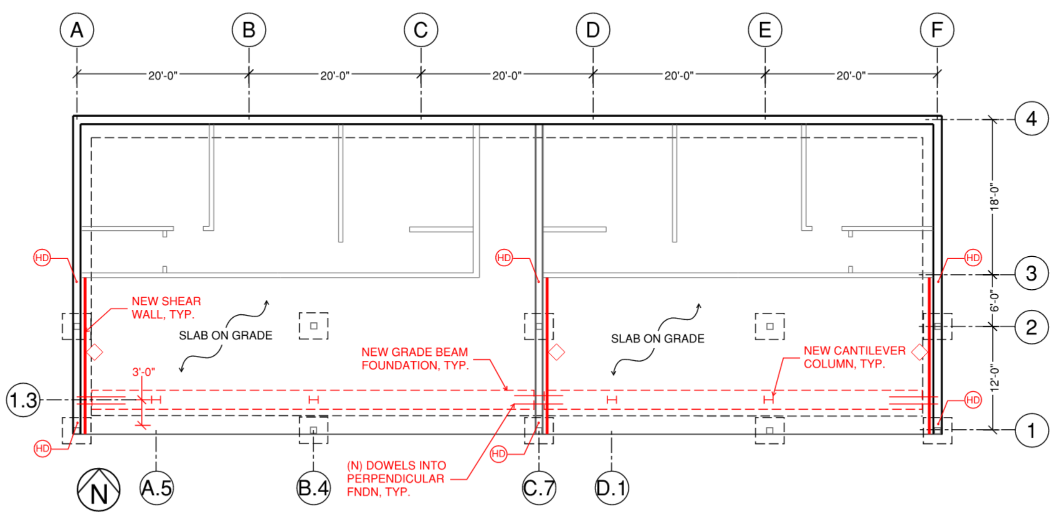


Figure 5-24 **First-story plan showing wood-structural-panel shear wall locations in addition to steel cantilever columns.**

5.4.10 Selecting Cantilever Column Sections

Sections for the steel cantilever columns need to be selected in order that they can be added to the Weak-Story Tool model and adequacy of the retrofit determined. In practice to date the majority of the column sections used have been steel wide flanges, although HSS sections are used occasionally. There are several competing criteria that drive the selection of steel cantilever columns. The final selected column section includes a balance of the following:

- **Minimum Strength.** Section strength must be adequate to provide the required story strength. It often takes several trials of different sections to find the minimum section size that provides the required capacity,
- **Maximum Strength.** Load-path detailing that develops the expected capacity of the vertical elements, increasing the size of load path elements and connections. As a result, the provided strength should not be substantially more than the minimum required.
- **Bracing.** Lateral-torsional bracing requirements of AISC 341 must be met. Because adequate lateral-torsional restraint is extremely hard to provide in wood-frame buildings, sections are often selected so that column lateral-torsional bracing can be eliminated. This requires a larger radius of gyration about the y-axis, which requires use of larger sections,
- **Accommodation of Weak-Axis Deflection.** Building drift in the column weak-axis direction most often results in loading of the steel vertical elements in the weak-axis direction (unless detailing is provided for a slip connection or similar). This weak-axis load needs to be taken into consideration in design of the connection between the vertical element and the diaphragm. As a result, the strength of the column in the weak direction should be kept as low as possible.

When considered together, the cantilever columns might be selected to be slightly larger than required to meet the strength requirement, but not significantly larger. See further discussion in following sections.

Recommendation Note

The following criteria should be balanced in determining the best section for cantilever steel columns:

- Minimum strength,
- Maximum strength,
- Lateral-torsional bracing requirements, and
- Design for weak-axis deflection.

5.4.11 Verification of the Retrofit Design

The Weak-Story Tool is run with additional retrofit elements (steel cantilever columns and wood-structural-panel sheathing) and repeated until the output indicates that the criteria have been met. For this design example, four W8×40 cantilever steel columns are used as the initial retrofit design, with adequacy to be determined by the Weak-Story Tool. For this design example, the Weak-Story Tool output showing acceptable retrofit is seen in Figure 5-24.

```
Executive Summary------------------------------------------------------------------------
X-Direction: Existing performance is NOT ADEQUATE; Retrofitted performance is ADEQUATE.
Y-Direction: Existing performance is NOT ADEQUATE; Retrofitted performance is ADEQUATE.
```

Detailed output for X-Direction:

```
X-Direction--------------------------------------------------------------
Controlling Upper Story: Level 02
Upper-story Strength (Vu)         99.7 kips
Upper-story Strength Ratio (Au)   0.358
Upper-story Strength Ratio (Cu)   0.589

Ground-Story Strength Limits
Target (Vr1)                      94.6 kips
Estimated Minimum (Vr,min)        94.6 kips
Maximum (Vr,max)                  125.6 kips
0.9 Vr,max                        113.0 kips

Est. of dV req'd (Vr1 - V1)       -0.1 kips
Added Retrofit Strength, (dV1)    39.2 kips

                                  Before Retrofit   After Retrofit
Ground-story Strength (V1)        94.7 kips         133.9 kips
Base Shear Ratio (C1)             0.340             0.480
Degradation Ratio (Cd)            0.323             0.950
Weak-story Strength Ratio (Aw)    0.950             1.343
Spectral Capacity (Sc: P20, OSL)  0.426             1.317


Retrofit REQUIRED:
Existing spectral capacity, Sc: P20, OSL ( = 0.426) < Sd ( = 0.985)

Acceptable range of retrofitted ground floor strength (existing plus
new) is approximately (Vr,min) 94.6 to (1.1 Vr,max) 138.1 kips.

Current retrofitted performance is ADEQUATE.
```

Figure 5-25 Weak-Story Tool output showing that the proposed retrofit is adequate.

In this trial retrofit, the minimum target strength has dropped from 113 kips in Figure 5-22 to 94.6 kips in Figure 5-25. This reduction in demand reflects that the added retrofit elements have reduced the torsion in the first story. (This target is the required minimum story strength from the Weak-Story Tool and is not related to ASCE/SEI 7 base shear calculations.)

5.4.12 Retrofit General Design Requirements

Now that a retrofit has been identified that meets the Weak-Story Tool Criteria, the next step is design and detailing of the vertical elements. As discussed in Section 5.1, FEMA P-807 will be used in combination with the 2018 IBC and associated standards.

Section 7.4 of FEMA P-807 Appendix B.3 provides more specific requirements for both the vertical elements and load-path detail as follows:

"Materials and systems for all retrofit elements shall be consistent with provisions of the building code for new construction. Detailing of retrofit wall and frame elements shall be consistent with that applied to special seismic force-resisting systems used in new construction for the corresponding occupancy and risk category."

"Design criteria for load-path components and connections shall be appropriate to the performance objective and shall be based on the building code for new construction, ASCE/SEI 41 or principles of capacity design."

FEMA P-807 Appendix B Section 7.5 goes on to require that load-path designs using building code provisions are to use overstrength loads.

For purposes of this design example and as recommended, capacity design methods are used for design of the load path for new vertical elements. This is most consistent with the expectation that the vertical elements will experience inelastic behavior and is anticipated to be more efficient than design using load combinations with overstrength.

Recommendation Note

Capacity design methods are recommended for design of the load path for new vertical elements.

For purposes of this design example, capacity design methods are used with resistance factors (phi factors) taken as 1.0, consistent with the concept of expected strength. This is done in part because there are no widely accepted procedures for assigning resistance factors in capacity-based design. In reviewing the demand-to-capacity factors that result, they are judged to provide adequate allowance for material variability. Should demand-to-capacity ratios be near one, the designer may want to look at the element involved on a case-by-case basis and judge acceptability.

5.4.13 Design of the Steel Cantilever Columns

FEMA P-807 Appendix B.3 Section 7.4.2 requires that the steel cantilever columns conform to the Special Cantilevered Column requirements of AISC 341 and further requires that the elements have a strength degradation ratio (strength at 3% drift divided by peak strength) of 0.8 or greater. Developed as a measure to help ensure ductility in new vertical elements, this means that the cantilever column needs to yield at approximately 3% drift. Because the drift at yield can be influenced by foundation flexibility, this flexibility is incorporated into the column back-bone curve. See FEMA P-807 Calculation Package 2 for further discussion.

5.4.13.1 STEEL SECTION CRITERIA

Based on the use of the 2018 IBC, AISC 341-16 is applicable for design of the steel cantilever columns once they are proportioned using the FEMA P-807 Weak-Story Tool. Discussion of

AISC 341-10 is also included because FEMA P-807 specifically cites this edition (Appendix B.3 Section 7.4.2).

Based on AISC 341-10 and AISC 341-16 Section E6, the following requirements are applicable for steel special cantilever column systems. It needs to be verified that the selected steel section meets these requirements.

Seismic *b/t* Ratio

The following calculations check the width-to-thickness ratio of the column flanges. The expected capacity is also calculated for future use.

Table D1.1 gives the limiting width/thickness ratios for highly ductile members. These ratios represent the upper bound, meaning that the ratios for the columns should stay below the tabulated values in order to be compliant.

$\frac{b_f}{2t_f} = 7.1$	W8×40 b/t ratio
$\lambda_{hd.flange} = 0.32\sqrt{\frac{E}{R_y \times F_{ye}}} = 7.3$	OK
Assuming $P_u \sim 0$	The retrofit columns for this example building are designed not to take any significant gravity loads
$h / t_w = 16.0$	W8×40 *h/t* ratio
$\lambda_{hd.web} = 2.57\sqrt{\frac{E}{R_y \times F_{ye}}} = 59.0$	OK

The selected W8×40 meets applicable requirements.

Expected Capacity

From AISC construction manual:

$Z_x = 39.8$ in.^{3}	Plastic section modulus
$I = 146.0$ in.	Moment of inertia

Section is compact for compression and flexure (see AISC 360-16 Table B4.1 for more details).

$height = 8.0$ ft	Estimated height, based on 8-ft clear story height
$M_{fy} = R_y \times Z_x \times F_{ye} = 182.4$ ft-kips	Expected yield moment

$V_{fy} = M_{fy} / height = 22.8$ kips Expected yield shear (per column)

The expected share strength of the cantilever columns is larger than the difference between the post- and pre-retrofit strengths of the ground story provided by the Weak-Story Tool. This discrepancy is due to the deformation incompatibility between the ductile cantilever columns, which yield around 2% drift, and the more brittle existing wall finishes, most of which peak at or below 1% drift. The peak ground story strength for the retrofit condition occurs around 1% drift, at which point the columns are at only about half of their yield strength, equating to about 40 kips of shear strength, which matches the difference between the post- and pre-retrofit strength of the ground story.

Stability Bracing

Under AISC 341-10 and AISC 314-16, Section E6.4a references Section D1.2a (stability of moderate ductility beams) for special cantilever column stability requirements. The two versions of AISC 341 have slightly different equations, which require the unbraced length for the W8×40 section to not exceed 16.8 feet or 17.1 feet, respectively. Two currently published documents have further considered lateral-torsional buckling of steel cantilever columns when used in the configuration of these examples (i.e., two or more cantilever columns joined by a common grade beam). The direction they provide is that lateral-torsional buckling be checked using two times the column height. This is interpreted to mean that where the distance between required bracing is not less than twice the column height, lateral-torsional bracing is only required at the base of the column. The published documents are the 2021 IEBC (Appendix A4, Section A403.10.2) and City of San Francisco Administrative Bulletin AB 107. This approach will be used for the design example. For the example column with the maximum permitted distance between lateral bracing calculated to be more than twice the column height, lateral bracing is not required at the top of the column, leaving the top connection only required to transfer shear into the diaphragm.

$r_y = 2.04$ in. Radius of gyration in the weak direction

$$L_b = 0.19 \frac{r_y E}{R_y F_{ye}} = 17.0 \text{ ft}$$ Equation D1-2 (AISC 341-16)

$$L_b = 0.17 \frac{r_y E}{F_{ye}} = 16.8 \text{ ft}$$ Equation D1-2 (AISC 341-10)

New criteria for unbraced length and bracing requirements are being balloted by AISC at time of writing as an update to Section E6; this update is anticipated to be adopted into the 2024 IBC. The currently balloted future AISC formulation will decrease the permitted unbraced length as shown.

$r_y = 2.04$ in. Radius of gyration in the weak direction

$M_2 = M_{fy} = 182.4$ kip-ft — Larger moment at end of unbraced length (positive in all cases)

$M'_1 = 0$ — Effective moment at end of unbraced length opposite from M_2

$$L_b = \left[0.12 - 0.076\frac{M'_1}{M_2}\right]\frac{r_y E}{R_y F_{ye}} = 10.8 \text{ ft}$$

Because this is less than twice the cantilever column height, bracing would be required in accordance with these future provisions. Torsional bracing is not, however, required by current provisions and so is not provided as part of this design examples

At the same time, three exceptions are provided where lateral-torsional bracing is not required. The first exception allows bracing to be omitted for round and square HSS sections. The second allows bracing to be omitted for weak-axis bending. The third exception mirrors the IEBC and San Francisco provisions, allowing bracing to be omitted where the required maximum permitted unbraced length is twice the column height. While it is helpful to have the third exception formalize this criteria, the change in the bracing length will mean that commonly used W-sections may no longer be permitted in the future when the 2024 IBC is the adopted building code. At this time based on the applicable building code being the 2018 IBC or 2021 IBC, conformance with AISC 341-16 as modified by the IEBC and AB 106 is judged to be acceptable. It is suggested that this topic be revisited when adoption of the 2024 IBC is imminent.

Although the future AISC provisions are likely to push engineers to use HSS sections, there are concerns that the behavior of HSS sections at the steel-to-concrete interface will not be ductile due to local buckling behavior. There is no known testing addressing this connection. Where concerns regarding buckling of HSS section walls have arisen in the past (for concentric braced frames), casting concrete or grout inside of the HSS has been one suggested approach to mitigation of local buckling. The designer is cautioned against use of HSS sections, but if they are to be used, casting concrete or grout inside of the HSS at the column-to-foundation interface is recommended.

Recommendation Note

If HSS sections are to be used for cantilever columns, it is recommended to cast concrete or grout inside of the HSS at the column to foundation interface.

5.4.13.2 STEEL MOMENT TRANSFER TO THE FOUNDATION

The most commonly used connection for the steel cantilever column to the foundation involves embedding the steel column into a large continuous grade beam (Figure 5-26). With the pair of steel columns acting in combination with the grade beam, the behavior is somewhat equivalent to a moment frame turned upside down. Figure 5-26 shows a housekeeping pad of concrete below the structural grade beam. This is a small, isolated footing that allows the base of the column to be secured in position while the reinforcing steel and concrete for the grade beam are installed. The pad serves only as an erection aid and does not play any structural role.

Where the column can be located two feet or more from the end of the grade beam, moment transfer from steel to foundation can occur using a couple with steel bearing on concrete near the top and bottom of the grade beam, as shown in the calculation below. The use of compression for moment transfer requires that the longitudinal reinforcing in the foundation be developed beyond a critical section at either face of the column. Where the steel column is pushed to the end of the new grade beam (Figure 5-27), the reinforcing can no longer be developed, and an alternative transfer mechanism will be necessary, such as welding rebar to the embedded steel section face to transfer tension forces.

Recommendation Note

Cantilever columns should be located several feet from the end of the grade beam such that the grade beam longitudinal reinforcing can be developed at a critical section at the face of the column. Where the steel column is pushed to the end of the new grade beam, the reinforcing can no longer be developed, and an alternative transfer mechanism will be necessary, such as welding rebar to the embedded steel face to transfer forces.

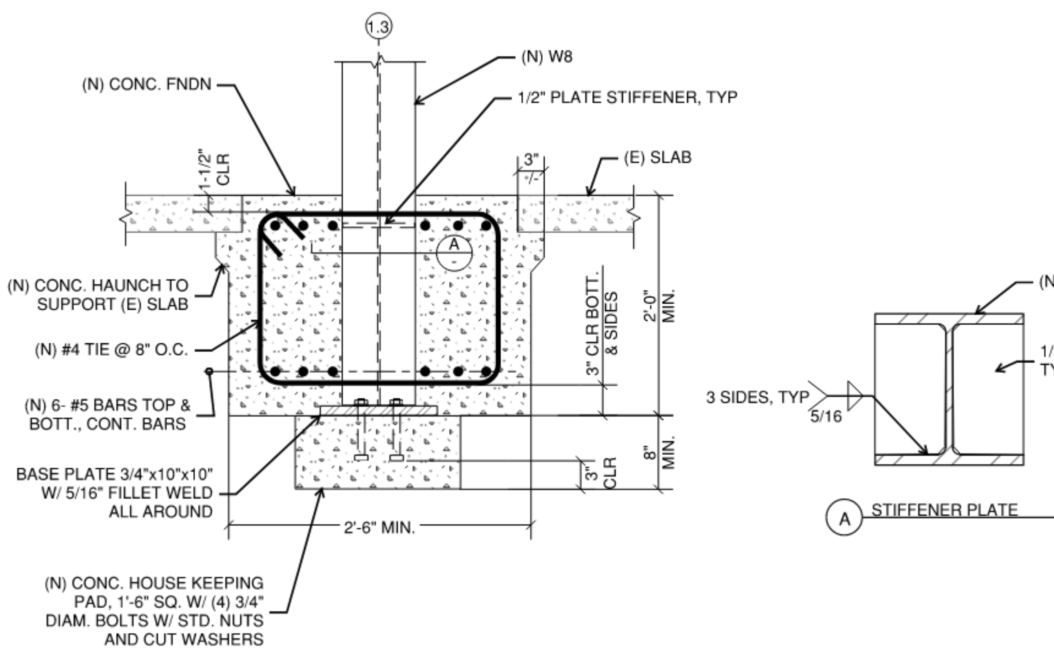


Figure 5-26 Common steel-to-concrete foundation detail.

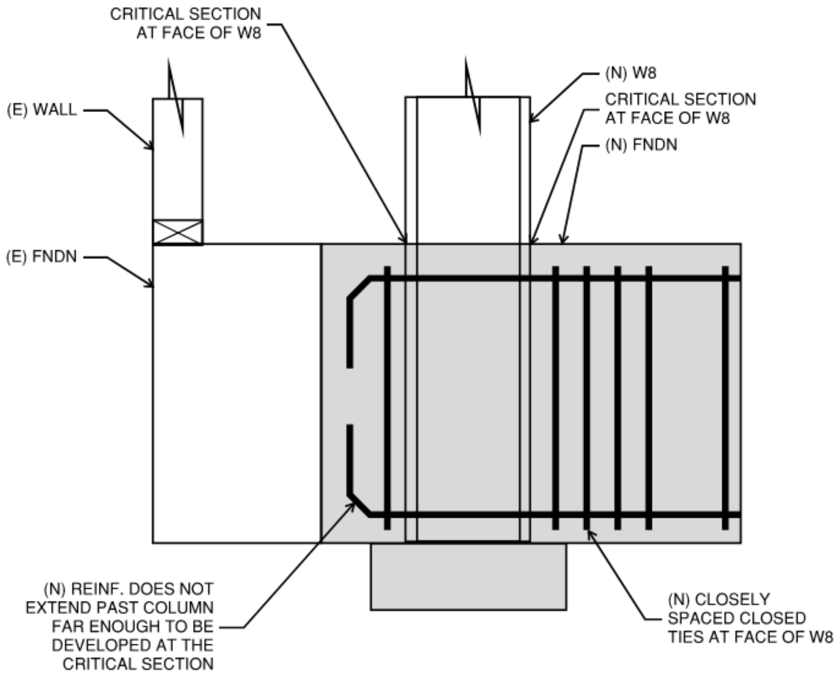


Figure 5-27 Elevation of new grade beam. Note closely spaced ties are added at each face of the W8 columns.

$M_{fy} = 182.4\,\text{kip-ft}$ — Expected moment capacity of cantilever column

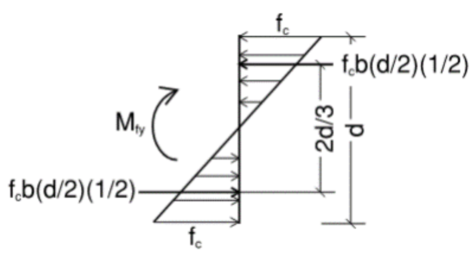


Figure 5-28 Bearing stress distribution in concrete grade beam.

$$f_c = \frac{M_{fy}}{b\dfrac{d}{4}\left(\dfrac{2d}{3}\right)} = 992.7 \text{ psi}$$

Expected maximum concrete stress in grade beam (Figure 5-28)

Check concrete bearing per ACI 318-14 Section 22.8

$b_n = 0.85f'_c = 2550.0$ psi — Nominal bearing stress capacity

$f_c / b_n = 0.4$ — < 1 OK

Because the first steel column in this design example is located approximately 10 feet from the end of the grade beam for this retrofit, as seen in Figure 5-23, moment transfer through compression blocks is adequate for the example. The calculation provided shows that the bearing stress is low compared to that permitted by ACI 318, providing adequate transfer at capacity-level forces.

5.4.13.3 FOUNDATION DESIGN

Using capacity-based design methods in combination with the 2018 IBC, the foundation is checked for soil bearing pressures and designed for shear and flexure. The bearing pressures, shear and flexure are calculated based on the steel columns reaching expected flexural capacity. This foundation design uses half of the overall 100-foot length of the new foundation, stopping at the point where the new foundation intersects and is doweled into the existing foundation. A similar design is to be provided for the other half of the foundation unless its design can be determined to be less critical. Either shorter or longer portions of the foundation can be used where justified by the foundation calculations.

As a first step, bearing pressures are checked as follows:

Check longitudinal reinforcement based on moment demand in grade beam to transfer the column overturning load into the soil below

$V_{fy} = 22.8$ kips

$M_{OT} = 2V_{fy} \times (height + h/2) = 410.4\ kip\text{-}ft$ Overturning moment from cantilever column (Figure 5-29)

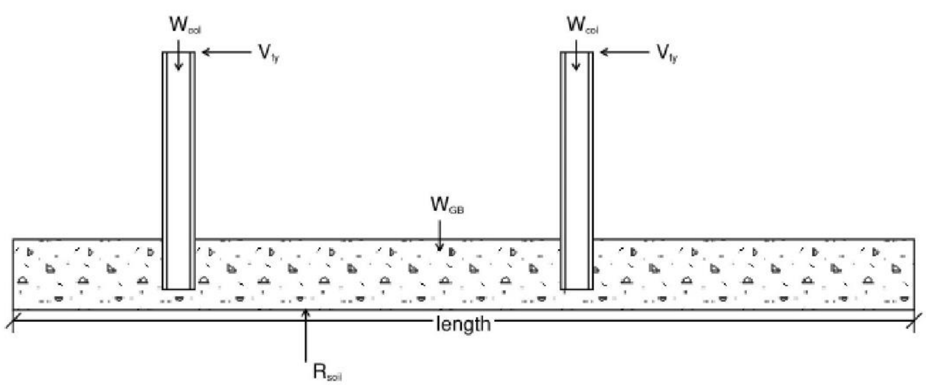


Figure 5-29 Cantilever column pair with grade beam and soil reaction resultant.

$length = 45$ ft

$W_{col} = 40\text{plf}\ (height + d) = 390.0$ lbf Self weight of column

$W_{GB} = 150\ \text{pcf} \times b \times h \times length = 33.7$ kips Self weight of grade beam

$$M_{grav} = W_{GB}\left(\frac{length}{2}\right) + W_{col}(10\text{ft} + 30\text{ft})$$
$$= 775.0 \text{ kip-ft}$$

Per FEMA P-807 Appendix B Section 7.5.1, only the dead load of the retrofit elements shall be included in the design unless the design explicitly transfers existing dead load to the retrofit element or incorporates existing gravity framing into the retrofit element.

$$R_{soil} = W_{col} + W_{GB} = 34.1 \text{ kip}$$

In this case, the example incorporates only the weight of the cantilevered columns and the new grade beam.

$$x_{soil} = \frac{\left(M_{grav} - M_{OT}\right)}{R_{soil}} = 10.7 \text{ ft}$$

Soil resultant is outside of the middle third of the grade beam, which means that the soil pressure goes to zero at some point along the length of the grade beam and the force has a triangular distribution. The centroid of a triangle is at the third point, which allows the length of the triangular distribution to be calculated.

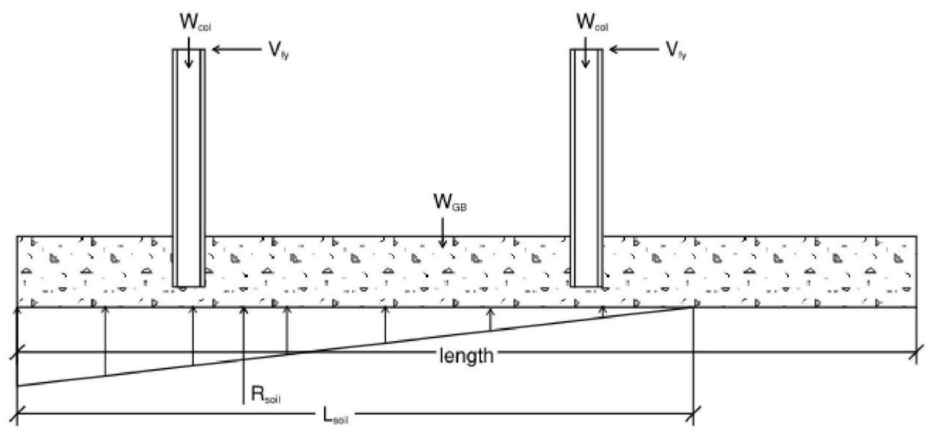


Figure 5-30 Cantilever column pair with grade beam and soil pressure distribution.

$$L_{soil} = 3 \times x_{soil} = 32.0 \text{ ft}$$

$$f_{max} = \frac{2R_{soil}}{L_{soil}b} = 852.6 \text{ psf}$$

Maximum expected bearing pressure (Figure 5-30)

In this example the calculated 853 psf soil bearing pressure easily falls within the IBC prescriptive bearing values at allowable stress design level. When using capacity-based design, this is a conservative criterion to use. It should be permissible to exceed this value by a substantial amount. Where bearing pressures are significantly greater than code prescriptive values (i.e., more than double), consultation with a geotechnical engineer is recommended.

Recommendation Note

It is recommended that closed ties be provided at a spacing of not greater than *d*/2 for the full length of the new grade beam, even where not required for shear capacity. It also is recommended that four closely spaced closed ties be placed on either side of each W8 to provide extra confinement where load transfer from the steel to concrete occurs.

Next, a check of the foundation shear demand against ACI nominal capacity shows that the unit shear is low enough that shear reinforcing is not required. However, closed ties are recommended at a spacing of not greater than *d*/2 where *d* is the effective depth of the grade beam as defined by ACI 318. The closed ties will provide confinement and crack control and will aid in construction by helping to maintian intended longitudinal rebar position while the concrete is placed. Four closely spaced closed ties will be placed on either side of each W8 to provide extra confinement where load tranfer from the steel column to concrete occurs (Figure 5-26). For new construction, ACI 318 Section 18.13.3.3 would require that the grade beam reinforcement use the confinement requirements for concrete special moment frame beams; this requirement would be triggered when concentrated moments from seismic forces are transferred to the grade beams. As discussed in Chapter 4, for purposes of SWOF bulding retrofit using capacity-based methods, this ACI requirement is not deemed to be necessary.

The maximum vertical shear occurs at the point where the soil bearing pressures equal the self weight of the foundation (Figure 5-31). The following calculation identifies the design shear based on this criterion.

$$q_{soil} = \frac{f_{max}}{length} = 18.9 \ \frac{\text{psf}}{\text{ft}}$$ Soil loading gradient

$$w_{gb} = b \times h \times 150 \text{ pcf} = 750.0 \text{ plf}$$ Self weight of grade beam

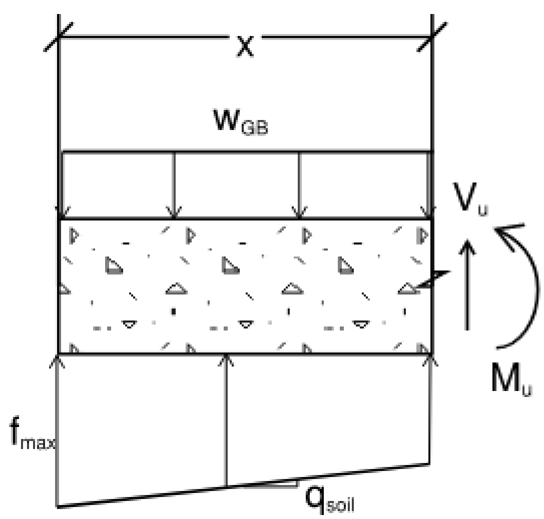


Figure 5-31 Free-body diagram of a section of the concrete grade beam.

$$V_{soil}(x) = b\left(f_{max}x - \frac{q_{soil}x^2}{2}\right)$$

$$V_u(x) = V_{weight}(x) - V_{soil}(x)$$

Ignoring the contribution of the columns, which will have minimal effect on the shear in the foundation

$$x_V = L_{soil} - \frac{w_{gb}}{q_{soil}b} = 16.2 \text{ ft}$$

$$V_{max} = V_u(x_V) = \text{-}16.2 \text{ kip}$$

Maximum shear value occurs at the point where the soil line load equals the foundation weight line load

$$V_c = 2\sqrt{f'_c \text{ psi}} \times b \times d = 69.0 \text{ kip}$$

$$\frac{|V_{max}|}{\frac{1}{2}V_c} = 0.5$$

No shear reinforcement is required for strength

Provide minimal ties for grade beams per ACI 318-14 Section 18.13.3.2

$$s_{min} = \min\left(\frac{b}{2},\ 12 \text{ in.}\right) = 12.0 \text{ in.}$$

Provide closed ties at 12" o.c.
Ties are not required but are recommended

Finally, the moment in the grade beam is calculated and used to determine the required longitudinal reinforcing (Figure 5-32).

$$M_{OT} = V_{fy}\,(height + h/2) = 205.2 \text{ kip-ft}$$

Overturning moment due to a single column

$$M_{soil}(x) = b\left(\frac{f_{max}x^2}{2} - q_{soil}\frac{x^3}{6}\right)$$

Moment due to soil reaction

$$M_{weight}(x) = w_{gb}\frac{x^2}{2}$$

Moment due to weight of grade beam

Figure 5-32 Moment distribution in the concrete grade beam.

$$M_u(x) = \begin{vmatrix} M_{soil}(x) - M_{weight}(x) & \text{if } x \le 10 \text{ ft} \\ M_{soil}(x) - M_{weight}(x) - M_{OT} - W_{OT}\left(x - 10\text{ft}\right) & \text{if } x > 10 \text{ ft} \\ M_{soil}(x) - M_{weight}(x) - 2M_{OT} - W_{OT}\left(2x - 40\text{ft}\right) & \text{if } x > 30 \text{ ft} \end{vmatrix}$$

Check four possible peaks in moment diagram

$$M_{test} = \begin{pmatrix} M_u(10 \text{ ft}) \\ M_u(10.1 \text{ ft}) \\ M_u(30 \text{ ft}) \\ M_u(30.1 \text{ ft}) \end{pmatrix} = \begin{pmatrix} 61.2 \\ -142.9 \\ 195.5 \\ -7.8 \end{pmatrix} \text{kip-ft}$$

$$M_{umax} = \max\left(\left|\vec{M}_{test}\right|\right) = 195.5 \text{ kip-ft}$$

Maximum moment demand on foundation

Capacity of grade beam

$A_s = 2.2$ in.^{2} — 5 #6 bars top and bottom

$$a = \frac{A_s f_y}{0.85 f'_c b} = 1.7 \text{ in.}$$

Depth of equivalent concrete compression block

$M_n = A_s \times f_y\,(d - a/2) = 221.5$ kip-ft

$M_{umax} / M_n = 0.9$ — < 1 OK

Recommendation Note

Extend the collector at the top of the open-front vertical elements for the entire available length of the diaphragm. This helps to ensure that the seismic capacity of the vertical elements can be developed without having the diaphragm serve as a weak link.

When designing a new building in accordance with the building code, a typical next step would be to check resistance to sliding between the foundation and the supporting soils. Friction and possibly lateral bearing on soils typically provide sliding resistance. For soft-story retrofits in general and in particular at lines of resistance where steel cantilever columns or moment frames are provided, it is difficult to provide adequate resistance to sliding. This is further exasperated when capacity design procedures are used, creating increased demand. At the same time, inherent connectivity at the foundation level makes local sliding of a frame line unlikely, and global sliding is not viewed as likely to pose a life-safety hazard. For these reasons, it is recommended to omit calculation of sliding resistance, provided that new foundations are extended to and doweled into existing perpendicular foundations at each end, or are doweled at a regular interval to existing parallel foundations along their full length.

Recommendation Note

Omit calculation of sliding resistance provided that new foundations are extended to and doweled into existing perpendicular foundations at each end, or are doweled at a regular interval to existing parallel foundations.

5.4.13.4 COLLECTOR DESIGN

Following capacity-based methods, the capacity of the vertical elements is used to design a collector along Line 1, serving to transfer loads between the new vertical elements and the second-floor diaphragm above. In accordance with Chapter 4 recommendations, the collector is designed to extend the full length of Line 1. Figure 5-33 and Figure 5-34 illustrate the collector and the connection of the vertical element to the collector. The collector addresses loading in the strong direction of the cantilever columns; later discussion will address additional load transfer considerations in the column weak-axis direction (see Section 5.4.13.6).

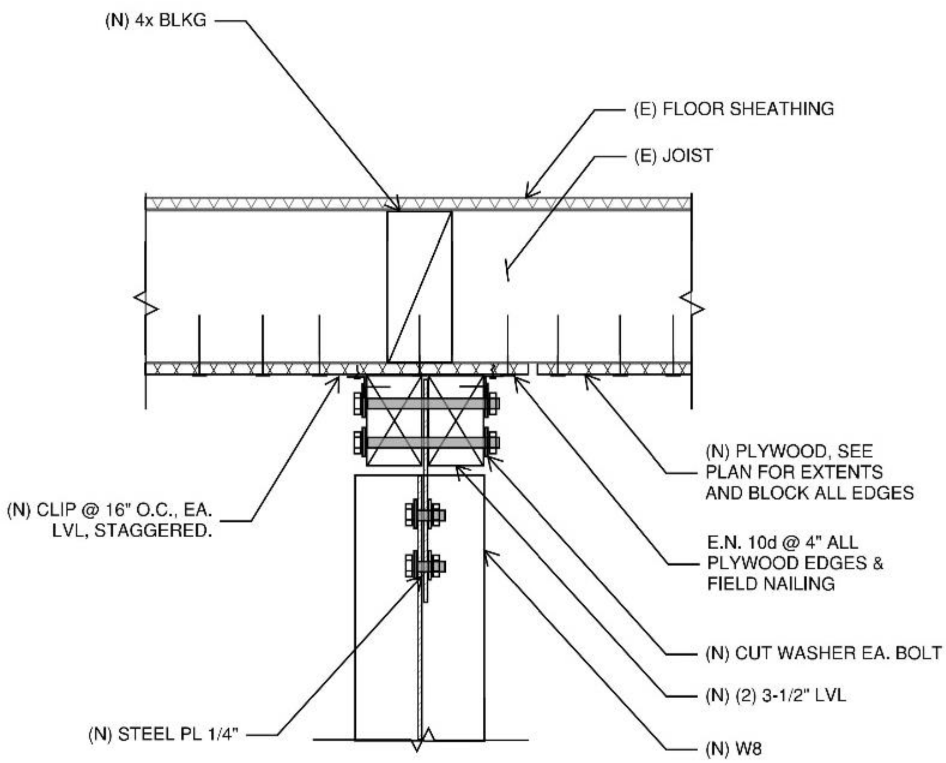


Figure 5-33 Load path connection from top of steel cantilever column to LVL collector.

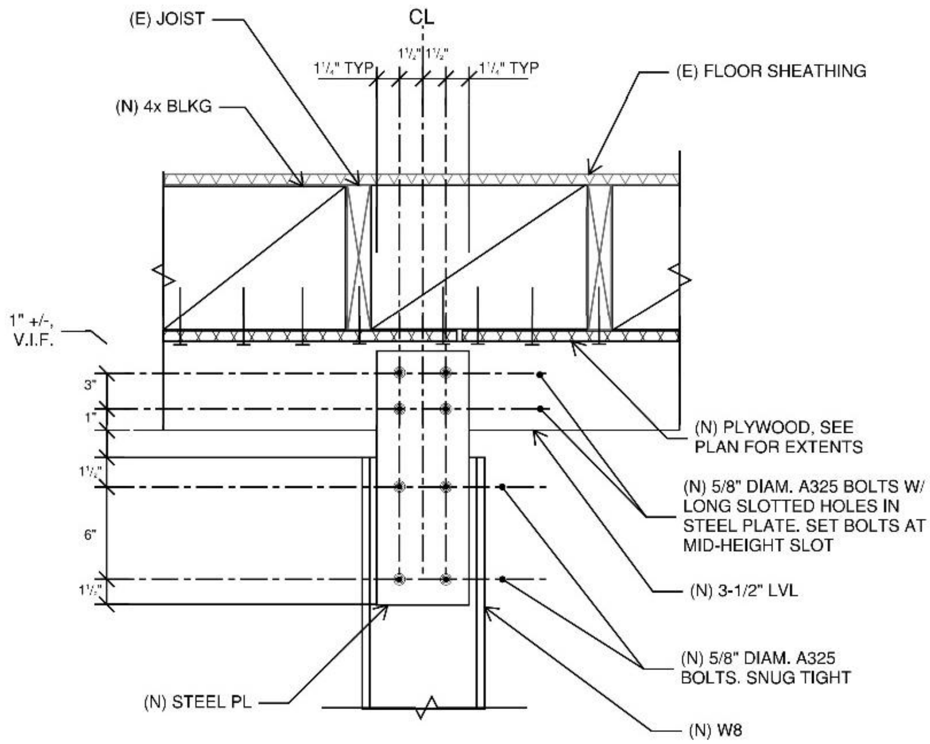


Figure 5-34 Load path connection from top of steel cantilever column to LVL collector.

The collector consists of two laminated veneer lumber (LVL) members designed for tension and compression using the horizontal reaction from the steel columns corresponding to the steel column strong-axis expected capacity. The LVL collector in turn transfers the horizontal reaction to a wood-structural-panel diaphragm added to the underside of the second-floor framing; this aids in transferring the reaction to the existing floor diaphragm above. While it might be possible to select a single LVL section to serve as the collector, the use of two LVLs keeps the eccentricity of the W8-to-LVL connection at a negligible level.

The first step in design of the collector is determination of the axial forces over the length of the collector, as illustrated in Figure 5-35.

Figure 5-35 Tension demand diagram for collector (compression similar).

$V_u = (2 \times V_{fy}) / (45 \text{ ft}) = 1013.4$ plf	Unit shear transferred through collector
$T_u = v_u \times 25 \text{ ft} = 25.3$ kips	Tension/compression demand of collector beam

Once the collector force distribution is known, the collector member is designed for tension and compression in accordance with the 2018 NDS.

Two new 3-1/2 × 5-1/2 LVL

$b = 3.5$ in.

$d = 5.5$ in.

Perform a preliminary sizing check on new collector beam (later also check for net section after designing bolted connection to column).

$F_t = 1300$psi	From TruJoist product manufacturer literature
$K_f = 2.70$	LRFD factor for F_t AWC NDS Table N1
$T_n = F_t\, K_f\, b\, d$	Tension capacity
$T_u\, /\, (2T_n) = 0.19$	< 1 OK

Next the bolted connection between the steel connection plate and the LVLs is designed based on the NDS. A double-shear connection is used with two LVL side members and a steel plate main member. The four bolts connecting between the steel plates and the LVLs are provided with vertical slotted holes in the steel plate. This will restrict the bolts to only transmitting horizontal loads to the LVLs. Avoiding vertical components in the bolt loads simplifies wood design requirements. This also allows for bolt movement should the moisture content of the LVLs change in the future, thereby reducing the likelihood of splitting the LVLs.

$V_u = V_{fy} = 22.8$ kips

Plate size: 5-1/2" x 18" x 1/4"

$t = 1/4$ in.	Plate thickness
$b = 5.5$ in.	Plate width

Bolts - 3/4" diameter

$d_{bolt} = 0.75$ in.

Plate Shear Strength

$A_s = t(b - 2d_{bolt}) = 1.0{\cdot}\text{in.}^2$	Steel cross sectional area

$f_y = 1.3 \times 36$ ksi = 46.8 ksi	Expected strength of A36 plate (AISC 341-16 Table A3.1)
$C_v = 1.0$	Web shear coefficient
$V_n = 0.6A_sf_yC_v = 28.1$ kips	Steel nominal shear strength
$V_u / V_n = 0.8$	< 1 OK

Bolts to LVL

Use 3/4" A325N bolts

Bolt in double shear at steel plate:

$V_n = (35.8 \text{ kip})/0.75 = 47.7$ kip	Nominal shear strength per bolt from Table 7-1 in the AISC Steel Construction Manual (Group A, threads not excluded from shear plane)
$V_u / (4V_n) = 0.1$	<1 OK

Bearing of bolts in LVL

$Z_{//} = 8800$ lbf	Per bolt. Value is found using the AWC connection calculator, with a 1/4" steel plate main member and 3.5" DF/L side plate members, which is an acceptable approximation of LVLs with a $G = 0.5$ for lateral connection design, as given by the TrusJoist catalog
$K_f = 3.32$	LRFD coefficient for connections (AWC NDS Table N1)

The bolted connection between the steel connection plates and the W8 is designed based on AISC 360. These bolts are in standard holes. In this example, the bolts are treated as two lines providing a couple to resist the moment (Figure 5-36), although they could also be treated as a typical four-bolt group.

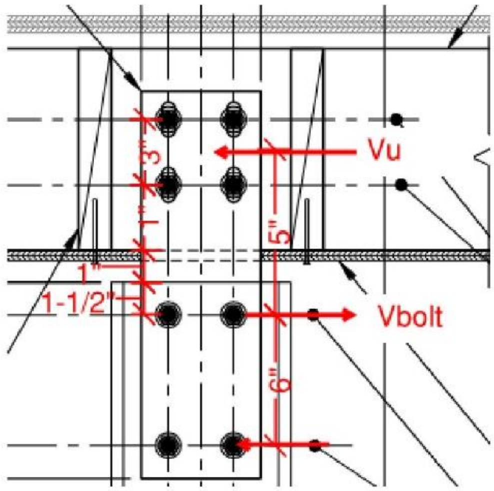


Figure 5-36 Bolts acting as a force couple to resist the shear from the collector.

The bolts at the column will create a couple to resist the combined shear and moment from the collector connection. The top row of bolts will resist the highest shear loads.

V_{bolt} = (V_u × 11 in.)/6 in. = 41.8 kips	Maximum shear in bolts (top row)
V_{bolt} / 2(V_n) = 0.9	OK
Required detailing dimensions	AISC 360 Section J3
End = 7/8 in. + (3/4)×*diam* = 1.44 in.	
Spacing = 3 × *diam* = 2.3 in.	AISC 360 Section J3.3
Edge = 1 in.	AISC 360 Table J3.4

Use 1-1/2" end distance, 1" edge distance, 3" spacing

As a final step in the collector design, the LVL adequacy is checked considering the net section at the bolt holes.

Net Section of Collector in Tension

T_u = 25.3 kips

b = 3.5 in.

d = 5.5 in.

F_t = 1300 psi — From TruJoist product manufacturer literature

K_f = 2.70 LRFD factor for F_t AWC NDS Table N1

$A_{bolt} = 2b\,(diam + 1/16 \text{ in.}) = 5.7 \text{ in.}^2$ Area lost due to bolt holes

$T_n = F_t K_f\,(b \times d - A_{bolt}) = 47.6$ kip Tension capacity

$T_u / (2\,T_n) = 0.27$ < 1 OK

Compressive section ok by inspection (unit stress capacity is higher)

5.4.13.5 SHEAR TRANSFER INTO THE SECOND FLOOR DIAPHRAGM

Even when a collector is provided extending the full length of the wall line, it is often necessary to provide a wood-structural-panel soffit on the underside of the second floor in order to help transfer load from the collector into the second-floor diaphragm. This is particularly true when using capacity design methods in combination with lumber-sheathed diaphragms, as occurs in this example. For this example, a wood-structural-panel ceiling diaphragm approximately 12 feet wide (from Line 1 to Line 2) will be provided for the length of Line 1 in combination with the collector. The 12 feet is selected as a convenient width for the example building configuration. Figure 5-37 illustrates the wood-structural-panel soffit and collector, along with shear transfer to the diaphragm above.

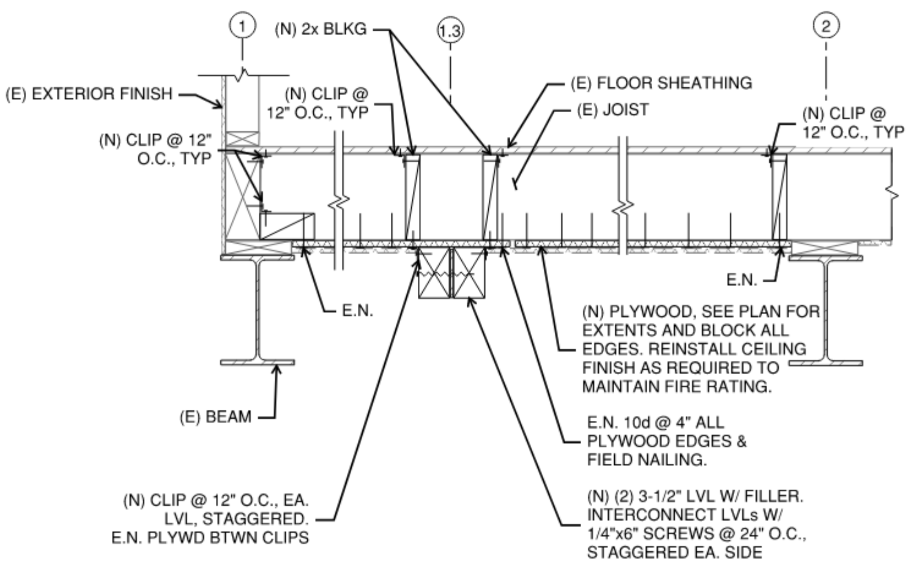


Figure 5-37 Wood-structural-panel soffit on underside of second-floor framing and load path connections from collector to diaphragm.

Recommendation Note

Because the second-story walls are not being evaluated or retrofit, the load path from the second-story walls to the collector is not well defined. Lacking a better understanding of the load path to the collector, it is reasonable to design for half of the load to come from the right of the collector and the other half to come from the left. The inclusion of the wood-structural-panel soffit and multiple lines of shear clips between the wood-structural-panel diaphragm and existing diaphragm help to avoid stress concentrations in the existing diaphragm that might otherwise occur.

Because the second-story walls are not being evaluated or retrofit, the load path from the second-story walls to the collector is not well defined. Because of this, judgement is needed in design of the load path from the collector to the diaphragm above. It is assumed that a good portion of the load to the collector comes from the second-floor exterior wall seen at the left-hand side of Figure 5-37. A portion of the load will also come from the right-hand side of the collector. Lacking a better understanding of the load path to the collector, it is reasonable to design for half of the load to come from the right of the collector and the other half to come from the left. In order to allow for these multiple load paths, multiple lines of blocking and shear clips are provided between the existing diaphragm sheathing above and the new diaphragm sheathing below. In Figure 5-36 three lines of transfer are designed, one on the left, one at the collector, and one to the right of the collector. This helps to avoid the stress concentration that would occur if a single line of transfer were provided.

Per the calculations below, the unit shear transferred to the wood-structural-panel soffit should be about half of the 1013 plf, or about 507 plf, which is within the expected capacity of a blocked wood-structural-panel diaphragm. The wood-structural-panel soffit then spreads the load over a number of resisting lines of joists or blocking, so that the unit shear transferred to the existing lumber diaphragm should be less than half of 1013 plf. With three lines of transfer the unit shear is about 338 plf in this example. This is less than the expected capacity of the diagonal-lumber-sheathed diaphragm as described in Chapter 2 (505, 1024 plf) and is acceptable. If the unit shear calculated were to be much larger than the capacity of the lumber-sheathed diaphragm, the extent of the wood-structural-panel diaphragm sheathing should be increased.

Plywood Soffit

The exact load path from the cantilever column to the diaphragm is an uncertainty in the load-path calculations. It is recommended to assume that 50% of the collector force goes to each side of the LVL collector.

$$v_u = n_{col} \times V_{fy} / (45 \text{ ft} \times 2) = 506.7 \text{ plf}$$

v_n = 755 plf	Blocked plywood diaphragm sheathing with 8d nailing at 6" o.c. – AWC NDS SDPWS Table 4.2A nominal wind capacity. Value assumes a 3/8" nominal thickness and 2x supporting framing. The nominal capacity for wind loading given by the NDS SDPWS is assumed to be approximately the expected capacity of the plywood.

The nominal capacity for wind loading given by the NDS SDPWS is assumed to be approximately the expected capacity of the plywood.

v_u / v_n = 0.67	< 1 OK

Transfer to Second-Floor Diaphragm

$v_u = n_{col} \times V_{fy}$ /(45 ft) =1013.4 plf	Unit shear to transfer into diaphragm

As with the plywood soffit above, the exact load path is unknown, so it is assumed that the plywood soffit spreads the load transfer out to at least three lines of joists or blocking.

$v_{u.diaph}= v_u$ / 3 = 337.8 plf	Unit shear to transfer to diaphragm (each side of collector)
v_n = 507plf	Strength of diagonally sheathed diaphragm (See Chapter 2). Tensile capacity is lower than compression capacity.
$v_{u.diaph}$ / v_n = 0.67	<1 OK

5.4.13.6 DEFORMATION COMPATIBILITY CONSIDERATIONS

Consideration needs to be given to deformation compatibility of the cantilever steel column elements as they drift in the weak-axis direction. Because the steel columns are cast into the foundation providing full fixity at the column-to-foundation interface, the column will try to act as a cantilever resisting element in the weak-axis direction, as well as the strong-axis direction.

With foundation detailing of the type shown in this example, the connections between the cantilevered steel columns and the concrete foundation are believed to generally adequate to develop the weak-axis expected capacity of the column. As a result, it is reasonable to assume that the weak-axis expected moment is transferred to the foundation. Should the connection not be adequate, premature failure of the retrofit columns in the weak axis could occur. This would be unacceptable performance and should be avoided.

The extent to which the expected moment can be transferred from the foundation to a combination of surrounding soils and foundations is much more variable and less certain. Where there are weak soils and minimal restraint from surrounding foundations and slabs, the foundation may tend to rock in the column weak-axis direction while developing little restraint. With either stronger soils or more foundation restraint, higher weak-axis moments could develop. It is recommended that it be assumed that some degree of restraint will occur and some resisting moment will develop at the foundation.

The issue of primary concern is the shear force that develops at the top of the steel cantilever column due to story drift in the weak direction. In particular, it is important that this force does not damage the connection of the cantilever column to the diaphragm above. If damage were to occur, the connection might no longer function for the strong-axis load path. This was discussed in Chapter 4.

It is recommended that the connection from the steel column to the diaphragm above develop the shear generated by the column reaching expected capacity in the weak axis direction. This will serve to protect the load path connections, regardless of the level of restraint developed at the cantilevered column foundation. Alternatively, a specific mechanism could be detailed to avoid shear transfer in the weak axis direction. It is cautioned that the expected capacity of the foundation restraint, however, is difficult to determine and detailing to accomplish this is complex and might not be readily constructable.

Recommendation Note

It is recommended that the connection from the steel column to the diaphragm above develop the shear generated by the column reaching expected capacity in the weak-axis direction.

Based on the recommendation for providing a top-of-column connection to develop the column weak-axis expected moment, design is required for a 10.6-kip reaction at the top of each column. The transfer will occur through the steel plate seen in Figure 5-33 and Figure 5-34. This plate will need to be checked per AISC provisions for shear and flexure in the plate weak-axis direction. The force transfer mechanism also involves tension in the steel-to-steel bolts and the steel plate bearing on the face of the LVLs.

In addition, a secondary collector is provided, perpendicular to the primary collector and extending the 12 feet across the wood-structural-panel ceiling soffit (Figure 5-38). Load is transmitted to the secondary collector through a combination of bearing between the primary and secondary collector framing members and the steel strap at the bottom of the secondary collector, which lets the sections of the secondary collector on either side of the main collector act in unison. Shear clips are provided between the collector and the wood-structural-panel soffit. The unit shear in the soffit is checked (assuming 50% of the load goes to each side of the paired collector members), resulting in a unit shear of 10.6 kips/ (2 sides x 12 ft) giving 442 plf, which is less than the expected capacity of the wood-structural-panel diaphragm. The calculation of the load path is not taken any further than this, as the design provided succeeds in eliminating the potential for local failure of the column connection and distributes the reaction into the combined existing and new diaphragm.

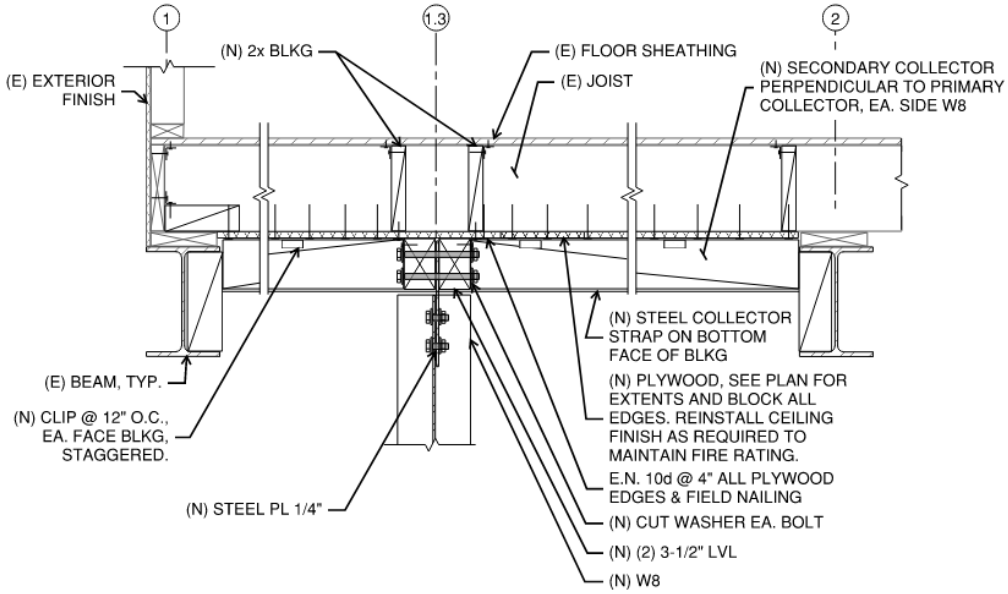


Figure 5-38 Wood-structural-panel soffit on underside of second-floor framing with secondary collector shown.

5.4.14 Design of Shear Walls

FEMA P-807 Appendix B.3 Section 7.4.1 requires other retrofit elements to be wood-structural-panel sheathed shear walls. The design and detailing of the shear walls will follow the same capacity-based approach as the steel cantilever columns. The sheathing and nail spacing has already been selected for the shear walls and determined adequate in the Weak-Story Tool. The selected sheathing and nailing is 15/32 rated sheathing with 8d common at 4 inches on center.

5.4.14.1 SHEAR WALL DESIGN

The provisions of the building code and 2015 SDPWS will be used for the details of shear wall construction.

5.4.14.2 SHEAR TRANSFER TO FOUNDATION

Shear anchorage at the base of the wall is designed using capacity design methods and the provisions of the 2018 IBC and ACI 318. Section 1905.1.8 of the 2018 IBC modifies ACI 318 section 17.2.3.5.2 to allow anchor bolts resisting in-plane shear to be designed based on the values given in NDS Table 12E without considering ACI anchorage to concrete provisions (ACI 318-14 Chapter 17), as long as certain requirements are met with regards to anchor bolt spacing and end/edge distances. See the 2018 IBC for more details. Anchor bolts connecting the shear wall to the existing foundation are shown in Figure 5-39.

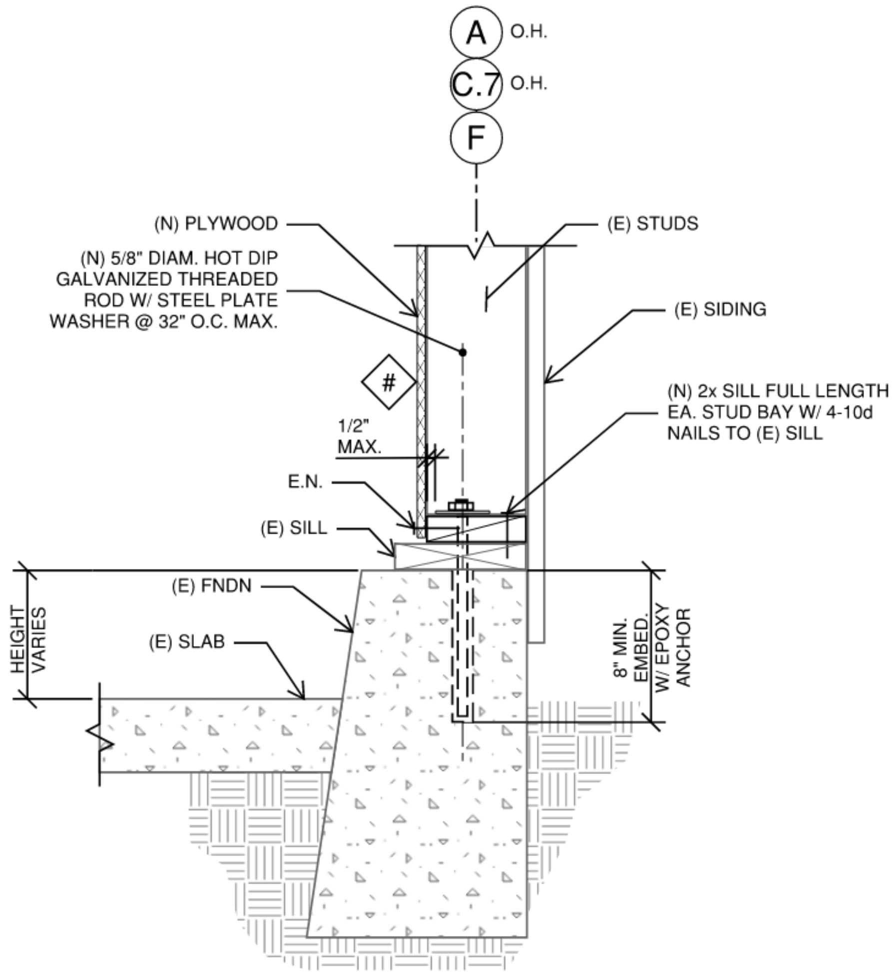


Figure 5-39 Shear load path connection at base of retrofit shear wall.

Per 2015 SDPWS Section 4.3.6.4.3, steel plate washers not less than 0.229 × 3 × 3 inches are required on all anchor bolts and should extend to within 1/2 inch of the back face of the wood-structural-panel sheathing. While it is possible that there are some existing anchor bolts that could be used towards the quantity of required bolts, the existing anchor bolts may not be in an adequate condition to be used, and it may not be possible to add the required plate washers to the existing anchor bolts. For these reasons, it is best to plan on adding new bolts to meet the calculated requirements.

5.4.14.3 FOUNDATION HOLD-DOWNS

Hold-downs (or tie-downs) for shear wall overturning and their anchorage to the foundation are designed using capacity-design methods and ACI 318 implemented using the software provided by the anchorage manufacturer. Because the loading is at a capacity level, overstrength factors are not required per ACI 318-14 Section 17.2.3.4.2(c). Where multiple hold-down anchors are required to resist the overturning tension, group action should be considered for the anchorage. Hold-downs connecting the shear wall to the existing foundation are shown in Figure 5-40.

For purposes of calculating the tension load to be carried by the hold-down, the overturning forces on the second and third stories are assumed to be substantially resisted by the self-weight of the second and third stories and therefore to not contribute net uplift forces to the new hold-downs being added at first story shear walls. This is consistent with the simplified overturning assumption of the Weak-Story Tool and the FEMA P-807 analytical studies that assumption is based on. Should there be circumstances where it is believed that consideration of the upper story overturning is needed, guidance is provided in FEMA P-1100, *Vulnerability-Based Assessment and Retrofit of One- and Two-Family Dwellings* (FEMA, 2019).

The first story hold-down force is calculated as the nominal unit shear capacity of the wood-structural-panel sheathing times the height of the wall, with no reduction based on resisting dead load.

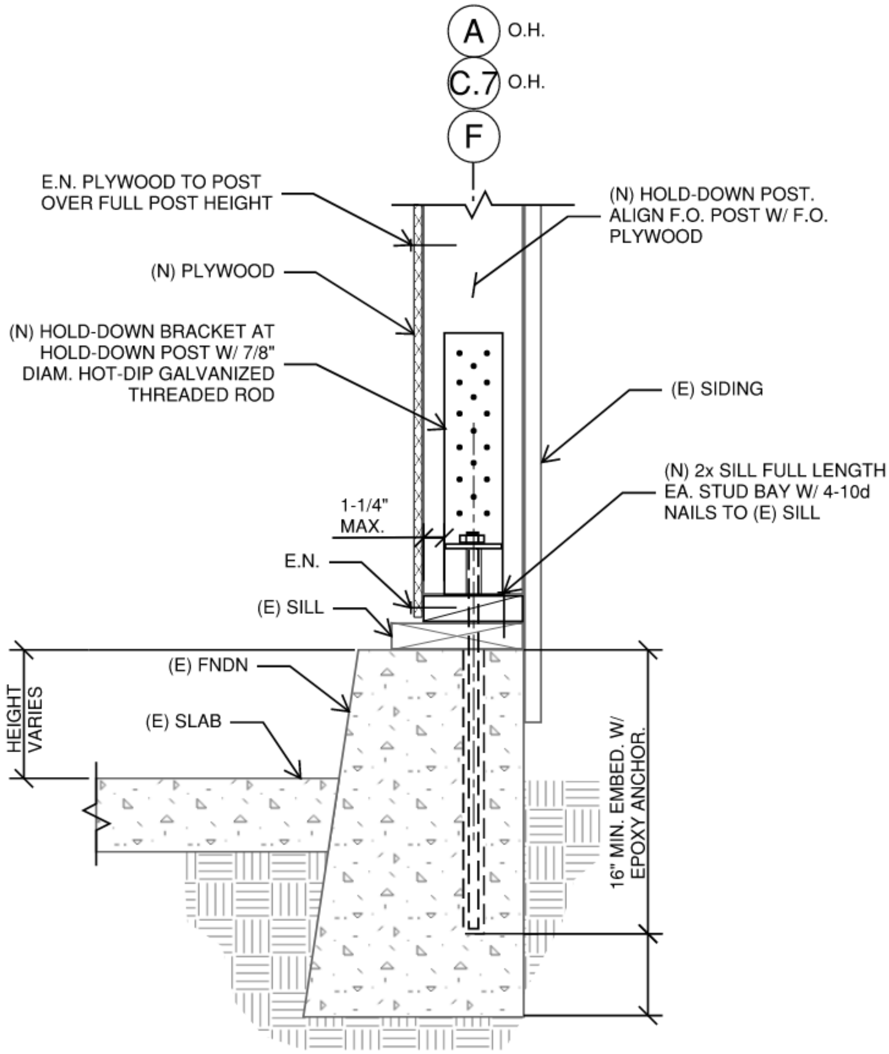


Figure 5-40 Uplift load path connection at the base of retrofit shear wall.

Hold-down Brackets

Provide new hold-down brackets at ends of new plywood shear walls to provide overturning resistance. Epoxy hold-down threaded rods into existing foundations below.

$height = 8.0$ ft Interstory height

$OT = ShearWall_{expected} \times height = 8.9$ kip

The expected overturning force is approximate because it neglects both dead load resisting overturning and any overturning force from the story above. This assumes that the upper stories will act as a box system and that the weight of the upper stories will provide overturning resistance for the seismic forces in the upper stories. This assumption is consistent with the use of the simplified approach in the Weak-Story Tool and the analytical studies behind the approach. It is recommended that the appropriateness of this approach be evaluated by the designer on a case-by-case basis.

$HD_{expected}$ = 15275 lbf	Hold-down bracket expected tension capacity Source: Simpson Strong-Tie ASCE/SEI 41 expected tension capacity of HDU8 hold-down (Figure 5-40)
$OT / HD_{expected}$ = 0.6	<1 OK

Hold-down Rod Anchorage

7/8" diameter threaded rod, anchored into concrete with epoxy.

$TD_{expected}$ = 2170 lbf / 0.65 = 3338.5 lbf	Expected capacity of 7/8" epoxy anchor with 16" minimum embedment, based on ACI 318-14 Chapter 17, calculated by Simpson Anchor Designer Software. (φ = 0.65 not included in capacity for expected strength)

When determining the expected strength of an anchor into existing concrete, be sure to assume cracked concrete, likely with no supplementary reinforcement. In seismic design category C, D, E, or F, an additional reduction factor of 0.75 is applied for seismic design. For adhesive anchors, the adhesive strength may be increased for anchors that resist only wind or seismic load, based on manufacturer literature.

$OT / TD_{expected}$ = 2.7	Try three anchors (check group action)

Demand is based on the expected capacity of the shear walls—no need for the overstrength factor based on ACI 318-14 17.2.3.4.3(c).

$TD3_{expected}$ = 5640 lbf / 0.65= 8676.9 lbf	Expected capacity of (3) 7/8" epoxy anchors with 16" minimum embedment and 16" spacing, based on ACI 318-14 Chapter 17, calculated by Simpson Anchor Designer Software. (φ=0.65 not included in capacity for expected strength)
$OT / TD3_{expected}$ = 1.03	OK for capacity level design

Check of Existing Foundation

In Section 5.4.14.3, the capacity level tension force for the hold-down is calculated to be 8.9 kips. This is a concentrated uplift load transmitted to the existing foundation in the example design. The foundation will need to be checked for shear and flexure to make sure that this load can be resisted without causing a local foundation failure.

When checking shear, it is reasonable to assume a shear demand of 4.5 kips to either side of the hold-down. With a conservative concrete shear capacity of 50 psi and a foundation 16 inches by 18 inches, the nominal shear capacity is 14.4 kips, more than adequate for the hold-down load.

For flexure, assumptions regarding loading and existing reinforcing are required to check adequacy. For the design example, an assumption of 600 plf load and simple span foundation beams extending 15 feet to either side of the hold-down result in the required reaction and in moments less than the nominal capacity of the foundation, assuming two No. 4 bars top and bottom in the foundation. In this case, the foundation alone is adequate. Where it is not, the walls above might be used as deep beams to mobilize required resisting load.

5.4.14.5 SHEAR TRANSFER TO SECOND-FLOOR DIAPHRAGM

Shear transfer from the top of the shear wall into the second-floor diaphragm is designed using capacity design, the 2015 SDPWS provisions, and manufacturer published literature on capacity values for manufactured shear clips. Shear clips provide a load-path connection from the first-story wall top plate to the rim joist or blocking above, and from the rim joist or blocking to the second-story bottom plate above (Figure 5-41). Manufactured products, such as clips and hold-down brackets, are typically required to be tested to at least 3x the published allowable stress design value therefore it is assumed the capacity of these elements is approximately 3x the published ASD value in the manufacturer's literature. Some manufacturers also publish ASCE/SEI 41 expected strength values, which are also reasonable to use for a capacity-based design. This ASCE/SEI 41 literature gives very similar values to 3x the published allowable value.

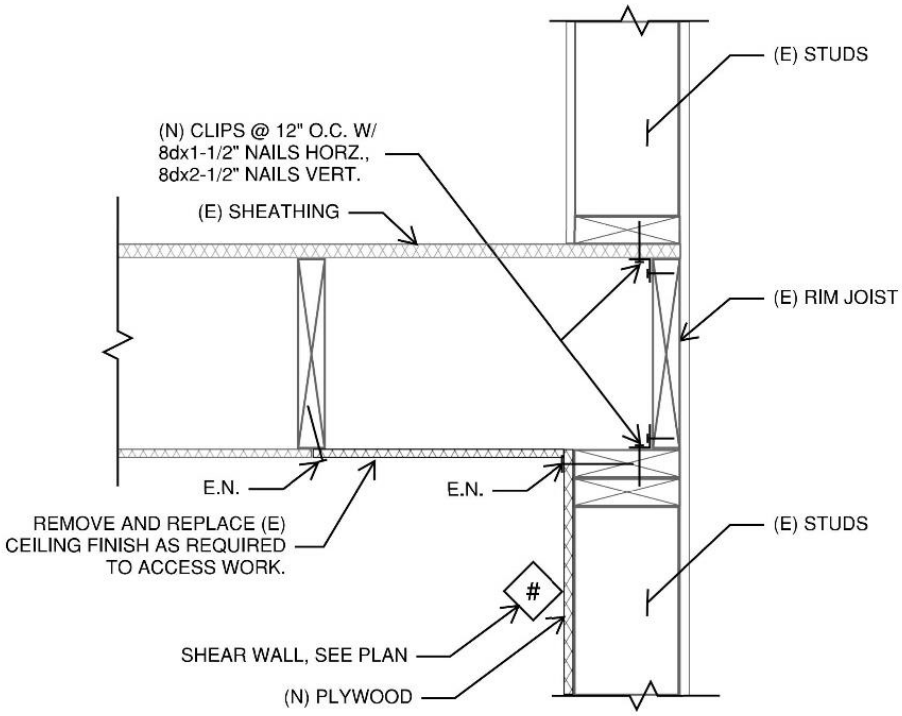


Figure 5-41 Load-path connection at the top of the retrofit shear wall.

5.4.14.6 SECOND-FLOOR DIAPHRAGM CHECK

The load path into the top of the wood-structural-panel sheathed shear wall includes a load path from the wall directly above and a second load path from the diaphragm. While the relative distribution of the load between these two sources is not known, it should be checked to see that the expected capacity for the sum of the two is equal to or greater than the expected capacity of the first story wall.

From earlier shear wall design information, we know that the expected capacity of the first-story wall is 1112 plf. Also, from earlier information, we know that the expected capacity of the diaphragm is approximately 500 plf in the weak direction and 1,000 plf in the strong direction. From Table 4-1 of FEMA P-807, we can also identify that the expected capacity of the second-story wall is 535 plf, summing stucco and gypboard. The expected capacity for the second story wall from this project is much higher. Because the lower end of the second-floor wall and diaphragm capacities (500 and 535 plf) approximately match the expected capacity of the first-story wall, load transfer from the first-story wall to these two elements is adequate. If the first-story wall were to be stronger than the sum, a wood-structural-panel ceiling soffit should be installed to help distribute loads, similar to the soffit that occurs at the cantilever column collector.

5.4.15 Load Path Connections for Existing Vertical Elements to Remain

FEMA P-807 Appendix B.3 Section 4.2.4 discusses verification of load-path connections for existing vertical elements to remain. The purpose is to verify a minimum level of load path interconnectivity for elements that are being relied upon. Although the survey can be conducted during the design of the retrofit, it is common practice to make an assumption during design and verify the existing condition during construction of the retrofit.

Where existing first-story walls included in the Weak-Story Tool model have existing wood-structural-panel sheathing, the load path at the top and bottom of the wall should be checked for the expected capacity of the combined wall materials in the same way the wall with new wood-structural-panel sheathing was checked.

For the rest of the existing first-story walls to remain, FEMA P-807 requires verification of the load path at the wall top and bottom. This entails verification that the framing nailing at the top of the wall is generally in accordance with conventional construction practice (e.g., joists toenailed to supporting wall top plates). This also entails making sure that there are anchor bolts or equivalent anchorage at the bottom of the wall. Where anchor bolts have been provided, 1/2-inch diameter at 6-feet on center are commonly found. FEMA P-807 is not specific about size or spacing, nor does it require any calculation, so the presence of some systematic anchorage is all that is required. If there is no systematic anchorage, however, new anchorage should be provided. FEMA P-807 includes guidance on the number of locations where load-path connections in existing walls need to be observed. There is no requirement that steel plate washers be added to existing anchor bolts that will remain.

5.4.16 Implications of FEMA P-807 Retrofit Design Using Vertical Elements Located Outside the Building Footprint

Where vertical elements are moved outside of the building footprint, the level of effort required to transfer forces to the vertical elements increases significantly. This section illustrates the detailing of force transfer to special steel cantilever columns located outside of the building footprint. For details of calculations the reader is referred to Calculation Package 2.

Figure 5-42 and Figure 5-43 illustrate conditions where the columns are located outside the building footprint. Placement of the cantilever column in the center of the new grade beam results in the column being pushed away from the face of the building, as seen in Figure 5-43. The column being pushed away creates a detailing challenge for the load path from the second-floor diaphragm into the cantilever column. One possible solution to this geometry is seen in Figure 5-42, where an HSS section is extended the 20 feet between cantilever columns and provides a stiff load path to transfer the torsion/moment generated by the eccentricity between the center of the cantilever column and the diaphragm edge. In Figure 5-42 a steel plate on the exterior face of the diaphragm serves as the collector and extends for the full length of Line 1.

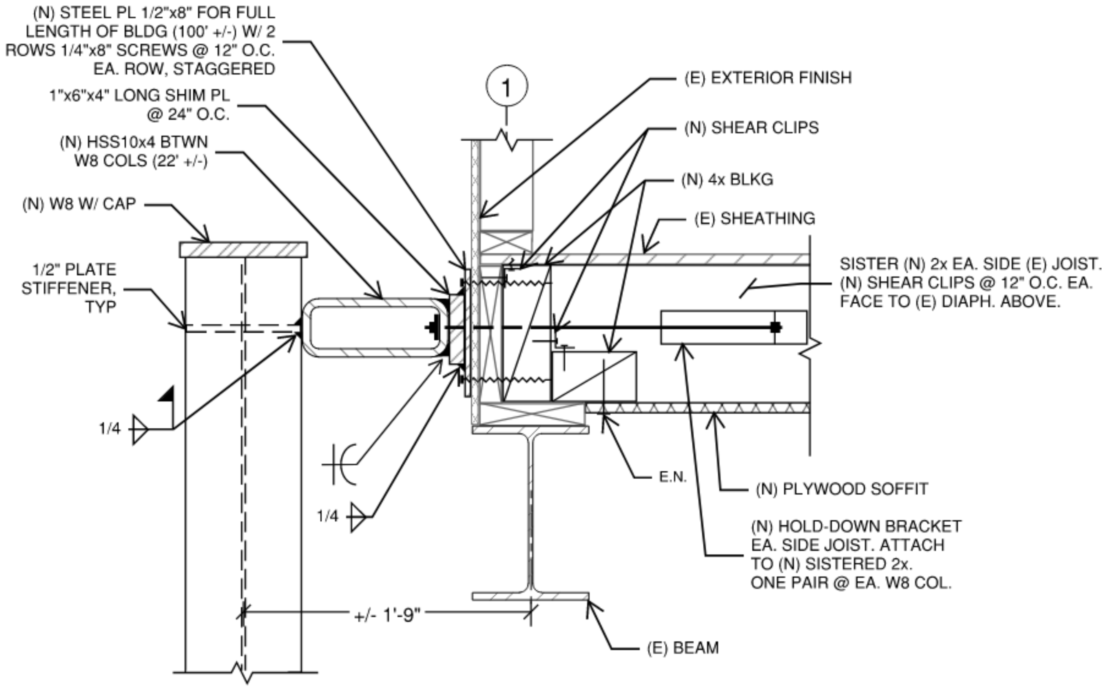


Figure 5-42 Top of steel cantilever column located outside the building footprint.

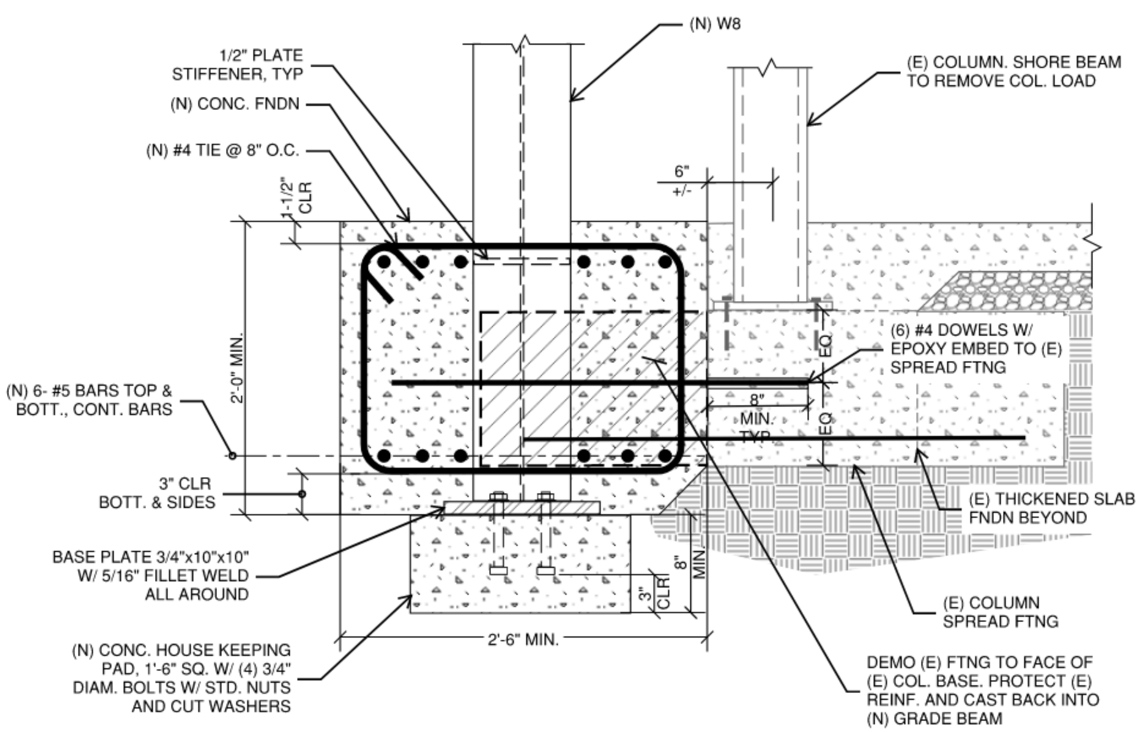


Figure 5-43 Bottom of steel cantilever column located outside of the building footprint.

5.4.16.1 COLLECTOR DESIGN

As a first step in design of the steel plate and HSS collector assembly, the collector axial force over the length of the diaphragm is determined. With these forces the collector steel plate can be checked.

$v_{u.col} = (2V_{fy})/(45\text{ ft}) = 1013.4$ plf	Unit shear transferred from HSS to steel plate collector
$v_{u.HSS} = (2V_{fy})/(22\text{ ft}) = 2072.9$ plf	Unit shear transferred from W8 to HSS
$P_u = v_{u.col} \times 25\text{ ft} = 25.3$ kips	Axial force demand of collector

Steel Plate

$t = 0.5$ in.	Thickness of plate
$d = 8$ in.	Depth of plate
$Area = t \times d$ 4.0 in.^{2}	
$f_y = 36$ ksi	Yield strength of plate
$R_{y.plate} = 1.3$	
$T_{capacity} = R_{y.plate} \times f_y \times Area = 187.2$ kips	Expected tension capacity of collector plate
$P_u / T_{capacity} = 0.14$	< 1 OK

The collector plate connection to the second floor is then designed. The plate is fastened through the existing wall stucco to the framing behind. This is based on the Chapter 4 recommendation that existing stucco be maintained so that the second-story stucco wall capacity is not compromised. Because of this approach, design will also need to be provided for the building envelop to maintain weather resistance at these connections.

The HSS member that spans the 20 feet between cantilever columns is used to resist both axial collector forces and the moment that occurs because of the eccentricity between the centerline of the W8 and the face of the second-floor framing (about 18 inches). As a result, the HSS resists a combination of axial and flexural loads.

5.4.16.2 RESOLUTION OF HSS FORCES INTO SECOND FLOOR

Once the HSS has been checked for collector forces, the next step is resolving the forces from the HSS into the second-floor system. The forces will include a moment couple to resolve the moment due to eccentricity and also the connection for the weak-axis column reaction discussed in Section 5.4.13.6 of this guideline.

As seen in Figure 5-42, a pair of hold-downs is provided at each end of the HSS section, tying the HSS to built-up floor framing members that are then tied into the wood-structural-panel ceiling soffit. This assembly provides the tension load path. The HSS bearing against the second floor is adequate for the compression load path.

5.4.16.3 INTERRUPTION OF EXISTING FOUNDATIONS

Where installation of the new cantilever columns and their grade beam interrupts existing foundations, it is necessary to address the effect on the existing footing. Based on allowable bearing pressures prescribed by the Uniform Building code in the 1960s and 1970s (when the example building is assumed to be constructed), it is estimated that the spread footings under these columns are approximately 5 feet by 5 feet. In the example detail shown in Figure 5-43, approximately 10 square feet of existing foundation (2 feet by 5 feet) are removed to allow construction of the new grade beam.

Prior to this work occurring, the beam that the existing column supports is shored to remove the beam load during demolition and reconstruction. Based on calculations of the load tributary to the existing footing, it is found that the soil bearing pressure for the existing footing is approximately 1000 psf. When multiplied by removed area of 10 square feet, this results in a capacity of 10 kips that is lost when the existing footing is cut back. Dowels are provided between the existing footing and new grade beam to ensure that this load can be transferred from the existing footing to the new grade beam. As a result, the required capacity of the existing foundation is maintained. In addition, the existing foundation rebar is maintained where the exiting foundation is partially demolished, and the rebar is cast into the new grade beam.

5.4.17 Implications of FEMA P-807 Retrofit Design Using Steel Special Moment Frames

Section 5.4.1 through Section 5.4.16 have illustrated retrofit design using steel cantilever columns at the open front. Steel moment frames are another commonly used retrofit element. The following sections discuss differences in the retrofit design using FEMA P-807 where special steel moment frames are used. For details of calculations, the reader is referred to Calculation Package 2, which includes detailed calculations related to use of the steel moment frame.

5.4.17.1 STEEL MOMENT FRAME DESIGN

The FEMA P-807 methodology requires that steel moment frames serving as new retrofit elements be designed as special moment frames; this is because the retrofit design relies on a high level of ductility in new elements. As a result, the AISC 341 provisions for special moment frames will be used. For projects of this size, use of prequalified moment connections (in accordance with AISC 358) or proprietary connections are the most practical approaches. For this design example, the prequalified reduced-beam-section (RBS) connection is selected. In addition to the details matching the prequalification criteria, AISC 341 design requirements around geometry of the steel moment frames will need to meet.

5.4.17.2 BASE FIXITY OF STEEL MOMENT FRAMES

The special steel moment frame base can be designed as either fixed or pinned. FEMA P-807 does not impose drift limits on new vertical elements in the way that code design for new structures would. Both the strength and stiffness of each new element is directly input into the FEMA P-807 Weak-Story Tool, and the effect of stiffness is directly considered in the adequacy of a proposed retrofit. Use of fixed-base column sections should result in lighter column sections being required in order to have an acceptable retrofit. The benefit of lighter sections needs to be balanced against two drawbacks of using fixed-base connections. First, the fixed-base connection can complicate detailing and construction, depending on the type of detail used, and second, a fixed base will prompt consideration of deformation compatibility and top of column connection for weak-axis reactions, as discussed in Section 5.4.13.6.

For purposes of this design example, it is selected to design a pinned-base moment frame, providing the heavier steel section but simplifying other aspects of detailing. Testing of steel retrofit frames (Mosalam et al., 2002) found that under cyclic loading typical pinned connection base plates readily deform such that minimal base fixity occurs, suggesting that modeling the base as a pinned condition is appropriate.

5.4.17.3 STEEL MOMENT FRAME MODELING IN THE WEAK-STORY TOOL

Within the FEMA P-807 Weak-Story Tool, there is little in the modeling that would differentiate treatment of the special steel cantilevered columns from the special steel moment frames. Vertical element modeling in the Weak-Story Tool would include steel frames with both the beams and columns, somewhat modifying the load-deflection plot. With pinned base columns, there is little reason to include foundation flexibility in the frame push-over curve as the effect of foundation fixity is negligible.

5.4.17.4 LATERAL TORSIONAL BRACING

The most significant difference with the use of special steel moment frames in place of cantilever columns is the design and detailing of lateral-torsional bracing at the moment frame beams. Lateral bracing requirements are found in AISC 358 Section D1.2c, with brace stiffness and capacity controlled by a combination of AISC 358 and AISC 360 provisions. For this example, lateral-torsional bracing is required at each column, just past the RBS location at each end of the beam, and at beam mid-span. The AISC stiffness requirements for lateral-torsional bracing are very difficult to meet with wood framing due to the combination of the inherent flexibility of wood structures and the local crushing and displacement that occurs where fasteners bear on wood. For these reasons, the lateral-torsional bracing method provided for this design example exclusively uses steel members and welded connections (Figure 5-44). Proprietary moment frame systems have also developed methods to address the difficulty of implementing lateral-torsional bracing stiffness requirements in wood-frame buildings.

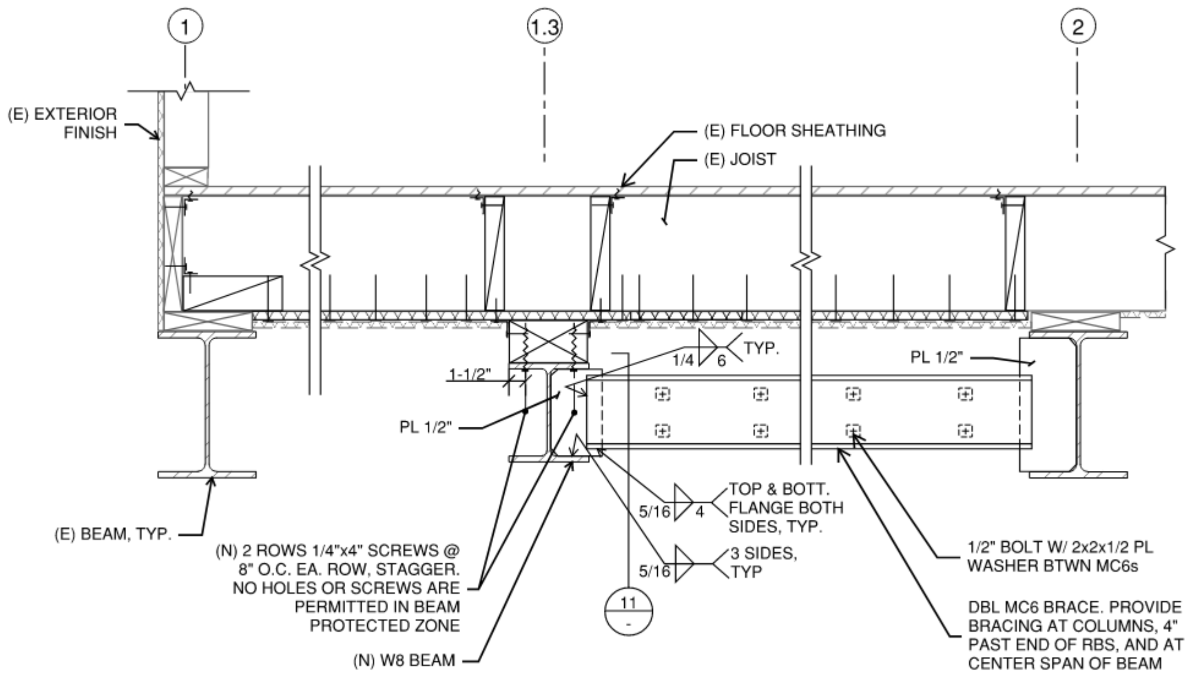


Figure 5-44 Lateral-torsional bracing of moment frame beam.

5.4.18 Implications of FEMA P-807 Retrofit of a Short-Side-Open Building

Sections 5.4.1 through Section 5.4.17 have focused on the design of a seismic retrofit for the three-story LO building. This section discusses implications of a retrofit for the short-side-open building. The project analytical studies have shown that three-story, short-side-open buildings are more vulnerable to collapse than the corresponding long-side-open building archetypes, underlining the need for retrofit. Many aspects of the seismic retrofit design for long-side-open and short-side-open buildings are the same. This section discusses aspects that vary.

5.4.18.1 LOCATION OF NEW VERTICAL ELEMENTS

A FEMA P-807 retrofit for the short-side-open example building was taken to a conceptual level. The resulting retrofit plan is seen in Figure 5-45. Similar to the long-side-open example building, the retrofit involved addition of steel cantilever columns at the open front line (Line 1), along with a new grade beam. In this case three columns are needed in place of the four for the long-side-open building. Also similar to the long-side-open building, addition of wood-structural-panel sheathing to the wing walls at each side of the garage is required (Line A and Line E). Unlike the long-side-open building, it was identified that wood-structural-panels sheathing is also needed on the transverse wall at the opposite end of the building from the steel cantilever columns (Line 9). A likely influence on this added retrofit scope is that with this building plan, Line 1 and Line 9 are better positioned to resist torsion than Line A and Line E.

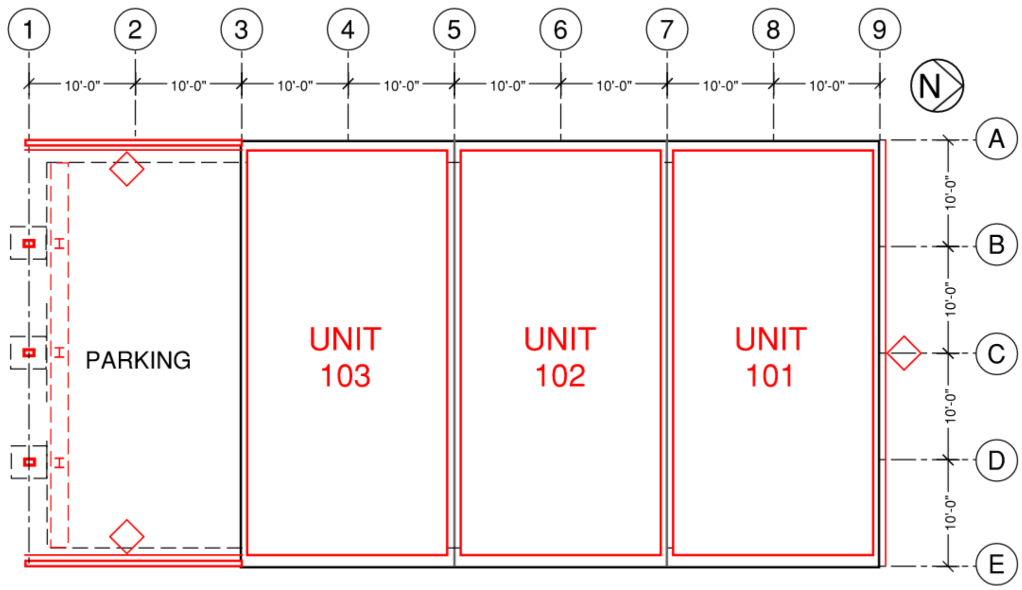


Figure 5-45 First-story plan showing steel cantilever column and grade beam retrofit location for the short-side-open building. I-shaped symbols depict new cantilever columns. Diamond symbols and associated lines depict new wood-structural-panel shear walls.

5.5 Retrofit Cost Estimates

Construction costs estimates for the optimized line and FEMA P-807 example retrofits for the three-story, long-side-open building were prepared by design-build contractor Optimum Seismic. The building is assumed to have 12 units with plan dimensions of 36 feet by 100 feet (see Table 5-1). As described in Section 5.3 and Section 5.4, the retrofits consist of special cantilever columns set in grade beams in the direction parallel to the open front and, for the FEMA P-807 retrofit, wood-structural-panel shear walls in the direction perpendicular to the open front. The following qualifications are noted:

- The cost of retrofit construction can vary dramatically based on many factors, including location, ease of access, size of building, materials, and dates of construction. As a result, the noted costs should be considered illustrative only.
- Where retrofits require work in occupied areas, there are additional costs that can be significantly greater than the cost of physical construction. These additional costs can include tenant relocation, lost tenant revenue, and an extended construction schedule.

- Besides the cost of construction of the retrofit itself, additional costs can be incurred related to disruption or relocation of utility lines (e.g., water, gas, electricity, sewer) to allow for installation of the retrofit work.
- Other potential additional costs relate to shoring of the existing structure, where required for installation of the retrofit work.
- Other additional costs that should be planned for include retrofit design, inspection during construction, and fees for building permits.

The following are incorporated into the cost estimates:

- The costs were developed for a downtown Los Angeles location with construction by a non-union contractor, starting in January of 2023. The size and configuration of the building is as presented in this chapter.
- The cost estimates apply where retrofit work does not occur in occupied areas of the building.
- Potential additional costs related to disruption or relocation of utility lines to allow for installation of the retrofit work are not included.
- Potential additional costs related to shoring of the existing structure are not included.
- Costs related to design of the retrofit, inspection during construction, and permitting fees are not included.

The resulting estimated costs of retrofit construction are:

- Section 5.3 Optimized Line Retrofit Estimated Cost: $65,000
- Section 5.4 FEMA P-807 Retrofit Estimated Cost: $135,000 ($100,000 steel frame, $35,000 shear walls perpendicular).

Chapter 6: Conclusions

SWOF buildings can be found across the United States, most notably along the West Coast. There are lots of them and the risks associated with the variations in configuration and construction materials are becoming more well understood. Regardless of these variations, the structural vulnerabilities of SWOF buildings make them prone to collapse during earthquakes. As a result, municipalities in California increasingly have enacted seismic retrofit ordinances for these types of buildings. The ordinances reflect regional differences in their approaches. The purpose of this report is to advance the understanding of the behavior of SWOF buildings and to encourage improved practice in the design of retrofits. The report also is intended to be used by jurisdictions and their consultants to inform decisions regarding ordinance scope and retrofit methods.

The analytical studies presented in Chapter 2 and the subsequent key findings given in Section 3.2 provide the basis for the recommendations for seismic retrofit ordinances given in Section 3.3.

FEMA P-807 was shown to generate full-story seismic retrofit designs that provide significant benefits in terms of reducing probabilities of collapse for all types of SWOF buildings. A few suggestions for future FEMA P-807 enhancements are given. Both line and optimized line retrofits were shown to provide mixed benefits in terms of reducing probabilities of collapse. For some archetypes, the reductions were moderate, whereas for other archetypes the reductions were negligible.

Practical recommendations for engineering retrofit designs of SWOF buildings are given in Chapter 4. Two design examples—one using an optimized line retrofit and the other using FEMA P-807—are presented in Chapter 5. These design examples include conceptual construction details and illustrate implementation of the recommendations from Chapter 4.

No change to the FEMA P-807 methodology is deemed necessary. Where evaluation of a building is desired before a retrofit is designed, the FEMA P-807 methodology and accompanying Weak-Story Tool are believed to be the best available tools.

Appendix A: Building Inventory

A.1 Overview

The purpose of the building inventory effort was to gain a better understanding of the SWOF building stock in California in order to inform the characteristics of archetype buildings used for the Chapter 2 analytical studies. A list of characteristics of interest was established and used as the basis for data collection. Data collection took somewhat different approaches in Southern California versus Northern California, based on differences in available data. In Southern California, a significant body of information had already been collected from jurisdictions by Degenkolb Engineers as part of the firm's work assisting in the development of retrofit ordinances. Also, in Southern California, information was collected from design-build contractor Optimum Seismic. In Northern California, the cities of Berkeley and Oakland provided data for use by the project team. This appendix discusses the data that were sought, the Southern California jurisdiction data, the Southern California contractor data, and the Northern California jurisdiction data. The appendix finishes with conclusions drawn from these data, including the influence on the selection of archetype buildings for the analytical studies.

A.2 Building Data Sought

The following list of building characteristics of interest was used as a starting point in requests for data:

- Number of stories
- Number of units
- Area (building footprint)
- Date of construction/applicable code edition
- Plan configuration
- Open front configuration
- Wall and floor finish materials
- Diaphragm type
- Vulnerabilities other than one open front
- Configuration of interior walls

The building-type classifications being used in Southern California were helpful in understating common configurations and their propensity in the building population. Where possible, buildings were categorized by "building type" based on the West Hollywood Seismic Retrofit Program Screening Report configurations (Figure A-1). West Hollywood Building Type A corresponds to the long-side-open (LO) archetype used in project analytical studies, while Type B corresponds to the short-side-open (SO) archetype. An additional category, "CS", was added to capture buildings in which a ground-floor crawlspace was identified to be a soft or weak story in the City of Oakland retrofit ordinance.

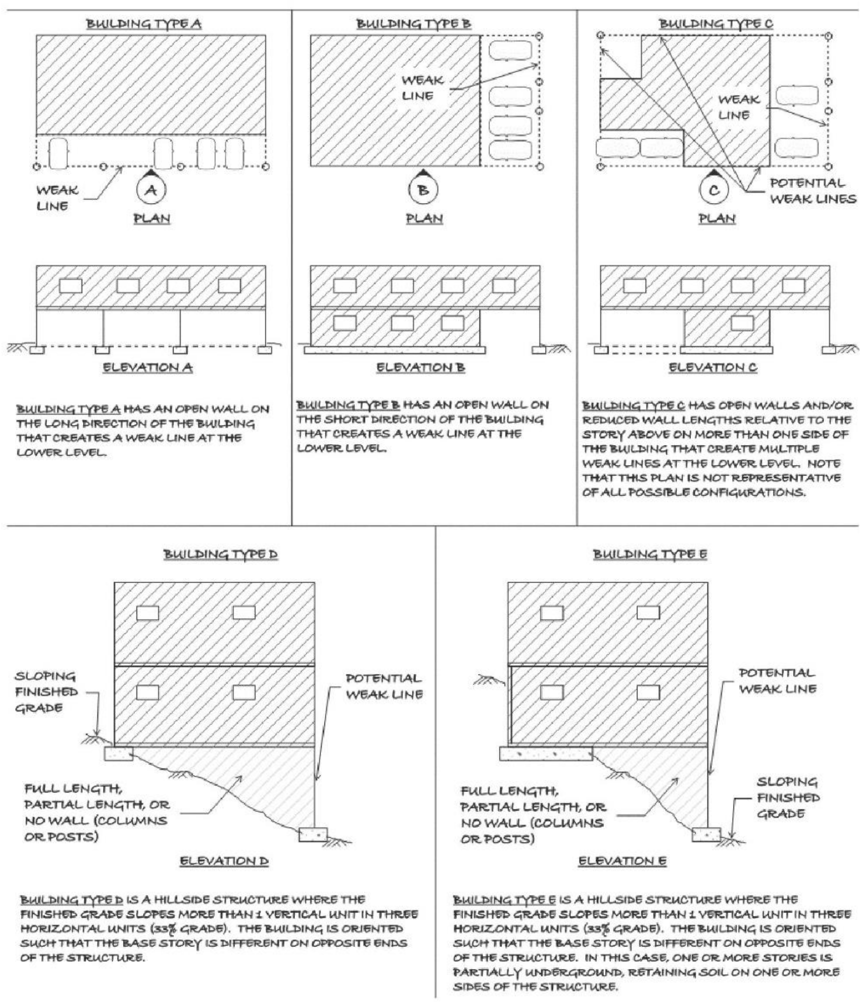


Figure A-1 SWOF building types as assigned by the City of West Hollywood screening form (image credit: CWH, 2019).

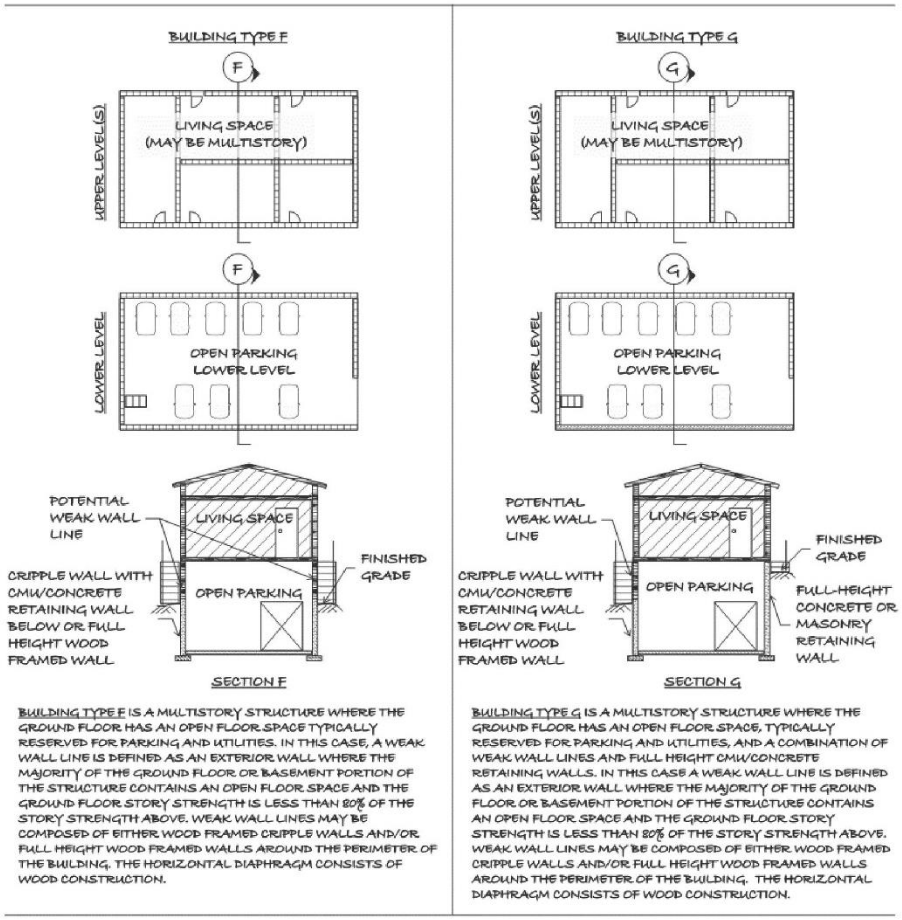


Figure A-1 SWOF building types as assigned by the City of West Hollywood screening form (continued) (image credit: CWH, 2019).

A.3 Southern California Jurisdictions

A.3.1 Data Collected

The cities of Santa Monica, Beverly Hills, and West Hollywood in the Los Angeles metropolitan area have enacted mandatory soft-story retrofit programs. These programs targeted multi-family residential buildings that potentially have a soft-or-weak first story. Buildings were identified through a visual survey from the public right-of-way to ascertain whether they had a potential soft-or-weak story, and those that had visible features indicating a potential weakness were included in the inventory of subject properties.

A.3.2 Data Summary

The inventory data of buildings with a potential soft-or-weak first story were aggregated and summarized in the following figures, which quantify distributions for the year of construction, number of units, number of stories, and building type. Building-type notation in Figure A-4 was taken from

City of West Hollywood SWOF Screening Report issued March 15, 2019. Most building types are representative of the short-side-open or long-side-open cases, or some combination of those (Type C).

The large majority of soft-or-weak story buildings were found to be built between 1950 and 1970, have two or three stories, and have 10 or fewer units (Figure A-2 and Figure A-3).

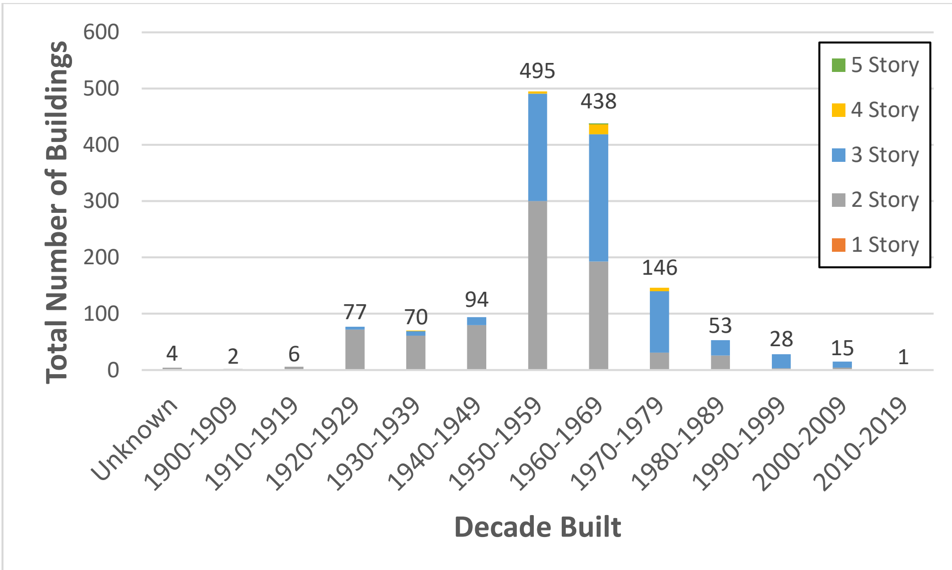


Figure A-2 **Decade built vs. number of buildings and number of stories in the Southern California building inventory study.**

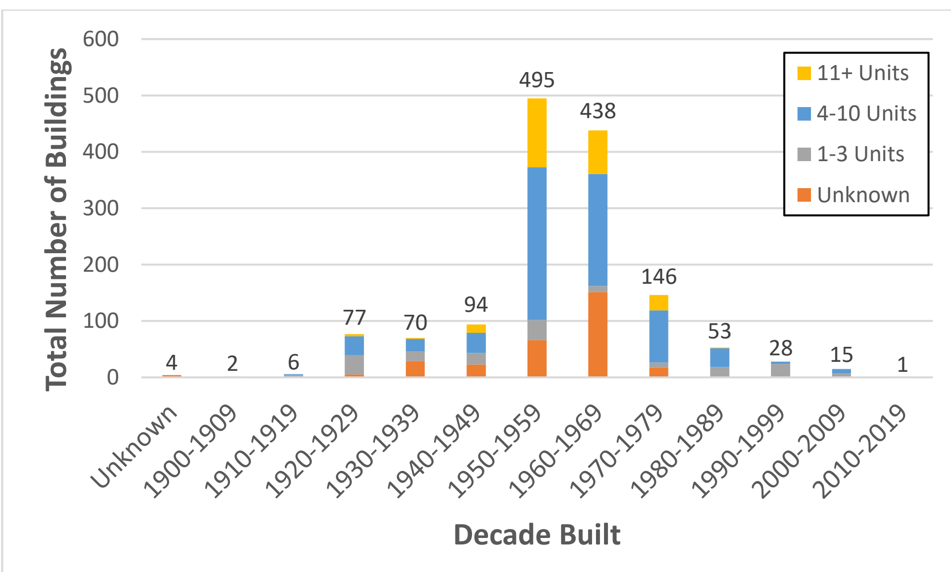


Figure A-3 **Decade built vs. number of buildings and number of units in the Southern California building inventory study.**

The great majority of SWOF buildings in the Southern California inventory survey were found to be Type A, consistent with the long-side-open (LO) archetype (Figure A-4).

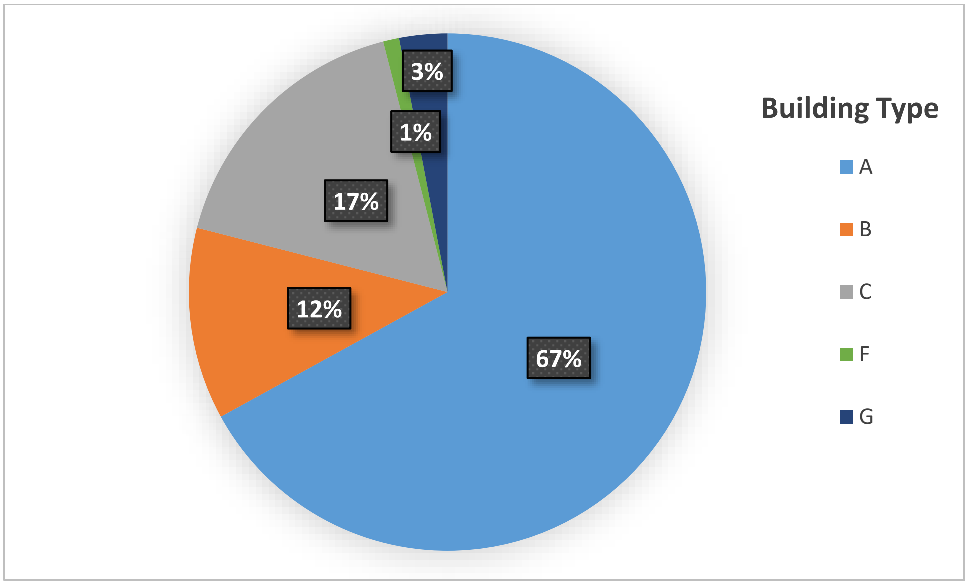


Figure A-4 Building types in the Southern California inventory survey based on Figure A-1 categories. Note that Type D and Type E buildings are omitted from the figure due to having zero units.

A.4 Southern California Design-Build Contractor

Southern California design-build contractor Optimum Seismic shared data from more than 900 soft-story retrofits in which they were involved, approximately 85% of which were located in the greater Los Angeles area. Optimum Seismic also participated in an interview with the authors of this appendix. The following information was shared.

For the approximately 900 buildings, Optimum Seismic provided an estimate of the decade of construction, as shown in Figure A-5. The great majority of the buildings were estimated to have been constructed in the 1950s and 1960s.

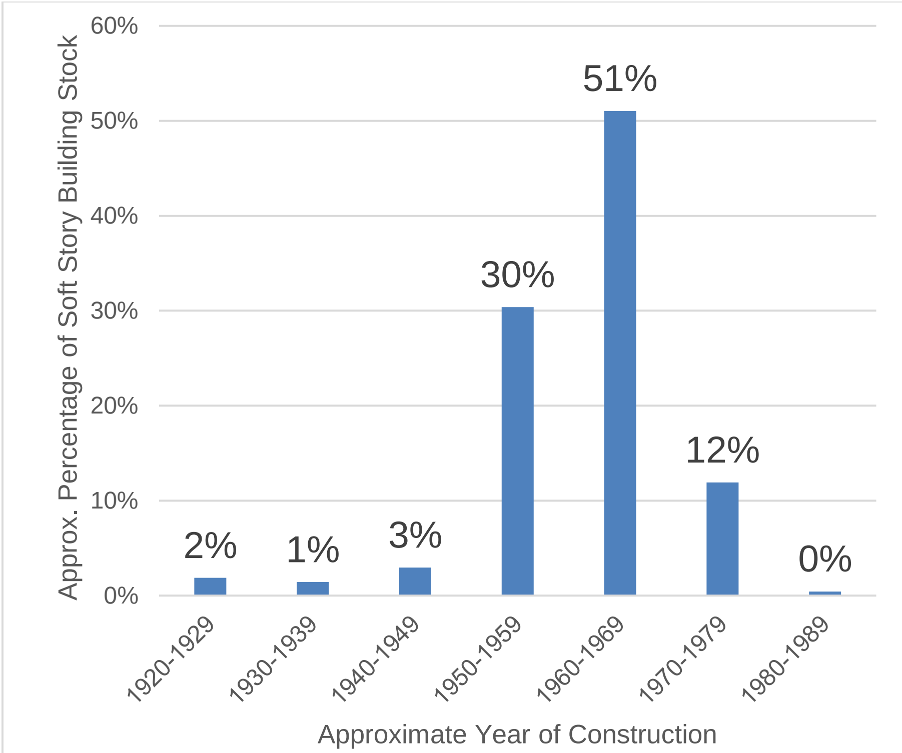


Figure A-5 Estimated decade of construction for buildings retrofit by Optimum Seismic.

The number of stories was identified to be two or three for the great majority of the buildings (Figure A-6).

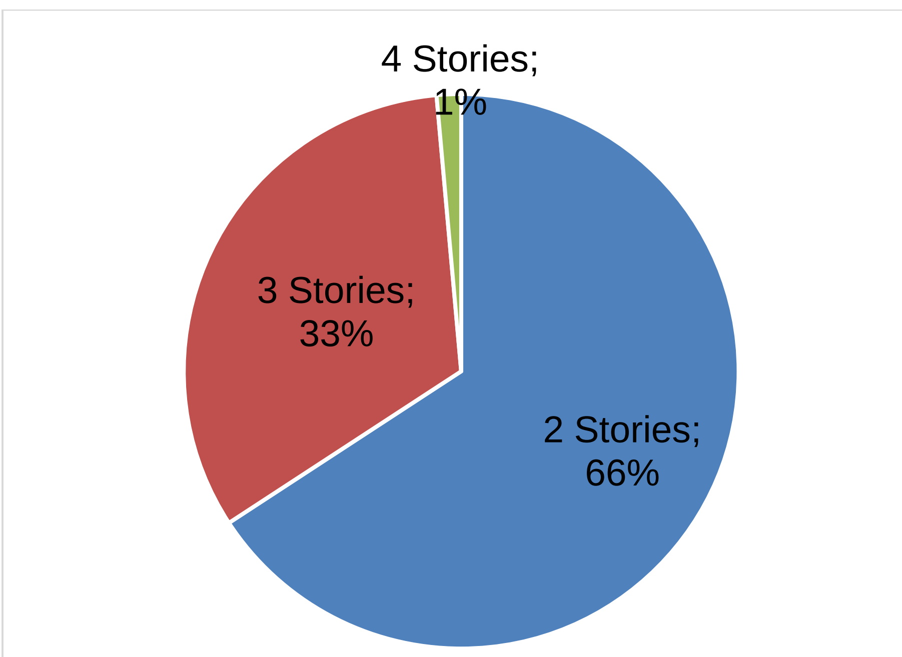


Figure A-6 Number of stories for soft-story buildings retrofit by Optimum Seismic.

Building Type B, with tuck-under parking on a short side of the building, was identified as the most prevalent. This corresponds to the short-side-open archetype used for project analytical studies. Building Type A, corresponding to the long-side-open archetype, was the next most prevalent.

The construction used for the second-floor diaphragm (floor diaphragm immediately above the soft or weak story) was discussed. This was of interest in order to determine when a transition might have occurred from lumber-sheathed floor diaphragms to plywood-sheathed floor diaphragms. Optimum Seismic indicated that they believed lumber sheathed diaphragms to have been prevalent up through the 1960s, and plywood prevalent starting in the 1970s. They also reported being able to see carpet floor finishes rather than hardwood in the gaps between the lumber of second-floor sheathing boards.

Optimum Seismic also shared that they believe use of gypsum wallboard wall and ceiling finishes became prevalent in place of plaster starting in the 1960s. Optimum Seismic noted that they are not systematically going into the occupied upper floors, so their opportunities to observe wall and ceiling finishes have been limited.

A.5 Northern California Jurisdictions

A.5.1 City of Berkeley

Berkeley's mandatory soft-story retrofit program was put into effect in 2014. The program criteria include multi-family residential buildings with five or more residential units, constructed before 1978, and identified as potentially having a soft-or-weak first story. It is understood that Berkeley's list of potential soft-or-weak story buildings was created through a visual "windshield survey" of multi-unit residential properties. Properties were observed from the street in order to ascertain whether they had a potential soft-or-weak story. Those that had visible features indicating a potential SWOF vulnerability were included in the city's inventory of subject properties.

The City of Berkeley has data on the 357 buildings in its mandatory soft-story retrofit program. The data include building address, number of stories, number of residential units, and year built. The building type, as per Figure A-1, was not included in the data provided by the City of Berkeley. Because these categorizations were found useful, the authors of this report used Google Earth and Google Street View to assign a building type to each building. The ability to assign building types based on these tools was limited. Open-front line locations could generally be identified, but it was difficult to determine the extent to which the first floor included occupied units. As a result, the amount of first-floor area with occupied units is not known. However, the portion of the first-floor area that is occupied is believed to be notably lower in Northern California SWOF buildings than in their Southern California counterparts.

In addition to tabulated data, Berkeley also provided approximately 40 "building cards"—paper forms used to record and track information about properties, such as assessments and addition or alteration permits. Each card includes a record of the finish materials present in each room of the building, providing information on how typical wall and floor finishes change by year of construction. An example building card is shown in Figure A-7.

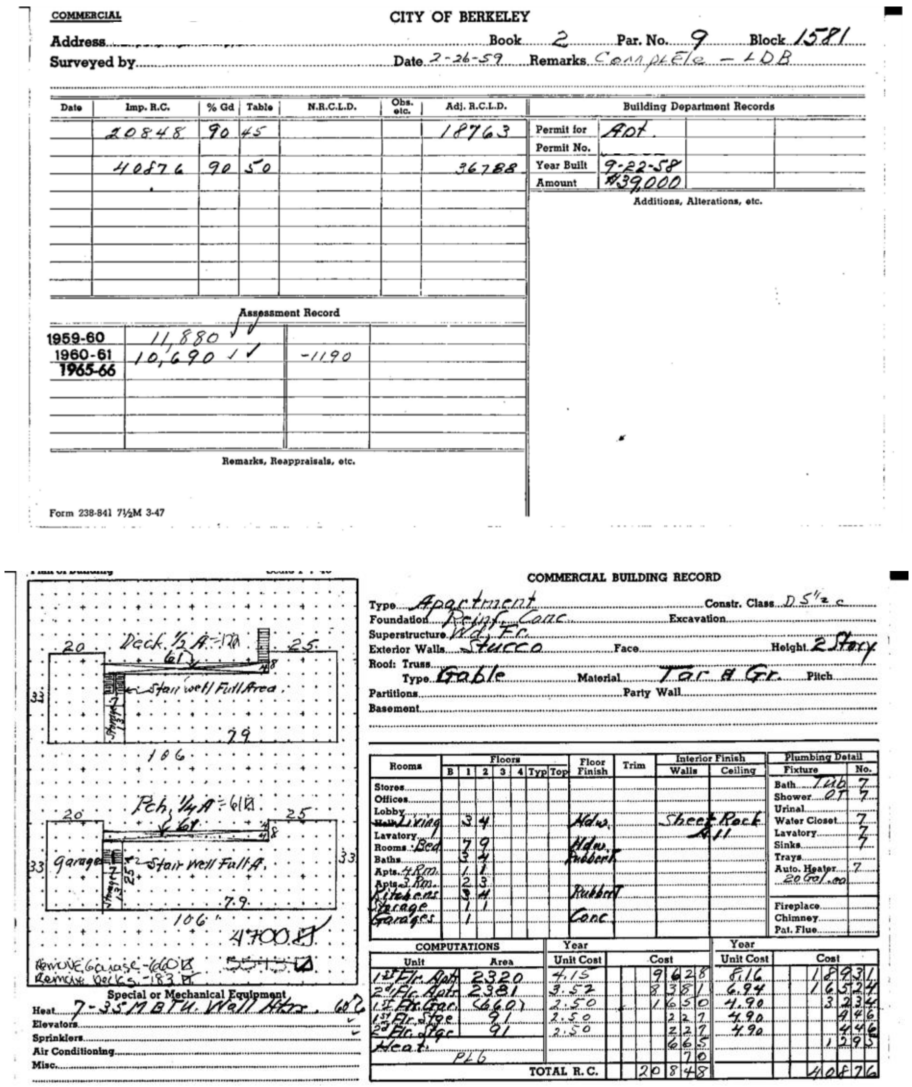


Figure A-7 Example Berkeley building card.

A.5.2 City of Oakland

Oakland's mandatory soft-story retrofit program was put into effect in 2019. The buildings requiring evaluation and retrofit included multi-family residential buildings with two or more stories, five or more residential units, constructed before 1991, and having a soft-or-weak story. Rather than identifying soft-or-weak story buildings through a visual survey as was done in Berkeley, Oakland included all wood-framed residential buildings with five or more residential units in their program notifications. Owners that believed their buildings did not meet the criteria for soft-or-weak story could have a screening performed by a licensed design professional. If the screening concluded that the building was not soft story, a petition providing justification was prepared by the licensed design professional and the property removed from the soft-story program. At this time, it is believed that most or all of the buildings that do not meet the city criteria for soft story have been removed from the list through this screening report process.

The description of soft-story buildings used in City of Oakland included both the typical occurrence of the soft story at the first story as well as cripple-wall stories designated as soft stories.

The City of Oakland had available data on the 230 properties that had already applied for retrofits under their program as of June 2021. The information provided by the spreadsheet included the building address, number of stories, number of residential units, year built, and partial data on the building type, as per Figure A-1. For buildings with no building type data provided, the authors of this report used Google Earth and Google Street View to assign a building type. As with City of Berkeley data, this information is limited in that building-type assignments were made based on open-front lines visible from street views. Again, the amount of first-floor area with occupied units is not known but is believed to be notably lower than Southern California counterparts.

A.5.3 Data Summary

The Northern California inventory data, as collected from Berkeley and Oakland, are summarized in the following figures describing the number of stories, number of units, year of construction, and building types.

The large majority of soft-or-weak story buildings were found to have two or three stories (Figure A-8).

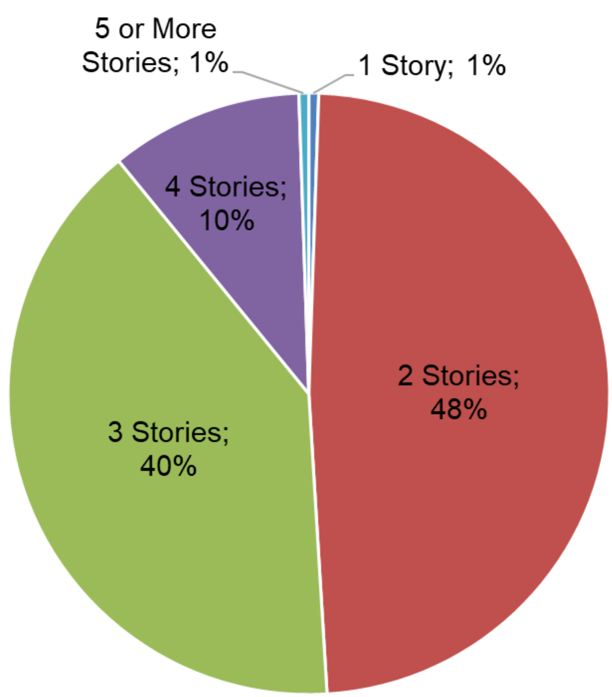


Figure A-8 Number of stories in the Northern California building inventory studied.

The number of units in soft-or-weak story buildings from the Berkeley and Oakland data is shown in Figure A-9. The most common range is 6 to 10 units.

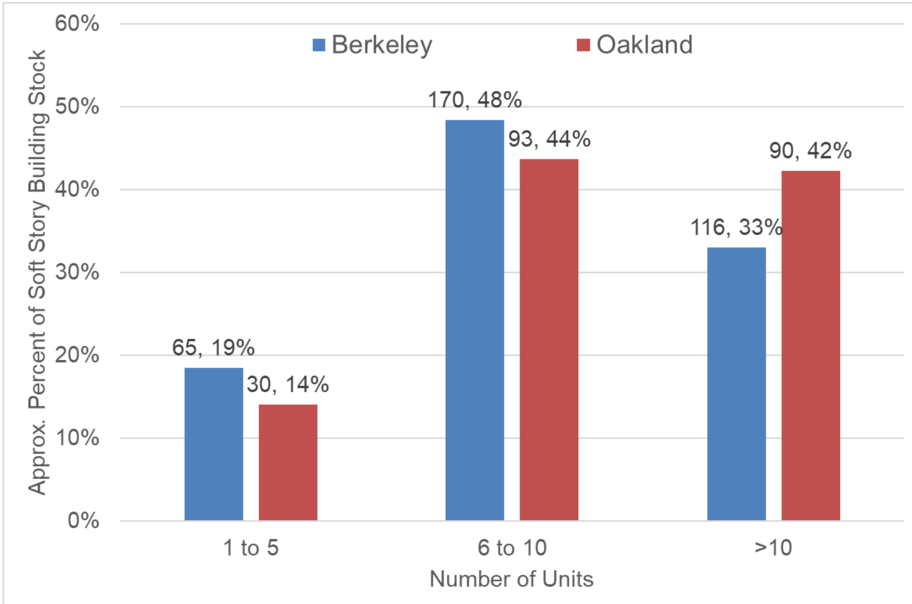


Figure A-9 Number of units in the Northern California building inventory studied.

Northern California shows large quantities of soft or weak story buildings built in the 1920s and between approximately 1950 and 1970 (Figure A-10).

Most buildings included in the Northern California inventory are either Type A or Type B using Figure A-1 categories (Figure A-11). A smaller percentage of the inventory is categorized as Type C, which is typically a combination of Type A and Type B. As previously noted, building-type categorization is based on buildings having open-front lines visible from the street; the amount of occupied first-floor area is not known and believed to be notably lower for Northern California buildings than for their Southern California counterparts.

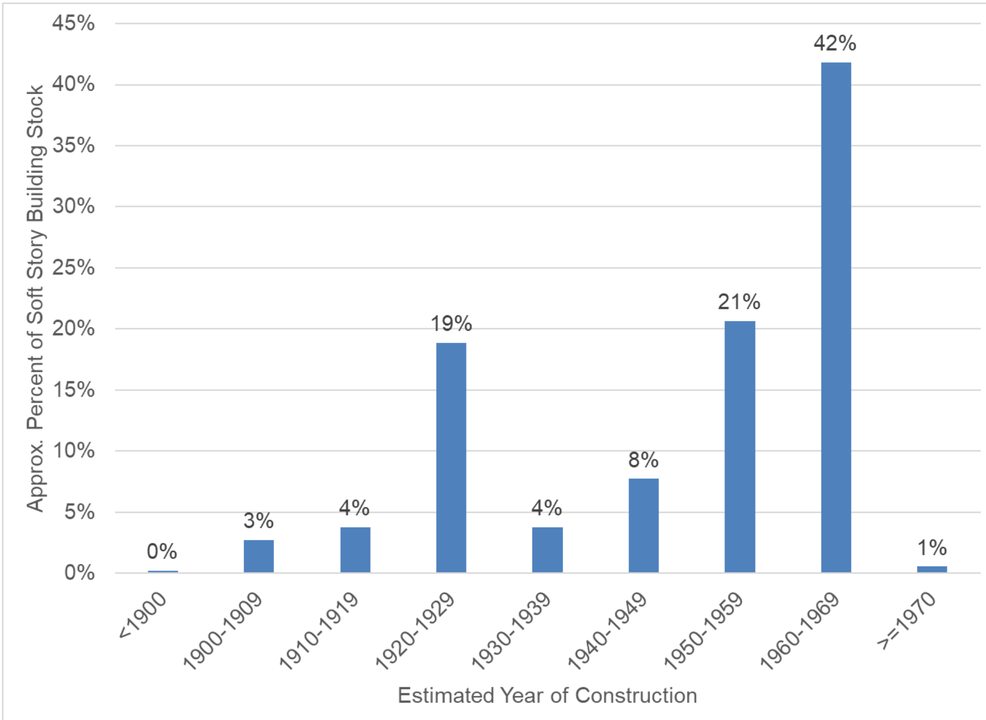


Figure A-10 Year of construction in the Northern California building inventory studied.

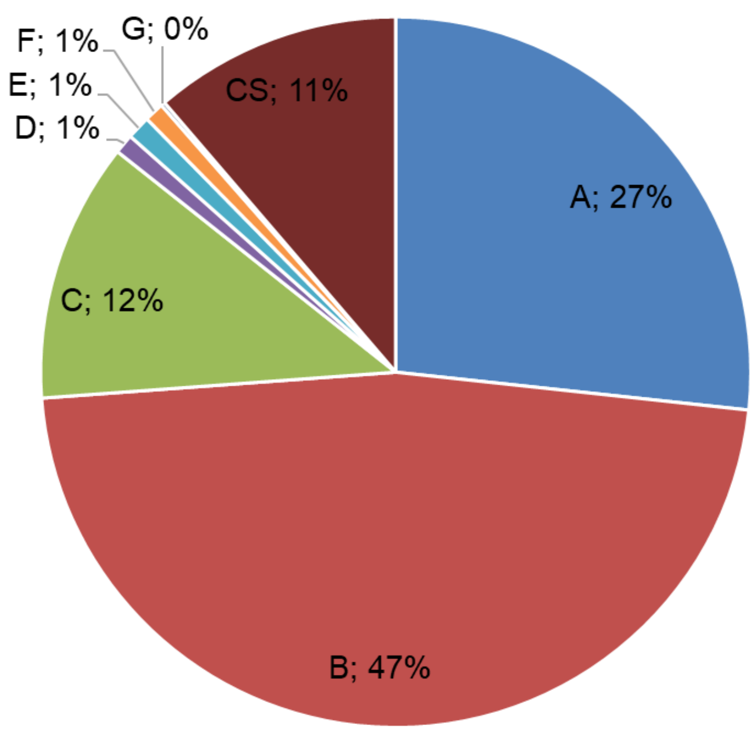


Figure A-11 Building type in the Northern California building inventory studied.

The Northern California inventory data were grouped by year of construction and building type to determine if there was correlation between the two factors. Table A-1 shows a breakdown of the data. For each range of construction dates, the proportion of Type A and Type B buildings is similar.

Table A-1 Quantity of Type A and Type B by Year of Construction

Date of Construction	Type A Buildings	Type B Buildings
1900-1909	5%	2%
1910-1919	3%	2%
1920-1929	14%	20%
1930-1939	2%	5%
1940-1949	8%	8%
1950-1959	21%	23%
1960-1969	47%	41%
Total	100%	100%

Data from the "building cards" supplied by the City of Berkeley was plotted against year of construction to observe trends in typical finishes for different eras of construction. In general, hardwood floors were most common until the late 1950s and early 1960s, at which point carpet floors became the norm (Figure A-12). The switch from plaster to gypsum wallboard wall and ceiling finishes appears to occur a few years earlier in the mid-to-late 1950s (Figure A-13).

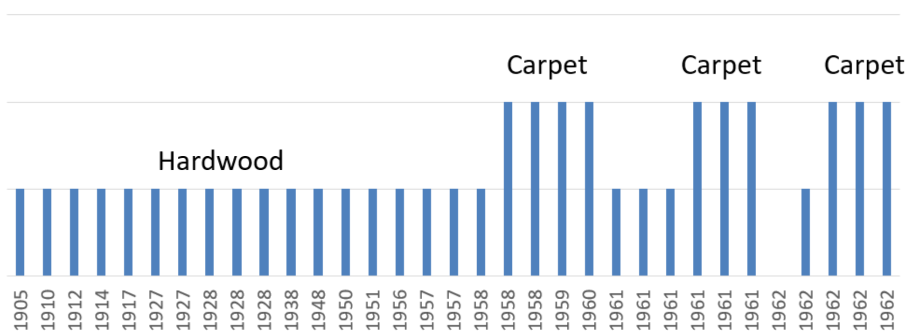


Figure A-12 Floor finishes for Berkeley buildings ordered by date of construction. Tall bars indicate carpet. Short bars indicate hardwood.

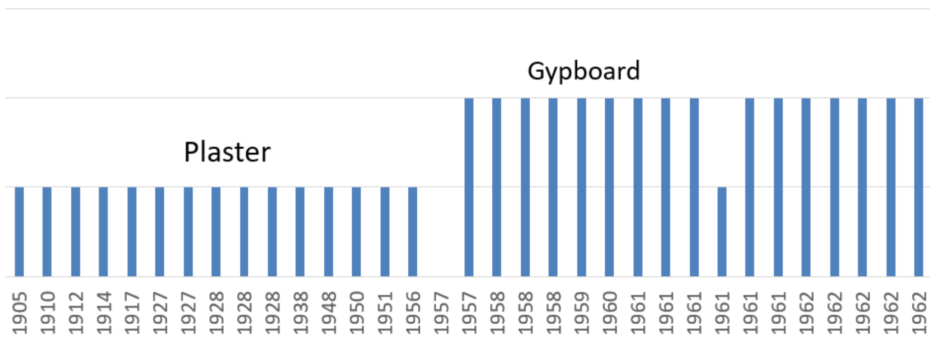


Figure A-13 Wall and ceiling finishes for Berkeley buildings ordered by date of construction. Tall bars indicate gypsum wallboard. Short bars indicate plaster.

A.6 Modeling Decisions Based on Inventory Data

The following discusses the collected data from Southern California inventories, Northern California inventories, and Optimum Seismic. Based on these data, building characteristics recommended for use in the Chapter 2 analytical modeling are identified.

A.6.1 Date of Construction

Across all three inventory datasets, a large portion of the wood soft-or-weak story buildings were identified to be constructed in the1950s and 1960s. Optimum Seismic reported that more than 80% of the buildings that the company has retrofitted were built in the 1950s and 1960s. In the combined Northern California data, 63% of the buildings were identified to be built in the 1950s and 1960s. In the combined Southern California dataset, 65% of the buildings were identified to have been built in the 1950s or 1960s. Based on these data, construction representative of this era was identified to be of highest priority for inclusion in the project analytical studies. Construction from the 1920s to 1940s was identified to be the next highest priority. These construction eras were used in part to determine the prevalent materials of construction. This is discussed further in Section A.6.5.

A.6.2 Building Type (Configuration)

Across all three inventory datasets, a large portion of the wood soft-or-weak story buildings were identified to be of building Type A and Type B (corresponding to LO and SO archetypes). In the combined Northern California dataset, Type A and Type B make up 74% of the buildings, with categorization based on street view of open-front lines. Optimum Seismic reported a prevalence of Type B buildings. Based on this information, it was decided that the project analytical studies would address Type A and Type B.

A.6.3 Number of Stories

Across all three inventory datasets, a large portion of the wood soft-or-weak story buildings were identified to be two or three stories. In the Northern California dataset, 88% of the buildings were two or three stories. In the Southern California dataset, more than 90% of the buildings were two or three stories. In the Optimum Seismic dataset, 99% of the buildings were two or three stories. Based on this information, it was decided that the project analytical studies would address two- and three-story buildings.

A.6.4 Building Plan Dimensions

While there was considerable variability, a large portion of the weak-or-soft story buildings had lengths estimated to be on the order of 100 feet to 120 feet. The widths were more variable, ranging from 20 feet to more than 120 feet. A building width on the order of 36 feet was judged to be reasonable for use as an archetype for analytical modeling. The building configuration used in the FEMA P-2006 soft-story design example was selected as the basis for the Type A long-side-open archetype, and a similar plan size was selected for the Type B short-side-open archetype.

A.6.5 Materials of Construction

Two combinations of materials of construction were selected for the analytical studies. For exterior wall finishes, stucco is by far the predominant finish observed in apartment buildings in the years of interest. The project analytical studies use stucco for all archetypes. For interior wall and ceiling finishes, data available from Optimum Seismic and the City of Berkeley indicated that those buildings constructed in the 1960s predominantly have gypsum wallboard interior finishes. This finish was selected for the building archetypes representing the 1950s and 1960s and is reflected in the archetype weak wall (WW) designation. Available data indicate that plaster-on-wood lath was predominant in the 1920s through 1950s, and it is reflected in the archetype strong wall (SW) designation.

The experience of the project team is that in Northern California lumber sheathing transitioned to plywood sheathing around 1960, at the same time that the City of Berkeley data show the change from hardwood floor to carpet finishes. The Optimum Seismic experience in Southern California is that lumber sheathing continued through the 1960s in combination with carpet. As a result, a straight-lumber-sheathed diaphragm without hardwood is reflected in the archetype weak diaphragm (WD) designation. In addition, a diagonal-lumber-sheathed diaphragm without hardwood is reflected in the archetype strong diaphragm (SD) designation. See Appendix D for further discussion of diaphragm modeling properties.

Information from these material descriptions above was combined to guide the selection of grouping of properties and determination of primary study archetypes. Figure A-14 provides an overview of this compilation. Based on this information, primary combinations of properties for analytical study archetypes were selected to be weak walls in combination with strong diaphragms (most representative of construction from the 1950s and 1960s) and strong walls in combination with weak diaphragms (most representative of construction from the 1920s through 1940s).

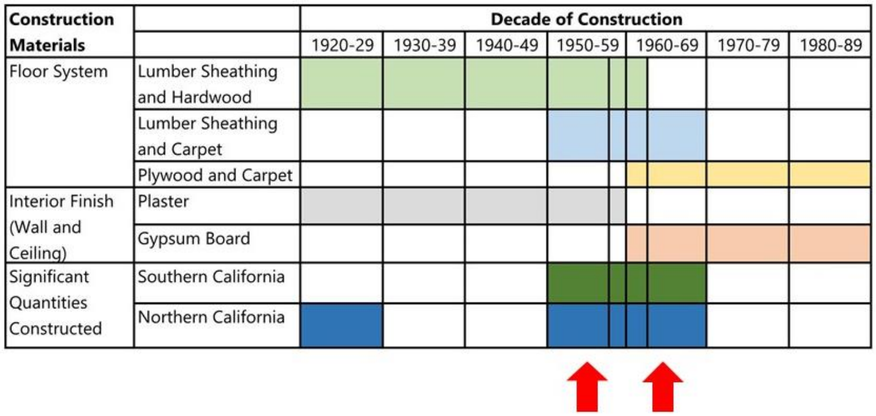


Figure A-14 Compilation of material description information, combined with information on time periods in which a significant portion of the building construction occurred. The red arrows show time periods determined to be of primary interest for development of analytical study archetypes.

A.6.6 Interior Wall Density

Because the FEMA P-2006 building was selected as the basis for the Type A archetype, the interior wall layout for the building was incorporated into the analytical studies. In order to support the use of this wall layout, a study was conducted concerning the linear feet of interior wall per square foot of floor area for a limited number of available soft-or-weak story building plans (See Section 5.2 of this guideline for further details). Ultimately, the study found that the FEMA P-2006 wall layout was representative and appropriate to use.

Appendix B: Building Code Evolution for SWOF Buildings

B.1 Overview

This appendix reviews the evolution of building code provisions related to SWOF buildings in areas of high seismicity. The focus is on the provisions of the Uniform Building Code (UBC) (ICBO, 1997) because it was the governing code in the western United States prior to the introduction of the International Building Code.

Table B-1 outlines changes in code provisions related to seismic engineering, including the introduction of provisions for building irregularities.

Table B-2 outlines changes in code provisions related to wood diaphragms, limitations on diagonal and special diagonally sheathed diaphragms, and prohibition of straight sheathing for cantilever diaphragms (Steinbrugge et al., 1994).

Table B-3 outlines changes in code provisions related to shear wall capacities (Steinbrugge et al., 1994).

Table B-4 outlines changes in code provisions related to *R* factors used for plywood and stucco walls.

Table B-5 outlines changes in code provisions related to the *K* factor, with a graphical figure of those changes.

Table B-6 outlines changes in the maximum base shear coefficient over time, with a graphical display of those changes.

B.2 Summary of Code Evolution

B.2.1 Seismic Engineering Provisions

Prior to the 1976 UBC, there existed no code provisions for structures having irregular shapes or framing systems. Starting in 1976, vague provisions requiring irregular structures to be analyzed considering the dynamic characteristics of the structure were added to the code; however, it was not until the 1988 UBC that provisions for structural irregularities were codified in detail. In 1988, in addition to providing clear, detailed descriptions for classifying building irregularities, structures having certain stiffness, weight, or geometric irregularities were required to undergo dynamic analysis, and limitations were placed on structures with certain vertical discontinuities and irregularities.

Table B-1 Building Code Evolution: Structural Provisions Related to SWOF Buildings

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1976) Section 2312	***3. Structures having irregular shapes or framing systems.*** *The distribution of lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.*	Provisions were added for structures having plan irregularities or vertical irregularities (i.e., soft stories).
Uniform Building Code (1979) Section 2312	***3. Structures having irregular shapes or framing systems.*** *The distribution of lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.*	No change.
Uniform Building Code (1982) Section 2312	***3. Structures having irregular shapes or framing systems.*** *The distribution of lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.*	No change.
Uniform Building Code (1985) Section 2312	***3. Structures having irregular shapes or framing systems.*** *The distribution of lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.*	No change.

Table B-1 Building Code Evolution: Structural Provisions Related to SWOF Buildings (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1988) Section 2312	*5. Configuration Requirements. A. General. Each structure shall be designated as being structurally regular or irregular.* *B. Regular Structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral force-resisting system such as the irregular features described below.* *C. Irregular structures.* *(i) Irregular structures have significant physical discontinuities in configuration or in their lateral force-resisting systems. Irregular features include but are not limited to, those described in Tables Nos. 23-M and 23-N. Structures in Seismic Zone No. 1 and in Occupancy Category IV in Seismic Zone No. 2 need be evaluated only for vertical irregularities of Type E (Table No. 23-M) and horizontal irregularities of Type A (Table No. 23-N).* *(ii) Structures having one or more of the features listed in Table No. 23-M shall be designated as of having a vertical irregularity.* *EXCEPTION: Where no story drift ratio under design lateral loads is greater than 1.3 times the story drift ratio of the story above the structure may be deemed to not have the structural irregularities of Types A or B in Table No. 23-M. The drift ratio relationship for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.* *(iii) Structures having one or more of the features listed in Table No. 23-N shall be designated as having a plan irregularity.*	Regular and Irregular structures are defined, and their classification is required in design of buildings.

Table B-1 Building Code Evolution: Structural Provisions Related to SWOF Buildings (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1988) Section 2312	*C. Dynamic. The dynamic lateral force procedure of Section 2312 (f) shall be used for all other structures, including the following: (i) Structures 240 feet or more in height except as permitted by Section 2312 (d) 8, Item B (i). (ii) Structures having a stiffness, weight, or geometric vertical irregularity of Type A, B, or C as defined in Table No. 23-M or structures having irregular features not described in Table No 23-M or 23-N except as permitted by Section 2312 (e) 3B. (iii) Structures over five stories or 65 feet in height in Seismic Zones Nos. 3 and 4 not having the same structural system throughout their height except as permitted by Section 2312 (e) 3 B.*	Structures having some stiffness, weight, or geometric vertical irregularities are required to undergo dynamic analysis.
	9. System Limitations. A. Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type E as defined in Table No 23-M, shall not be over two stories or 30 feet in height where the weak story has a calculated strength of less than 65 percent of the story above. Exception: Where the weak story is capable of resisting a total lateral seismic force of 3 (R_w/8) times the design force prescribed in section 2312 (e)	Limitations are placed on structures with certain vertical discontinuities or irregularities.
	C. Irregular Features. All structures having irregular features described in Table No. 23-M or 23-N shall be designed to meet the additional requirements of those sections referenced in the tables.	Additional requirements are instated for structures having irregular features.

Table B-1 Building Code Evolution: Structural Provisions Related to SWOF Buildings (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1991) Section 2330	*B. Structures having one or more of the features listed in Table No. 23-M shall be designated as if having a vertical irregularity.* *EXCEPTION: Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above the structure may be deemed to not have the structural irregularity of Type A or B in Table No. 23-M. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.*	Exception for story drift ratio added for designation as having vertical irregularity.
Uniform Building Code (1994) Chapter 16	(not shown)	Minor changes.
Uniform Building Code (1997) Chapter 16	(not shown)	Chapter 16 is revised in its entirety and looks very similar to today's code.

B.2.2 Wood Diaphragm Provisions

The evolution of the UBC wood diaphragm provisions is important for understanding soft-story construction, since many soft-story buildings consisting of tuck-under parking rely on wood diaphragms. Provisions for wood diaphragms were first introduced in the 1937 UBC and slowly developed until the 1952 UBC, when allowable shear values were specified for plywood diaphragm sheathing for wind and seismic loads dependent on the nailing and blocking provided. In the 1955 UBC, provisions for wood diaphragms were greatly expanded: diagonally sheathed and special diagonally sheathed diaphragms were defined, allowable shears were limited to 300 plf and 600 plf, respectively, and nailing requirements were defined; maximum diaphragm ratios for cantilever diaphragms were specified for diagonally and special diagonally sheathed diaphragms; plywood diaphragm blocking and nailing provisions were defined, and reductions in load capacity were defined for when blocking was omitted. In the 1967 UBC, allowable shear stresses for blocked and un-blocked plywood diaphragms were tabulated, and allowable shear stresses for plywood shear walls were added to the code. In the 1970 UBC, straight-lumber sheathing was prohibited for resisting shears in cantilever diaphragms. Between 1970 and 1994, UBC wood diaphragm tables and provisions saw revisions to plywood allowable stresses, blocking layouts, and permitted materials (such as the introduction of particleboard sheathing), but remained largely the same.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1937) Section 2528	**Wood Diaphragms** **Sec. 2528.** Wood diaphragms may be used to distribute horizontal forces to resisting elements such as walls or partitions, provided the maximum deflection in the plane of the diaphragm, as determined by tests or analogies drawn therefrom, does not exceed the permissible deflection of such wall or partition. In determining the permissible deflection of walls or partitions, the actual elastic properties of the materials (modulus of elasticity, allowable extreme fiber stresses, etc.) may be determined by tests or other data acceptable to the Building Inspector, or the assigned values for such properties elsewhere herein provided shall be used. In determining the maximum horizontal deflection of a proposed wood diaphragm under assumed design loads, data from actual tests of diaphragms corresponding to the type proposed may be used or an analogy may be drawn from data furnished in an article entitled "Tests Indicate Design Methods for Earthquake-Proof Timber Floors" appearing in the Engineering News-Record for June 20, 1935, or in "The Rigidity and Strength of Frame Walls" published by the U. S. Forest Products Laboratory. Connections and anchorage of wood diaphragms to resisting elements shall be provided along all the margins of the diaphragm. Such connections shall be capable of resisting the design loads or forces elsewhere herein prescribed.	Provisions for wood diaphragms were added to the code.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1940) Section 2528	(d) **Plywood.** The term "Plywood" as used in this code shall mean a built-up board or piece of wood made of three or more plies of veneer joined with glue and so laid that the grain of adjoining plies is at right angles. An odd number of plies shall be used. For the purpose of this code all plywood shall conform to the U. S. Commercial Standard CS 45-38.	The term "plywood" was defined.
Uniform Building Code (1946) Section 2528	(e) **Plywood Stresses.** Working stresses of plywood shall not exceed the values set forth in the bulletin, "Methods of Calculating the Strength of Plywood," issued by the Forest Products Laboratory, April 17, 1942. Plywood of Douglas fir shall conform to U. S. Commercial Standard CS 45-45. Plywood of other species, when used structurally, shall be identified as to veneer grade and glue type by an approved agency and shall meet the performance standards in U. S. Commercial Standard CS 45-45 for its type.	Provisions for plywood allowable stresses were added to the code.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1952)
Section 2503</td>
<td>
1952 EDITION
107
TABLE NO. 25-B—RECOMMENDED WORKING STRESSES FOR PLYWOOD (DOUGLAS FIR)
In bending, tension, and compression (except bearing and 45-degree stresses) consider only those plies with their grain direction parallel to the principal stress
<table>
<tr><th colspan="5">DRY LOCATION</th></tr>
<tr><th>TYPE OF STRESS</th><th>Exterior A-A (So2S)</th><th>Exterior A-B (So/Sld)
Exterior A-C (So1S)</th><th>Exterior B-C
Exterior Sheathing (C-C)
Interior Sheathing (C-D)
Exterior Concrete Form (B-B)
Interior Concrete Form (B-B)</th><th>Interior A-A (So2S)
Interior A-B (So/Sld)
Interior A-D (So1S)
Interior B-D (Sld/1S)
(Apply the following percentages to the stresses for the corresponding Exterior grade.)</th></tr>
<tr><td>EXTREME FIBER in bending
Face grain // to span
Face grain ⊥ to span</td><td>
2188
1875</td><td>
2000
1875</td><td>
1875
1875</td><td>
100%
80%</td></tr>
<tr><td>TENSION
// to face grain (3-ply only*)
⊥ to face grain
± 45° to face grain</td><td>
2188
1875
337</td><td>
2000
1875
320</td><td>
1875
1875
310</td><td>
100%**
80%
85%</td></tr>
<tr><td>COMPRESSION
// to face grain (3-ply only*)
⊥ to face grain
± 45° to face grain</td><td>
1605
1375
496</td><td>
1460
1375
472</td><td>
1375
1375
460</td><td>
100%**
70%
80%</td></tr>
<tr><td>BEARING (on face)</td><td>405</td><td>405</td><td>405</td><td>100%</td></tr>
<tr><td>SHEAR, rolling, in plane of plies:
// or ⊥ to face grain
± 45° to face grain</td><td>
79
105</td><td>
72
96</td><td>
68
90</td><td>
75%
75%</td></tr>
<tr><td>SHEAR, in plane ⊥ to plies:
// or ⊥ to face grain
± 45° to face grain</td><td>
210
420</td><td>
192
384</td><td>
180
360</td><td>
85%
85%</td></tr>
<tr><td>MODULUS OF ELASTICITY in bending
Face grain // to span
Face grain ⊥ to span</td><td>
1,600,000
1,600,000</td><td>
1,600,000
1,600,000</td><td>
1,600,000
1,600,000</td><td>
100%
70%</td></tr>
<tr><td colspan="5">* For tension or compression, // to grain, in 5-ply or thicker, use values for 3-ply, but in next lower grade.
** For 5 or more plies use 90%.</td></tr>
<tr><th colspan="5">DAMP OR WET LOCATION</th></tr>
<tr><td colspan="5">Where moisture content will exceed 16 per cent, decrease by 20 per cent values shown for Dry Location for following properties: Extreme Fiber in Bending, Tension and Compression both parallel and perpendicular to grain and at 45 degrees, and Bearing. (No change in values for shear or modulus of elasticity.)
Only Exterior Type plywood should be used where moisture content will exceed 18 per cent.</td></tr>
</table>
Table No. 25-B

(e) Plywood Stresses. Working stresses of Douglas fir plywood shall not exceed the values set forth in Table No. 25-B. Working stresses of plywood other than Douglas fir shall be determined according to the species.
Plywood of Douglas fir shall conform to U.B.C. Standard No. 25-3. Plywood of other species, when used structurally, shall be identified as to veneer grade and glue type by an approved agency and shall meet the performance standards in U.B.C. Standard No. 25-3 for its type.
</td>
<td>Tables for plywood allowable stresses were added to the code. UBC Standard for plywood performance & specification was added to the code.</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1952)
Section 2524</td><td>

(b) **Plywood Diaphragms.** Wood diaphragms sheathed with plywood may be used to resist horizontal forces not exceeding those set forth in Table No. 25-I. Plywood thickness shall not be less than that set forth in Table No. 31-B for corresponding joist spacing and live loads.

All boundary members shall be proportioned and spliced

1952 EDITION Sections 2524-2526

TABLE NO. 25-I—ALLOWABLE SHEARS* FOR WIND OR SEISMIC LOADINGS ON HORIZONTAL PLYWOOD DIAPHRAGMS

(Pounds per Foot of Width)
For Douglas Fir and Southern Pine Framing
(For other species adjust values accordingly)

<table>
<tr><th rowspan="3">PLYWOOD THICKNESS</th><th rowspan="3">NAIL SIZE</th><th colspan="6">Width of Framing Member</th></tr>
<tr><th colspan="3">Nail Spacing on All Panel Edges 2⅝" or more</th><th colspan="3">Less than 2⅝" but Not Less than 1⅝"</th></tr>
<tr><th>6"</th><th>4"</th><th>3"</th><th>6"</th><th>4"</th><th>3"</th></tr>
<tr><td>5/16"-3/8"</td><td>6d com.</td><td>185</td><td>280</td><td>315</td><td>165</td><td>250</td><td>280</td></tr>
<tr><td>3/8"-1/2"-5/8"</td><td>8d com.</td><td>265</td><td>400</td><td>450</td><td>240</td><td>355</td><td>405</td></tr>
<tr><td>1/2"-5/8"</td><td>10d com.</td><td>320</td><td>480</td><td>545</td><td>285</td><td>425</td><td>485</td></tr>
</table>

*Tabulated shears shall be reduced one-fourth for other than wind or seismic loads.

where necessary to transmit direct stresses. Boundary nail spacing shall not exceed one-half that set forth in Table No. 25-I.

End joints of plywood panels shall be staggered. All panel edges shall be nailed to framing members at least one and five-eighths inches (1⅝") thick. When blocking is omitted, loads shall be determined in accordance with engineering analysis. Panel edges shall bear on the framing members and in general butt along their center lines. Nails shall be placed not less than three-eighths inch (⅜") in from the panel edge, not more than twelve inches (12") apart along intermediate joists, and shall be firmly driven into the framing members.

</td><td>Plywood diaphragms are added to the wood diaphragm provisions, and Allowable shears for wind and seismic loading are defined for plywood diaphragms. When blocking is omitted, loads shall be determined in accordance with engineering analysis.</td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1955)
Section 2405</td>
<td>
All plywood when used structurally, including, among others, use for siding, roof and wall sheathing, subflooring, diaphragms, and built-up beams, shall conform to performance standards for its type in U.B.C. Standard No. 25-2; it shall be identified as to grade and glue type by an approved agency. In addition to the above requirements all plywood when permanently exposed in outdoor applications shall be of exterior type.
Wood Diaphragms
Sec. 2511. (a) General. Wood and plywood diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragm, as determined by calculations, tests, or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements.
Permissible deflection shall be that deflection up to which the diaphragm and any attached distributing or resisting element will maintain its structural integrity under assumed load conditions, i. e., continue to support assumed loads without danger to occupants of the structure.
Connections and anchorages capable of resisting the design forces shall be provided between the diaphragms and the resisting elements. Openings in diaphragms which materially affect their strength shall be fully detailed on the plans, and shall have their edges adequately reinforced to transfer all shearing stresses.
Size and shape of diaphragms shall be limited as set forth in Table No. 25-J.
139
Section 2511 UNIFORM BUILDING CODE
TABLE NO. 25-J—MAXIMUM DIAPHRAGM DIMENSION RATIOS
<table>
<tr><th></th><th>HORIZONTAL DIAPHRAGMS
Maximum Span-Width Ratios</th><th>VERTICAL DIAPHRAGMS
Maximum Height-Width Ratios</th></tr>
<tr><td>1. Diagonal sheathing, conventional</td><td>3:1</td><td>2:1</td></tr>
<tr><td>2. Diagonal sheathing, special</td><td>4:1</td><td>3½:1</td></tr>
<tr><td>3. Plywood, nailed all edges</td><td>4:1</td><td>3½:1</td></tr>
<tr><td>4. Plywood, blocking omitted at intermediate joints</td><td>4:1</td><td>2:1</td></tr>
</table>
Wood Diaphragms (Cont'd.)
In buildings of wood construction where rotation is provided for, transverse shear resisting elements normal to the longitudinal element shall be provided at spacings not exceeding 1½ times the width for conventional diagonally sheathed diaphragms or two times the width for special diagonally sheathed or plywood diaphragms.
In masonry or concrete buildings wood and plywood diaphragms shall not be considered as transmitting lateral forces by rotation.
</td>
<td>
Plywood Interior & exterior types are defined.
Size and shape of diaphragms is limited to maximum ratios.
Diagonally sheathed and Special diagonally sheathed diaphragms are defined & allowable shears are specified at 300 plf and 600 plf, respectively. Nailing requirements are defined.
Diaphragm ratios are specified for rotation for diagonally and special diagonally sheathed diaphragms.
Vertical diaphragms may be sheathed with plywood.
</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1955) Section 2511	All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least one and five-eighths inches (1⅝") wide. In general, panel edges shall bear on the framing members and butt along their center lines. Nails shall be placed not less than three-eighths inch (⅜") in from the panel edge, not more than twelve inches (12") apart along intermediate supports and six inches (6") along panel edge-bearings, and shall be firmly driven into the framing members. No unblocked panels less than twelve inches (12") wide shall be used. When blocking is omitted and the panels are arranged so that load is applied perpendicular to the unblocked edges and to the continuous panel joints, shears shall not exceed two-thirds of the values given for six-inch (6") nail spacing in Table No. 25-K. For other panel arrangements shears shall not exceed one-half of the tabulated values for six-inch (6") nail spacing. (b) **Diagonally Sheathed Diaphragms. 1. Conventional construction.** Such wood diaphragms shall be made up of one-inch (1") nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards shall be directly nailed to each intermediate bearing member with not less than two 8d nails for one-inch by six-inch (1"x 6") boards and three 8d nails for boards eight inches (8") or wider, and in addition three 8d nails and four 8d nails shall be used for six-inch (6") and eight-inch (8") boards, respectively, at the diaphragm boundaries. End joints in adjacent boards shall be separated by at least one joist or stud space, and there shall be at least two boards between joints on the same support. Boundary members at edges of diaphragms shall be designed to resist direct tensile or compressive chord stresses and shall be adequately tied together at corners. Conventional wood diaphragms may be used to resist shears, due to wind or seismic forces, not exceeding 300 pounds per lineal foot of width. **2. Special construction.** Special diagonally sheathed diaphragms shall conform to conventional construction and, in addition, shall have all elements designed in conformance with the provisions of this Code. Each chord or portion thereof may be considered as a beam, loaded with a uniform load per foot equal to 50 per cent of the unit shear due to diaphragm action. The load shall be assumed as acting normal to the chord, in the plane of the diaphragm and either toward or away from the dia- 140	Plywood diaphragm blocking and nailing provisions are defined. Reductions in load capacity are defined for when blocking is omitted. Plywood diaphragm minimum thickness is specified. Requirements for boundary members are defined. Tabulated allowable shear for wind or seismic loading on blocked plywood diaphragms are revised.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1955)
Section 2511</td>
<td>
phragm. The span of the chord, or portion thereof, shall be the distance between structural members of the diaphragm such as the joists, studs, and blocking, which serve to transfer the assumed load to the sheathing.
Special diagonally sheathed diaphragms shall include conventional diaphragms sheathed with two layers of diagonal sheathing at 90 degrees to each other and on the same face of the supporting members.
Special diagonally sheathed diaphragms may be used to resist shears, due to wind or seismic loads, provided such shears do not stress the nails beyond their allowable safe lateral strength and do not exceed 600 pounds per lineal foot of width.

(c) **Plywood Diaphragms.** Horizontal and vertical diaphragms sheathed with plywood may be used to resist horizontal forces not exceeding those set forth in Table No. 25-K, or may be calculated by principles of mechanics without limitation by using values of nail strength and plywood shear values as given elsewhere in this Code. Plywood thickness for horizontal diaphragms shall be not less than that set forth in Tables No. 25-L and No. 25-M for corresponding joist spacing and loads, except that one-fourth inch (¼") plywood may be used where perpendicular loads permit.

TABLE NO. 25-K—ALLOWABLE SHEAR FOR WIND OR SEISMIC LOADINGS ON BLOCKED DOUGLAS FIR PLYWOOD DIAPHRAGMS
(Pounds per Foot)
For Douglas Fir and Southern Pine Framing
(For other species adjust values accordingly)
<table>
<tr><th rowspan="3">MINIMUM PLYWOOD* THICKNESS</th><th rowspan="3">COMMON NAIL SIZE</th><th colspan="6">NAIL SPACING ON ALL PLYWOOD PANEL EDGES†</th></tr>
<tr><th colspan="3">For Framing Member 2⅝ Inches or More in Width</th><th colspan="3">For Framing Member Less than 2⅝ Inches but not Less than 1⅝ Inches in Width</th></tr>
<tr><th>6"</th><th>4"</th><th>3"</th><th>6"</th><th>4"</th><th>3"</th></tr>
<tr><td>5/16"**</td><td>6d</td><td>280</td><td>420</td><td>475</td><td>250</td><td>375</td><td>420</td></tr>
<tr><td>⅜"</td><td>8d</td><td>400</td><td>600</td><td>675</td><td>360</td><td>530</td><td>600</td></tr>
<tr><td>½"</td><td>10d</td><td>480</td><td>720</td><td>820</td><td>425</td><td>640</td><td>730</td></tr>
</table>
†NOTE: When the force acting along either boundary or any line of continuous panel joints exceeds three-fourths of the tabulated value, nail spacing along such boundary or line shall be reduced by one-third.
*For Douglas fir plywood grades having inner plies of species other than Douglas fir use next greater thickness or reduce shears one-fourth.
**These values may be used with ¼" plywood where perpendicular loads permit its use.
</td>
<td>Tabulated allowable shear for wind or seismic loading on blocked plywood diaphragms are revised.</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1955)
Section 2511</td><td>
TABLE NO. 25-K—ALLOWABLE SHEAR FOR WIND OR SEISMIC LOADINGS ON BLOCKED DOUGLAS FIR PLYWOOD DIAPHRAGMS
(Pounds per Foot)
For Douglas Fir and Southern Pine Framing
(For other species adjust values accordingly)
<table>
<tr><th rowspan="3">MINIMUM PLYWOOD* THICKNESS</th><th rowspan="3">COM-MON NAIL SIZE</th><th colspan="6">NAIL SPACING ON ALL PLYWOOD PANEL EDGES†</th></tr>
<tr><th colspan="3">For Framing Member 2⅝ Inches or More in Width</th><th colspan="3">For Framing Member Less than 2⅝ Inches but not Less than 1⅝ Inches in Width</th></tr>
<tr><th>6″</th><th>4″</th><th>3″</th><th>6″</th><th>4″</th><th>3″</th></tr>
<tr><td>5/16″**</td><td>6d</td><td>280</td><td>420</td><td>475</td><td>250</td><td>375</td><td>420</td></tr>
<tr><td>⅜″</td><td>8d</td><td>400</td><td>600</td><td>675</td><td>360</td><td>530</td><td>600</td></tr>
<tr><td>½″</td><td>10d</td><td>480</td><td>720</td><td>820</td><td>425</td><td>640</td><td>730</td></tr>
</table>
†NOTE: When the force acting along either boundary or any line of continuous panel joints exceeds three-fourths of the tabulated value, nail spacing along such boundary or line shall be reduced by one-third.
*For Douglas fir plywood grades having inner plies of species other than Douglas fir use next greater thickness or reduce shears one-fourth.
**These values may be used with ¼″ plywood where perpendicular loads permit its use.
</td><td>Tabulated allowable shear for wind or seismic loading on blocked plywood diaphragms are revised.</td></tr>
<tr><td>Uniform Building Code (1958)
Section 2405</td><td>
TABLE NO. 25-M—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR PLYWOOD DIAPHRAGM
<table>
<tr><th rowspan="2">THICKNESS OF PLYWOOD (In Inches)</th><th rowspan="2">COMMON NAIL SIZE</th><th rowspan="2">NOMINAL WIDTH OF FRAMING MEMBERS (In Inches)</th><th rowspan="2">PLYWOOD SPECIES[1]</th><th colspan="3">BLOCKED DIAPHRAGMS[2] UNIFORM NAIL SPACING ON ALL PANEL EDGES</th><th colspan="2">UNBLOCKED DIAPHRAGM WITH NAILS SPACED 6″ O.C. AT SUPPORTED EDGES</th></tr>
<tr><th>6″</th><th>4″</th><th>3″</th><th>Load Perpendicular to Unblocked Edges and Continuous Panel Joints</th><th>All Other Panel Arrangements</th></tr>
<tr><td rowspan="4">5/16</td><td rowspan="4">6d</td><td rowspan="2">Not less than 2</td><td>Douglas Fir</td><td>188</td><td>281</td><td>315</td><td>167</td><td>125</td></tr>
<tr><td>Western Softwood</td><td>125</td><td>183</td><td>210</td><td>111</td><td>84</td></tr>
<tr><td rowspan="2">3 or More</td><td>Douglas Fir</td><td>210</td><td>315</td><td>356</td><td>187</td><td>140</td></tr>
<tr><td>Western Softwood</td><td>140</td><td>210</td><td>238</td><td>125</td><td>93</td></tr>
<tr><td rowspan="4">⅜</td><td rowspan="4">8d</td><td rowspan="2">Not less than 2</td><td>Douglas Fir</td><td>270</td><td>398</td><td>450</td><td>240</td><td>180</td></tr>
<tr><td>Western Softwood</td><td>180</td><td>265</td><td>300</td><td>160</td><td>120</td></tr>
<tr><td rowspan="2">3 or More</td><td>Douglas Fir</td><td>300</td><td>450</td><td>506</td><td>267</td><td>200</td></tr>
<tr><td>Western Softwood</td><td>200</td><td>300</td><td>338</td><td>178</td><td>133</td></tr>
<tr><td rowspan="4">½</td><td rowspan="4">10d</td><td rowspan="2">Not less than 2</td><td>Douglas Fir</td><td>319</td><td>480</td><td>548</td><td>283</td><td>212</td></tr>
<tr><td>Western Softwood</td><td>213</td><td>320</td><td>365</td><td>189</td><td>141</td></tr>
<tr><td rowspan="2">3 or More</td><td>Douglas Fir</td><td>360</td><td>540</td><td>615</td><td>320</td><td>240</td></tr>
<tr><td>Western Softwood</td><td>240</td><td>360</td><td>410</td><td>213</td><td>160</td></tr>
</table>
[1]Plywood grades shall be interior structural grade or exterior grade or better. For Douglas fir interior type Grades A-A, A-B, A-D and B-D reduce values 25 per cent.
[2]These values may be increased one-third provided the nail spacing is reduced one-third at the diaphragm boundary and at continuous panel joints.
174
</td><td>Minor changes to wood diaphragm provisions.</td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1960)
Section 2511</td><td></td><td>Western Softwood added to plywood materials (previously only Douglas Fir).</td></tr>
<tr><td>Uniform Building Code (1964)
Section 2501</td><td>
158
TABLE NO. 25-B
TABLE NO. 25-B—ALLOWABLE UNIT STRESSES FOR PLYWOOD (DOUGLAS FIR AND WESTERN LARCH)[1]
<table>
<tr><th>TYPE OF STRESS</th><th>EXTERIOR A-A</th><th>EXTERIOR A-B
EXTERIOR A-C</th><th>EXTERIOR CONCRETE FORM B-B; EXTERIOR B-C; EXTERIOR C-C; INTERIOR CONCRETE FORM B-B; INTERIOR SHEATHING C-D; AND C-D PLUGGED INTERIOR SHEATHING C-D (EXTERIOR GLUE)</th><th>ALL OTHER GRADES CONFORMING TO U.B.C. STANDARD NO. 25-9-64[2] APPLY THE FOLLOWING PERCENTAGES TO STRESSES FOR CORRESPONDING EXTERIOR GRADE</th></tr>
<tr><td>EXTREME FIBER in bending:
Face grain // to span
Face grain ⊥ to span</td><td>
2188
1875</td><td>
2000
1875</td><td>
1875
1875</td><td>
100
65</td></tr>
<tr><td>TENSION
// to face grain (3-ply only[3])
⊥ to face grain
± 45° to face grain</td><td>
2188
1875
337</td><td>
2000
1875
320</td><td>
1875
1875
310</td><td>
100[4]
65
75</td></tr>
<tr><td>COMPRESSION
// to face grain (3-ply only[3])
⊥ to face grain
± 45° to face grain</td><td>
1605
1375
496</td><td>
1460
1375
472</td><td>
1375
1375
460</td><td>
100[4]
60
70</td></tr>
<tr><td>BEARING (on face)</td><td>405</td><td>405</td><td>405</td><td>100</td></tr>
<tr><td>SHEAR rolling in plane of plies[5]:
// or ⊥ to face grain
± 45° to face grain</td><td>
79
105</td><td>
72
96</td><td>
68
90</td><td>
75
75</td></tr>
</table>
UNIFORM BUILDING CODE
</td><td>Tables for plywood revised for Western Softwood and Western Larch.</td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1964)
Section 2501</td>
<td>
1964 EDITION
159
TABLE NO. 25-B—Continued
<table>
<tr><th>TYPE OF STRESS</th><th>EXTERIOR A-A</th><th>EXTERIOR A-B
EXTERIOR A-C</th><th>EXTERIOR CONCRETE FORM B-B; EXTERIOR B-C; EXTERIOR C-C; INTERIOR CONCRETE FORM B-B; INTERIOR SHEATHING C-D; AND C-D PLUGGED INTERIOR SHEATHING C-D (EXTERIOR GLUE)</th><th>ALL OTHER GRADES CONFORMING TO U.B.C. STANDARD NO. 25-9-64[2] APPLY THE FOLLOWING PERCENTAGES TO STRESSES FOR CORRESPONDING EXTERIOR GRADE</th></tr>
<tr><td>SHEAR in plane ⊥ to plies:
∥ or ⊥ to face grain
± 45° to face grain</td><td>
260
520</td><td>
240
480</td><td>
225
450</td><td>
80
80</td></tr>
<tr><td>MODULUS OF ELASTICITY in bending:
Face grain ∥ to span
Face grain ⊥ to span</td><td>
1,600,000
1,600,000</td><td>
1,600,000
1,600,000</td><td>
1,600,000
1,600,000</td><td>
100
60</td></tr>
<tr><td colspan="5">WET OR DAMP LOCATION</td></tr>
<tr><td colspan="5">Where moisture content will exceed 16 per cent, decrease by 20 per cent values shown for Dry Location for following properties: Extreme fiber in bending, tension and compression both parallel and perpendicular to grain and at 45 degrees, and bearing. (No change in values for shear or modulus of elasticity.)
Only Exterior Type plywood should be used where moisture content will exceed 18 per cent.</td></tr>
</table>
[1]In bending, tension, and compression (except bearing and 45-degree stresses) consider only those plies with their grain direction parallel to the principal stress.
[2]With Douglas fir or Western larch faces and backs, and inner plies of any Western softwood species listed in Groups I, II, or III of U.B.C. Standard No. 25-9-64.
[3]For tension or compression, ∥ to grain, in five-ply or thicker, use values for three-ply, but in next lower grade.
[4]For five or more plies use 80 per cent.
[5]The working stresses for rolling shear in glued joints shall be reduced by 50 per cent for flange web joints of beams having plywood webs, plywood gusset, plates and framing members located at the edges of stressed-skin plywood panels.
TABLE NO. 25-B (Continued)

TABLE NO. 25-C — UNIFORM BUILDING CODE
TABLE NO. 25-C—ALLOWABLE UNIT STRESSES FOR PLYWOOD, WESTERN SOFTWOOD, GROUPS I AND II SPECIES ONLY[1]
<table>
<tr><th colspan="4">DRY LOCATION</th></tr>
<tr><th>TYPE OF STRESS</th><th>EXTERIOR A1-A1, A-A INTERIOR A1-A1, A-A</th><th>EXTERIOR A1-C, A-B, A-C INTERIOR A1-D, A-B, A-D</th><th>EXTERIOR B-B, B-C, C-C INTERIOR B-B, B-D AND INTERIOR C-D</th></tr>
<tr><td>EXTREME FIBER in bending:
Face grain ∥ to span
Face grain ⊥ to span</td><td>
1600
1350</td><td>
1450
1350</td><td>
1350
1350</td></tr>
<tr><td>TENSION
∥ to face grain (3-ply only[2]):
⊥ to face grain
± 45° to face grain</td><td>
1600
1350
250</td><td>
1450
1350
240</td><td>
1350
1350
230</td></tr>
<tr><td>COMPRESSION
∥ to face grain (3-ply only[2]):
⊥ to face grain
± 45° to face grain</td><td>
1000
900
320</td><td>
950
900
300</td><td>
900
900
290</td></tr>
<tr><td>BEARING (on face)</td><td>310</td><td>310</td><td>310</td></tr>
<tr><td>SHEAR, rolling, in plane of plies[3]:
∥ or ⊥ to face grain
± 45°</td><td>
60
80</td><td>
55
75</td><td>
50
70</td></tr>
<tr><td>SHEAR, in plane ⊥ to plies:
∥ or ⊥ to face grain
± 45°</td><td>
200
400</td><td>
185
370</td><td>
170
340</td></tr>
<tr><td>MODULUS OF ELASTICITY in bending:
Face grain ∥ or ⊥ to span:
Group I
Group II</td><td>

1,300,000
1,100,000</td><td>

1,300,000
1,100,000</td><td>

1,300,000
1,100,000</td></tr>
<tr><td colspan="4">WET OR DAMP LOCATION</td></tr>
<tr><td colspan="4">Where moisture content will exceed 16 per cent, decrease by 20 per cent values shown for dry location for following properties: Extreme fiber in bending, tension and compression both parallel and perpendicular to the grain and at 45 degrees, and bearing. (No change in values for rolling shear or modulus of elasticity. Reduce shear in plane perpendicular to plies by 13 per cent.)</td></tr>
</table>
[1]For grades, thicknesses and species of Western softwood plywood listed in U.B.C. Standard No. 25-9-64.
[2]For tension or compression parallel to grain in five-ply or thicker, use values for three-ply, but in next lower grade.
[3]The working stresses for rolling shear in glued joints shall be reduced by 50 per cent for flange web joints of beams having plywood webs, plywood gusset plates and framing members located at the edges of stressed-skin plywood panels.
160
</td>
<td>Tables for plywood revised for Western Softwood and Western Larch.</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1967) Section 2511	See tables No. 25-P and No. 25-Q below	Allowable shear stresses for blocked and unblocked plywood diaphragms are tabulated. Allowable shear stresses for plywood shear walls are tabulated. .

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TABLE NO. 25-P—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS[1]

PLYWOOD GRADE	COMMON NAIL SIZE	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBER (Inches)	BLOCKED DIAPHRAGMS: Nail Spacing at Diaphragm Boundaries (All Cases) and Continuous Panel Edges Parallel to Load (Cases 3 and 4)[2] 6 / Nail Spacing at Other Plywood Panel Edges 6	4 / 6	2½ / 4	2 / 3	UNBLOCKED DIAPHRAGMS: Nails Spaced 6" Maximum at Supported Edges — Load Perpendicular to Unblocked Edges and Continuous Panel Joints (Case 1)	All Other Configurations (Cases 2, 3 and 4)
STRUCTURAL I	6d	1¼	5/16	2 3	188 210	250 280	375 420	420 475	167 187	125 140
	8d	1½	⅜	2 3	270 300	360 400	530 600	600 675	240 267	180 200
	10d	1⅝	½	2 3	318 360	425 480	640 720	730 820	283 320	212 240

TABLE NO. 25-P—Continued

STRUCTURAL II, C-C Exterior, Standard Sheathing and Other grades covered in U.B.C. Standard No. 25-9-67	6d	1¼	5/16	2 3	125 140	167 187	250 280	280 317	111 125	84 93
			⅜	2 3	150 168	201 224	300 336	336 380	133 150	101 112
	8d	1½	⅜	2 3	180 200	240 267	353 400	400 450	160 178	120 133
			½	2 3	215 240	288 320	424 480	480 540	192 214	144 160
	10d	1⅝	⅝	2 3	255 288	340 384	512 576	584 656	227 256	169 192

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[1]These values are for short time loads due to wind or earthquake and must be reduced 25 per cent for normal loading.
[2]Space nails at twelve inches (12") on center along intermediate framing members.

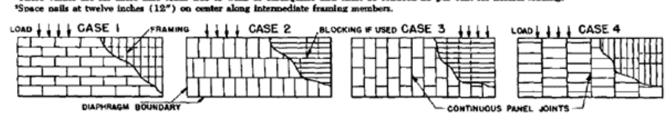

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TABLE NO. 25-Q—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS[1]

PLYWOOD GRADE	NAIL SIZE (Common or Galvanized Box)	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	PLYWOOD APPLIED DIRECT TO FRAMING — Nail Spacing at Plywood Panel Edges 6	4	2½	2	NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED OVER ½-INCH GYPSUM SHEATHING — Nail Spacing at Plywood Panel Edges 6	4	2½	2
STRUCTURAL I	6d 8d 10d	1¼ 1½ 1⅝	5/16 ⅜ ½	200 280 340	300 430 510	450 640 770	510 730 870	8d 10d —	200 280 —	300 430 —	450 640 —	510 730 —
STRUCTURAL II, C-C Exterior, Standard Sheathing, Panel Siding Plywood and other grades Covered in U.B.C. Standard No. 25-9-67	6d 8d 10d	1¼ 1½ 1⅝	5/16 ⅜ ½	180 260 310	270 380 460	400 570 690	450 640 770	8d 10d —	180 260 —	270 380 —	400 570 —	450 640 —
	NAIL SIZE (Galvanized Casing)							NAIL SIZE (Galvanized Casing)				
Plywood Panel Siding in Grades Covered in U.B.C. Standard No. 25-9-67	6d 8d	1¼ 1½	5/16 ⅜	140 160	210 240	320 360	360 410	8d 10d	140 160	210 240	320 360	360 410

[1]All panel edges backed with two-inch (2") nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at twelve inches (12") on center along intermediate framing members. These values are for short time loads due to wind or earthquake and must be reduced 25 per cent for normal loading.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1970) Section 2514	**Wood Diaphragms** **Sec. 2514** (a) **General.** Lumber and plywood diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragm, as determined by calculations, tests, or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements. See U.B.C. Standard No. 25-9 for a method of calculating the deflection of a blocked plywood diaphragm. In buildings of wood frame construction where rotation is provided for, the depth of the diaphragm normal to the open side shall not exceed 25 feet nor two-thirds the diaphragm width, whichever is the smaller depth. Straight sheathing shall not be permitted to resist shears in diaphragms acting in rotation. **EXCEPTIONS:** 1. One-story, wood-framed structures with the depth normal to the open side not greater than 25 feet, may have a depth equal to the width. 2. Where calculations show that diaphragm deflections can be tolerated, the depth normal to the open end may be increased to a depth to width ratio not greater than 1½:1 for diagonal sheathing or 2:1 for special diagonal sheathed or plywood diaphragms. (c) **Plywood Diaphragms.** Horizontal and vertical diaphragms sheathed with plywood may be used to resist horizontal forces not exceeding those set forth in Table No. 25-L for horizontal diaphragms, and Table No. 25-M for vertical diaphragms, or may be calculated by principles of mechanics without limitation by using values of nail strength and plywood shear values as specified elsewhere in this Code. Plywood for horizontal diaphragms shall be as set forth in Table No. 25-Q for corresponding joist spacing and loads. Maximum spans for plywood subfloor-underlayment shall be as set forth **SECTIONS 2514-2516** **UNIFORM BUILDING CODE** **Wood Diaphragms (Continued)** in Table No. 25-R. Plywood used for horizontal and vertical diaphragms shall conform to U.B.C. Standard No. 25-9. All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least 2-inch nominal in the dimension to which the plywood is attached and such members shall be limited to a maximum spacing of 16 inches on center for vertical diaphragms. In general, panel edges shall bear on the framing members and butt along their center lines. Nails shall be placed not less than ⅜ inch in from the panel edge, nor more than 12 inches apart along intermediate supports and 6 inches along panel edge bearings, and shall be firmly driven into the framing members. No unblocked panels less than 12 inches wide shall be used.	Reference for plywood diaphragm deflection calculations is added to the code. Straight lumber sheathing is prohibited for resisting shears in diaphragms "acting in rotation." Exceptions to the limitations on size and shape of wood diaphragms are added to the code. Allowable shear stresses for plywood sheathed diaphragms are revised.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1970) Section 2514		

TABLE NO. 25-L — ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS[1]

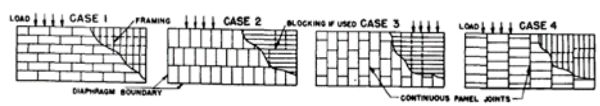
PLYWOOD GRADE	Common Nail Size	Minimum Nominal Penetration in Framing (in inches)	Minimum Nominal Plywood Thickness (in inches)	Minimum Nominal Width of Framing Member (in inches)	BLOCKED DIAPHRAGMS: Nail Spacing at diaphragm boundaries (all cases) and continuous panel edges parallel to load (cases 3 & 4)				UNBLOCKED DIAPHRAGM: Nails spaced 6" max. at supported end	
					6	4	2½	2	Load perpendicular to unblocked edges and continuous panel joints (case 1)	All other configurations (cases 2, 3 & 4)
					Nail spacing at other plywood panel edges					
					6	6	4	3		
STRUCTURAL I	6d	1¼	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1½	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1⅝	½	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240
STRUCTURAL II, C-C Exterior, Standard Sheathing and Other Grades Covered in U.B.C. Standard No. 25-9	6d	1¼	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1½	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			½	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1⅝	½	2	290	385	575[2]	655[2]	255	190
				3	325	430	650	735	290	215
			⅝	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240

(Continued)

TABLE NO. 25-L (Continued)

[1]These values are for short time loads due to wind or earthquake and must be reduced 25 per cent for normal loading. Space nails 12 inches on center along intermediate framing members.

[2]Reduce tabulated allowable shears 10 per cent when boundary members provide less than 3-inch nominal nailing surface.



NOTE: Framing may be located in either direction for blocked diaphragms.

TABLE NO. 25-M — ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS[1]

PLYWOOD GRADE	NAIL SIZE (Common or Galvanized Box)	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	PLYWOOD APPLIED DIRECT TO FRAMING: Nail Spacing at Plywood Panel Edges				NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED OVER ½-INCH GYPSUM SHEATHING: Nail Spacing at Plywood Panel Edges			
				6	4	2½	2		6	4	2½	2
STRUCTURAL I	6d 8d 10d	1¼ 1½ 1⅝	5/16 3/8 ½	200 280 340	300 430 510	450 640 770[2]	510 730 870[2]	8d 10d —	200 280 —	300 430 —	450 640[2] —	510 730[2] —
STRUCTURAL II, C-C Exterior, Standard Sheathing, Panel Siding Plywood and Other Grades Covered in U.B.C. Standard No. 25-9	6d 8d 10d	1¼ 1½ 1⅝	5/16 3/8 ½	180 260 310	270 380 460	400 570 690[2]	450 640 770[2]	8d 10d —	180 260 —	270 380 —	400 570[2] —	450 640[2] —
	NAIL SIZE (Galvanized Casing)							NAIL SIZE (Galvanized Casing)				
Plywood Panel Siding in Grades Covered in U.B.C. Standard No. 25-9	6d 8d	1¼ 1½	5/16 3/8	140 160	210 240	320 360	360 410	8d 10d	140 160	210 240	320 360	360 410

[1]All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 12 inches on center along intermediate framing members. These values are for short time loads due to wind or earthquake and must be reduced 25 per cent for normal loading.

[2]Reduce tabulated allowable shears 10 per cent when boundary members provide less than 3-inch nominal nailing surface.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1973)
Section 2514</td>
<td>
(c) Plywood Diaphragms. Horizontal and vertical diaphragms sheathed with plywood may be used to resist horizontal forces not exceeding those set forth in Table No. 25-J for horizontal diaphragms, and Table No. 25-K for vertical diaphragms, or may be calculated by principles of mechanics without limitation by using values of nail strength and plywood shear values as specified elsewhere in this Code. Plywood for horizontal diaphragms shall be as set forth in Table No. 25-R for corresponding joist spacing and loads. Plywood in shear walls shall be at least 5/16 inch thick for studs spaced 16 inches on center and 3/8 inch thick where studs are spaced 24 inches on center.

Maximum spans for plywood subfloor underlayment shall be as set forth in Table No. 25-S. Plywood used for horizontal and vertical diaphragms shall conform to U.B.C. Standard No. 25-9.

All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least 2-inch nominal in the dimension to which the plywood is attached. In general, panel edges shall bear on the framing members and butt along their center lines. Nails shall be placed not less than 3/8 inch in from the panel edge, nor more than 12 inches apart along intermediate supports and 6 inches apart along panel edge bearings, and shall be firmly driven into the framing members. No unblocked panels less than 12 inches wide shall be used.

25-K

TABLE NO. 25-K—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS[1]
<table>
<tr><th rowspan="3">PLYWOOD GRADE</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th rowspan="3">MINIMUM NAIL PENETRATION IN FRAMING (Inches)</th><th rowspan="3">MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)</th><th colspan="4">PLYWOOD APPLIED DIRECT TO FRAMING</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED OVER 1/2-INCH GYPSUM SHEATHING</th></tr>
<tr><th colspan="4">Nail Spacing at Plywood Panel Edges</th><th colspan="4">Nail Spacing at Plywood Panel Edges</th></tr>
<tr><th>6</th><th>4</th><th>2 1/2</th><th>2</th><th>6</th><th>4</th><th>2 1/2</th><th>2</th></tr>
<tr><td>STRUCTURAL I</td><td>6d
8d
10d</td><td>1 1/4
1 1/2
1 5/8</td><td>5/16
3/8
1/2</td><td>200
230[3]
340</td><td>300
360[3]
510</td><td>450
530[3]
770[2]</td><td>510
610[3]
870[2]</td><td>8d
10d
—</td><td>200
280
—</td><td>300
430
—</td><td>450
640[2]
—</td><td>510
730[2]
—</td></tr>
<tr><td>STRUCTURAL II, C-C Exterior, Standard Sheathing, Panel Siding Plywood and Other Grades Covered in U.B.C. Standard No. 25-9</td><td>6d
8d
10d</td><td>1 1/4
1 1/2
1 5/8</td><td>5/16
3/8
1/2</td><td>180
220[3]
310</td><td>270
320[3]
460</td><td>400
470[3]
690[2]</td><td>450
530[3]
770[2]</td><td>8d
10d
—</td><td>180
260
—</td><td>270
380
—</td><td>400
570[2]
—</td><td>450
640[2]
—</td></tr>
<tr><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Plywood Panel Siding in Grades Covered in U.B.C. Standard No. 25-9</td><td>6d
8d</td><td>1 1/4
1 1/2</td><td>5/16
3/8</td><td>140
130[3]</td><td>210
200[3]</td><td>320
300[3]</td><td>360
340[3]</td><td>8d
10d</td><td>140
160</td><td>210
240</td><td>320
360</td><td>360
410</td></tr>
</table>
[1]All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch plywood installed with face grain parallel to studs spaced 24 inches on center and 12 inches on center for other conditions and plywood thicknesses. These values are for short time loads due to wind or earthquake and must be reduced 25 percent for normal loading.
[2]Reduce tabulated allowable shears 10 percent when boundary members provide less than 3-inch nominal nailing surface.
[3]The values for 3/8-inch thick plywood applied direct to framing may be increased 20 percent provided studs are spaced a maximum of 16 inches on center or plywood is applied with face grain across studs.
</td>
<td>Plywood in shear walls shall be at least 5/16" thick for studs spaced at 16" o.c. and 3/8" thick for studs spaced at 24" o.c.

Allowable shear values for plywood shear walls are revised.</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1976) Chapter 25		Tables for allowable stresses for plywood are revised. Tables for allowable stresses for plywood diaphragms are revised. Tables for allowable stresses for plywood shear walls are revised.

1976 EDITION

TABLE NO. 25-B—ALLOWABLE UNIT STRESSES FOR CONSTRUCTION AND INDUSTRIAL SOFTWOOD PLYWOOD
(In Pounds per Square Inch—Normal Loading)
(To be used with section properties in Plywood-Design Specification—See U.B.C. Standard No. 25-9)

STRESS	SPECIES[1] GROUP OF FACE PLY	EXTERIOR A-A, A-C, C-C STRUCTURAL I A-C, C-C (Use Group I Stresses)		EXTERIOR A-B, B-B, B-C, C-C (PLUGGED) STRUCTURAL I C-D (Use Group 1 Stresses) STRUCTURAL II C-D (Use Group 3 Stresses) C-D SHEATHING (Exterior Glue) ALL INTERIOR GRADES WITH EXTERIOR GLUE		ALL OTHER GRADES OF INTERIOR INCLUDING C-D SHEATHING
		Wet[2]	Dry[3]	Wet[2]	Dry[3]	Dry[3]
1. Extreme fiber stress in bending (F_b)	1	1430	2000	1190	1650	1650
Tension in plane of plies (F_t)	2, 3	980	1400	820	1200	1200
Face grain parallel or perpendicular to span (at 45° to face grain use 1/6 F_t)	4	940	1330	780	1110	1110
2. Compression in plane of plies (F_c)	1	970	1640	900	1540	1540
Parallel or perpendicular to	2	730	1200	680	1100	1100
face grain	3	610	1060	580	990	990
(at 45° to face grain use 1/3 F_c)	4	610	1000	580	950	950
3. Shear in plane perpendicular to plies	1	205	250	205	250	210
Parallel or perpendicular to face grain	2, 3	160	185	160	185	160
(at 45° to face grain use 2 F_v)	4	145	175	145	175	155

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(Continued)

266 **TABLE NO. 25-B—ALLOWABLE UNIT STRESSES FOR CONSTRUCTION AND INDUSTRIAL SOFTWOOD PLYWOOD (continued)** 25-B
(In Pounds per Square Inch—Normal Loading)
(To be used with section properties in Plywood-Design Specification—See U.B.C. Standard No. 25-9)

4. Shear, rolling, in the plane of plies	Marine and Structural I	63	75	63	75	
Parallel or perpendicular to face grain	Structural II	49	56	49	56	
(at 45° to face grain use 1 1/3 F_s)	All Other	44	53	44	53	48
5. Bearing (on face)	1	210	340	210	340	340
Perpendicular to plane	2, 3	135	210	135	210	210
of plies	4	105	160	105	160	160
6. Modulus of elasticity	1	1,500,000	1,800,000	1,500,000	1,800,000	1,800,000
In bending in plane of plies	2	1,300,000	1,500,000	1,300,000	1,500,000	1,500,000
Face grain parallel or	3	1,100,000	1,200,000	1,100,000	1,200,000	1,200,000
perpendicular to span	4	900,000	1,000,000	900,000	1,000,000	1,000,000

[1]See U.B.C. Standard No. 25-9 for plywood species groups. For C-C and C-D, the combination of Identification Index and panel thickness determines the species group and therefore the stress permitted, as in the following table:

[2]Wet condition of use corresponds to a moisture content of 16 percent or more.

[3]Dry condition of use corresponds to a moisture content of less than 16 percent.

IDENTIFICATION INDEX*

THICKNESS (inches)	12/0	16/0	20/0	24/0	32/16	42/20	48/24
5/16	4	3	1				
3/8		4	3	1			
1/2				4	1		
5/8					3	1	
3/4						3	1
7/8						4	3

*30/12-5/8", and 36/16-3/4"—Use Group 4 stresses.

UNIFORM BUILDING CODE

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1976) Chapter 25	See excerpt below	

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TABLE NO. 25-J—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

PLYWOOD GRADE	Common Nail Size	Minimum Nominal Penetration in Framing (in inches)	Minimum Nominal Plywood Thickness (in inches)	Minimum Nominal Width of Framing Member (in inches)	BLOCKED DIAPHRAGMS: Nail Spacing at diaphragm boundaries (all cases) and continuous panel edges parallel to load (cases 3, 4, 5 & 6) 6 / Nail spacing at other plywood panel edges 6	4 / 6	2½ / 4	2 / 3	UNBLOCKED DIAPHRAGM: Nails spaced 6" max. at supported end — Load perpendicular to unblocked edges and continuous panel joints (case 1)	Other configurations (cases 2, 3, 4)
STRUCTURAL I	6d	1¼	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1½	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1⅝	½	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240
C-D, C-C, STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9	6d	1¼	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1½	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			½	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1⅝	½	2	290	385	575[2]	655[2]	255	190
				3	325	430	650	735	290	215
			5/8	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240

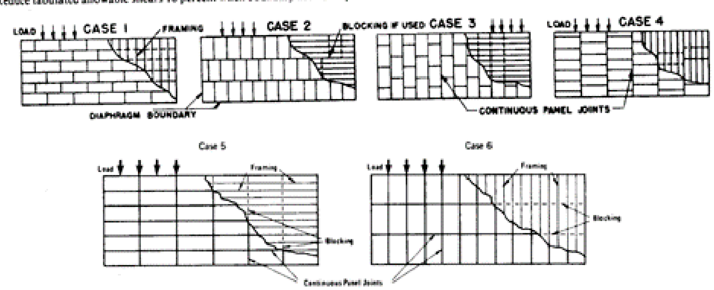
UNIFORM BUILDING CODE

1976 EDITION

[1]These values are for short time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 10 inches on center for floors and 12 inches on center for roofs along intermediate framing members.

Allowable shear values for nails in framing members of other species set forth in Table No. 25-17-J of U.P.C. Standards shall be calculated for all grades by multiplying the values for nails in STRUCTURAL I by the following factors: Group III, 0.82 and Group IV, 0.65.

[2]Reduce tabulated allowable shears 10 percent when boundary members provide less than 3-inch nominal nailing surface.



NOTE: Framing may be located in either direction for blocked diaphragms.

25-J

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1976)
Chapter 25</td><td>
282 — 25-K
TABLE NO. 25-K—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]
<table>
<tr><th rowspan="3">PLYWOOD GRADE</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th rowspan="3">MINIMUM NAIL PENETRATION IN FRAMING (Inches)</th><th rowspan="3">MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)</th><th colspan="4">PLYWOOD APPLIED DIRECT TO FRAMING</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED OVER ½-INCH GYPSUM SHEATHING</th></tr>
<tr><th colspan="4">Nail Spacing at Plywood Panel Edges</th><th colspan="4">Nail Spacing at Plywood Panel Edges</th></tr>
<tr><th>6</th><th>4</th><th>2½</th><th>2</th><th>6</th><th>4</th><th>2½</th><th>2</th></tr>
<tr><td>STRUCTURAL I</td><td>6d
8d
10d</td><td>1¼
1½
1⅝</td><td>5/16
3/8
½</td><td>200
230[3]
340</td><td>300
360[3]
510</td><td>450
530[3]
770[2]</td><td>510
610[3]
870[2]</td><td>8d
10d
—</td><td>200
280
—</td><td>300
430
—</td><td>450
640[2]
—</td><td>510
730[2]
—</td></tr>
<tr><td>C-D, C-C, STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9</td><td>6d
8d
10d</td><td>1¼
1½
1⅝</td><td>5/16
3/8
½</td><td>180
220[3]
310</td><td>270
320[3]
460</td><td>400
470[3]
690[2]</td><td>450
530[3]
770[2]</td><td>8d
10d
—</td><td>180
260
—</td><td>270
380
—</td><td>400
570[2]
—</td><td>450
640[2]
—</td></tr>
<tr><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Plywood Panel Siding in Grades Covered in U.B.C. Standard No. 25-9</td><td>6d
8d</td><td>1¼
1½</td><td>5/16
3/8</td><td>140
130[3]</td><td>210
200[3]</td><td>320
300[3]</td><td>360
340[3]</td><td>8d
10d</td><td>140
160</td><td>210
240</td><td>320
360</td><td>360
410</td></tr>
</table>
[1]All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for ⅜-inch plywood installed with face grain parallel to studs spaced 24 inches on center and 12 inches on center for other conditions and plywood thicknesses. These values are for short time loads due to wind or earthquake and must be reduced 25 percent for normal loading.
Allowable shear values for nails in framing members of other species set forth in Table No. 25-17-J of U.B.C. Standards shall be calculated for all grades by multiplying the values for common and galvanized box nails in STRUCTURAL I and galvanized casing nails in other grades by the following factors: Group III, 0.82 and Group IV, 0.65.
[2]Reduce tabulated allowable shears 10 percent when boundary members provide less than 3-inch nominal nailing surface.
[3]The values for ⅜-inch thick plywood applied direct to framing may be increased 20 percent provided studs are spaced a maximum of 16 inches on center or plywood is applied with face grain across studs.
UNIFORM BUILDING CODE
</td><td></td></tr>
<tr><td>Uniform Building Code (1979)
Section 2514</td><td>
Diaphragm sheathing nails or other approved sheathing connectors shall be driven flush but shall not fracture the surface of the sheathing.
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TABLE NO. 25-J—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]
<table>
<tr><th rowspan="5">PLYWOOD GRADE</th><th rowspan="5">Common Nail Size</th><th rowspan="5">Minimum Nominal Penetration in Framing (in inches)</th><th rowspan="5">Minimum Nominal Plywood Thickness (in inches)</th><th rowspan="5">Minimum Nominal Width of Framing Member (in inches)</th><th colspan="4">BLOCKED DIAPHRAGMS</th><th colspan="2">UNBLOCKED DIAPHRAGM</th></tr>
<tr><th colspan="4">Nail spacing at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6)</th><th colspan="2">Nails spaced 6" max. at supported end</th></tr>
<tr><th>6</th><th>4</th><th>2½</th><th>2</th><th rowspan="3">Load perpendicular to unblocked edges and continuous panel joints (Case 1)</th><th rowspan="3">Other configurations (Cases 2, 3 & 4)</th></tr>
<tr><th colspan="4">Nail spacing at other plywood panel edges</th></tr>
<tr><th>6</th><th>6</th><th>4</th><th>3</th></tr>
<tr><td rowspan="3">STRUCTURAL I</td><td>6d</td><td>1¼</td><td>5/16</td><td>2
3</td><td>185
210</td><td>250
280</td><td>375
420</td><td>420
475</td><td>165
185</td><td>125
140</td></tr>
<tr><td>8d</td><td>1½</td><td>3/8</td><td>2
3</td><td>270
300</td><td>360
400</td><td>530
600</td><td>600
675</td><td>240
265</td><td>180
200</td></tr>
<tr><td>10d</td><td>1⅝</td><td>½</td><td>2
3</td><td>320
360</td><td>425
480</td><td>640[2]
720</td><td>730[2]
820</td><td>285
320</td><td>215
240</td></tr>
<tr><td rowspan="6">C-D, C-C, STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9</td><td rowspan="2">6d</td><td rowspan="2">1¼</td><td>5/16</td><td>2
3</td><td>170
190</td><td>225
250</td><td>335
380</td><td>380
430</td><td>150
170</td><td>110
125</td></tr>
<tr><td>3/8</td><td>2
3</td><td>185
210</td><td>250
280</td><td>375
420</td><td>420
475</td><td>165
185</td><td>125
140</td></tr>
<tr><td rowspan="2">8d</td><td rowspan="2">1½</td><td>3/8</td><td>2
3</td><td>240
270</td><td>320
360</td><td>480
540</td><td>545
610</td><td>215
240</td><td>160
180</td></tr>
<tr><td>½</td><td>2
3</td><td>270
300</td><td>360
400</td><td>530
600</td><td>600
675</td><td>240
265</td><td>180
200</td></tr>
<tr><td rowspan="2">10d</td><td rowspan="2">1⅝</td><td>½</td><td>2
3</td><td>290
325</td><td>385
430</td><td>575[2]
650</td><td>655[2]
735</td><td>255
290</td><td>190
215</td></tr>
<tr><td>5/8</td><td>2
3</td><td>320
360</td><td>425
480</td><td>640[2]
720</td><td>730[2]
820</td><td>285
320</td><td>215
240</td></tr>
</table>
UNIFORM BUILDING CODE
</td><td>Provisions for diaphragm sheathing connectors are added to the code & table for blocked diaphragm is modified.</td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1982) Chapter 25	308 TABLE NO. 25-K—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1] 25-K UNIFORM BUILDING CODE	Table for allowable shear for plywood shear walls is modified

PLYWOOD GRADE	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED DIRECT TO FRAMING Nail Spacing at Plywood Panel Edges 6	4	3	2[2]	NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED OVER ½-INCH GYPSUM SHEATHING Nail Spacing at Plywood Panel Edges 6	4	3	2[2]
STRUCTURAL I	5/16	1¼	6d	200	300	390	510	8d	200	300	390	510
	3/8	1½	8d	230[3]	360[3]	460[3]	610[3]	10d	280	430	550[2]	730
	½	1⅝	10d	340	510	665[2]	870	—	—	—	—	—
C-D, C-C STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9.	5/16	1¼	6d	180	270	350	450	8d	180	270	350	450
	3/8	1½	8d	220[3]	320[3]	410[3]	530[3]	10d	260	380	490[2]	640
	½	1⅝	10d	310	460	600[2]	770	—	—	—	—	—
			NAIL SIZE (Galvanized Casing)					NAIL SIZE (Galvanized Casing)				
Plywood panel siding in grades covered in U.B.C. Standard No. 25-9	5/16	1¼	6d	140	210	275	360	8d	140	210	275	360
	3/8	1½	8d	130[3]	200[3]	260[3]	340[3]	10d	160	240	310[2]	410

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1985) Chapter 25	**Wood Diaphragms** **Sec. 2513.** (a) **General.** Lumber, plywood and particleboard diaphragms may be used to resist horizontal forces in horizontal and vertical distributing or resisting elements, provided the deflection in the plane of the diaphragm, as determined by calculations, tests or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements. See U.B.C. Standard No. 25-9 for a method of calculating the deflection of a blocked plywood diaphragm. Where plywood is applied on both faces of a shear wall in accordance with Table No. 25-K-1, allowable shear for the wall may be taken as twice the tabulated shear for one side, except that where the shear capacities are not equal, the allowable shear shall be either the shear for the side with the higher capacity or twice the shear for the side with the lower capacity, whichever is greater. (d) **Particleboard Diaphragms.** Horizontal and vertical diaphragms sheathed with particleboard may be used to resist horizontal forces not exceeding those set forth in Table No. 25-J-2 for horizontal diaphragms and Table No. 25-K-2 for vertical diaphragms. All boundary members shall be proportioned and spliced where necessary to transmit direct stresses. Framing members shall be at least 2-inch nominal in the dimension to which the particleboard is attached. In general, panel edges shall bear on the framing members and butt along their center lines. Nails shall be placed not less than 3/8 inch in from the panel edge, shall be spaced not more than 6 inches on center along panel edge bearings, and shall be firmly driven into the framing members. No unblocked panels less than 12 inches wide shall be used.	Particleboard is added for the same uses as plywood. Particle board is permitted for wood horizontal and vertical diaphragms. Note for plywood applied on both faces of shear wall is added. Tables for particleboard diaphragm and shear wall capacities are added.

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1988)
Chapter 25</td>
<td>
ROTATION is the torsional movement of a diaphragm about a vertical axis.
SUBDIAPHRAGM is a portion of a larger diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.
TABLE NO. 25-V—WALL BRACING
<table>
<tr><th rowspan="2">SEISMIC ZONE</th><th rowspan="2">CONDITION</th><th colspan="8">TYPE OF BRACE[1]</th><th rowspan="2">AMOUNT OF BRACING[2]</th></tr>
<tr><th>A</th><th>B</th><th>C</th><th>D</th><th>E</th><th>F</th><th>G</th><th>H</th></tr>
<tr><td rowspan="3">0, 1 and 2</td><td>One Story Top of Two or Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td rowspan="3">Each end and each 25′ of wall.</td></tr>
<tr><td>First Story of Two Story or Second Story of Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td>First Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td rowspan="3">3 and 4</td><td>One Story Top of Two or Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>Each end and each 25′ of wall.</td></tr>
<tr><td>First Story of Two Story or Second Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td><td>Each end. 25% of wall length to be sheathed.</td></tr>
<tr><td>First Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td><td>Each end. 40% of wall length to be sheathed.</td></tr>
</table>
[1]See Section 2517 (g) 3 for full description.
[2]Bracing at ends shall be near thereto as possible. Braces shall be installed so that there is no unbraced section along the wall exceeding 25 feet.
[3]Gypsum wallboard applied to supports at 16 inches on center.
</td>
<td>
The term “subdiaphragm” is defined.
The term “rotation” is defined.
Requirements for wood design in Zones 3 & 4 are added, including collectors being required that cannot be spliced via diaphragm sheathing.
Provisions for wall bracing are added.
</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1988)
Chapter 25</td><td>

TABLE NO. 25-V—WALL BRACING

<table>
<tr><th rowspan="2">SEISMIC ZONE</th><th rowspan="2">CONDITION</th><th colspan="8">TYPE OF BRACE[1]</th><th rowspan="2">AMOUNT OF BRACING[2]</th></tr>
<tr><th>A</th><th>B</th><th>C</th><th>D</th><th>E</th><th>F</th><th>G</th><th>H</th></tr>
<tr><td rowspan="3">0, 1 and 2</td><td>One Story Top of Two or Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td rowspan="3">Each end and each 25′ of wall.</td></tr>
<tr><td>First Story of Two Story or Second Story of Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td>First Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td rowspan="3">3 and 4</td><td>One Story Top of Two or Three Story</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td><td>Each end and each 25′ of wall.</td></tr>
<tr><td>First Story of Two Story or Second Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td><td>Each end. 25% of wall length to be sheathed.</td></tr>
<tr><td>First Story of Three Story</td><td></td><td>X</td><td>X</td><td>X</td><td>X[3]</td><td>X</td><td>X</td><td>X</td><td>Each end. 40% of wall length to be sheathed.</td></tr>
</table>

[1]See Section 2517 (g) 3 for full description.
[2]Bracing at ends shall be near thereto as possible. Braces shall be installed so that there is no unbraced section along the wall exceeding 25 feet.
[3]Gypsum wallboard applied to supports at 16 inches on center.

</td><td></td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1991) Chapter 25	Diaphragms with panel edges supported in accordance with Tables Nos. 25-S-1, 25-S-2 and 25-T-1 shall not be considered as blocked diaphragms unless blocking or other means of shear transfer is provided. Diaphragms with panel edges supported in accordance with Tables Nos. 25-S-3 and 25-T-2 shall not be considered as blocked diaphragms unless blocking or other means of shear transfer is provided. E. **Particleboard.** Particleboard shall be Grade 2-M-W using exterior glue. Particleboard diaphragms and shear walls shall be c ... ucted of sheets not less than 4 feet by 8 feet except at boundaries and changes in framing where minimum sheet dimension shall be 24 inches unless all edges of the undersized sheets are supported by framing members or blocking. Framing members or blocking shall be provided at the edges of all sheets in shear walls.	Additional provisions for diaphragm blocking are added to the code for particle board and plywood.

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25-J-1

TABLE NO. 25-J-1—ALLOWABLE SHEAR IN POUNDS PER FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

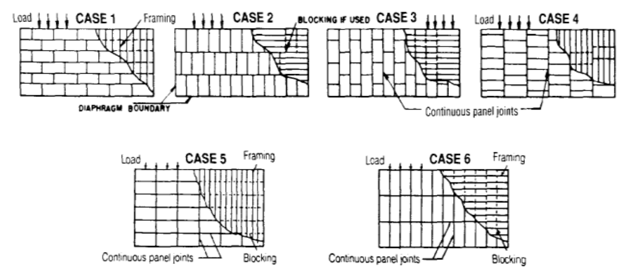
PLYWOOD GRADE	Common Nail Size	Minimum Nominal Penetration in Framing (In Inches)	Minimum Nominal Plywood Thickness (In Inches)	Minimum Nominal Width of Framing Member (In Inches)	BLOCKED DIAPHRAGMS: Nail spacing at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6)				UNBLOCKED DIAPHRAGM: Nails spaced 6" max. at supported end	
					6	4	2 1/2[2]	2[2]	Load perpendicular to unblocked edges and continuous panel joints (Case 1)	Other configurations (Cases 2, 3, 4, 5 and 6)
					Nail spacing at other plywood panel edges					
					6	6	4	3		
STRUCTURAL I	6d	1 1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1 1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	1 5/8	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
C-D, C-C, STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9	6d	1 1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1 1/2	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			15/32	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[3]	1 5/8	15/32	2	290	385	575	655	255	190
				3	325	430	650	735	290	215
			19/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240

1991 UNIFORM BUILDING CODE

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr>
<td>Uniform Building Code (1991)
Chapter 25</td>
<td>
1991 UNIFORM BUILDING CODE

[1]These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 12 inches on center along intermediate framing members.
Allowable shear values for nails in framing members of other species set forth in Table No. 25-17-J of the U.B.C. Standards shall be calculated for all grades by multiplying the values for nails in Structural I by the following factors: Group III, 0.82 and Group IV, 0.65.
[2]Framing at adjoining panel edges shall be 3-inch nominal or wider and nails shall be staggered where nails are spaced 2 inches or $2^1/_2$ inches on center.
[3]Framing at adjoining panel edges shall be 3-inch nominal or wider and nails shall be staggered where 10d nails having penetration into framing of more than $1^5/_8$ inches are spaced 3 inc[illegible]r less on center.



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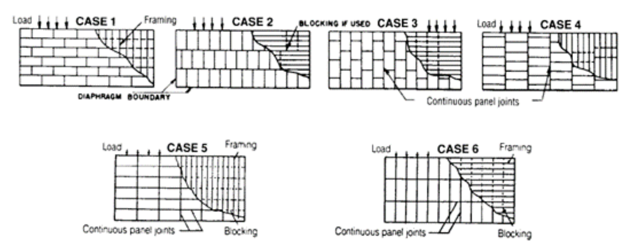
Note: Framing may be oriented in either direction for diaphragms, provided sheathing is properly designed for vertical loading.
25-J-1

1991 UNIFORM BUILDING CODE

[1]These values are for short-time loads due to wind or earthquake and must be reduced 25 percent for normal loading. Space nails 12 inches on center along intermediate framing members.
Allowable shear values for nails in framing members of other species set forth in Table No. 25-17-J of the U.B.C. Standards shall be calculated for all grades by multiplying the values for nails by the following factors: Group III, 0.82 and Group IV, 0.65.
[2]Framing at adjoining panel edges shall be 3-inch nominal or wider and nails shall be staggered where nails are spaced 2 inches or $2^1/_2$ inches on center.
[3]Framing at adjoining panel edges shall be 3-inch nominal or wider and nails shall be staggered where 10d nails having penetration into framing of more than $1^5/_8$ inches are spaced 3 inches or less on center.



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Note: Framing may be oriented in either direction for diaphragms, provided sheathing is properly designed for vertical loading.
25-J-2
</td>
<td>Tables for diaphragm capacities are updated for particle board and plywood blocking cases.</td>
</tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

<table>
<tr><th>UBC Code Year</th><th>Code Excerpts</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1991)
Chapter 25</td><td>
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25-K-1

TABLE NO. 25-K-1—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1 4]

<table>
<tr><th rowspan="3">PLYWOOD GRADE</th><th rowspan="3">MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)</th><th rowspan="3">MINIMUM NAIL PENETRATION IN FRAMING (Inches)</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED DIRECT TO FRAMING</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED OVER 1/2-INCH OR 5/8-INCH GYPSUM SHEATHING</th></tr>
<tr><th colspan="4">Nail Spacing at Plywood Panel Edges</th><th colspan="4">Nail Spacing at Plywood Panel Edges</th></tr>
<tr><th>6</th><th>4</th><th>3</th><th>2[2]</th><th>6</th><th>4</th><th>3</th><th>2[2]</th></tr>
<tr><td rowspan="4">STRUCTURAL I</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>200</td><td>300</td><td>390</td><td>510</td><td>8d</td><td>200</td><td>300</td><td>390</td><td>510</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>230[3]</td><td>360[3]</td><td>460[3]</td><td>610[3]</td><td>10d[5]</td><td>280</td><td>430</td><td>550</td><td>730[3]</td></tr>
<tr><td>15/32</td><td>1 1/2</td><td>8d</td><td>280</td><td>430</td><td>550</td><td>730</td><td>10d[5]</td><td>280</td><td>430</td><td>550</td><td>730</td></tr>
<tr><td>15/32</td><td>1 5/8</td><td>10d[5]</td><td>340</td><td>510</td><td>665</td><td>870</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td rowspan="6">C-D, C-C STRUCTURAL II, plywood panel siding and other grades covered in U.B.C. Standard No. 25-9.</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>180</td><td>270</td><td>350</td><td>450</td><td>8d</td><td>180</td><td>270</td><td>350</td><td>450</td></tr>
<tr><td>3/8</td><td>1 1/4</td><td>6d</td><td>200</td><td>300</td><td>390</td><td>510</td><td>8d</td><td>200</td><td>300</td><td>390</td><td>510</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>220[3]</td><td>320[3]</td><td>410[3]</td><td>530[3]</td><td>10d[5]</td><td>260</td><td>380</td><td>490</td><td>640</td></tr>
<tr><td>15/32</td><td>1 1/2</td><td>8d</td><td>260</td><td>380</td><td>490</td><td>640</td><td>10d[5]</td><td>260</td><td>380</td><td>490</td><td>640</td></tr>
<tr><td>15/32</td><td>1 5/8</td><td>10d[5]</td><td>310</td><td>460</td><td>600</td><td>770</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>19/32</td><td>1 5/8</td><td>10d[5]</td><td>340</td><td>510</td><td>665</td><td>870</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Plywood panel siding in grades covered in U.B.C. Standard No. 25-9</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>140</td><td>210</td><td>275</td><td>360</td><td>8d</td><td>140</td><td>210</td><td>275</td><td>360</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>130[3]</td><td>200[3]</td><td>260[3]</td><td>340[3]</td><td>10d[5]</td><td>160</td><td>240</td><td>310</td><td>410</td></tr>
</table>

1991 UNIFORM BUILDING CODE
</td><td></td></tr>
<tr><td>Uniform Building Code (1994)
Chapter 25</td><td>
Changes are substantial, see code for changes.

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25-K-1

TABLE NO. 25-K-1—ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1 4]

<table>
<tr><th rowspan="3">PLYWOOD GRADE</th><th rowspan="3">MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)</th><th rowspan="3">MINIMUM NAIL PENETRATION IN FRAMING (Inches)</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED DIRECT TO FRAMING</th><th rowspan="3">NAIL SIZE (Common or Galvanized Box)</th><th colspan="4">PLYWOOD APPLIED OVER 1/2-INCH OR 5/8-INCH GYPSUM SHEATHING</th></tr>
<tr><th colspan="4">Nail Spacing at Plywood Panel Edges</th><th colspan="4">Nail Spacing at Plywood Panel Edges</th></tr>
<tr><th>6</th><th>4</th><th>3</th><th>2[2]</th><th>6</th><th>4</th><th>3</th><th>2[2]</th></tr>
<tr><td rowspan="4">STRUCTURAL I</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>200</td><td>300</td><td>390</td><td>510</td><td>8d</td><td>200</td><td>300</td><td>390</td><td>510</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>230[3]</td><td>360[3]</td><td>460[3]</td><td>610[3]</td><td>10d[5]</td><td>280</td><td>430</td><td>550</td><td>730[3]</td></tr>
<tr><td>15/32</td><td>1 1/2</td><td>8d</td><td>280</td><td>430</td><td>550</td><td>730</td><td>10d[5]</td><td>280</td><td>430</td><td>550</td><td>730</td></tr>
<tr><td>15/32</td><td>1 5/8</td><td>10d[5]</td><td>340</td><td>510</td><td>665</td><td>870</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td rowspan="6">C-D, C-C STRUCTURAL II, plywood panel siding and other grades covered in U.B.C. Standard No. 25-9.</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>180</td><td>270</td><td>350</td><td>450</td><td>8d</td><td>180</td><td>270</td><td>350</td><td>450</td></tr>
<tr><td>3/8</td><td>1 1/4</td><td>6d</td><td>200</td><td>300</td><td>390</td><td>510</td><td>8d</td><td>200</td><td>300</td><td>390</td><td>510</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>220[3]</td><td>320[3]</td><td>410[3]</td><td>530[3]</td><td>10d[5]</td><td>260</td><td>380</td><td>490</td><td>640</td></tr>
<tr><td>15/32</td><td>1 1/2</td><td>8d</td><td>260</td><td>380</td><td>490</td><td>640</td><td>10d[5]</td><td>260</td><td>380</td><td>490</td><td>640</td></tr>
<tr><td>15/32</td><td>1 5/8</td><td>10d[5]</td><td>310</td><td>460</td><td>600</td><td>770</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td>19/32</td><td>1 5/8</td><td>10d[5]</td><td>340</td><td>510</td><td>665</td><td>870</td><td>—</td><td>—</td><td>—</td><td>—</td><td>—</td></tr>
<tr><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td><td>NAIL SIZE (Galvanized Casing)</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Plywood panel siding in grades covered in U.B.C. Standard No. 25-9</td><td>5/16</td><td>1 1/4</td><td>6d</td><td>140</td><td>210</td><td>275</td><td>360</td><td>8d</td><td>140</td><td>210</td><td>275</td><td>360</td></tr>
<tr><td>3/8</td><td>1 1/2</td><td>8d</td><td>130[3]</td><td>200[3]</td><td>260[3]</td><td>340[3]</td><td>10d[5]</td><td>160</td><td>240</td><td>310</td><td>410</td></tr>
</table>

1991 UNIFORM BUILDING CODE
</td><td>Overhaul of code format.

Term “plywood” is replaced with “wood structural panel.”

Section 2326 “Conventional Light-Frame Construction Provisions” is added to the code (see code, this is a long section), including provisions for bracing.</td></tr>
</table>

Table B-2 Building Code Evolution: Wood Diaphragm Provisions (continued)

UBC Code Year	Code Excerpts	Notes & Observations
Uniform Building Code (1994) Chapter 25	(e) **Requirements for Wood Design—Seismic Zones Nos. 3 and 4.** 1. **Wood shear walls and diaphragms.** A. **General.** Design and construction of wood shear walls and diaphragms in Seismic Zones Nos. 3 and 4 shall conform to the requirements of this section. B. **Framing.** Collector members shall be provided to transmit tension and compression forces. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Diaphragm sheathing shall not be used to splice these members. Diaphragm chords and ties shall be placed in, or tangent to, the plane of the diaphragm framing unless it can be demonstrated that the moments, shears and deflections and deformations resulting from other arrangements can be tolerated. C. **Plywood.** Plywood shall be manufactured using exterior glue. Plywood diaphragms and shear walls shall be constructed with plywood sheets not less than 4 feet by 8 feet, except at boundaries and changes in framing where minimum sheet dimension shall be 24 inches unless all edges of the undersized sheets are supported by framing members or blocking. Framing members or blocking shall be provided at the edges of all sheets in shear walls. Plywood sheathing may be used for splicing members, other than those noted in Section 2513 (e) 1 B, where the additional nailing required to develop the transfer of forces will not cause cross-grain bending or cross-grain tension in the nailed member. D. **Heavy wood panels.** Diagonally sheathed panels utilizing 2-inch nominal boards may be used to resist the same permissible shears as 1-inch nominal lumber, except that 16d nails shall be used instead of 8d. Panels utilizing straight decking overlaid with plywood may be used to resist shear forces using the same shear values as permitted for the plywood alone. Plywood joints parallel to the decking shall be located at least 1 inch offset from any parallel decking joint. Heavy decking panels utilizing dowel pins, or vertically laminated panels connected by nailing units to one another, resist shear forces based on the permissible shear values of their connectors.	

B.2.3 Shear Wall Capacity Provisions

Table B-3 Development of UBC Earthquake Provisions, 1949 to Present

Edition	Basic Formula for Design Base Shear	Vertical Distribution	Straight Sheath	Gyp Lath & Pl (plf)	Metal Lath & Cem Pl (plf)	Let-in Brace	Foot-notes
1949	$F = \frac{60}{N+4.5}W$	Approximately Triangular	-	-	-	-	3, 6, 7
1955							3, 6
1961	$V = KC_1W$ $C_1 = \frac{0.05}{\sqrt[3]{T}}$	1 & 2 Story – Uniform; Others Triangular $F_x = \frac{Vw_xh_x}{\sum wh}$	Note 3	100		Note 3	3, 4, 8
1964							3, 4
1967		1 & 2 Story – Uniform; Others Triangular $F_x = \frac{(V-F_t)w_xh_x}{\sum_{i=1}^{n} w_ih_i}$					1, 3, 4
1973					180		
1976	$V = ZIKCSW$ $C_1 = \frac{1}{15\sqrt{T}}$	Triangular Distribution Only					
1985							2, 3, 4
1988	$V = \frac{ZIC}{R_w}W$ $C = \frac{1.25S}{T^{2/3}}$			50, Note 5			2, 3, 4, 5

Table B-3 Development of UBC Earthquake Provisions, 1949 to Present (continued)

<table>
<tr><th>Edition</th><th>Gypsum Sheath (plf)</th><th colspan="2">Gypsum Board 1/2" & 5/8" (plf)</th><th>1" Diag. Sheath</th><th>Combine Matls</th><th>Nails</th><th>Footnotes</th></tr>
<tr><td>1949</td><td rowspan="2">-</td><td rowspan="2">-</td><td rowspan="4">-</td><td rowspan="2">-</td><td rowspan="4">-</td><td>Common Only</td><td>3, 6, 7</td></tr>
<tr><td>1955</td><td rowspan="8">Common and Box</td><td>3, 6</td></tr>
<tr><td>1961</td><td rowspan="6">75 to 175</td><td rowspan="6">100 to 150</td><td rowspan="7">300</td><td>3, 4, 8</td></tr>
<tr><td>1964</td><td>3, 4</td></tr>
<tr><td>1967</td><td rowspan="4">175 to 250</td><td rowspan="3">Note 1</td><td rowspan="3">1, 3, 4</td></tr>
<tr><td>1973</td></tr>
<tr><td>1976</td></tr>
<tr><td>1985</td><td rowspan="2">Note 2</td><td>2, 3, 4</td></tr>
<tr><td>1988</td><td>38 to 88, Note 5</td><td>50 to 75, Note 5</td><td>88 to 125, Note 5</td><td>2, 3, 4, 5</td></tr>
</table>

Footnotes:

1. Shear values may not be cumulative for different materials. May be *doubled* for identical materials.
2. Shear values may not be cumulative for different materials. May be *additive* for identical materials.
3. Historically, there have been no values in the UBC for straight sheathing or let-in braces. However, many jurisdictions in the Los Angeles basin accepted the values in the Los Angeles City Building Code for these elements.
4. Rotation only permitted in wood frame buildings.
5. Values for gypsum materials are reduced 50% as required in seismic zones Nos. 3 & 4.
6. Earthquake provisions were placed in an appendix that provided suggestive provisions only for those jurisdictions desirous of enforcing earthquake provisions.
7. N = Number of stories above the story under consideration.
8. C_1 = 0.10 for all 1 & 2 story buildings.

B.2.4 *R* Factor Provisions

Table B-4 Building Code Evolution: *R* Factors

<table>
<tr><th>UBC/IBC Code Year</th><th>Code Excerpts</th><th>Plywood Factor</th><th>Stucco Factor</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1973)</td><td>
TABLE NO. 23-I—HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]
<table>
<tr><th>TYPE OR ARRANGEMENT OF RESISTING ELEMENTS</th><th>VALUE[2] OF K</th></tr>
<tr><td>All building framing systems except as hereinafter classified</td><td>1.00</td></tr>
<tr><td>Buildings with a box system as specified in Section 2314 (b)</td><td>1.33</td></tr>
<tr><td>Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls using the following design criteria:
(1) The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames
(2) The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces
(3) The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force</td><td>0.80</td></tr>
<tr><td>Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force</td><td>0.67</td></tr>
<tr><td>Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building[3]</td><td>3.00[4]</td></tr>
<tr><td>Structures other than buildings and other than those set forth in Table No. 23-J</td><td>2.00</td></tr>
</table>
</td><td>K = 1.33</td><td>K = 1.33</td><td></td></tr>
</table>

Table B-4 Building Code Evolution: *R* Factors (continued)

<table>
<tr><th>UBC/IBC Code Year</th><th>Code Excerpts</th><th>Plywood Factor</th><th>Stucco Factor</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1979)</td><td>
TABLE NO. 23-I—HORIZONTAL FORCE FACTOR K FOR BUILDINGS OR OTHER STRUCTURES[1]
<table>
<tr><th>TYPE OR ARRANGEMENT OF RESISTING ELEMENTS</th><th>VALUE[2] OF K</th></tr>
<tr><td>1. All building framing systems except as hereinafter classified</td><td>1.00</td></tr>
<tr><td>2. Buildings with a box system as specified in Section 2312 (b)</td><td>1.33</td></tr>
<tr><td>3. Buildings with a dual bracing system consisting of a ductile moment-resisting space frame and shear walls or braced frames using the following design criteria:
a. The frames and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames.
b. The shear walls or braced frames acting independently of the ductile moment-resisting portions of the space frame shall resist the total required lateral forces.
c. The ductile moment-resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.</td><td>0.80</td></tr>
<tr><td>4. Buildings with a ductile moment-resisting space frame designed in accordance with the following criteria: The ductile moment-resisting space frame shall have the capacity to resist the total required lateral force</td><td>0.67</td></tr>
<tr><td>5. Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building.</td><td>2.5[1]</td></tr>
<tr><td>6. Structures other than buildings and other than those set forth in Table No. 23-J</td><td>2.00</td></tr>
</table>
</td><td>K = 1.33</td><td>K = 1.33</td><td></td></tr>
<tr><td>Uniform Building Code (1985)</td><td>
TABLE NO. 23-I—HORIZONTAL FORCE FACTOR K FOR BUILDINGS OR OTHER STRUCTURES[1]
<table>
<tr><th>TYPE OR ARRANGEMENT OF RESISTING ELEMENTS</th><th>VALUE[2] OF K</th></tr>
<tr><td>1. All building framing systems except as hereinafter classified</td><td>1.00</td></tr>
<tr><td>2. Buildings with a box system as specified in Section 2312 (b)
EXCEPTION: Buildings not more than three stories in height with stud wall framing and using plywood horizontal diaphragms and plywood vertical shear panels for the lateral force system may use K = 1.0.</td><td>1.33</td></tr>
<tr><td>3. Buildings with a dual bracing system consisting of a ductile moment-resisting space frame and shear walls or braced frames using the following design criteria:
a. The frames and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames
b. The shear walls or braced frames acting independently of the ductile moment-resisting portions of the space frame shall resist the total required lateral forces
c. The ductile moment-resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force</td><td>0.80</td></tr>
<tr><td>4. Buildings with a ductile moment-resisting space frame designed in accordance with the following criteria: The ductile moment-resisting space frame shall have the capacity to resist the total required lateral force</td><td>0.67</td></tr>
<tr><td>5. Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building</td><td>2.5[3]</td></tr>
<tr><td>6. Structures other than buildings and other than those set forth in Table No. 23-J</td><td>2.00</td></tr>
</table>
</td><td>K = 1.0</td><td>K = 1.33</td><td>First time factors diverge</td></tr>
</table>

Table B-4 Building Code Evolution: *R* Factors (continued)

<table>
<tr><th>UBC/IBC Code Year</th><th>Code Excerpts</th><th>Plywood Factor</th><th>Stucco Factor</th><th>Notes & Observations</th></tr>
<tr><td>Uniform Building Code (1994)</td><td>
TABLE 16-N—STRUCTURAL SYSTEMS
<table>
<tr><th>BASIC STRUCTURAL SYSTEM[1]</th><th>LATERAL-FORCE-RESISTING SYSTEM—DESCRIPTION</th><th>R_w[2]</th><th>H[3]
× 304.8 for mm</th></tr>
<tr><td>1. Bearing wall system</td><td>1. Light-framed walls with shear panels</td><td></td><td></td></tr>
<tr><td></td><td>a. Wood structural panel walls for structures three stories or less</td><td>8</td><td>65</td></tr>
<tr><td></td><td>b. All other light-framed walls</td><td>6</td><td>65</td></tr>
<tr><td></td><td>2. Shear walls</td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete</td><td>6</td><td>160</td></tr>
<tr><td></td><td>b. Masonry</td><td>6</td><td>160</td></tr>
<tr><td></td><td>3. Light steel-framed bearing walls with tension-only bracing</td><td>4</td><td>65</td></tr>
<tr><td></td><td>4. Braced frames where bracing carries gravity loads</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>6</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[4]</td><td>4</td><td>—</td></tr>
<tr><td></td><td>c. Heavy timber</td><td>4</td><td>65</td></tr>
<tr><td>2. Building frame system</td><td>1. Steel eccentrically braced frame (EBF)</td><td>10</td><td>240</td></tr>
<tr><td></td><td>2. Light-framed walls with shear panels</td><td></td><td></td></tr>
<tr><td></td><td>a. Wood structural panel walls for structures three stories or less</td><td>9</td><td>65</td></tr>
<tr><td></td><td>b. All other light-framed walls</td><td>7</td><td>65</td></tr>
<tr><td></td><td>3. Shear walls</td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete</td><td>8</td><td>240</td></tr>
<tr><td></td><td>b. Masonry</td><td>8</td><td>160</td></tr>
<tr><td></td><td>4. Ordinary braced frames</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>8</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[4]</td><td>8</td><td>—</td></tr>
<tr><td></td><td>c. Heavy timber</td><td>8</td><td>65</td></tr>
<tr><td></td><td>5. Special concentrically braced frames</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>9</td><td>240</td></tr>
<tr><td>3. Moment-resisting frame system</td><td>1. Special moment-resisting frames (SMRF)</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>12</td><td>N.L.</td></tr>
<tr><td></td><td>b. Concrete</td><td>12</td><td>N.L.</td></tr>
<tr><td></td><td>2. Masonry moment-resisting wall frame</td><td>9</td><td>160</td></tr>
<tr><td></td><td>3. Concrete intermediate moment-resisting frames (IMRF)[5]</td><td>8</td><td>—</td></tr>
<tr><td></td><td>4. Ordinary moment-resisting frames (OMRF)</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel[6]</td><td>6</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[7]</td><td>5</td><td>—</td></tr>
<tr><td>4. Dual systems</td><td>1. Shear walls</td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete with SMRF</td><td>12</td><td>N.L.</td></tr>
<tr><td></td><td>b. Concrete with steel OMRF</td><td>6</td><td>160</td></tr>
<tr><td></td><td>c. Concrete with concrete IMRF[5]</td><td>9</td><td>160</td></tr>
<tr><td></td><td>d. Masonry with SMRF</td><td>8</td><td>160</td></tr>
<tr><td></td><td>e. Masonry with steel OMRF</td><td>6</td><td>160</td></tr>
<tr><td></td><td>f. Masonry with concrete IMRF[4]</td><td>7</td><td>—</td></tr>
<tr><td></td><td>2. Steel EBF</td><td></td><td></td></tr>
<tr><td></td><td>a. With steel SMRF</td><td>12</td><td>N.L.</td></tr>
<tr><td></td><td>b. With steel OMRF</td><td>6</td><td>160</td></tr>
<tr><td></td><td>3. Ordinary braced frames</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel with steel SMRF</td><td>10</td><td>N.L.</td></tr>
<tr><td></td><td>b. Steel with steel OMRF</td><td>6</td><td>160</td></tr>
<tr><td></td><td>c. Concrete with concrete SMRF[4]</td><td>9</td><td>—</td></tr>
<tr><td></td><td>d. Concrete with concrete IMRF[4]</td><td>6</td><td>—</td></tr>
<tr><td></td><td>4. Special concentrically braced frames</td><td></td><td></td></tr>
<tr><td></td><td>a. Steel with steel SMRF</td><td>11</td><td>N.L.</td></tr>
<tr><td></td><td>b. Steel with steel OMRF</td><td>6</td><td>160</td></tr>
<tr><td>5. Undefined systems</td><td>See Sections 1627.8.3 and 1627.9.2</td><td>—</td><td>—</td></tr>
</table>
</td><td>$R_w = 8$</td><td>$R_w = 6$</td><td>Factors change to “R_w”</td></tr>
<tr><td>Uniform Building Code (1997)</td><td>
TABLE 16-N—STRUCTURAL SYSTEMS[1]
<table>
<tr><th>BASIC STRUCTURAL SYSTEM[2]</th><th>LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION</th><th>R</th><th>Ω_o</th><th>HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet)
× 304.8 for mm</th></tr>
<tr><td>1. Bearing wall system</td><td>1. Light-framed walls with shear panels</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Wood structural panel walls for structures three stories or less</td><td>5.5</td><td>2.8</td><td>65</td></tr>
<tr><td></td><td>b. All other light-framed walls</td><td>4.5</td><td>2.8</td><td>65</td></tr>
<tr><td></td><td>2. Shear walls</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete</td><td>4.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>b. Masonry</td><td>4.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>3. Light steel-framed bearing walls with tension-only bracing</td><td>2.8</td><td>2.2</td><td>65</td></tr>
<tr><td></td><td>4. Braced frames where bracing carries gravity load</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>4.4</td><td>2.2</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[3]</td><td>2.8</td><td>2.2</td><td>—</td></tr>
<tr><td></td><td>c. Heavy timber</td><td>2.8</td><td>2.2</td><td>65</td></tr>
<tr><td>2. Building frame system</td><td>1. Steel eccentrically braced frame (EBF)</td><td>7.0</td><td>2.8</td><td>240</td></tr>
<tr><td></td><td>2. Light-framed walls with shear panels</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Wood structural panel walls for structures three stories or less</td><td>6.5</td><td>2.8</td><td>65</td></tr>
<tr><td></td><td>b. All other light-framed walls</td><td>5.0</td><td>2.8</td><td>65</td></tr>
<tr><td></td><td>3. Shear walls</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete</td><td>5.5</td><td>2.8</td><td>240</td></tr>
<tr><td></td><td>b. Masonry</td><td>5.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>4. Ordinary braced frames</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>5.6</td><td>2.2</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[3]</td><td>5.6</td><td>2.2</td><td>—</td></tr>
<tr><td></td><td>c. Heavy timber</td><td>5.6</td><td>2.2</td><td>65</td></tr>
<tr><td></td><td>5. Special concentrically braced frames</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>6.4</td><td>2.2</td><td>240</td></tr>
<tr><td>3. Moment-resisting frame system</td><td>1. Special moment-resisting frame (SMRF)</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel</td><td>8.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>b. Concrete[4]</td><td>8.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>2. Masonry moment-resisting wall frame (MMRWF)</td><td>6.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>3. Concrete intermediate moment-resisting frame (IMRF)[5]</td><td>5.5</td><td>2.8</td><td>—</td></tr>
<tr><td></td><td>4. Ordinary moment-resisting frame (OMRF)</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel[6]</td><td>4.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>b. Concrete[7]</td><td>3.5</td><td>2.8</td><td>—</td></tr>
<tr><td></td><td>5. Special truss moment frames of steel (STMF)</td><td>6.5</td><td>2.8</td><td>240</td></tr>
<tr><td>4. Dual systems</td><td>1. Shear walls</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Concrete with SMRF</td><td>8.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>b. Concrete with steel OMRF</td><td>4.2</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>c. Concrete with concrete IMRF[5]</td><td>6.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>d. Masonry with SMRF</td><td>5.5</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>e. Masonry with steel OMRF</td><td>4.2</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>f. Masonry with concrete IMRF[3]</td><td>4.2</td><td>2.8</td><td>—</td></tr>
<tr><td></td><td>g. Masonry with masonry MMRWF</td><td>6.0</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>2. Steel EBF</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. With steel SMRF</td><td>8.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>b. With steel OMRF</td><td>4.2</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>3. Ordinary braced frames</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel with steel SMRF</td><td>6.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>b. Steel with steel OMRF</td><td>4.2</td><td>2.8</td><td>160</td></tr>
<tr><td></td><td>c. Concrete with concrete SMRF[3]</td><td>6.5</td><td>2.8</td><td>—</td></tr>
<tr><td></td><td>d. Concrete with concrete IMRF[3]</td><td>4.2</td><td>2.8</td><td>—</td></tr>
<tr><td></td><td>4. Special concentrically braced frames</td><td></td><td></td><td></td></tr>
<tr><td></td><td>a. Steel with steel SMRF</td><td>7.5</td><td>2.8</td><td>N.L.</td></tr>
<tr><td></td><td>b. Steel with steel OMRF</td><td>4.2</td><td>2.8</td><td>160</td></tr>
<tr><td>5. Cantilevered column building systems</td><td>1. Cantilevered column elements</td><td>2.2</td><td>2.0</td><td>35[6]</td></tr>
<tr><td>6. Shear wall-frame interaction systems</td><td>1. Concrete[8]</td><td>5.5</td><td>2.8</td><td>160</td></tr>
<tr><td>7. Undefined systems</td><td>See Sections 1629.6.7 and 1629.9.2</td><td>—</td><td>—</td><td>—</td></tr>
</table>
</td><td>$R = 5.5$</td><td>$R = 4.5$</td><td>Factors change to “R”</td></tr>
</table>

Table B-4 Building Code Evolution: *R* Factors (continued)

UBC/IBC Code Year	Code Excerpts	Plywood Factor	Stucco Factor	Notes & Observations
International Building Code (2000)	TABLE 1617.6 DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS	*R* = 6	*R* = 2	Beginning of large increase in factor difference

TABLE 1617.6
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_0^g	DEFLECTION AMPLIFICATION FACTOR, C_d^b	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
1. Bearing Wall Systems									
A. Ordinary steel braced frames	(14)[j] 2211	4	2	3 1/2	NL	NL	160	160	160
B. Special reinforced concrete shear walls	1910.2.4	5 1/2	2 1/2	5	NL	NL	160	160	100
C. Ordinary reinforced concrete shear walls	1910.2.3	4 1/2	2 1/2	4	NL	NL	NP	NP	NP
D. Detailed plain concrete shear walls	1910.2.2	2 1/2	2 1/2	2	NL	NP	NP	NP	NP
E. Ordinary plain concrete shear walls	1910.2.1	1 1/2	2 1/2	1 1/2	NL	NP	NP	NP	NP
F. Special reinforced masonry shear walls	2106.1.1.5	5	2 1/2	3 1/2	NL	NL	160	160	100
G. Intermediate reinforced masonry shear walls	2106.1.1.4	3 1/2	2 1/2	2 1/4	NL	NL	NP	NP	NP
H. Ordinary reinforced masonry shear walls	2106.1.1.2	2 1/2	2 1/2	1 3/4	NL	160	NP	NP	NP
I. Detailed plain masonry shear walls	2106.1.1.3	2	2 1/2	1 3/4	NL	NP	NP	NP	NP
J. Ordinary plain masonry shear walls	2106.1.1.1	1 1/2	2 1/2	1 1/4	NL	NP	NP	NP	NP
K. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	6	3	4	NL	NL	65	65	65
L. Light frame walls with shear panels—all other materials	2306.4.5	2	2 1/2	2	NL	NL	35	NP	NP
2. Building Frame Systems									
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	(15)[j]	8	2	4	NL	NL	160	160	100
B. Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	(15)[j]	7	2	4	NL	NL	160	160	100
C. Special steel concentrically braced frames	(13)[j]	6	2	5	NL	NL	160	160	100
D. Ordinary steel concentrically braced frames	(14)[j]	5	2	4 1/2	NL	NL	160	100	100
E. Special reinforced concrete shear walls	1910.2.4	6	2 1/2	5	NL	NL	160	160	100
F. Ordinary reinforced concrete shear walls	1910.2.3	5	2 1/2	4 1/2	NL	NL	NP	NP	NP
G. Detailed plain concrete shear walls	1910.2.2	3	2 1/2	2 1/2	NL	NP	NP	NP	NP
H. Ordinary plain concrete shear walls	1910.2.1	2	2 1/2	2	NL	NP	NP	NP	NP
I. Composite eccentrically braced frames	(14)[k]	8	2	4	NL	NL	160	160	100

B.2.5 *K* Factor Provisions

Table B-5 Building Code Evolution: *K* Factors

<table>
<tr><th>UBC/IBC Code Year</th><th>Plywood Factor</th><th>Stucco Factor</th></tr>
<tr><td>1973</td><td>1.33</td><td rowspan="2">1.33</td></tr>
<tr><td>1979</td><td>1.00</td></tr>
<tr><td>1985</td><td>0.13</td><td>0.17</td></tr>
<tr><td>1994</td><td>0.18</td><td>0.22</td></tr>
<tr><td>1997</td><td>0.17</td><td rowspan="3">0.50</td></tr>
<tr><td>2000</td><td rowspan="2">0.15</td></tr>
<tr><td>2006</td></tr>
</table>

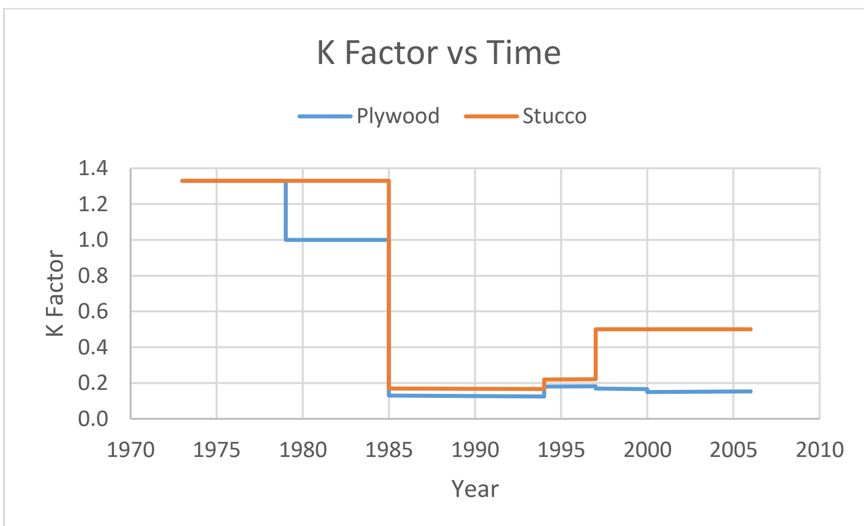

Figure B-1 *K* Factor versus time.

B.2.6 Base Shear Coefficient Provisions

Table B-6 Building Code Evolution: Maximum Base Shear Coefficient

UBC/IBC Code Year	Plywood Coefficient	Stucco Coefficient
1973	0.133	0.133
1979	0.186	0.186
1985	0.140	
1994	0.138	0.184
1997	0.157	0.192
2000	0.194	0.582
2006	0.179	

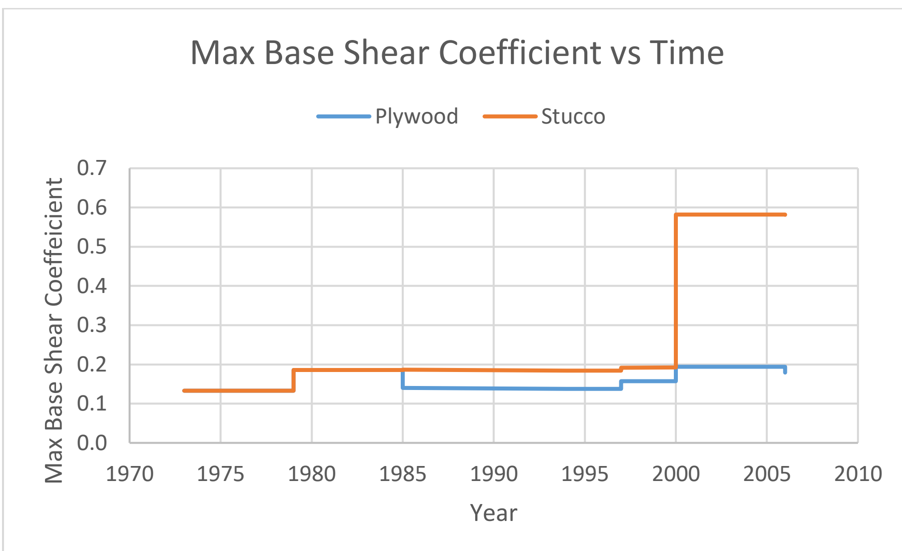

Figure B-2 Maximum base shear coefficient versus time.

Appendix C: Modeling Report

C.1 Overview

Two categories of archetype buildings (long-side open, LO, and short-side open, SO) were analyzed to assess the seismic performance of SWOF buildings. Each category has two- and three-story versions, a variety of wall and diaphragm materials, and various types of retrofits. The walls and diaphragms were modeled with nonlinear material properties, except when rigid diaphragms were used. The analyses combined state-of-the-art information about SWOF building material properties with advanced methods in nonlinear dynamic analyses. Three-dimensional models were used to run pushover studies and incremental dynamic analyses, per the FEMA P-695 protocol. Collapse was modeled explicitly up to 20% drift. In the few cases where models had capacity at 20% drift, the analyses were terminated to account for non-simulated failure modes of the gravity system. This appendix describes the structural modeling and documents model performance in terms of probability of collapse given different levels of spectral acceleration.

C.2 General Modeling Strategy

The structures were modeled with the OpenSees software platform (McKenna et al., 2000). The walls, diaphragms, and retrofit frames were represented as assemblages of lumped-plasticity nonlinear springs connected to lumped masses. The springs were calibrated to physical tests of the representative wall and diaphragm materials (Welch and Deierlein, 2020, and other sources noted in Chapter 2). The springs have appropriate nonlinear behavior for in-plane shear (the behavior of interest), high elastic stiffness for in-plane flexural and axial modes, and negligible stiffness for out-of-plane modes. This is a widely used and computationally efficient approach to simulate the seismic behavior of wood-structures (Rosowsky, 2002; Folz and Filiatrault, 2004(a)(b); Christovasilis and Filiatrault, 2009; Goda and Atkinson, 2010; van de Lindt et al., 2010; Welch and Deierlein, 2020). Figure C-1 shows the layout and geometry of an idealized structure, and it illustrates the overall modeling approach. The detailed geometry idealization and material calibration of each element is described in the following subsections.

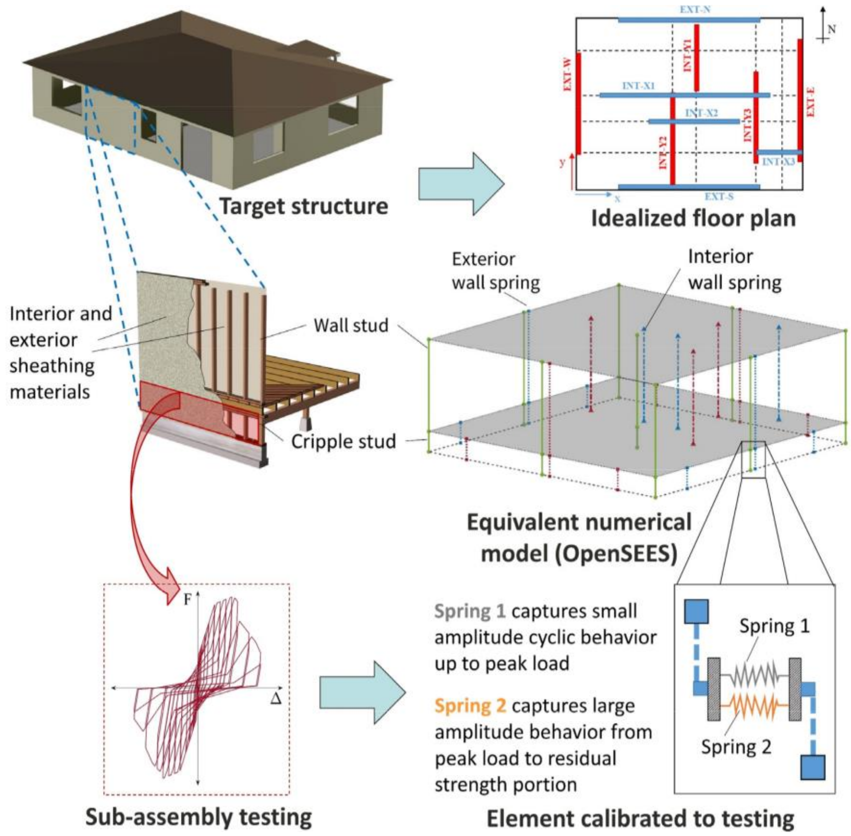

Figure C-1 Illustration of the three-dimensional macro-element modeling concept (image credit: Welch and Deierlein, 2020).

C.3 Modeling of Superstructure

The lumped masses are assigned to the nodes. The nodes are connected by the nonlinear springs (walls and diaphragms). Figure C-2 shows the SO3 archetype on the right and the LO3 archetype on the left. The nodes are connected using OpenSees *twoNodeLink* elements (nonlinear springs). The red springs are exterior walls, the blue springs are interior walls, and the yellow springs are diaphragms. The open fronts of both archetypes (LO and SO) are in the X-direction. The story heights are 9 feet and the global coordinate origins are the bottom left corners of the buildings. The node naming convention is a six-digit number. The first two digits represent the vertical level of the node (10: base; 11: second level; 12: third level; 13: fourth level).The next two digits represent nodes located on gridlines along the Y axis (e.g., GL A: 01, GL B: 02). The next single digit represents the nodes located on gridlines along the X axis (e.g., GL 1: 1, GL 2: 2). The last digit is set to be zero unless multiple nodes are required at the same location. Base nodes (nodes at the 0 height) are fixed in all six degrees of freedom.

The steel frame used in the retrofitted structures is also modeled with a *twoNodeLink* element. The element has the nonlinear behavior assigned to the translational X degree of freedom. The spring has no out-of-plane stiffness, emulating the flexible out-of-plane behavior of the frame.

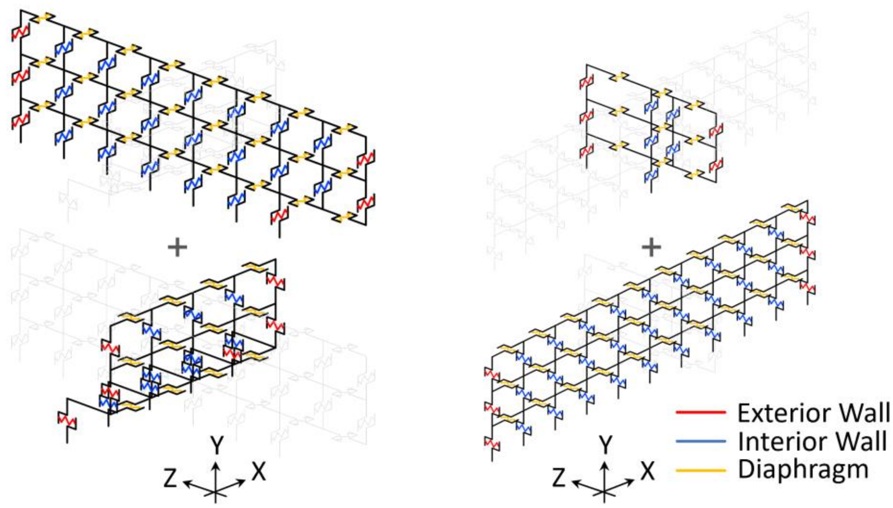

Figure C-2 Model of SO3 archetype (left) and model of LO3 archetype (right).

C.3.1 Modeling Walls and Frames

Table C-1 and Table C-2 provide node connectivity, element names, element lengths, wall types, and element directions for the walls and frames of the three-story LO and SO archetypes. The two-story archetypes are similar.

Table C-1 Summary of Wall Elements for Modeling the LO Archetype

Wall Type	Story	Direction	Element ID	iNode	jNode	X (ft)	Y (ft)	L_{wall} (ft)
Exterior	1	Y	W1-A	100130	110130	0.0	18.0	36.0
Interior	1	Y	W1-B	100230	110230	10.0	18.0	6.0
Interior	1	Y	W1-C	100330	110330	20.0	18.0	6.0
Interior	1	Y	W1-D	100430	110430	30.0	18.0	16.0
Interior	1	Y	W1-E	100530	110530	40.0	18.0	5.4
Interior	1	Y	W1-F	100630	110630	50.0	18.0	43.2
Interior	1	Y	W1-G	100730	110730	60.0	18.0	5.4
Interior	1	Y	W1-H	100830	110830	70.0	18.0	16.0
Interior	1	Y	W1-I	100930	110930	80.0	18.0	6.0
Interior	1	Y	W1-J	101030	111030	90.0	18.0	6.0
Exterior	1	Y	W1-K	101130	111130	100.0	18.0	36.0

Table C-1 Summary of Wall Elements for Modeling the LO Archetype (continued)

Wall Type	Story	Direction	Element ID	iNode	jNode	X (ft)	Y (ft)	L_{wall} (ft)
Interior	1	X	W1-3	100630	110630	50.0	18.0	118.0
Exterior	1	X	W1-4	100640	110640	50.0	36.0	60.0
Exterior	2	Y	W2-A	110130	120130	0.0	18.0	36.0
Interior	2	Y	W2-B	110230	120230	10.0	18.0	20.0
Interior	2	Y	W2-C	110330	120330	20.0	18.0	36.0
Interior	2	Y	W2-D	110430	120430	30.0	18.0	20.0
Interior	2	Y	W2-E	110530	120530	40.0	18.0	36.0
Interior	2	Y	W2-F	110630	120630	50.0	18.0	20.0
Interior	2	Y	W2-G	110730	120730	60.0	18.0	36.0
Interior	2	Y	W2-H	110830	120830	70.0	18.0	20.0
Interior	2	Y	W2-I	110930	120930	80.0	18.0	36.0
Interior	2	Y	W2-J	111030	121030	90.0	18.0	20.0
Exterior	2	Y	W2-K	111130	121130	100.0	18.0	36.0
Exterior	2	X	W2-1	110610	120610	50.0	0.0	40.0
Interior	2	X	W2-2	110620	120620	50.0	12.0	60.0
Interior	2	X	W2-3	110630	120630	50.0	18.0	80.0
Exterior	2	X	W2-4	110640	120640	50.0	36.0	60.0
Exterior	3	Y	W3-A	120130	130130	0.0	18.0	36.0
Interior	3	Y	W3-B	120230	130230	10.0	18.0	20.0
Interior	3	Y	W3-C	120330	130330	20.0	18.0	36.0
Interior	3	Y	W3-D	120430	130430	30.0	18.0	20.0
Interior	3	Y	W3-E	120530	130530	40.0	18.0	36.0
Interior	3	Y	W3-F	120630	130630	50.0	18.0	20.0
Interior	3	Y	W3-G	120730	130730	60.0	18.0	36.0
Interior	3	Y	W3-H	120830	130830	70.0	18.0	20.0
Interior	3	Y	W3-I	120930	130930	80.0	18.0	36.0

Table C-1 Summary of Wall Elements for Modeling the LO Archetype (continued)

Wall Type	Story	Direction	Element ID	iNode	jNode	X (ft)	Y (ft)	L_{wall} (ft)
Interior	3	Y	W3-J	121030	131030	90.0	18.0	20.0
Exterior	3	Y	W3-K	121130	131130	100.0	18.0	36.0
Exterior	3	X	W3-1	120610	130610	50.0	0.0	40.0
Interior	3	X	W3-2	120620	130620	50.0	12.0	60.0
Interior	3	X	W3-3	120630	130630	50.0	18.0	80.0
Exterior	3	X	W3-4	120640	130640	50.0	36.0	60.0

Table C-2 Summary of Wall Elements for Modeling the SO Archetype

Wall Type	Story	Direction	Element ID	iNode	jNode	X (ft)	Y (ft)	L_{wall} (ft)
Exterior	1	Y	W1-A	100160	110160	0.0	50.0	30.0
Interior	1	Y	W1-B	100260	110260	10.0	50.0	28.6
Interior	1	Y	W1-C	100360	110360	20.0	50.0	28.6
Interior	1	Y	W1-D	100460	110460	30.0	50.0	28.6
Exterior	1	Y	W1-E	100560	110560	40.0	50.0	30.0
Exterior	1	X	W1-3	100330	110330	20.0	20.0	40.0
Interior	1	X	W1-4	100340	110340	20.0	30.0	19.05
Interior	1	X	W1-5	100350	110350	20.0	40.0	19.05
Interior	1	X	W1-6	100360	110360	20.0	50.0	19.05
Interior	1	X	W1-7	100370	110370	20.0	60.0	19.05
Interior	1	X	W1-8	100380	110380	20.0	70.0	19.05
Exterior	1	X	W1-9	100390	110390	20.0	80.0	20.0
Exterior	2	Y	W2-A	110150	120150	0.0	40.0	40.0
Interior	2	Y	W2-B	110250	120250	10.0	40.0	38.1
Interior	2	Y	W2-C	110350	120350	20.0	40.0	38.1
Interior	2	Y	W2-D	110450	120450	30.0	40.0	38.1
Exterior	2	Y	W2-E	110550	120550	40.0	40.0	40.0

Table C-2 Summary of Wall Elements for Modeling the SO Archetype (continued)

Wall Type	Story	Direction	Element ID	iNode	jNode	X (ft)	Y (ft)	L_{wall} (ft)
Exterior	2	X	W2-1	110310	120310	20.0	00.0	20.0
Interior	2	X	W2-2	110320	120320	20.0	10.0	19.05
Interior	2	X	W2-3	110330	120330	20.0	20.0	19.05
Interior	2	X	W2-4	110340	120340	20.0	30.0	19.05
Interior	2	X	W2-5	110350	120350	20.0	40.0	19.05
Interior	2	X	W2-6	110360	120360	20.0	50.0	19.05
Interior	2	X	W2-7	110370	120370	20.0	60.0	19.05
Interior	2	X	W2-8	110380	120380	20.0	70.0	19.05
Exterior	2	X	W2-9	110390	120390	20.0	80.0	20.0
Exterior	3	Y	W3-A	120150	130150	0.0	40.0	40.0
Interior	3	Y	W3-B	120250	130250	10.0	40.0	38.1
Interior	3	Y	W3-C	120350	130350	20.0	40.0	38.1
Interior	3	Y	W3-D	120450	130450	30.0	40.0	38.1
Exterior	3	Y	W3-E	120550	130550	40.0	40.0	40.0
Exterior	3	X	W3-1	120310	130310	20.0	00.0	20.0
Interior	3	X	W3-2	120320	130320	20.0	10.0	19.05
Interior	3	X	W3-3	120330	130330	20.0	20.0	19.05
Interior	3	X	W3-4	120340	130340	20.0	30.0	19.05
Interior	3	X	W3-5	120350	130350	20.0	40.0	19.05
Interior	3	X	W3-6	120360	130360	20.0	50.0	19.05
Interior	3	X	W3-7	120370	130370	20.0	60.0	19.05
Interior	3	X	W3-8	120380	130380	20.0	70.0	19.05
Exterior	3	X	W3-9	120390	130390	20.0	80.0	20.0

C.3.2 Modeling Diaphragms

Table C-3 and Table C-4 provide node connectivity, diaphragm element names, effective lengths, diaphragm types, and element directions for the diaphragms for the three-story LO and SO archetypes. The two-story archetypes are similar.

Table C-3 Summary of Diaphragm Elements for Modeling the LO Archetype

Level	Direction	Element ID	iNode	X (ft)	Y (ft)	jNode	X (ft)	Y (ft)	L_{Diaph} (ft)
2nd	Y	Diaph1-AB	110130	0.0	18.0	110230	10.0	18.0	36.0
2nd	Y	Diaph1-BC	110230	10.0	18.0	110330	20.0	18.0	36.0
2nd	Y	Diaph1-CD	110330	20.0	18.0	110430	30.0	18.0	36.0
2nd	Y	Diaph1-DE	110430	30.0	18.0	110530	40.0	18.0	36.0
2nd	Y	Diaph1-EF	110530	40.0	18.0	110630	50.0	18.0	36.0
2nd	Y	Diaph1-FG	110630	50.0	18.0	110730	60.0	18.0	36.0
2nd	Y	Diaph1-GH	110730	60.0	18.0	110830	70.0	18.0	36.0
2nd	Y	Diaph1-HI	110830	70.0	18.0	110930	80.0	18.0	36.0
2nd	Y	Diaph1-IJ	110930	80.0	18.0	111030	90.0	18.0	36.0
2nd	Y	Diaph1-JK	111030	90.0	18.0	111130	100.0	18.0	36.0
2nd	X	Diaph1-12	110610	50.0	0.0	110620	50.0	12.0	100.0
2nd	X	Diaph1-23	110620	50.0	12.0	110630	50.0	18.0	100.0
2nd	X	Diaph1-34	110630	50.0	12.0	110640	50.0	36.0	100.0
3rd	Y	Diaph2-AB	120130	0.0	18.0	120230	10.0	18.0	36.0
3rd	Y	Diaph2-BC	120230	10.0	18.0	120330	20.0	18.0	36.0
3rd	Y	Diaph2-CD	120330	20.0	18.0	120430	30.0	18.0	36.0
3rd	Y	Diaph2-DE	120430	30.0	18.0	120530	40.0	18.0	36.0
3rd	Y	Diaph2-EF	120530	40.0	18.0	120630	50.0	18.0	36.0
3rd	Y	Diaph2-FG	120630	50.0	18.0	120730	60.0	18.0	36.0
3rd	Y	Diaph2-GH	120730	60.0	18.0	120830	70.0	18.0	36.0
3rd	Y	Diaph2-HI	120830	70.0	18.0	120930	80.0	18.0	36.0
3rd	Y	Diaph2-IJ	120930	80.0	18.0	121030	90.0	18.0	36.0
3rd	Y	Diaph2-JK	121030	90.0	18.0	121130	100.0	18.0	36.0

Table C-3 Summary of Diaphragm Elements for Modeling the LO Archetype (continued)

Level	Direction	Element ID	iNode	X (ft)	Y (ft)	jNode	X (ft)	Y (ft)	L_{Diaph} (ft)
3rd	X	Diaph2-12	120610	50.0	0.0	120620	50.0	12.0	100.0
3rd	X	Diaph2-23	120620	50.0	12.0	120630	50.0	18.0	100.0
3rd	X	Diaph2-34	120630	50.0	12.0	120640	50.0	36.0	100.0
4th	Y	Diaph3-AB	130130	0.0	18.0	130230	10.0	18.0	36.0
4th	Y	Diaph3-BC	130230	10.0	18.0	130330	20.0	18.0	36.0
4th	Y	Diaph3-CD	130330	20.0	18.0	130430	30.0	18.0	36.0
4th	Y	Diaph3-DE	130430	30.0	18.0	130530	40.0	18.0	36.0
4th	Y	Diaph3-EF	130530	40.0	18.0	130630	50.0	18.0	36.0
4th	Y	Diaph3-FG	130630	50.0	18.0	130730	60.0	18.0	36.0
4th	Y	Diaph3-GH	130730	60.0	18.0	130830	70.0	18.0	36.0
4th	Y	Diaph3-HI	130830	70.0	18.0	130930	80.0	18.0	36.0
4th	Y	Diaph3-IJ	130930	80.0	18.0	131030	90.0	18.0	36.0
4th	Y	Diaph3-JK	131030	90.0	18.0	131130	100.0	18.0	36.0
4th	X	Diaph3-12	130610	50.0	0.0	130620	50.0	12.0	100.0
4th	X	Diaph3-23	130620	50.0	12.0	130630	50.0	18.0	100.0
4th	X	Diaph3-34	130630	50.0	12.0	130640	50.0	36.0	100.0

Table C-4 Summary of Diaphragm Elements for Modeling SO Archetype

Level	Direction	Element ID	iNode	X (ft)	Y (ft)	jNode	X (ft)	Y (ft)	L_{Diaph} (ft)
2nd	Y	Diaph1-AB	110150	0.0	40.0	110250	10.0	40.0	80.0
2nd	Y	Diaph1-BC	110250	10.0	40.0	110350	20.0	40.0	80.0
2nd	Y	Diaph1-CD	110350	20.0	40.0	110450	30.0	40.0	80.0
2nd	Y	Diaph1-DE	110450	30.0	40.0	110550	40.0	40.0	80.0
2nd	X	Diaph1-12	110310	20.0	00.0	110320	20.0	10.0	40.0
2nd	X	Diaph1-23	110320	20.0	10.0	110330	20.0	20.0	40.0
2nd	X	Diaph1-34	110330	20.0	20.0	110340	20.0	30.0	40.0

Table C-4 Summary of Diaphragm Elements for Modeling SO Archetype (continued)

Level	Direction	Element ID	iNode	X (ft)	Y (ft)	jNode	X (ft)	Y (ft)	L_{Diaph} (ft)
2nd	X	Diaph1-45	110340	20.0	30.0	110350	20.0	40.0	40.0
2nd	X	Diaph1-56	110350	20.0	40.0	110360	20.0	50.0	40.0
2nd	X	Diaph1-67	110360	20.0	50.0	110370	20.0	60.0	40.0
2nd	X	Diaph1-78	110370	20.0	60.0	110380	20.0	70.0	40.0
2nd	X	Diaph1-89	110380	20.0	70.0	110390	20.0	80.0	40.0
3rd	Y	Diaph2-AB	120150	0.0	40.0	120250	10.0	40.0	80.0
3rd	Y	Diaph2-BC	120250	10.0	40.0	120350	20.0	40.0	80.0
3rd	Y	Diaph2-CD	120350	20.0	40.0	120450	30.0	40.0	80.0
3rd	Y	Diaph2-DE	120450	30.0	40.0	120550	40.0	40.0	80.0
3rd	X	Diaph2-12	120310	20.0	00.0	120320	20.0	10.0	40.0
3rd	X	Diaph2-23	120320	20.0	10.0	120330	20.0	20.0	40.0
3rd	X	Diaph2-34	120330	20.0	20.0	120340	20.0	30.0	40.0
3rd	X	Diaph2-45	120340	20.0	30.0	120350	20.0	40.0	40.0
3rd	X	Diaph2-56	120350	20.0	40.0	120360	20.0	50.0	40.0
3rd	X	Diaph2-67	120360	20.0	50.0	120370	20.0	60.0	40.0
3rd	X	Diaph2-78	120370	20.0	60.0	120380	20.0	70.0	40.0
3rd	X	Diaph2-89	120380	20.0	70.0	120390	20.0	80.0	40.0
4th	Y	Diaph3-AB	130150	0.0	40.0	130250	10.0	40.0	80.0
4th	Y	Diaph3-BC	130250	10.0	40.0	130350	20.0	40.0	80.0
4th	Y	Diaph3-CD	130350	20.0	40.0	130450	30.0	40.0	80.0
4th	Y	Diaph3-DE	130450	30.0	40.0	130550	40.0	40.0	80.0
4th	X	Diaph3-12	130310	20.0	00.0	130320	20.0	10.0	40.0
4th	X	Diaph3-23	130320	20.0	10.0	130330	20.0	20.0	40.0
4th	X	Diaph3-34	130330	20.0	20.0	130340	20.0	30.0	40.0
4th	X	Diaph3-45	130340	20.0	30.0	130350	20.0	40.0	40.0
4th	X	Diaph3-56	130350	20.0	40.0	130360	20.0	50.0	40.0

Table C-4 Summary of Diaphragm Elements for Modeling SO Archetype (continued)

Level	Direction	Element ID	iNode	X (ft)	Y (ft)	jNode	X (ft)	Y (ft)	L_{Diaph} (ft)
4th	X	Diaph3-67	130360	20.0	50.0	130370	20.0	60.0	40.0
4th	X	Diaph3-78	130370	20.0	60.0	130380	20.0	70.0	40.0
4th	X	Diaph3-89	130380	20.0	70.0	130390	20.0	80.0	40.0

C.4 Wall and Diaphragm Materials

The wall and diaphragm material properties represent construction materials from the 1930s through the 1970s. Two primary material sets were used for the walls and diaphragms. One set has strong walls, SW (stucco and plaster), and weak diaphragms, WD (straight wood sheathing). The second set has weak walls, WW (stucco and gypsum wallboard), and strong diaphragms, SD (diagonal wood sheathing). The material load-deformation backbone curves for wall and diaphragm materials are expressed in normalized units. Shear capacity is expressed in pounds per linear foot (plf; lb/ft). Displacements are expressed in terms of drift ratio (e.g., displacement divided by wall height).

Exterior strong walls (SW) have a stucco exterior layer and a gypsum plaster on wood lath interior layer. The peak strength is 1050 plf at 1.2% of drift. Interior strong walls (SW) have two layers of plaster on wood lath with the peak strength of 890 plf at 1.2% drift. Exterior weak walls (WW) have a stucco exterior layer and a gypsum wallboard interior layer. The peak strength is 800 plf at 1.5% of drift. Interior weak walls (WW) have two layers of gypsum wallboard and a peak strength of 420 plf at 0.8% drift.

The weak diaphragms (WD) are based straight wood sheathing and have a peak strength of 177 plf at 4.0% drift. The strong diaphragms (SD) are based diagonal wood sheathing. The diaphragms have asymmetric behavior in tension and compression to the orientation of the lumber sheathing. The peak strength in compression is 1028 plf at 1.87%. The peak strength in tensional is 507 plf at 3.2%. Table C-5 summarizes the peak capacities of the diaphragms used for the primary study.

Table C-5 Summary of Diaphragm Types and Peak Strengths

Element			Peak Strength (plf)		Drift @ Peak (%)	
			+	-	+	-
Wall	Exterior	Strong	1050	1050	1.2	1.2
		Weak	800	800	1.5	1.5
	Interior	Strong	890	890	1.2	1.2
		Weak	420	420	0.8	0.8
Diaphragm		Strong	507	1028	3.2	1.87
		Weak	177	177	4.0	4.0

C.4.1 Hysteretic Material Behavior

The analysis utilized the OpenSees *Pinching4* material backbone (Lowes et al., 2004). The backbone of the material is specified by four stress and strain values in each direction (positive and negative). As depicted in Figure C-3, the material allows for pinching behavior to vary based on maximum displacement histories during the back and forth loading through the three controlling parameters: *rForce*, *rDisp*, and *uForce*.

In addition, the *Pinching4* material employs three different types of degradation: (1) reloading stiffness degradation, (2) unloading stiffness degradation, and (3) force (strength) degradation. Each of these degradation types can be controlled by four parameters. The general damage index associated with each of these degradation types is calculated based on the following equation:

$$\delta_i = g_1\left(d_{\max}\right)^{g_3} + g_2\left(E_i/E_{mono}\right)^{g_4} < g_{\lim} \tag{C-1}$$

Where δ_i is the damage index of the i^{th} increment between 0 to 1, and d_{max}, E_i, and E_{mono} are the maximum displacement (strain) in the history, the hysteretic energy dissipated in the i^{th} increment, and the total monotonic energy of the material backbone, respectively. The user can control the rate of the degradation using parameters g_1 through g_4 and limit the total degradation using the g_{lim} parameter. These parameters provide a wide range of control, making *Pinching4* well suited to emulate complex hysteretic behavior.

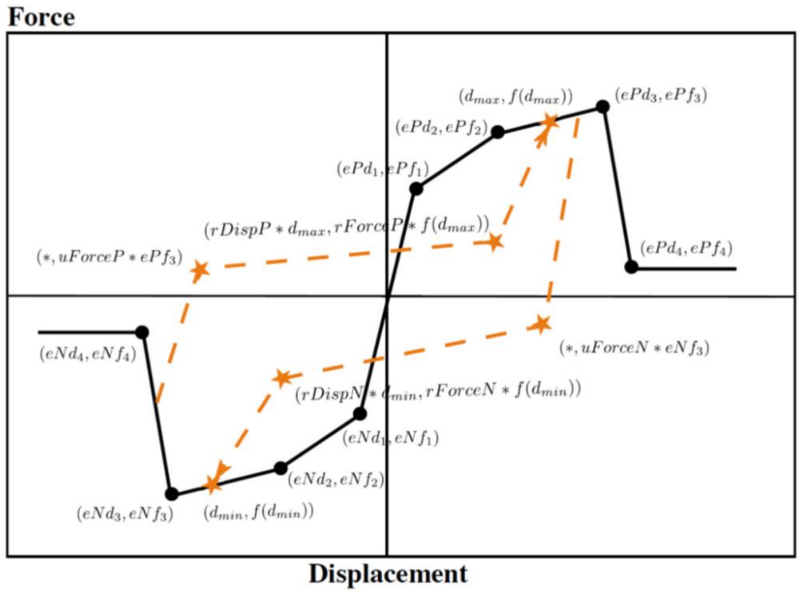

Figure C-3 ***Pinching4*** **material backbone and cyclic loading definitions (image credit: Acevedo, 2018).**

C.4.1.1 PARALLEL SPRING APPROACH

Physical testing of the representative wall and diaphragm materials used for the archetypes reveals brittle behavior of the cyclic load-deformation patterns. These architectural finish materials are stiff and strong up to the peak point and after that, the strength drops significantly. To emulate the rapid changes in the cyclic behavior before and after the peak point, the models use two *Pinching4* springs in parallel, as first proposed by Acevedo (2018).

Figure C-4 shows the combined backbone of the two parallel springs. Spring 1 is used to capture the cyclic behavior of smaller drifts, prior to significant damage. Spring 2 is used to emulate the behavior from peak load to the residual strength stage of the response. The two springs are combined using the parallel *uniaxialMaterial* feature in OpenSees. Four weighted stresses (forces) were used with the two springs, where the *a*, *b*, *c*, and *d* factors were set to 0.8, 0.75, 0.3, and 0.1, respectively. These values were adopted from Welch and Deierlein (2020). Figure C-4 shows the effect of these factors on a sample material backbone.

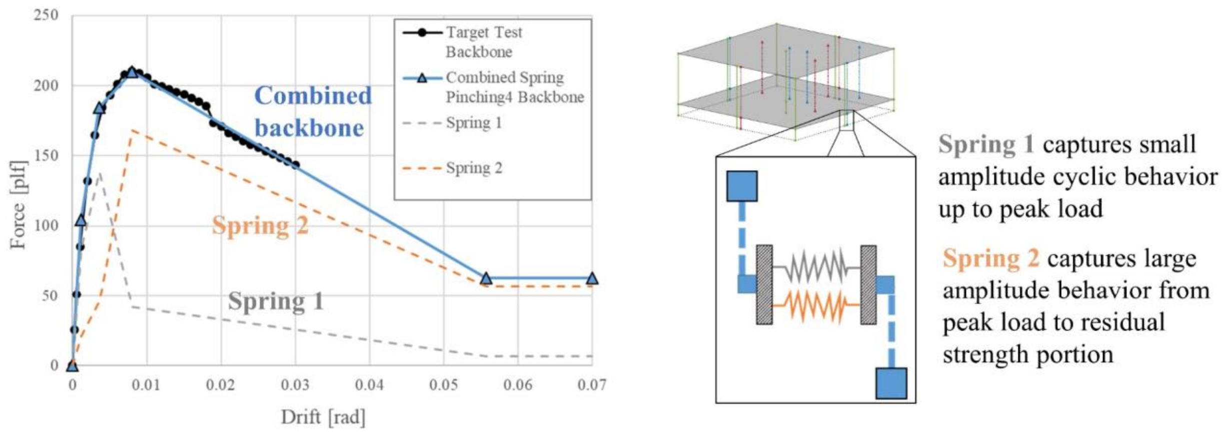

Figure C-4 Illustration of the parallel spring concept used to capture small and large displacement cyclic behavior for wall materials (image credit: Welch and Deierlein, 2020).

Table C-6 describes the modeling parameters used to emulate experimental material backbones in the material models.

Table C-6 Definition of Material Modeling Parameters using *Pinching4* Material Models

Pinching4 Parameter		Description
Backbone	(ed_1, ef_1)	Deformation (d) and force (f) defining initial stiffness of backbone curve
	(ed_2, ef_2)	Deformation (d) and force (f) defining "cracked" portion of backbone curve
	(ed_3, ef_3)	Deformation (d) and force (f) defining ultimate strength point on backbone curve
	(ed_4, ef_4)	Deformation (d) and force (f) defining the residual strength portion of backbone curve
Spring 1	$rDisp_1$	Ratio of deformation at which reloading occurs to the maximum historic deformation demand
	$rForce_1$	Ratio of force at which reloading occurs to the force corresponding to the maximum historic deformation demand
	$uForce_1$	Ratio of strength developed upon reversal of loading to the peak strength developed
	gD_{11}	Reloading stiffness degradation coefficient
	$gDLim_1$	Reloading stiffness degradation limit
	gK_{11}	Unloading stiffness degradation coefficient
	$gKLim_1$	Unloading stiffness degradation limit

Table C-6 Definition of Material Modeling Parameters using *Pinching4* Material Models (continued)

Pinching4 Parameter		Description
Spring 2	$rDisp_2$	Ratio of deformation at which reloading occurs to the maximum historic deformation demand
	$rForce_2$	Ratio of force at which reloading occurs to the force corresponding to the maximum historic deformation demand
	$uForce_2$	Ratio of strength developed upon reversal of loading to the peak strength developed
	gD_{12}	Reloading stiffness degradation coefficient
	$gDLim_2$	Reloading stiffness degradation limit
	gK_{12}	Unloading stiffness degradation coefficient
	$gKLim_2$	Unloading stiffness degradation limit

C.4.2 Numeric Material Calibration

The analyses employed reloading and unloading degradation at the material level as a function of displacement excursions. Only the scalar degradation factor (g_1) and degradation limit (g_{Lim}) were used for each of the two types of degradation used (where these parameters are the factors used in Equation C-1).

The first point on the *Pinching4* material (ed_1, ef_1) represents the initial stiffness of the experimental results. The second point on the backbone (ed_2, ef_2) is an intermediate point between the first and peak point on the backbone and can be thought of as the softening point in the material. ed_3 and ef_3 are used to match the strength of the material. The fourth point (ed_4, ef_4) is used to capture the residual strength. In the cases where experimental loading did not extend to the residual strength region, 30% of peak strength was used for the residual strength, as recommended in FEMA P-2139-2, *Short-Period Building Collapse Performance and Recommendations for Improving Seismic* Design (FEMA, 2020). The specific modeling parameters and force-versus-displacement plots are provided below for each material used.

C.4.3 Material Modeling Inputs

C.4.3.1 STRONG EXTERIOR WALLS

The modeling parameters for strong exterior walls, SW (exterior stucco with interior lath and plaster), were adopted from Welch and Deierlein (2020) and are provided in Table C-7. The resulting behavior is shown in Figure C-5.

Table C-7 Modeling Parameters for Exterior Stucco with Plaster on Wood Lath Interior

Material	Backbone deformation points (% drift)				Backbone force points (plf)			
	ed_1	ed_2	ed_3	ed_4	ef_1	ef_2	ef_3	ef_4
SLP3 (best estimate)	0.08	0.36	1.20	3.70	357	829	1050	315
Cyclic properties	Spring 1	$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
		0.06	0.26	-0.20	0	0	0.13	2.0
	Spring 2	$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
		0.06	0.17	-0.23	0.3	2.0	0.13	2.0

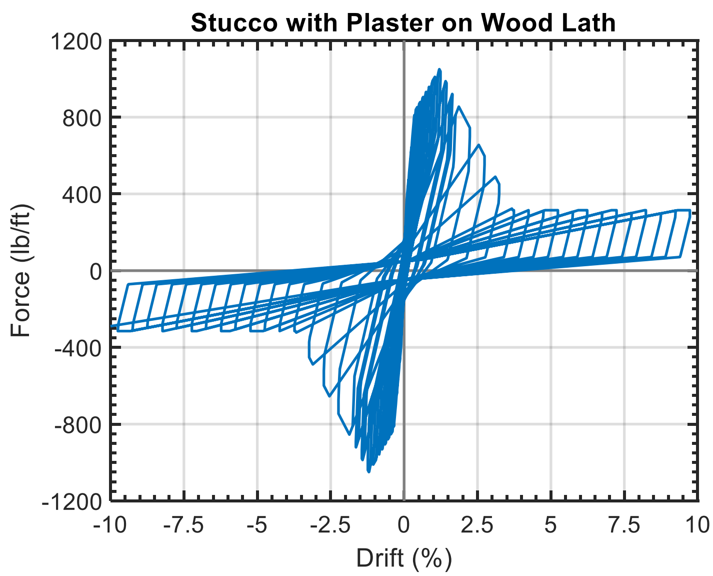

Figure C-5 Behavior of exterior stucco with plaster on wood lath interior material using *Pinching4* model.

C.4.3.2 STRONG INTERIOR WALLS

The modeling parameters for strong interior walls, SW (two layers of gypsum plaster on wood lath), were calibrated to best match the experimental results by Carroll (2006), and reported in Welch and Deierlein (2020). These were based on one layer of plaster on wood lath. The backbone force points were doubled for this project to reflect the two layers of material. See Table C-8 for the parameters used. The resulting behavior is shown in Figure C-6.

Table C-8 Modeling Parameters for Two Layers of Plaster on Wood Lath

Material		Backbone deformation points (% drift)				Backbone force points (plf)			
		ed_1	*ed_2*	*ed_3*	*ed_4*	*ef_1*	*ef_2*	*ef_3*	*ef_4*
LP2 (best estimate)		0.08	0.28	1.20	2.90	230	572	890	256
Cyclic properties	Spring 1	*$rDisp_1$*	*$rForce_1$*	*$uForce_1$*	*gK_{11}*	*$gKLim_1$*	*gD_{11}*	*$gDLim_1$*	
		0.06	0.31	-0.10	-0.07	-0.50	0.14	0.30	
	Spring 2	*$rDisp_2$*	*$rForce_2$*	*$uForce_2$*	*gK_{12}*	*$gKLim_2$*	*gD_{12}*	*$gDLim_2$*	
		0.28	0.18	-0.11	-0.05	-0.20	0.11	0.30	

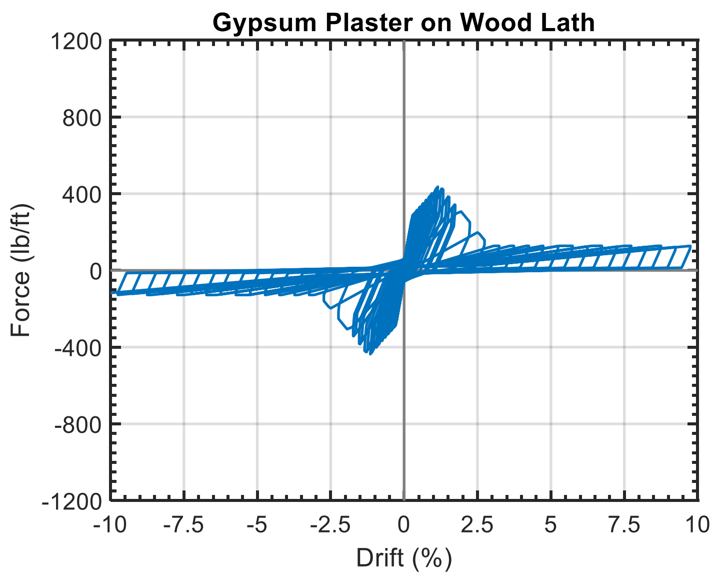

Figure C-6 Behavior of stucco plus plaster on wood lath material using the *Pinching4* model.

C.4.3.3 WEAK EXTERIOR WALLS

The modeling parameters for exterior weak walls, WW (stucco and gypsum wallboard), were based on experimental behavior obtained from FEMA P-1100, *Vulnerability-Based Seismic Assessment and Retrofit of One- and Two-Family* Dwellings (FEMA, 2019). That study interpreted available testing of stucco and gypsum wallboard panels and generated best-estimate values, which were used here. The values are shown in Table C-9. The resulting behavior is shown in Figure C-7.

Table C-9 Modeling Parameters for Stucco Plus Gypsum Wallboard

Material	Backbone deformation points (% drift)				Backbone force points (plf)			
	ed_1	*ed_2*	*ed_3*	*ed_4*	*ef_1*	*ef_2*	*ef_3*	*ef_4*
S2 (best estimate)	0.08	0.72	1.5	5.4	257	731	800	240
Cyclic properties	Spring 1	*$rDisp_1$*	*$rForce_1$*	*$uForce_1$*	*gK_{11}*	*$gKLim_1$*	*gD_{11}*	*$gDLim_1$*
		0.06	0.26	-0.20	0	0	0.13	2.0
	Spring 2	*$rDisp_2$*	*$rForce_2$*	*$uForce_2$*	*gK_{12}*	*$gKLim_2$*	*gD_{12}*	*$gDLim_2$*
		0.06	0.17	-0.23	0.3	2.0	0.13	2.0

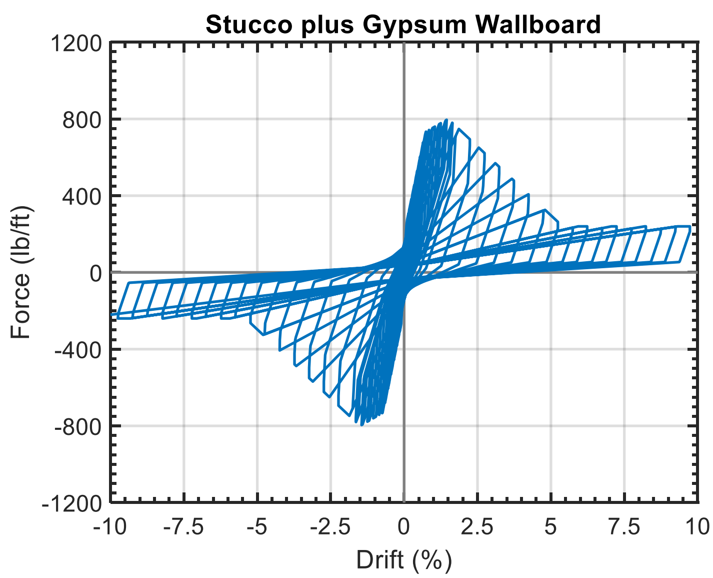

Figure C-7 Behavior of stucco plus gypsum material using *Pinching4* model.

C.4.3.4 WEAK INTERIOR WALLS

The modeling parameters for interior weak walls, WW (two layers of gypsum wallboard), were based on experimental behavior from FEMA P-1100. The FEMA P-1100 best-estimate values were used and these are shown in Table C-10. The resulting behavior is shown in Figure C-8.

Table C-10 Modeling Parameters for Two Layers of Gypsum Wallboard

	Backbone deformation points (% drift)				Backbone force points (plf)			
Material	ed_1	ed_2	ed_3	ed_4	ef_1	ef_2	ef_3	ef_4
G2 (best estimate)	0.12	0.36	0.8	5.65	210	370	420	126
Cyclic properties	Spring 1	$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
		0.15	0.22	-0.21	-0.3	-1.0	0.1	2.0
	Spring 2	$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
		0.4	0.12	-0.19	0.2	2.0	0.12	2.0

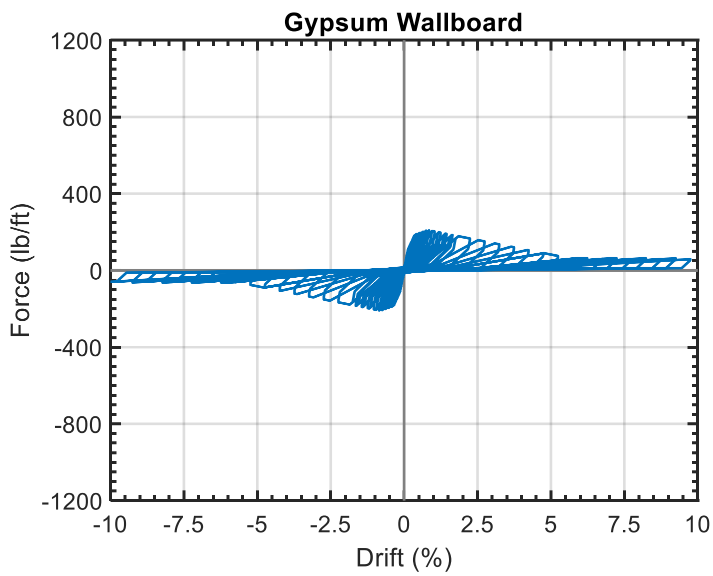

Figure C-8 Behavior of gypsum wallboard material using *Pinching4* model.

C.4.3.5 STRONG DIAPHRAGMS

The modeling parameters for strong diaphragms, SD (diagonal lumber sheathing), were based on experimental study by C. Ni and E. Karacabeyli (2007). The experiment was configured as diagonal sheathing walls. The values used are shown in Table C-11. The resulting behavior is shown in Figure C-9.

Table C-11 Modeling Parameters for the Strong Diaphragm

Material	Backbone deformation points (% drift)				Backbone force points (plf)			
	ped_1	ped_2	ped_3	ped_4	pef_1	pef_2	pef_3	pef_4
Wall #5	0.03	0.67	2.4	5.07	57.3	415	507	402
	ned_1	ned_2	ned_3	ned_4	nef_1	nef_2	nef_3	nef_4
	-0.016	-0.39	-1.4	-2.96	-115	-837	-1028	-811
Cyclic properties	Spring 1	$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
		0.18	0.37	-0.1	-0.1	-1.0	0.14	0.3
	Spring 2	$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
		0.4	0.34	-0.12	0	0	0.09	0.2

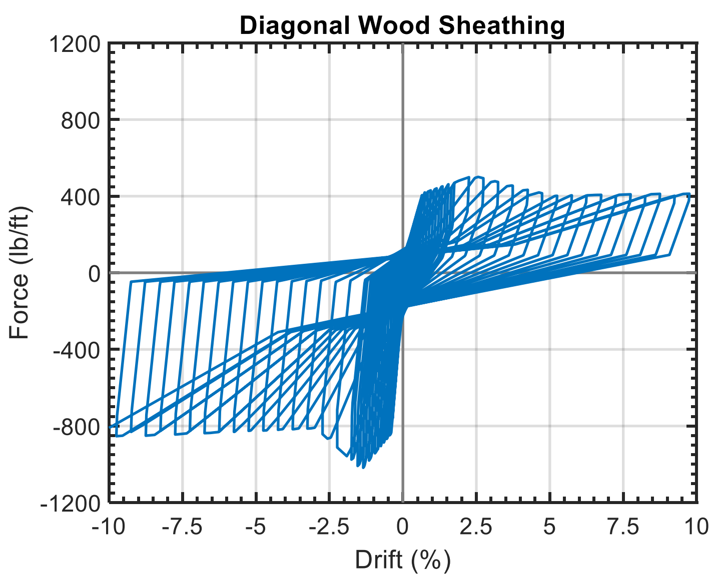

Figure C-9 Behavior of the strong diaphragm based on diagonal sheathing using the *Pinching4* model.

C.4.3.6 WEAK DIAPHRAGM

The modeling parameters for weak diaphragms, WD (straight lumber sheathing), were based on experimental work by Schiller et al. (2020). The experiment was configured as cripple walls with horizontal wood siding. The values used were obtained from Test A-7, and they are shown in Table C-12. The resulting behavior is shown in Figure C-10.

Table C-12 Modeling Parameters for Weak Diaphragms

Material		Backbone deformation points (% drift)				Backbone force points (plf)			
		ed_1	ed_2	ed_3	ed_4	ef_1	ef_2	ef_3	ef_4
CW-HS1		0.17	1.26	4.0	24.4	51	149	177	53
Cyclic properties	Spring 1		$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
			0.18	0.37	-0.1	-0.1	-1.0	0.14	0.3
	Spring 2		$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
			0.4	0.34	-0.12	0	0	0.09	0.2

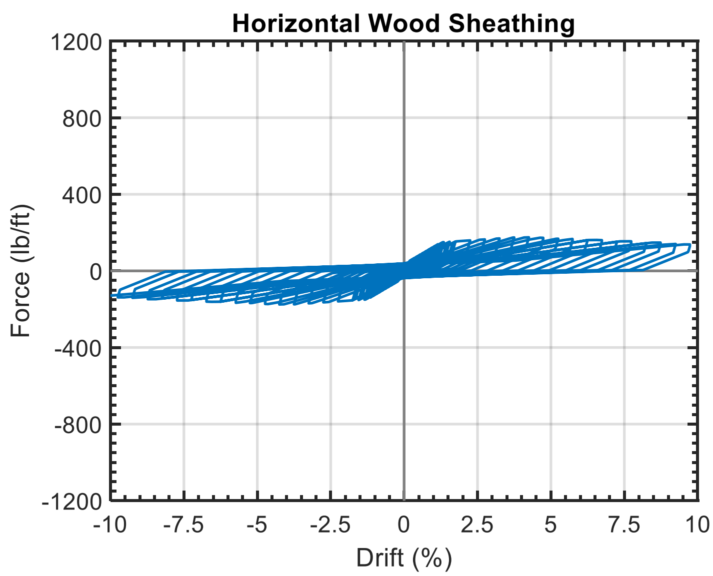

Figure C-10 Behavior of the weak diaphragm based on straight lumber sheathing using the *Pinching4* model.

C.4.3.7 BRITTLE DIAPHRAGMS

The modeling inputs for the brittle diaphragms (BD) were derived from the strong diaphragm, SD (diagonal lumber sheathing). The peak strength was reduced by half, and the strengths were reduced to zero at 5% drift. The values for this diaphragm were based on engineering judgment, in response to field observations of poor construction practices (e.g., misdriven nailing) and long-term deterioration from cracked wood due to drying. This diaphragm was used to study the effects of lower diaphragm strengths and brittle behavior. The modeling parameters are shown in Table C-13 below. The resulting behavior is shown in Figure C-11.

Table C-13 Modeling Parameters for Brittle Diaphragms

Material	Backbone deformation points (% drift)				Backbone force points (plf)			
	ped_1	ped_2	ped_3	ped_4	pef_1	pef_2	pef_3	pef_4
BD	0.014	0.33	1.2	5.0	28.7	207.7	254	201
	ned_1	ned_2	ned_3	ned_4	nef_1	nef_2	nef_3	nef_4
	-0.008	-0.2	-0.7	-5.0	-58	-418	-514	-406
Cyclic properties	Spring 1	$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
		0.18	0.37	-0.1	-0.1	-1.0	0.14	0.3
	Spring 2	$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
		0.4	0.34	-0.12	0	0	0.09	0.2

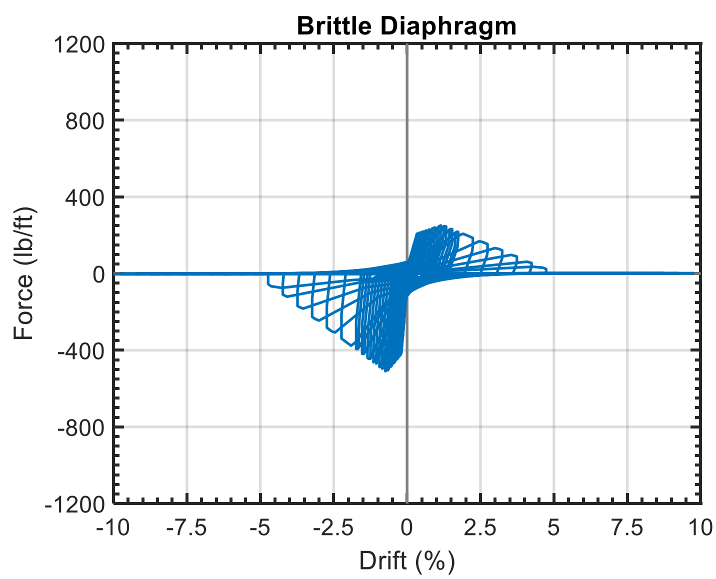

Figure C-11 Behavior of the brittle diaphragm based on diagonal sheathing but modified to reduce peak strength by half and reduce strength to zero at 5% drift using the *Pinching4* model.

C.4.3.8 VERY WEAK DIAPHRAGM

The modeling inputs for the very weak diaphragm (VWD) were derived from the weak diaphragm, WD (straight lumber sheathing). The peak strength was set to 100 plf (compared to 177 plf for the weak diaphragm). Like the brittle diaphragm, these values were based on engineering judgment in response to field observations. This diaphragm was used to study the effects of very low diaphragm strengths. The modeling parameters are shown in Table C-14 below. The resulting behavior is shown in Figure C-12.

Table C-14 Modeling Parameters for Very Weak Diaphragms

Material		Backbone deformation points (% drift)				Backbone force points (plf)			
		ed$_1$	*ed*$_2$	*ed*$_3$	*ed*$_4$	*ef*$_1$	*ef*$_2$	*ef*$_3$	*ef*$_4$
VWD		0.17	1.26	4.0	24.4	29	84	100	30
Cyclic properties	Spring 1	*rDisp*$_1$	*rForce*$_1$	*uForce*$_1$	*gK*$_{11}$	*gKLim*$_1$	*gD*$_{11}$	*gDLim*$_1$	
		0.18	0.37	-0.1	-0.1	-1.0	0.14	0.3	
	Spring 2	*rDisp*$_2$	*rForce*$_2$	*uForce*$_2$	*gK*$_{12}$	*gKLim*$_2$	*gD*$_{12}$	*gDLim*$_2$	
		0.4	0.34	-0.12	0	0	0.09	0.2	

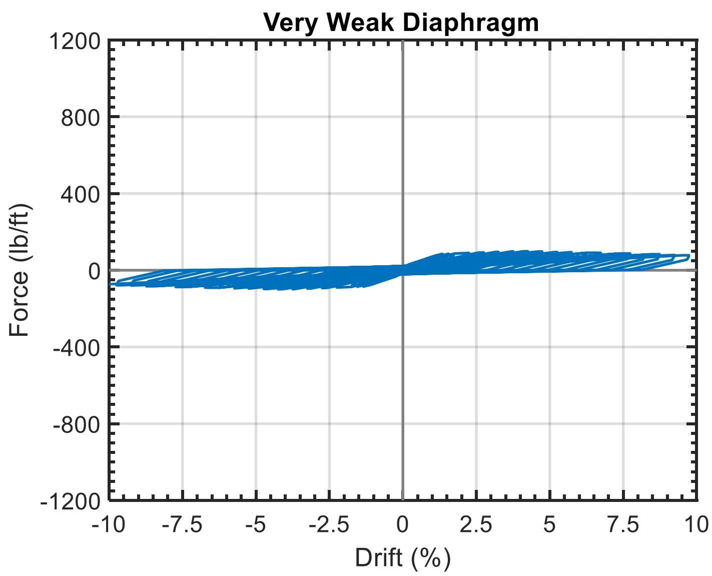

Figure C-12 Behavior of the very weak diaphragm based on reduced strength straight sheathing using the *Pinching4* model.

C.4.3.9 LOWER BOUND DIAPHRAGM

The lower bound diaphragm (LBD) is derived from the weakest diaphragm test found in the literature review (60% of VWD). The values have been further reduced for condition effects, such as poor construction or material degradation. The modeling parameters are shown in Table C-15 below. The resulting behavior is shown in Figure C-13.

Table C-15 Modeling Parameters for Lower Bound Diaphragms

Material		Backbone deformation points (% drift)				Backbone force points (plf)			
Material		ed_1	ed_2	ed_3	ed_4	ef_1	ef_2	ef_3	ef_4
LBD		0.17	1.26	4.0	24.4	17	50.4	60	18
Cyclic properties	Spring 1		$rDisp_1$	$rForce_1$	$uForce_1$	gK_{11}	$gKLim_1$	gD_{11}	$gDLim_1$
			0.18	0.37	-0.1	-0.1	-1.0	0.14	0.3
	Spring 2		$rDisp_2$	$rForce_2$	$uForce_2$	gK_{12}	$gKLim_2$	gD_{12}	$gDLim_2$
			0.4	0.34	-0.12	0	0	0.09	0.2

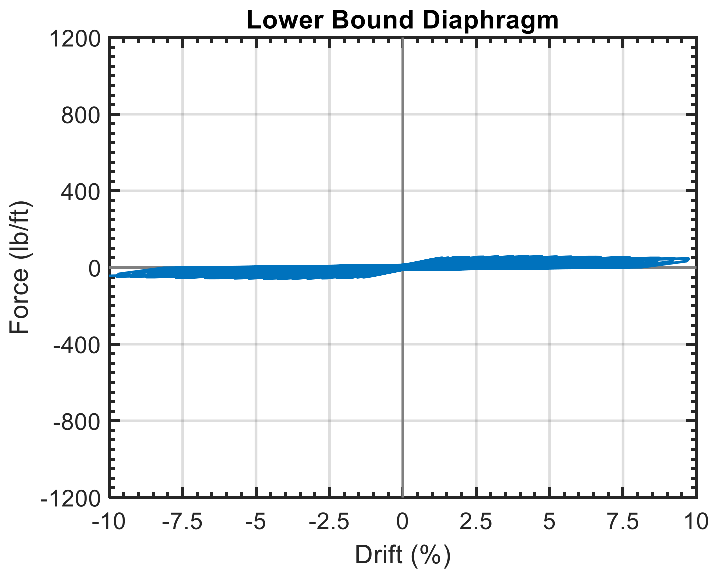

Figure C-13 Behavior of the LBD derived from the VWD reduced for conditions by 60% using the *Pinching4* model.

C.5 Seismic Mass Discretization

Seismic masses are assigned in the X and Y directions independently, and each discrete mass element corresponds to a wall line and assigned to each story. There are X-direction walls and Y-direction walls. Walls resist loads in their primary direction (in-plane) with no stiffness out-of-plane. The masses are assigned to walls in proportion to the wall tributary area. Table C-16 and Table C-17 show the assigned nodal seismic masses in kips for the LO and SO archetypes, respectively. (The values assigned within the models were divided by the acceleration of gravity, g.)

Table C-16 Seismic Masses for the Long-Side-Open Archetypes

Node Tag	Weak (kips)		Strong (kips)		Node Tag	Weak (kips)		Strong (kips)	
	X-dir.	Y-dir.	X-dir.	Y-dir.		X-dir.	Y-dir.	X-dir.	Y-dir.
110130	-	9.45	-	14.41	120830	-	10.28	-	17.85
110230	-	10.25	-	15.86	120930	-	11.42	-	20.76
110330	-	10.82	-	17.32	121030	-	10.28	-	17.85
110430	-	10.57	-	16.67	121130	-	9.27	-	14.92
110530	-	10.80	-	17.27	120610	26.86	-	44.79	-
110630	34.68	11.43	61.62	18.87	120620	23.92	-	44.03	-
110730	-	10.80	-	17.27	120640	31.37	-	50.55	-
110830	-	10.57	-	16.67	130130	-	5.37	-	8.41
110930	-	10.82	-	17.32	130230	-	6.62	-	10.83
111030	-	10.25	-	15.86	130330	-	7.18	-	12.29
111130	-	9.45	-	14.41	130430	-	6.62	-	10.83
110610	22.56	-	31.56	-	130530	-	7.18	-	12.29
110620	25.66	-	35.77	-	130630	21.66	6.62	37.74	10.83
110640	32.32	-	52.98	-	130730	-	7.18	-	12.29
120130	-	9.27	-	14.92	130830	-	6.62	-	10.83
120230	-	10.28	-	17.85	130930	-	7.18	-	12.29
120330	-	11.42	-	20.76	131030	-	6.62	-	10.83
120430	-	10.28	-	17.85	131130	-	5.37	-	8.41
120530	-	11.42	-	20.76	130610	15.89	-	25.57	-
120630	33.47	10.28	62.75	17.85	130620	15.65	-	26.78	-
120730	-	11.42	-	20.76	130640	19.37	-	30.05	-

Table C-17 Seismic Masses for the Short-Side-Open Archetypes

Node Tag	Weak (kips) X-dir.	Weak (kips) Y-dir.	Strong (kips) X-dir.	Strong (kips) Y-dir.	Node Tag	Weak (kips) X-dir.	Weak (kips) Y-dir.	Strong (kips) X-dir.	Strong (kips) Y-dir.
110160	-	17.99	-	26.73	120330	11.14	-	19.39	-
110260	-	21.00	-	35.15	120340	11.14	-	19.39	-
110360	-	21.00	-	35.15	120360	11.14	-	19.39	-
110460	-	21.00	-	35.15	120370	11.14	-	19.39	-
110560	-	17.99	-	35.15	120380	11.14	-	19.39	-
110310	7.99	-	10.56	-	120390	9.33	-	14.30	-
110320	12.13	-	16.52	-	130150	-	11.96	-	18.32
110330	13.56	-	20.26	-	130250	-	16.78	-	28.39
110340	11.17	-	19.45	-	130350	8.81	16.78	14.65	28.39
110350	11.17	-	19.45	-	130450	-	16.78	-	28.39
110370	11.17	-	19.45	-	130550	-	11.96	-	18.34
110380	11.17	-	19.45	-	130310	6.28	-	9.63	-
110390	9.33	-	14.30	-	130320	8.81	-	14.65	-
120150	-	17.44	-	26.73	130330	8.81	-	14.65	-
120250	-	20.60	-	36.95	130340	8.81	-	14.65	-
120350	-	20.60	-	36.95	130360	8.81	-	14.65	-
120450	11.14	20.60	19.39	36.95	130370	8.81	-	14.65	-
120550	-	17.44	-	26.73	130380	8.81	-	14.65	-
120310	9.33	-	14.30	-	130390	6.28	-	9.63	-
120320	11.14	-	19.39	-					

C.6 P-Delta Modeling

C.6.1 P-Delta at Material Level

The P-delta effects were assigned to each mass node and incorporated at the wall material level by modifying backbone curves with the appropriate negative stiffness. This formulation effectively captures the local P-delta effects in the nonlinear diaphragm responses. It also prevents numerical convergence problems, especially at high drift levels. The material backbones were modified using the following equation:

$$V_{final,j} = V_{initial,j} - \sum_{i=j}^{n} W_i \frac{\delta_i}{h_i} \tag{C-2}$$

Where $V_{final,j}$ is the implemented material backbone of story j in the numerical model; $V_{initial,j}$ is the unmodified material backbones (as discussed earlier); W_i, δ_i, and h_i are the weight, lateral displacement, and height from the base of story i, respectively.

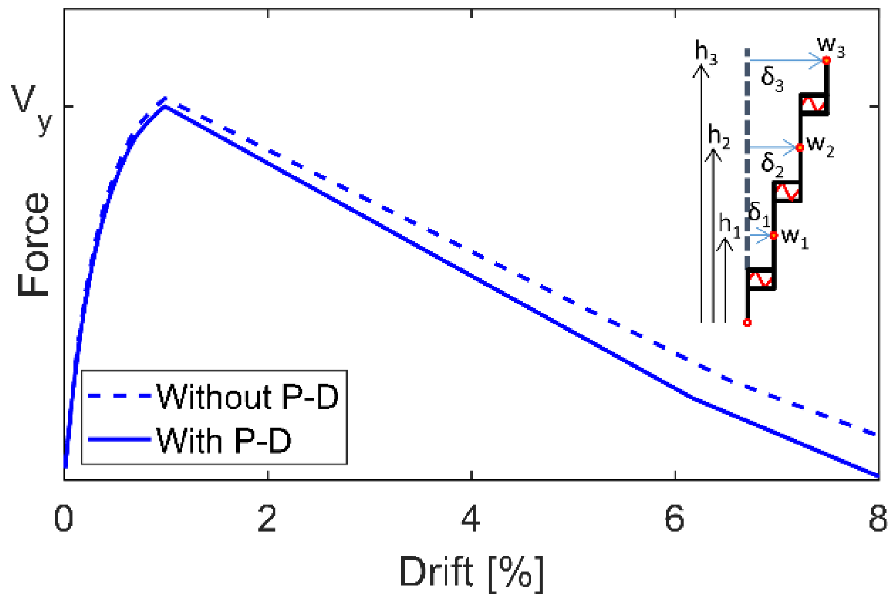

Figure C-14 The P-delta effects were captured at each mass node by modifying the wall material models. The building pushover plot shows the overall effects of P-delta.

C.6.2 P-Delta in Open-Front

At the first story of both the LO and SO archetypes, there are no walls at Lines 1 and 2. Therefore, to consider the P-delta effect, leaning columns with tributary masses were explicitly modeled at these positions.

C.7 Dynamic Characteristics of Numerical Models

C.7.1 Damping

Most of the hysteretic energy dissipation was directly captured by the nonlinearity of the wall and diaphragm elements. An equivalent viscous damping ratio of 0.5% at the elastic first and third modes was used to account for other sources of energy dissipation.

C.7.2 Modal Periods

The first three modal periods of all primary study and other selected archetypes are provided in Table C-18.

Table C-18 Archetype Modal Periods

Archetype	Mode 1 (seconds)	Mode 2 (seconds)	Mode 3 (seconds)
Three-Story, Long-Side-Open Archetypes			
LN3-WW-RD	0.303	0.281	0.246
LO3-WW-RD	0.352	0.326	0.265
LO3-WW-SD	0.371	0.334	0.270
LO3-WW-SD-L	0.334	0.291	0.212
LO3-WW-SD-OL	0.334	0.305	0.222
LO3-WW-SD-P807	0.282	0.253	0.205
LN3-SW-RD	0.324	0.295	0.262
LO3-SW-RD	0.371	0.343	0.284
LO3-SW-WD	0.761	0.439	0.410
LO3-SW-WD-L	0.625	0.439	0.336
LO3-SW-WD-OL	0.633	0.439	0.341
LO3-SW-WD-P807	0.605	0.355	0.326
Two-Story, Long-Side-Open Archetypes			
LO2-WW-RD	0.253	0.234	0.186
LO2-WW-SD	0.258	0.241	0.191
LO2-WW-SD-L	0.24	0.204	0.151
LO2-WW-SD-OL	0.24	0.225	0.167
LO2-WW-SD-P807	0.196	0.182	0.145
LO2-SW-RD	0.278	0.248	0.209
LO2-SW-WD	0.633	0.331	0.322
LO2-SW-WD-L	0.511	0.331	0.36
LO2-SW-WD-OL	0.555	0.331	0.282

Table C-18 Archetype Modal Periods (continued)

Archetype	Mode 1 (seconds)	Mode 2 (seconds)	Mode 3 (seconds)
Two-Story, Long-Side-Open Archetypes			
LO2-SW-WD-P807	0.501	0.264	0.255
Three-Story, Short-Side-Open Archetypes			
SN3-WW-RD	0.363	0.322	0.303
SO3-WW-RD	0.374	0.338	0.332
SO3-WW-SD	0.393	0.338	0.334
SO3-WW-SD-L	0.348	0.338	0.296
SO3-WW-SD-OL	0.346	0.338	0.294
SO3-WW-SD-P807	0.334	0.303	0.284
SN3-SW-RD	0.376	0.340	0.318
SO3-SW-RD	0.391	0.354	0.346
SO3-SW-WD	0.714	0.379	0.366
SO3-SW-WD-L	0.557	0.379	0.286
SO3-SW-WD-OL	0.551	0.379	0.282
SO3-SW-WD-P807	0.559	0.359	0.286
Two-Story, Short-Side-Open Archetypes			
SO2-WW-RD	0.276	0.250	0.242
SO2-WW-SD	0.284	0.242	0.234
SO2-WW-SD-L	0.242	0.228	0.188
SO2-WW-SD-OL	0.242	0.232	0.191
SO2-WW-SD-P807	0.234	0.217	0.193
SO2-SW-RD	0.267	0.241	0.232
SO2-SW-WD	0.612	0.269	0.262
SO2-SW-WD-L	0.445	0.269	0.19
SO2-SW-WD-OL	0.454	0.269	0.194
SO2-SW-WD-P807	0.459	0.228	0.197

C.8 FEMA P-695 Analysis

The analytical studies and the resulting probability of collapse data have been developed in accordance with the FEMA P-695 methodology. FEMA P-695 is a guidance document that standardizes the evaluation and quantification of the seismic safety performance of buildings. It outlines procedures in which an analytical model undergoes simulated ground motions of increasing intensity until collapse, and the results are processed to generate fragility functions. These functions quantify the probability of collapse based on response spectral acceleration values. The methodology recommends the following steps: (1) Select a nonlinear analytical procedure, (2) Select appropriate input ground motions, (3) Perform nonlinear pushover analyses, (4) Perform nonlinear response history analyses, (5) Determine the median collapse capacity, (6) Calculate the Collapse Margin Ration, (7) Calculate the Adjusted Collapse Margin Ratio, and (8) Calculate the probability of collapse.

The following summarizes details of the implementation of the FEMA P-695 methodology.

Step 1. The buildings were modeled with OpenSees using assemblages of nonlinear shear springs.

Step 2. For the IDA, the FEMA P-695 Far-Field Record set was used, consisting of 22 pairs of orthogonal ground motions, used as is and rotated 90 degrees, resulting in 44 pairs of ground motions. Each step in the IDA increased the intensity of the ground motion set based on the median value at a period of 0.25 seconds. All 44 ground motion pairs were scaled up until collapse was detected (Figure C-15).

Step 3. Pushover analyses are reported in Chapter 2.

Step 4. In this project, P-delta sidesway collapse was explicitly modeled, which was observed in a ground motion where drift increased continuously with a slight increase in the response spectral acceleration (represented as a flatline). Typically, collapse occurred when the inter-story drift reached between 5% and 10%. In a few cases, the models continued to support increasing spectral accelerations past the typical range of collapse. In these cases, the analyses were halted at 20% drift. This drift was chosen as the limit where non-simulated collapse failures would occur. The collapse intensity measure (S_a@T=0.25 sec.) was reported as the corresponding spectral acceleration at the incremental step just prior to reaching collapse.

Step 5. The median collapse response spectral acceleration, $\hat{S}_{CT}$, was calculated for the archetype from the IDA data.as illustrated below.

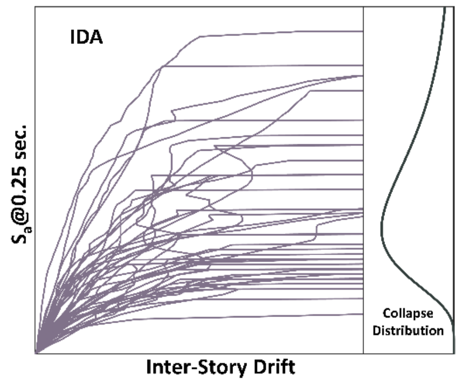

Figure C-15 IDA curves with lognormal distribution.

Step 6. The Collapse Margin Ratio, *CMR*, is calculated.

$$CMR = 1.2\frac{\hat{S}_{CT}}{S_{MT}} \quad \text{(FEMA P-695 Eq. 6-9)}$$

Where the CMR is obtained from $\hat{S}_{CT}$ (median collapse intensity) obtained from IDA curves. The multiplier of 1.2 has been applied to $\hat{S}_{CT}$, as specified in FEMA P-695 Section 6.4.5, because three-dimensional analytical models were employed in combination with pairs of orthogonal ground motions. S_{MT} represents the MCE_R of the selected ground motion intensity at the design period *T* = 0.25 seconds. S_{MT} = 1.978g is used for the study.

Step 7. The Adjusted Collapse Margin Ratio, *ACMR*, was calculated using the modification factor of the Spectral Shape Factor, *SSF*.

$$AMCR = SSF \times CMR \quad \text{(FEMA P-695 Eq. 7-1)}$$

SSF was obtained from Table B-8 of FEMA P-695 based on the period-based ductility (μ_T).

$$\mu_T = {}^{\delta_u}\!/_{\delta_{y,eff}} \quad \text{(FEMA P-695 Eq. 6-6)}$$

$$\delta_{y,eff} = C_0\frac{V_{max}}{W}\left[\frac{g}{4\pi^2}\right](max(T, T_1))^2 \quad \text{(FEMA P-695 Eq. 6-7)}$$

Where δ_U represents the roof displacement at 80% of the maximum base shear and $\delta_{y,eff}$ denotes the effective roof displacement at yield. C_0 is defined in ASCE/SEI 41-06 Section 3.3.3.3 and has a value of 1.2 for two-story and three-story shear-controlled buildings.

Step 8. The probability of collapse, denoted as P[Collapse|S_{MT}], was calculated assuming a lognormal distribution with a median value of *ACMR* multiplied by S_{MT} for the specific model under study.

$$P[Collapse|S_{MT}] = \Phi\left(\frac{\ln(1/_{ACMR})}{\beta_{TOT}}\right)$$

Where Φ(.) is the standard normal cumulative distribution function and β_{TOT} is the total collapse variability. The logarithmic standard deviation, β_{TOT}, is selected to define the overall uncertainty of the system. A constant value of β_{TOT} = 0.757 is used for all collapse calculations.

The value of β_{TOT} is determined by four parameters: β_{RTR} (uncertainty between records), β_{DR} (uncertainty in design requirements), β_{TD} (uncertainty in test data), and β_{MDL} (uncertainty in modeling). The last three parameters were selected from FEMA P-695 Table 3-1, Table 3-2, and Table 5-2. A confidence level of 0.5 for β_{DR}, indicating poor confidence in the basis of design requirements, was assigned for all archetype models. β_{TD} was set at a confidence level of 0.35, indicating fair confidence in the test results, and β_{MDL} was assigned a value of 0.2, reflecting good accuracy and robustness of the models. Lastly, β_{RTR} was specified as 0.4, in accordance with Chapter 6 of FEMA P-695. Using Equation 7-5 from FEMA P-695, the total system uncertainty was calculated to be 0.757.

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \qquad \text{(FEMA P-695 Eq. 7-5)}$$

The resulting parameters for the primary study archetypes are provided in Table C-19.

Table C-19 Selected FEMA P-695 Adjusting Parameters

Archetype	μ_T	S_{ct}	*CMR*	*SSF*	*ACMR*
Three-Story, Long-Side-Open Archetypes					
LO3-SW-WD	4.15	1.23	0.75	1.22	0.91
LO3-SW-WD-L	3.54	1.37	0.83	1.20	1.00
LO3-SW-WD-OL	3.59	1.44	0.88	1.20	1.05
LO3-SW-WD-P807	4.14	1.52	0.92	1.22	1.13
LO3-WW-SD	6.13	1.08	0.65	1.28	0.84
LO3-WW-SD-L	6.77	1.29	0.78	1.30	1.02
LO3-WW-SD-OL	6.74	1.25	0.76	1.30	0.99
LO3-WW-SD-P807	6.52	1.85	1.12	1.29	1.45
Three-Story, Short-Side-Open Archetypes					
SO3-SW-WD	4.68	0.85	0.52	1.24	0.64
SO3-SW-WD-L	3.75	0.89	0.54	1.21	0.66
SO3-SW-WD-OL	3.23	0.89	0.54	1.19	0.64
SO3-SW-WD-P807	3.74	1.67	1.01	1.21	1.23

Table C-19 Selected FEMA P-695 Adjusting Parameters (continued)

Archetype					
SO3-WW-SD	5.94	1.03	0.63	1.28	0.80
SO3-WW-SD-L	6.69	1.10	0.67	1.30	0.87
SO3-WW-SD-OL	7.20	1.08	0.66	1.31	0.86
SO3-WW-SD-P807	7.65	1.47	0.89	1.32	1.18
LO2-SW-WD	4.23	1.83	1.11	1.23	1.36
LO2-SW-WD-L	3.84	2.00	1.22	1.21	1.48
LO2-SW-WD-OL	3.93	2.01	1.22	1.22	1.49
LO2-SW-WD-P807	4.23	2.24	1.36	1.23	1.67
LO2-WW-SD	6.46	1.57	0.95	1.29	1.23
LO2-WW-SD-L	5.77	1.86	1.13	1.27	1.43
LO2-WW-SD-OL	6.79	1.84	1.12	1.30	1.45
LO2-WW-SD-P807	5.77	2.42	1.47	1.27	1.87
SO2-SW-WD	3.66	1.17	0.71	1.21	0.85
SO2-SW-WD-L	3.90	1.29	0.78	1.22	0.95
SO2-SW-WD-OL	3.93	1.29	0.78	1.22	0.95
SO2-SW-WD-P807	4.11	1.96	1.19	1.22	1.45
SO2-WW-SD	5.78	1.32	0.80	1.27	1.02
SO2-WW-SD-L	6.13	1.40	0.85	1.28	1.09
SO2-WW-SD-OL	6.79	1.41	0.86	1.30	1.11
SO2-WW-SD-P807	6.48	1.79	1.08	1.29	1.40

C.9 Seismic Retrofit Parameters

Table C-20 through Table C-22 provide details about the seismic retrofit parameters and retrofit elements used.

Table C-20 Three-Story, Long-Side-Open Retrofit Design Parameters and Elements

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
LO3-WW-SD-L	303.4	3.5	0.376	3	(4) W12x26	NA	NA
LO3-WW-SD-OL	303.4	3.5	0.376	1	(4) W10x22	NA	NA
LO3-WW-SD-P807	303.4	NA	NA	NA	(4) W10x22	20	72
LO3-WW-BD-L	303.4	3.5	0.376	1	(4) W12x26	NA	NA
LO3-WW-BD-P807	303.4	NA	NA	NA	(4) W10x22	20	72
LO3-WW-VWD-L	303.4	3.5	0.376	1	(4) W12x26	NA	NA
LO3-WW-VWD-P807	303.4	NA	NA	NA	(4) W10x22	20	72
LO3-SW-WD-L	504.2	3.5	0.376	3	(4) W14x45	NA	NA
LO3-SW-WD-OL	504.2	3.5	0.376	1	(4) W12x30	NA	NA
LO3-SW-WD-P807	504.2	NA	NA	NA	(4) W12x30	64	144

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

Table C-21 Three-Story, Short-Side-Open Retrofit Design Parameters and Elements

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
SO3-WW-SD-L	269.9	3.5	0.376	3	(2) W12x26	NA	NA
SO3-WW-SD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
SO3-WW-SD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
SO3-WW-BD-L	269.9	3.5	0.376	3	(2) W12x26	NA	NA
SO3-WW-BD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
SO3-WW-BD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
SO3-WW-WD-L	269.9	3.5	0.376	3	(2) W12x26	NA	NA
SO3-WW-WD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
SO3-WW-WD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
SO3-WW-VWD-L	269.9	3.5	0.376	3	(2) W12x26	NA	NA
SO3-WW-VWD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
SO3-WW-VWD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
SO3-WW-LBD-L	269.9	3.5	0.376	3	(2) W12x26	NA	NA

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

Table C-21 Three-Story, Short-Side-Open Retrofit Design Parameters and Elements (continued)

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
S03-WW-LBD-OL	269.9	3.5	0.376	1	(2) W10x22	NA	NA
S03-WW-LBD-P807	269.9	NA	NA	NA	(2) W10x22	64	120
S03-SW-WD-L	445.0	3.5	0.376	3	(2) W14x38	NA	NA
S03-SW-WD-OL	445.0	3.5	0.376	1	(2) W12x26	NA	NA
S03-SW-WD-P807	445.0	NA	NA	NA	(3) W12x26	64	120

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

Table C-22 Two-Story, Long-Side-Open Retrofit Design Parameters and Elements

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
LO2-WW-SD-L	187.8	3.5	0.376	3	(4) W12x19	NA	NA
LO2-WW-SD-OL	187.8	3.5	0.376	1	(4) W8x18	NA	NA
LO2-WW-SD-P807	187.8	NA	NA	NA	(4) W8x18	20	53
LO2-SW-WD-L	302.1	3.5	0.376	3	(4) W12x30	NA	NA
LO2-SW-WD-OL	302.1	3.5	0.376	1	(4) W10x22	NA	NA
LO2-SW-WD-P807	302.1	NA	NA	NA	(6) W10x22	40	126

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

Table C-23 Two-Story, Short-Side-Open Retrofit Design Parameters and Elements

Archetype	Building Seismic Weight, *W* (kips)	Response Modification Coefficient, *R*	Seismic Response Coefficient, C_s (g)	Deflection Amplification Factor, C_d	Retrofit Elements		
					Frame	Plywood X (ft)	Plywood Y (ft)
S02-WW-SD-L	173.2	3.5	0.376	3	(2) W12x19	NA	NA
S02-WW-SD-OL	173.2	3.5	0.376	1	(2) W12x16	NA	NA
S02-WW-SD-P807	173.2	NA	NA	NA	(2) W12x16	15	30
S02-SW-WD-L	280.7	3.5	0.376	3	(2) W12x26	NA	NA
S02-SW-WD-OL	280.7	3.5	0.376	1	(2) W12x22	NA	NA
S02-SW-WD-P807	280.7	NA	NA	NA	(2) W12x22	40	80

Note: NA refers to not applicable. The FEMA P-807 method does not use *R* or C_d. Instead, it uses pre-calculated backbone curves to estimate strength deficits, which are used to determine retrofit designs.

C.10 Analysis Results

Table C-24 through Table C-27 provide the probabilities of collapse (POC) for all models analyzed at a response spectral acceleration, S_a, of 1.0g. Table C-28 provides POC for the primary study archetypes at S_a equal to 2.0g.

Table C-24 Results for Three-Story, Long-Side-Open Archetypes

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
LN3-SW-RD	0.29	LN3	strong	rigid	-
LN3-WW-RD	0.19	LN3	weak	rigid	-
LN3-SW-WD	0.36	LN3	strong	weak	-
LN3-WW-SD	0.19	LN3	weak	strong	-
LO3-SW-RD	0.36	LO3	strong	rigid	-
LO3-WW-RD	0.27	LO3	weak	rigid	-

Table C-24 Results for Three-Story, Long-Side-Open Archetypes (continued)

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
LO3-SW-LBD	0.57	LO3	strong	lower bound	-
LO3-SW-VWD	0.34	LO3	strong	very weak	-
LO3-SW-WD	0.22	LO3	strong	weak	-
LO3-WW-SD	0.27	LO3	weak	strong	-
LO3-WW-BD	0.28	LO3	weak	brittle	-
LO3-WW-VWD	0.22	LO3	weak	very weak	-
LO3-SW-LBD-L	0.26	LO3	strong	lower bound	line
LO3-SW-VWD-L	0.21	LO3	strong	very weak	line
LO3-SW-WD-L	0.18	LO3	strong	weak	line
LO3-WW-SD-L	0.18	LO3	weak	strong	line
LO3-WW-BD-L	0.21	LO3	weak	brittle	line
LO3-WW-VWD-L	0.13	LO3	weak	very weak	line
LO3-SW-LBD-OL	0.21	LO3	strong	lower bound	opt. line
LO3-SW-VWD-OL	0.19	LO3	strong	very weak	opt. line
LO3-SW-WD-OL	0.17	LO3	strong	weak	opt. line
LO3-WW-SD-OL	0.19	LO3	weak	strong	opt. line
LO3-SW-LBD-P807	0.25	LO3	strong	lower bound	P807
LO3-SW-VWD-P807	0.20	LO3	strong	very weak	P807
LO3-SW-WD-P807	0.14	LO3	strong	weak	P807
LO3-WW-SD-P807	0.08	LO3	weak	strong	P807
LO3-WW-BD-P807	0.19	LO3	weak	brittle	P807
LO3-WW-VWD-P807	0.09	LO3	weak	very weak	P807

Table C-25 Results for Three-Story, Short-Side-Open Archetypes

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
SN3-SW-RD	0.38	SN3	strong	rigid	-
SN3-WW-RD	0.28	SN3	weak	rigid	-
SN3-SW-WD	0.42	SN3	strong	weak	-
SN3-WW-SD	0.28	SN3	weak	strong	-
SO3-SW-RD	0.39	SO3	strong	rigid	-
SO3-WW-RD	0.29	SO3	weak	rigid	-
SOW3-SW-RD	0.33	SO3	strong	rigid	-
SOW3-WW-RD	0.24	SO3	weak	rigid	-
SO3-SW-LBD	0.56	SO3	strong	lower bound	-
SO3-SW-VWD	0.43	SO3	strong	very weak	-
SO3-SW-VWD-s	0.44	SO3	strong	very weak	-
SO3-SW-WD	0.38	SO3	strong	weak	-
SO3-SW-WD-s	0.38	SO3	strong	weak	-
SO3-WW-SD	0.27	SO3	weak	strong	-
SO3-WW-BD	0.28	SO3	weak	brittle	-
SO3-WW-VWD	0.33	SO3	weak	very weak	-
SO3-WW-VWD-s	0.35	SO3	weak	very weak	-
SO3-WW-WD	0.29	SO3	weak	weak	-
SO3-WW-WD-s	0.29	SO3	weak	weak	-
SO3-WW-LBD	0.41	SO3	weak	lower bound	-
SOW3-WW-SD	0.21	SO3	weak	strong	-
SOW3-WW-BD	0.22	SO3	weak	brittle	-
SO3-SW-LBD-L	0.36	SO3	strong	lower bound	line
SO3-SW-VWD-L	0.35	SO3	strong	very weak	line

Table C-25 Results for Three-Story, Short-Side-Open Archetypes (continued)

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
SO3-SW-VWD-L-s	0.36	SO3	strong	very weak	line
SO3-SW-WD-L	0.37	SO3	strong	weak	line
SO3-SW-WD-L-s	0.37	SO3	strong	weak	line
SO3-WW-SD-L	0.24	SO3	weak	strong	line
SO3-WW-BD-L	0.25	SO3	weak	brittle	line
SO3-WW-VWD-L	0.25	SO3	weak	very weak	line
SO3-WW-VWD-L-s	0.26	SO3	weak	very weak	line
SO3-WW-WD-L	0.25	SO3	weak	weak	line
SO3-WW-WD-L-s	0.25	SO3	weak	weak	line
SO3-WW-LBD-L	0.22	SO3	weak	lower bound	line
SOW3-WW-SD-L	0.16	SO3	weak	strong	line
SOW3-WW-BD-L	0.19	SO3	weak	brittle	line
SO3-SW-LBD-OL	0.36	SO3	strong	lower bound	opt. line
SO3-SW-VWD-OL	0.36	SO3	strong	very weak	opt. line
SO3-SW-VWD-OL-s	0.37	SO3	strong	very weak	opt. line
SO3-SW-WD-OL	0.38	SO3	strong	weak	opt. line
SO3-SW-WD-OL-s	0.38	SO3	strong	weak	opt. line
SO3-WW-SD-OL	0.24	SO3	weak	strong	opt. line
SO3-WW-BD-OL	0.25	SO3	weak	brittle	opt. line
SO3-WW-VWD-OL	0.24	SO3	weak	very weak	opt. line
SO3-WW-VWD-OL-s	0.25	SO3	weak	very weak	opt. line
SO3-WW-WD-OL	0.25	SO3	weak	weak	opt. line
SO3-WW-WD-OL-s	0.25	SO3	weak	weak	opt. line
SO3-WW-LBD-OL	0.21	SO3	weak	lower bound	opt. line
SOW3-WW-SD-OL	0.16	SO3	weak	strong	opt. line

Table C-25 Results for Three-Story, Short-Side-Open Archetypes (continued)

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
SO3-SW-LBD-P807	0.20	SO3	strong	lower bound	P807
SO3-SW-VWD-P807	0.15	SO3	strong	very weak	P807
SO3-SW-VWD-P807-s	0.15	SO3	strong	very weak	P807
SO3-SW-WD-P807	0.12	SO3	strong	weak	P807
SO3-SW-WD-P807-s	0.13	SO3	strong	weak	P807
SO3-WW-SD-P807	0.13	SO3	weak	strong	P807
SO3-WW-BD-P807	0.17	SO3	weak	brittle	P807
SO3-WW-VWD-P807	0.14	SO3	weak	very weak	P807
SO3-WW-VWD-P807-s	0.14	SO3	weak	very weak	P807
SO3-WW-WD-P807	0.12	SO3	weak	weak	P807
SO3-WW-WD-P807-s	0.13	SO3	weak	weak	P807
SO3-WW-LBD-P807	0.18	SO3	weak	lower bound	P807
SOW3-WW-SD-P807	0.15	SO3	weak	strong	P807

Table C-26 Results for Two-Story, Long-Side-Open Archetypes

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
LO2-SW-WD	0.10	LO2	strong	weak	-
LO2-SW-VWD	0.12	LO2	strong	very weak	-
LO2-SW-LBD	0.14	LO2	strong	Lower Bound	-
LO2-SW-WD-L	0.08	LO2	strong	weak	line
LO2-SW-VWD-L	0.10	LO2	strong	very weak	line
LO2-SW-LBD-L	0.13	LO2	strong	Lower Bound	line
LO2-SW-WD-OL	0.08	LO2	strong	weak	opt. line
LO2-SW-VWD-OL	0.10	LO2	strong	very weak	opt. line
LO2-SW-LBD-OL	0.13	LO2	strong	Lower Bound	opt. line
LO2-SW-WD-P807	0.06	LO2	strong	weak	P807

Table C-26 Results for Two-Story, Long-Side-Open Archetypes (continued)

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
LO2-SW-VWD-P807	0.08	LO2	strong	very weak	P807
LO2-SW-LBD-P807	0.11	LO2	strong	Lower Bound	P807
LO2-WW-SD	0.12	LO2	weak	strong	-
LO2-WW-SD-L	0.08	LO2	weak	strong	line
LO2-WW-SD-OL	0.08	LO2	weak	strong	opt. line
LO2-WW-SD-P807	0.04	LO2	weak	strong	P807

Table C-27 Results for Two-Story, Short-Side-Open Archetypes

Archetype	POC @ 1g	Form	Wall	Diaphragms	Retrofit
SO2-SW-WD	0.24	SO2	strong	weak	-
SO2-SW-VWD	0.32	SO2	strong	very weak	-
SO2-SW-LBD	0.49	SO2	strong	Lower Bound	-
SO2-SW-WD-L	0.20	SO2	strong	weak	line
SO2-SW-VWD-L	0.18	SO2	strong	very weak	line
SO2-SW-LBD-L	0.23	SO2	strong	Lower Bound	line
SO2-SW-WD-OL	0.20	SO2	strong	weak	opt. line
SO2-SW-VWD-OL	0.18	SO2	strong	very weak	opt. line
SO2-SW-LBD-OL	0.20	SO2	strong	Lower Bound	opt. line
SO2-SW-WD-P807	0.08	SO2	strong	weak	P807
SO2-SW-VWD-P807	0.11	SO2	strong	very weak	P807
SO2-SW-LBD-P807	0.17	SO2	strong	Lower Bound	P807
SO2-WW-SD	0.18	SO2	weak	strong	-
SO2-WW-SD-L	0.15	SO2	weak	strong	line
SO2-WW-SD-OL	0.15	SO2	weak	strong	opt. line
SO2-WW-SD-P807	0.09	SO2	weak	strong	P807

Table C-28 Results for the Primary Study Archetypes at a Spectral Response Acceleration of 2.0g

Archetype	POC @ 2g	Form	Wall	Diaphragms	Retrofit
L03-SW-WD	0.55	L03	strong	weak	-
L03-SW-WD-L	0.50	L03	strong	weak	line
L03-SW-WD-OL	0.48	L03	strong	weak	opt. line
L03-SW-WD-P807	0.44	L03	strong	weak	P807
L03-WW-SD	0.60	L03	weak	strong	-
L03-WW-SD-L	0.50	L03	weak	strong	line
L03-WW-SD-OL	0.51	L03	weak	strong	opt. line
L03-WW-SD-P807	0.32	L03	weak	strong	P807
S03-SW-WD	0.73	S03	strong	weak	-
S03-SW-WD-L	0.71	S03	strong	weak	line
S03-SW-WD-OL	0.72	S03	strong	weak	opt. line
S03-SW-WD-P807	0.40	S03	strong	weak	P807
S03-WW-SD	0.62	S03	weak	strong	-
S03-WW-SD-L	0.58	S03	weak	strong	line
S03-WW-SD-OL	0.58	S03	weak	strong	opt. line
S03-WW-SD-P807	0.42	S03	weak	strong	P807
L02-SW-WD	0.35	L02	strong	weak	-
L02-SW-WD-L	0.30	L02	strong	weak	line
L02-SW-WD-OL	0.30	L02	strong	weak	opt. line
L02-SW-WD-P807	0.25	L02	strong	weak	P807
L02-WW-SD	0.40	L02	weak	strong	-
L02-WW-SD-L	0.32	L02	weak	strong	line
L02-WW-SD-OL	0.32	L02	weak	strong	opt. line
L02-WW-SD-P807	0.21	L02	weak	strong	P807

Table C-28 Results for the Primary Study Archetypes at a Spectral Response Acceleration of 2.0g (continued)

Archetype	POC @ 2g	Form	Wall	Diaphragms	Retrofit
S02-SW-WD	0.59	S02	strong	weak	-
S02-SW-WD-L	0.54	S02	strong	weak	line
S02-SW-WD-OL	0.54	S02	strong	weak	opt. line
S02-SW-WD-P807	0.32	S02	strong	weak	P807
S02-WW-SD	0.50	S02	weak	strong	-
S02-WW-SD-L	0.45	S02	weak	strong	line
S02-WW-SD-OL	0.45	S02	weak	strong	opt. line
S02-WW-SD-P807	0.34	S02	weak	strong	P807

Appendix D: Diaphragm Properties

D.1 Introduction and Purpose

The analytical modeling performed as part of the original development of the FEMA P-807 Weak-Story Tool treated floor diaphragms as rigid. More recent modeling (Anaraki et al., 2019) explored the effect on seismic performance of diaphragms modeled with flexibility and nonlinear properties, finding that diaphragm strength and stiffness affect analytical results, including the vulnerability of SWOF buildings prior to retrofit. As a result, the analytical modeling conducted for this guideline studied the effects of diaphragms modeled with flexibility (using nonlinear springs). This appendix provides an overview of considerations included in the selection of diaphragm model properties.

D.2 Diaphragm Strength and Hysteretic Behavior

Information collected and summarized in Appendix A shows that, in the existing SWOF building stock, the type of floor diaphragm structural sheathing varies over time. Up through the 1950s, floors and roofs were primarily lumber sheathed; in the 1960s, both lumber sheathing and plywood were used; and starting in the 1970s, the great majority of sheathing was plywood. The lumber-sheathed floors and roofs are a mix of straight and diagonal lumber sheathing, with indications that diagonal sheathing was more prevalent than straight sheathing in the 1960s. At the same time, floor and ceiling finish materials, acting in combination with the structural sheathing, varied over time. Of particular importance for floor strength and stiffness, hardwood floors are prevalent up through the 1950s, while during the 1960s, typical floor finishes transitioned to carpet. These variations and transitions complicated the choice of modeling properties.

Test data available to inform the selection of modeling properties for diaphragms are summarized in Table D-1. Because limited information is available from testing of floor diaphragm components, wall component test data have also been included.

Table D-1 Peak Capacities of Lumber Sheathed Diaphragms and Walls

Lumber Sheathing Type	Specimen Type	Testing Protocol	Dimension (ft)	Peak Tested Capacity (plf)	Reference and Test Specimen Designation
straight	wall	cyclic	8' × 16'	89	Ni and Karacabeyli, 2007- Wall 12
straight	wall	monotonic	7.3' × 12.1'	133	FPL, 1940 - Test 1
straight	wall	cyclic	2' × 12'	177	Schiller et al., 2020a, b, c, d - Specimen A7 & Welch and Deierlein, 2020
straight	wall	monotonic	7.3' × 12.1'	185	FPL, 1940 - Test 2
straight	wall	monotonic	8' × 8'	220	FPL, 1951 - Control
straight	wall	monotonic	8' × 12'	225	FPL, 1958 - Control
diagonal	diaphragm	monotonic	20' × 60'	325	FPL, 1957 - Test FA-1
diagonal	wall	monotonic	9' × 14'	397	FPL, 1956 - Test 31
diagonal	wall	cyclic	8' × 16'	505/1024	Ni and Karacabeyli, 2007 - Wall 5
straight	diaphragm	monotonic	24' × 40'	625	Green & Horner, 1934
diagonal	wall	monotonic	9' × 14'	658	FPL, 1956 - Test 5
diagonal	wall	monotonic	?	908	FPL,1940 - Test 3
diagonal	wall	monotonic	9' × 14'	1116	FPL, 1956 - Test 9A
diagonal	wall	monotonic	?	1133	FPL, 1940 - Test 7
diagonal	wall	monotonic	9' × 14'	1221/1436	FPL, 1956 - No. 896
diagonal	diaphragm	monotonic	24' × 40'	1250	Green & Horner, 1934
diagonal	wall	monotonic	?	1263	FPL, 1940 - Test 5

The data in Table D-1 are sorted from lowest to highest peak shear capacity. All diaphragm tests tabulated were of bare structural diaphragm assemblies with framing at the diaphragm boundaries consistent with conventional framing practices. Other diaphragm tests with supplemental framing at boundaries were reviewed, but because they had higher capacity and stiffness were not included. It was noted that five diaphragms from the FPL 1957 publication with supplemental boundary framing were tested to between 975 plf and 1000 plf. Based on the discussion in Section D.3.2, these higher capacities may be achievable with typical SWOF construction.

The tabulated wall component data is of interest for establishing diaphragm properties because wall sheathing and fastening for walls is essentially the same as provided for floor diaphragms and framing at boundaries is generally similar, with some variations.

The diaphragm test data are representative of the configuration of interest in that the test component is consistent with diaphragm construction and conventional diaphragm boundary conditions. The shear wall test data is representative of the configuration of interest in that the wall components emulate a cantilevered element, similar to the unretrofitted diaphragm cantilever to the open front (cantilever from Line 3 out to Line 1 in Figure D-1). This is of particular interest relative to unretrofitted building vulnerability in these studies. Further, cyclic data are generally preferred over monotonic or limited cyclic. The project team used consensus judgement in selecting representative diaphragm modeling properties based on the available data.

The project team included two sets of diaphragm properties in the primary study to represent the broader group of data. One set of properties was selected from the lower end of tabulated strengths, and one from the upper end. Use of properties towards the upper and lower ends of the strength range (not the highest and lowest values) was deemed to be a reasonable representation of the majority of the building stock. The property selected towards the lower end of strength is based on work by Schiller et al. (2020a,b,c,d) and Welch and Deierlein (2020), and it represents straight lumber sheathing with a peak shear strength of 177 plf. This is designated as the weak diaphragm (WD) in the analytical studies. The property selected near the upper end is based on work by Ni and Karacabeyli (2007) and it represents diagonal lumber sheathing with a peak strength 505 plf with diagonal lumber in tension and 1024 plf with diagonal lumber in compression. This is designated as the strong diaphragm (SD) in the analytical studies. Because both tests incorporated cyclic loading, a full set of hysteretic modeling parameters was derived from these tests. The project team choose not to include the floor or ceiling finish materials that might be acting in combination with the structural sheathing (even though finish materials were being included for the walls); this choice resulted in the modeled capacity typically being somewhat low and very low where hardwood floors might occur. The model hysteretic parameters and resulting cyclic behavior are illustrated in Section 2.3.

Interest in the effect of varying diaphragm modeling properties on the analytically predicted vulnerability of the SWOF buildings led the project team to further explore lower-end diaphragm properties. Three additional diaphragm properties were selected:

- Brittle Diaphragm (BD): This corresponds to a diaphragm with peak strength at 50% of the peak of the SD (252 plf tension, 512 plf compression), and with zero remaining capacity at 5% drift. This represents a diagonally sheathed diaphragm that due to poor initial construction or deterioration has less strength and less deformation capacity than the SD diaphragm.

- Very Weak Diaphragm (VWD): This corresponds to a diaphragm with a peak strength of 100 plf, very close to the lowest two capacities in Table D-1.

- Lower Bound Diaphragm (LBD): This corresponds to a diaphragm with a peak strength of 60 plf, the lowest tabulated strength in Table D-1, further reduced to 2/3 of this capacity due to poor initial construction or deterioration. In short-side-open archetypes with LBD properties, Line 3 bracing walls that are stacked from story to story in other archetypes were moved so they no longer stack. This was done to create higher diaphragm demands, in addition to use of lower bound properties.

The above added diaphragm properties were selected based on the consensus judgement of the project team, informed by the range of data in Table D-1. As with the previous diaphragm properties, it was selected to not include the strength of floor and ceiling finishes that might be acting in combination with the structural sheathing. While the prevalence of the LBD in the existing building stock is believed by the project team to be rare, the prevalence is not known and may vary by location. In order to achieve LBD properties, it is required that all the following conditions occur simultaneously:

- The diaphragm has straight lumber sheathing,
- The lowest tested capacity for straight lumber sheathing is representative,
- The diaphragm strength has been further reduced due to poor construction or deterioration,
- The floor finish is carpet rather than hardwood flooring,
- The ceiling construction is so poor that the ceiling does not contribute any strength, and
- Walls do not stack between the first and second floors.

Should either a jurisdiction or an individual engineer be concerned that these combined conditions are prevalent, investigation of building construction is encouraged. These studies incorporated lower bound properties for the diaphragm properties only, with walls retaining the originally assigned strength and hysteretic behavior; this was because project participants were particularly concerned about the potential for increased probability of collapse as a result of lower bound diaphragm properties.

Results from the additional analyses incorporating these additional diaphragm properties provide insight regarding SWOF buildings with diaphragms that might fall at the very lower bounds of strength.

D.3 Diaphragm Load Path for Unretrofitted Condition

In the process of selecting diaphragm properties for modeling, the adequacy of the load path to support the diaphragm strengths being modeled was evaluated. Evaluation focused on the cantilevered portion of the second-floor diaphragm (the cantilever from Line 3 to the open front at Line 1), subject to seismic loading parallel to the open front and parallel to Line 3. This portion of the diaphragm was identified as important to the performance of the existing building prior to retrofit. The following were evaluated for loading parallel to the open front (Figure D-1):

- The shear connection between the second-floor diaphragm and the top of the first-story Line 3 wall at the critical line of the cantilever (Line 3 wall in analysis models), and
- The diaphragm chords at the second floor at the location of peak diaphragm demand (Line 3 wall in analysis models).

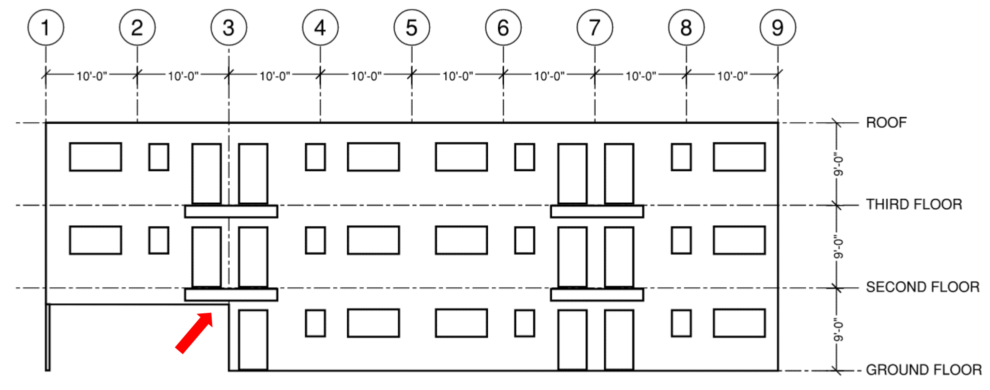

Figure D-1 Elevation of short-side-open archetype with arrow pointing at critical location for second-floor diaphragm shear and flexure.

The Line 3 wall in the analysis models was anticipated to correspond to the critical shear and flexure location for the existing building second-floor diaphragm prior to retrofit. With retrofit implemented, the shear and flexure demands at this location are significantly reduced and of limited concern. To help evaluate the existing framing configuration, minimum fastening provisions from Table 25-J of the 1958 UBC (ICBO, 1958) were used as an indication of the minimum framing fastening that might be expected to act in combination with continuity provided by other materials and systems.

D.3.1 Shear Load Path

For the shear transfer load path and forces parallel to Line 3, Table 25-J of the 1958 UBC indicates minimum fastening of two 16-penny common toe-nails between the floor joists (commonly located at sixteen inches on center) and the wall top plate below (Figure D-2). This would be the anticipated minimum fastening from joist to top plate, as this fastening is required in order that the joists retain their position and spacing as the joists are laid out during initial framing. Using the 2018 NDS (AWC, 2018), the nominal capacity of this anticipated minimum fastening is 777 pounds per joist, or 583 plf.

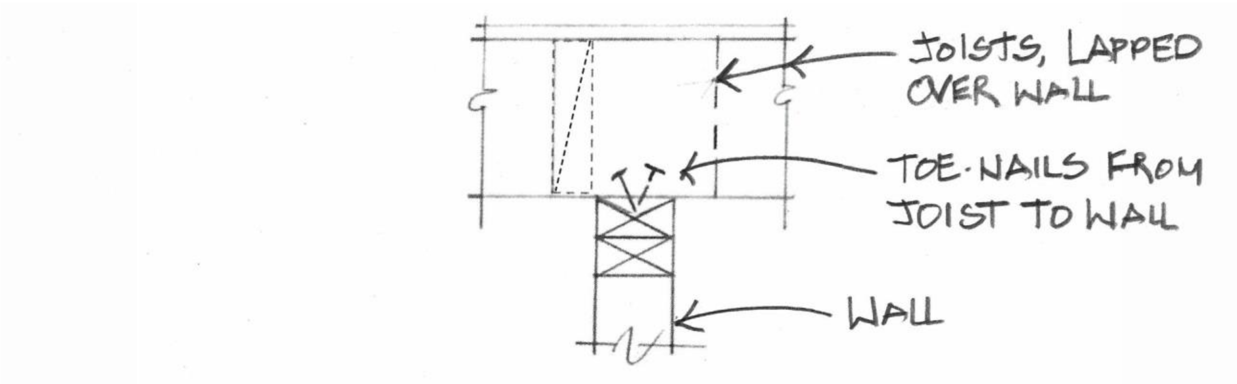

Figure D-2 Toe-nails fastening the second-floor joists to the Line 3 first-story wall.

Based on typical framing practice it is also anticipated that solid blocking is provided over the wall parallel to Line 3 (Figure D-3). The blocking would also commonly be fastened with two toe-nails. Assuming 16d common nails, this increases the shear capacity of the floor joist to top plate to 1554 pounds, or 1166 plf. This nominal capacity can be further confirmed by comparison to testing of conventional fastening load paths found in CUREE Report W-22 (Ryan et al., 2003). In Testing Scenario 2, with a configuration almost identical to that anticipated in the Line 3 connection, the tested peak capacity was 1209 plf. This test data confirms the reasonableness of the calculated 1166 plf nominal capacity for the bare framing condition.

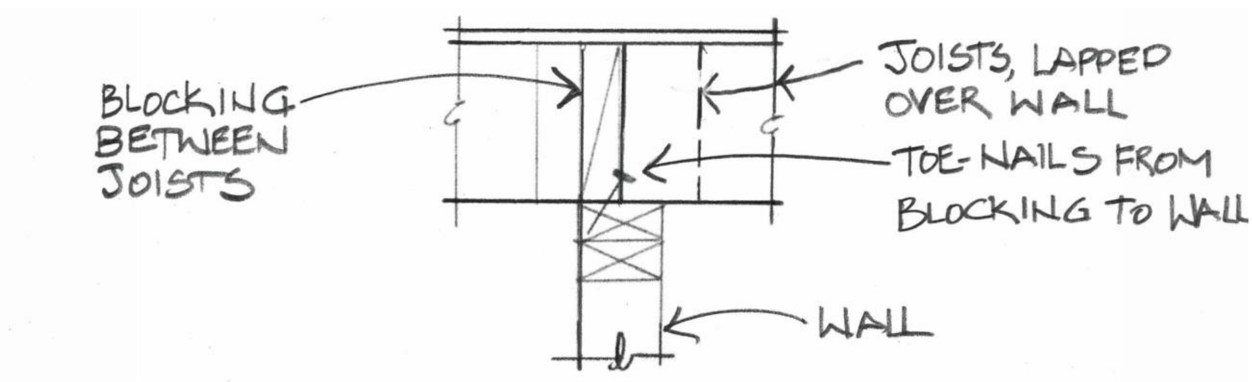

Figure D-3 Toe-nails fastening the second-floor blocking to the Line 3 first-story wall.

The 1166 plf shear transfer capacity calculated considering the bare framing and its fasteners ignores the additional shear transfer provided by the finish materials on the ceiling and each face of the wall. Prior testing has shown that gypboard and stucco can maintain their shear capacity through typical joints and around corners. If these capacities were to be added, the peak capacity for shear transfer into the first-story Line 3 wall would be on the order of 2000 plf, compared to the wall modeled peak capacity of approximately 800 plf.

Whether the capacity of this shear transfer is near the lower end or the upper end of the range identified, it is unlikely to be a weak link in the seismic performance of a SWOF building. Based on this, the analytical studies assumed the shear transfer to be adequate to develop the peak strength of the wall below.

D.3.2 Flexure Load Path

In the unretrofitted building configuration where the second-floor diaphragm cantilevers from Line 3 to line 1, the diaphragm is required by statics to carry flexural forces as well as shear. In accordance with common design assumptions, flexure is assumed to be primarily carried by tension and compression in chord members at each side of the diaphragm. Like the shear considerations, resistance of the existing construction to these chord forces was considered.

A lower bound calculation of chord capacity for the short-side-open archetype can be made considering typical framing and fastening (Figure D-4). For this calculation, the minimum fastening requirements of the 1958 UBC are again used as an indicator of minimum anticipated construction. The following nominal capacities are anticipated to provide tension and compression capacity for the chords at Line 3:

A. Three 16d common face nails between floor joists at the lap over the Line 3 wall: For three joists nominal tension (T_n) and compression (C_n) = 4,213 lb.

B. One 16d common toenail on each of the lapped joists, allowing the top plate to tie the joists together for three lapped joists: T_n & C_n = 1,166 lb.

C. Face nailing of the exterior wall top plate assuming a 16-foot-long plate with an 8-foot dimension to either side of Line 3 and face-nailed with 16d common at 16 inches on center: T_n & C_n = 2,802 lb.

When these three sources are combined, the total nominal capacity of the chord splice considering only the framing connections is 8,181 lb. Based on the analysis model that has second-story walls at 10 and 20 feet from Line 3 (located at Lines 1 and 2), the nominal chord capacity can be divided by an average of 15 feet to determine what unit shear can be supported, v = 8181/15 = 545 plf. With only the face nails from the first bullet considered, v = 280 plf.

Using these very conservative assumptions of what is acting, moderate diaphragm shears can be supported, recognizing that there are other significant contributors to chord capacity.

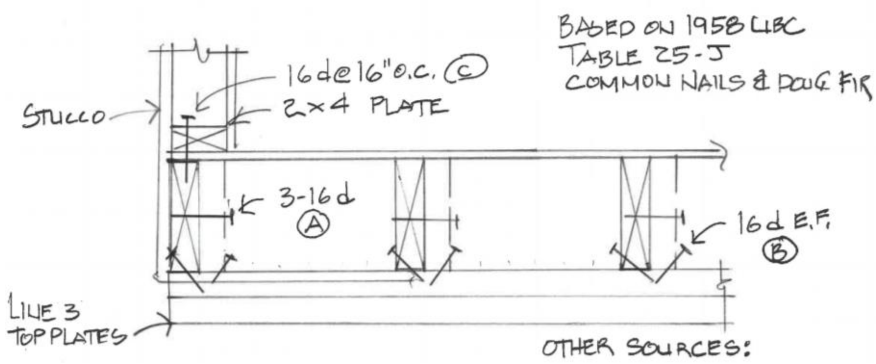

Figure D-4 Section cut through second floor diaphragm at Line 3.

Information from Table D-1 diaphragm tests suggests that diaphragms tested in the laboratory were able to withstand significant chord forces with very conventional framing and fastening details and without strength contributions of finish materials.

- FPL (1957) Specimen FA-1: *T* & *C* = 5,400 lb. (treating as uniformly loaded based on loading at 1/5 points)
- Green & Horner (1934) straight sheathed: *T* & *C* = 625 plf × 40 ft/3 = 8,333 lb. (concentrated loads at 1/3 points)
- Green & Horner (1934) diagonal sheathed: *T* & *C* = 1250 plf × 40 ft/3 = 16,666 lb. (concentrated loads at 1/3 points)

This again demonstrates that bare framing can withstand significant chord forces even when not considering contributions of surrounding materials and considering isolated component behavior rather than structure behavior.

Other sources of capacity include diagonal sheathing that provides continuity across Line 3 and will be particularly able to contribute tension and compression capacity at the intersection of Line 3 and the perpendicular exterior walls. Also included are both ceiling and wall finishes that extend across Line 3. See Dolan et al. (2003) for testing related to the effect of walls above on diaphragms. Of the finishes, the most significant contribution is anticipated to come from the exterior stucco applied vertically to the walls perpendicular to Line 3, as the stucco itself has considerable tensile capacity and stiffness. The stucco is in a single plane with distributed fastening to the framing over the 20 feet of the cantilever and the 80 feet beyond the cantilever. If the ability of the stucco nailing to transfer loading from framing to stucco were estimated at 600 plf based on estimated shear capacity, this would suggest that a chord force of approximately 12,000 lb. could be transferred to the stucco over the 20 feet between Line 1 and Line 3.

Prior testing of full structures (Fischer et al., 2001; Mosalam et al., 2002; Christovasilis et al., 2009) and large assemblies (Acevedo et al., 2017; Acevedo et al., 2018; Cobeen et al., 2020) has shown that wood structures in general, as well as those tested with finish materials in place, respond as an integrated box, with the strength and stiffness of the system significantly greater than the predicted strength and stiffness of the individual parts. Similarly, the portion of the SWOF building cantilevering from Line 3 toward Line 1 consists of multiple floors, walls, and a roof that behave more as an integrated structure than individual elements.

Based on the above discussion, the analytical studies assumed the chord capacity to be adequate to develop the diaphragm shear capacity.

D.3.3 Conclusions

Based on the described evaluation, the project team made the judgement that is not necessary to reduce diaphragm peak capacities in the analytical studies to account for second-floor diaphragm shear or chord capacities behaving as a weak link.

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