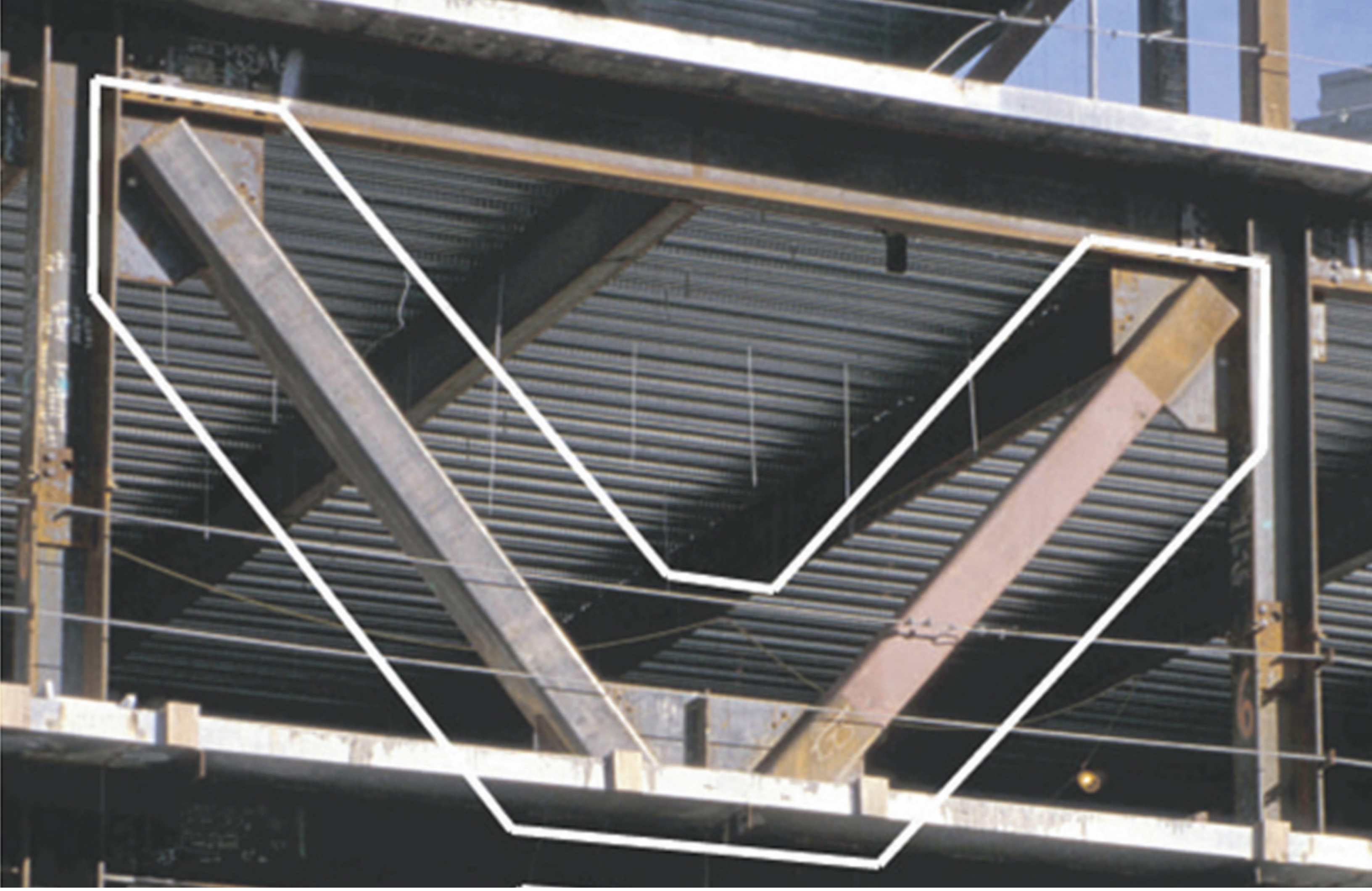

Quantification of Building Seismic Performance Factors: Component Equivalency Methodology

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Quantification of Building Seismic Performance Factors: Component Equivalency Methodology

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Foreword

The Federal Emergency Management Agency (FEMA) has the goal of reducing 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 and related multi-hazard 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. These voluntary resource guidance products present criteria for the design, construction, upgrade, and function of buildings subject to earthquake ground motions in order to minimize the hazard to life in all buildings and increase the expected performance of critical and higher occupancy structures.

This publication builds upon an earlier FEMA publication, FEMA P-695 *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). FEMA P-695 presents a procedural methodology for reliably quantifying seismic performance factors, including the response modification coefficient, R, the system overstrength factor, Ω_O, and the deflection amplification factor, C_d, used to characterize the global seismic response of a system.

While the methodology contained in FEMA P-695 provides a means to evaluate complete seismic-force-resisting systems proposed for adoption into building codes, a component-based methodology was needed to reliably evaluate structural elements, connections, or subassemblies proposed as substitutes for equivalent components in established seismic-force-resisting systems. The Component Equivalency Methodology presented in this document fills this need by maintaining consistency with the probabilistic, system-based collapse assessment concepts of FEMA P-695 while providing simple procedures for comparing the tested performance of different components. It is intended to be of assistance to organizations, such as the International Code Council Evaluation Service, who need to compare the seismic performance of alternate components to those contained in established seismic force resisting system.

FEMA wishes to express its sincere gratitude to Charlie Kircher, Project Technical Director, and to the members of the Project Team for their efforts in the development of this publication, including the Project Management Committee consisting of Greg Deierlein, Andre Filiatrault, Jim Harris, John Hooper, Helmut Krawinkler, and Kurt Stochlia; the Project Working Groups consisting of Curt Haselton, Abbie Liel, Jackie Steiner, and Seyed Hamid Shivaee; and the Project Review Panel consisting of S.K. Ghosh, Mark Gilligan, Ramon Gilsanz, Ron Hamburger, Rich Klingner, Phil Line, Bonnie Manley, Rawn Nelson, Andrei Reinhorn, and Rafael Sabelli. Without their dedication and hard work, this project would not have been possible.

Federal Emergency Management Agency

Preface

In 2008, the Applied Technology Council (ATC) was awarded a "Seismic and Technical Guidance Development and Support" contract (HSFEHQ-08-D-0726) by the Federal Emergency Management Agency (FEMA) to conduct a variety of tasks, including one entitled "Quantification of Building System Performance and Response Parameters." Designated the ATC-63-1 Project, this work was the continuation of the ATC-63 Project, funded under an earlier FEMA contract, which resulted in the publication of the FEMA P-695 report, *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). This report outlined a procedural methodology for reliably quantifying seismic performance factors, including the response modification coefficient, R factor, the system overstrength factor, Ω_0, and the deflection amplification factor, C_d, used to characterize the global seismic response of a system.

While the FEMA P-695 Methodology provided a means to evaluate complete seismic-force-resisting systems proposed for adoption into building codes, a component-based methodology was still needed that could reliably evaluate structural elements, connections, or subassemblies proposed as substitutes for equivalent components in current code-approved seismic-force-resisting systems. The purpose of the ATC-63-1 Project was to develop such a methodology.

The recommended Component Equivalency Methodology described in this report balances the competing objectives of: (1) maintaining consistency with the probabilistic, analytical, system-based collapse assessment concepts of the FEMA P-695 Methodology; and (2) providing simple procedures for comparing the tested performance of different components. It was developed based on probabilistic concepts using results from collapse sensitivity studies on key performance parameters.

ATC is indebted to the leadership of Charlie Kircher, Project Technical Director, and to the members of the ATC-63-1 Project Team for their efforts in the development of the recommended methodology. The Project Management Committee, consisting of Greg Deierlein, Andre Filiatrault, Jim Harris, John Hooper, Helmut Krawinkler, and Kurt Stochlia monitored and guided the technical development efforts. The Project Working Groups, which included Curt Haselton, Abbie Liel, Seyed Hamid Shivaee, and Jackie

Steiner, deserve special recognition for their contributions in developing, investigating, and testing the methodology, and in preparing this report. The Project Review Panel, consisting of S.K. Ghosh, Mark Gilligan, Ramon Gilsanz, Ronald Hamburger, Richard Klingner, Philip Line, Bonnie Manley, Rawn Nelson, Andrei Reinhorn, and Rafael Sabelli provided technical review, advice, and consultation at key stages of the work. Ayse Hortacsu served as ATC project manager for this work. The names and affiliations of all who contributed to this report are provided in the list of Project Participants.

ATC also gratefully acknowledges Michael Mahoney (FEMA Project Officer), Robert Hanson (FEMA Technical Monitor), and William Holmes (ATC Project Technical Monitor) for their input and guidance in the preparation of this report, Peter N. Mork for ATC report production services, and Ramon Gilsanz as ATC Board Contact.

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Chapter 1
Introduction

This report describes a recommended methodology for evaluating the seismic performance equivalency of *components*, which are structural elements, connections, or subassemblies experiencing inelastic response that controls the collapse performance of a seismic-force-resisting system. The recommended Component Equivalency Methodology (referred to as the Component Methodology) is a statistically based procedure for developing, evaluating, and comparing test data for new components (*proposed components*) that are proposed as substitutes for selected components (*reference components*) in a current code-approved seismic-force-resisting system (*reference SFRS*).

The Component Methodology is derived from the general methodology contained in FEMA P-695 *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). Like the general methodology in FEMA P-695, the intent of the Component Methodology is to ensure that code-designed buildings have adequate resistance to earthquake-induced collapse. In the case of component equivalency, this intent implies equivalent safety against collapse when proposed components are substituted for reference components in a reference SFRS.

Proposed components found to be equivalent using the Component Methodology can be substituted for components of the reference SFRS, subject to design requirements and seismic design category restrictions on the use of the reference SFRS. Reference systems include the seismic-force-resisting systems contained in ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). For clarity, it should be noted that the term “component” in ASCE/SEI 7-10 refers exclusively to nonstructural components, which is different from how the term is used in this report. In the Component Methodology, the term “component” refers to structural elements that are part of the seismic-force-resisting system.

1.1 Background and Purpose

The Applied Technology Council (ATC) was commissioned by the Federal Emergency Management Agency (FEMA) under the ATC-63 Project to develop a methodology for quantitatively determining the response modification coefficient, R, the system overstrength factor, Ω_O, and the deflection amplification factor, C_d, used in prescriptive seismic design procedures found in modern building codes. Collectively referred to as seismic performance factors, these factors are fundamentally critical in the specification of seismic design loading. They are used to estimate strength and deformation demands on seismic-force-resisting systems that are designed using linear methods of analysis, but are responding in the nonlinear range.

The resulting FEMA P-695 report outlines a procedural methodology for quantifying collapse behavior and establishing seismic performance factors for newly proposed structural systems. The FEMA P-695 Methodology relies on collapse simulation through nonlinear response history analysis of structural systems. It accounts for potential uncertainties in ground motions, component design parameters, structural configuration, and behavioral characteristics of structural elements based on available laboratory test data. It is anticipated that this methodology will be used by the nation's seismic code development committees to set minimum acceptable design criteria for code-approved systems, and to provide guidance in the selection of appropriate design criteria for other systems when linear design methods are utilized.

While complete systems have been proposed for adoption as new seismic-force-resisting systems, it is also common that new structural elements, connections, or subassemblies are proposed for use in current code-approved seismic-force-resisting systems. Such components, as they are referred to in this report, have been typically evaluated on the basis that their substitution for components in a reference system would result in equivalent (or better) seismic performance.

Although the FEMA P-695 Methodology could be used to evaluate the seismic performance capability of new components, FEMA initiated the ATC-63-1 Project to simplify and adapt the general methodology contained in FEMA P-695 for use in evaluating the specific case of component equivalency. Whereas the FEMA P-695 Methodology is based on both experimental testing and nonlinear dynamic analyses of archetypical structural systems, a key difference is that the Component Equivalency Methodology is based primarily on experimental testing of components.

The resulting Component Methodology described in this report is not intended to replace the FEMA P-695 Methodology for the evaluation of new systems. Instead, it is intended to provide an additional tool for evaluating the performance equivalency of components meeting the applicability criteria described herein.

1.2 Objectives and Scope

The Component Methodology measures the equivalency of proposed components and reference components by comparing key performance parameters, such as strength, stiffness, effective ductility, and deformation capacity. Values of these key parameters are determined from statistical evaluation of test data. The Component Methodology is based on the following two basic performance objectives:

- Proposed components can replace reference components in the reference SFRS without changing the seismic performance of the reference SFRS.
- The collapse performance of the reference SFRS is comparable (or assumed to be comparable) to seismic-force-resisting systems that comply with the collapse performance objectives of the FEMA P-695 Methodology.

The first objective is the basis of the quantitative acceptance criteria used in the Component Methodology. The second objective recognizes that many seismic-force-resisting systems in ASCE/SEI 7-10 have not been comprehensively evaluated using the FEMA P-695 Methodology. Evaluation of selected seismic-force-resisting systems has shown that current code-approved systems generally comply with the collapse performance objectives of the FEMA P-695 Methodology (FEMA, 2009b and NIST, 2010). Therefore, provided that currently approved seismic-force-resisting systems have well established design criteria and supporting test data, they are permitted to be used as a reference SFRS in the Component Methodology.

The scope of the Component Methodology is limited to proposed components that meet certain applicability criteria. These criteria determine the suitability of the seismic-force-resisting system for which the component is proposed (the reference SFRS), define minimum quality requirements for design and test data, and establish limits on the use of the procedures in terms of performance-related attributes. The Component Methodology applies to components that have well-defined boundaries within the reference SFRS, and where the overall seismic behavior of the reference SFRS is not otherwise changed by the replacement of reference components with proposed components. While the scope of the Component Methodology is intended for broad application, it may not be applicable to all types of proposed components. Where the Component Methodology does not apply, the more general procedures of the FEMA P-695 Methodology should be used to evaluate seismic performance equivalency.

The Component Methodology considers that proposed components might be used to replace some or all reference components within a reference SFRS. While partial replacement (i.e., "mixing") of proposed and reference components within a system might be desirable for design versatility, it could inadvertently create a vertical or horizontal irregularity in the seismic-force-resisting system if the proposed and reference components do not have sufficiently similar strength and stiffness. For this reason, the Component Methodology limits the differences in strength and stiffness between proposed and reference components to allow for partial replacement.

1.3 Assumptions and Limitations

Subject to the applicability criteria described in Chapter 2, the Component Methodology is intended to apply broadly to component types proposed for use in any of the currently approved seismic-force-resisting systems contained in ASCE/SEI 7-10. Practical application of the Component Methodology, however, will likely be limited to reference components for which there is available test data that is of sufficient quality and quantity for judging equivalency. This section summarizes key assumptions and potential limitations of the Component Methodology.

1.3.1 Equivalency Approach

The Component Methodology is based on the concept of component equivalency as a practical means of achieving an acceptable level of collapse safety for the seismic-force-resisting system of interest. The equivalency approach necessarily assumes that the collapse safety of the reference SFRS is adequate before proposed components are substituted for reference components. The acceptance criteria of the Component Methodology are intended to ensure that collapse safety will remain adequate when the proposed substitutions are made. Applicability criteria limit application of the Component Methodology to: (1) proposed component types that are considered suitable for evaluation using an equivalency approach; and (2) currently approved seismic-force-resisting systems that are considered suitable for use as a reference SFRS.

1.3.2 Suitability of Proposed Components

While the Component Methodology is intended to apply broadly to different types of components, equivalency concepts may not be applicable, or appropriate, in all cases. Figure 1-1 conceptually illustrates two fundamental issues regarding the applicability of the Component Methodology. The first is whether or not the proposed product is a new "system" or a new "component." The second is whether or not the new component has characteristics (and data) suitable for evaluation using an equivalency approach.

The boundaries shown in Figure 1-1 are defined by the applicability criteria in Section 2.3. It is possible that some new products will not meet these criteria. In such cases, the FEMA P-695 Methodology should be used for evaluation of new products that are deemed inappropriate for evaluation by equivalency.

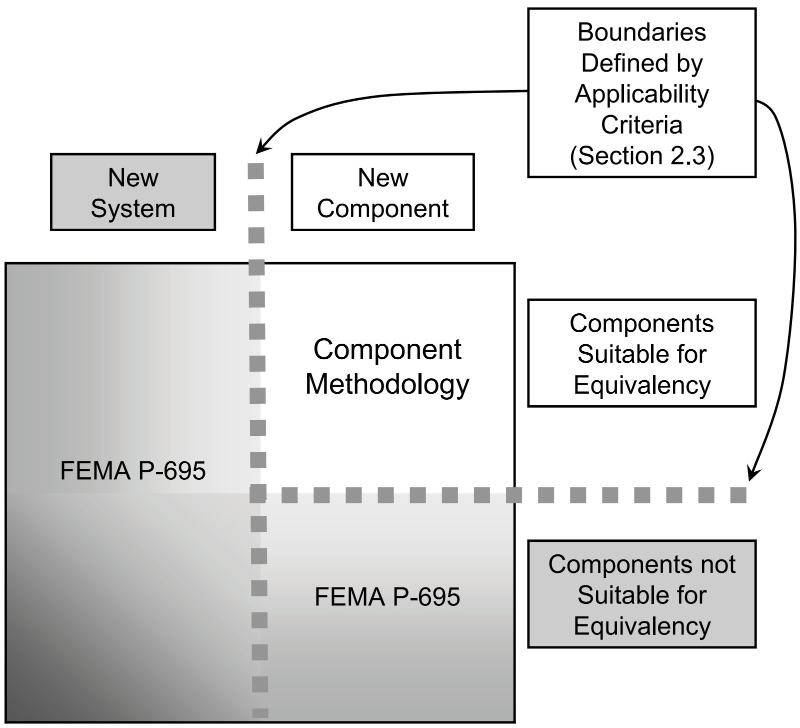


Figure 1-1 Conceptual boundaries defined by the Component Methodology applicability criteria of Section 2.3.

1.3.3 Suitability of the Reference Seismic-Force-Resisting System

In the FEMA P-695 Methodology, adequacy of collapse resistance is evaluated in an absolute sense using criteria that define an acceptable probability of collapse when subjected to Maximum Considered Earthquake ground motions. In the Component Methodology, adequacy of collapse resistance is evaluated in a relative sense using criteria that compare proposed and reference component performance, assuming that the reference SFRS complies with the collapse performance criteria of the FEMA P-695 Methodology.

Ideally, only systems with adequate collapse safety would be used as a reference SFRS. The Component Methodology, however, permits any system in ASCE/SEI 7-10 to be used as a reference system without being shown to comply with the FEMA P-695 Methodology. This is done for two reasons. First, it would not be practical for the Component Methodology to require implementation of the FEMA P-695 Methodology to evaluate a reference SFRS before evaluating component equivalency. Second, evaluations of several systems have shown that, with certain exceptions, current code-approved systems generally comply with the criteria of the FEMA P-695 Methodology.

One such exception includes short-period configurations of all types of seismic-force-resisting systems. This exception reflects a shortcoming in current seismic design requirements in general, and not in any one seismic-force-resisting system in particular. Such a shortcoming, while unfortunate, is not considered sufficient by itself to render any single system in ASCE/SEI 7-10 unsuitable for use as a reference SFRS.

1.3.4 Limitations on Test Data and Design Requirements

The Component Methodology requires a minimum quality (and quantity) of component test data and a minimum quality of component design information. Lack of availability of this information, in particular reference component test data, could limit use the Component Methodology for some reference systems.

Proposed component test data are expected to be developed as part of product development. Although the Component Methodology requires somewhat more extensive cyclic-load (and monotonic-load) testing than is typically used to support product development and approval, such testing is within the control of the product developer. Reference component test data are expected to be obtained from existing sources (previous tests of components within the reference system of interest). Unfortunately, sources of reference component test data can also be limited.

While results of laboratory tests of structural elements of different material types appear frequently and extensively in a number of technical publications, few research programs have comprehensively investigated any given system of ASCE/SEI 7-10. The vast majority of the systems in Table 12.2-1 of ASCE/SEI 7-10 do not have sufficient quality (or quantity) of data required for equivalency evaluation. Of the relatively small number systems with requisite test data, only a few (such as light-frame wood structural panels) have readily useable databases of test results.

While it would be possible for product developers to conduct the necessary reference component testing, it is recognized that such testing may not be desirable or practical. Lack of readily useable, quality reference component "benchmark" data is likely the most significant limitation on the use of the Component Methodology.

1.4 Anticipated Use and Implementation

The Component Methodology described in this report is intended as a technical resource for use by seismic codes and standards development committees, product evaluation services, product manufacturers, suppliers, and their consultants.

Although the Component Methodology is based on the FEMA P-695 Methodology, the two methodologies are fundamentally different in their application. The FEMA P-695 Methodology is intended for use in the development of seismic performance factors for new seismic-force-resisting systems for which seismic codes and standards committees are responsible for adoption. The Component Methodology is intended for use in establishing the equivalency of a new component for which product evaluation services have traditionally been responsible for issuing evaluation reports.

While seismic codes and standards committees may choose to reference, or possibly adopt, applicable portions of this report, it is envisioned that product evaluation services are the most likely immediate users of this information. In this case, the Component Methodology provides a technically sound basis for establishing acceptance criteria to be used in product evaluation reports.

How this document will be implemented by interested organizations and potential users is ultimately the responsibility of others, and is not known at this time. As part of on-going work by the National Institute of Building Science's Building Seismic Safety Council (BSSC) Provisions Update Committee (PUC) to develop the 2014 edition of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, a special Issue Team has been formed to study implementation of the FEMA P-695 Methodology and the Component Methodology described herein. This Issue Team will be responsible for deciding if, and in what manner, the PUC will make use of these methodologies.

1.5 Technical Approach

Development of the Component Methodology necessarily balanced two competing objectives: (1) maintaining consistency with the probabilistic, analytical, and system-based collapse assessment concepts of the FEMA P-695 Methodology; and (2) providing simple procedures for comparing the tested performance of different components. Work involved the following tasks designed to systematically investigate the trade-offs between these objectives:

- Identification of key component performance parameters
- Development of component testing requirements
- Development of probabilistic acceptance criteria

1.5.1 Identification of Key Component Performance Parameters

Key component performance parameters were identified through literature review and numerical collapse sensitivity studies on two- and three-dimensional models of wall and frame structures. Collapse sensitivity studies considered full replacement (i.e., proposed components being used throughout the reference SFRS), as well as partial replacement (i.e., the "mixing" of proposed components and reference components within the reference SFRS). The following parameters were identified as critical for establishing equivalency in seismic collapse resistance:

- Deformation capacity (ultimate deformation)
- Strength (ratio of measured ultimate strength to design strength)
- Initial stiffness (ratio of measured initial stiffness to design stiffness)
- Effective ductility (ratio of ultimate deformation to effective yield deformation)

Component deformation capacity and strength were found to be the most important parameters affecting the collapse safety of a seismic-force-resisting system. Initial stiffness, in general, had less of an effect on collapse safety, but was included as a key parameter because of its fundamental relation to ASCE/SEI 7-10 design processes, including: (1) story drift limits; (2) the second-order stability coefficient; (3) the distribution of force demands to components within a statically indeterminate structural system; and (4) other seismic checks such as those related to horizontal and vertical stiffness irregularities. Like initial stiffness, effective ductility had less of an effect on collapse safety, but was still considered important for preventing inconsistencies in the hysteretic behavior of components and potential stiffness and strength irregularities that can result when elastic code-based seismic design procedures are used.

1.5.2 Development of Component Testing Requirements

Component testing requirements were drawn from the requirements contained within the FEMA P-695 Methodology and ASTM E2126-09 *Standard Test Method for Cyclic (Reversed) Load Test for Shear Resistance* *of Vertical Elements of the Lateral Load Force Resisting System for Buildings* (ASTM, 2009), tailored to meet the needs of the Component Methodology. Cyclic-load and monotonic-load testing requirements address the number of component configurations that need to be tested, the number of test specimens per configuration, and the selection of load histories for cyclic-load testing.

Cyclic-load test data are the primary basis for establishing equivalency of proposed and reference components. Since the measured component strength and deformation capacity may differ depending on the cyclic-load history applied, guidelines for the selection and comparison of loading histories are needed to ensure that performance parameters of the proposed and reference components are appropriately compared. Accordingly, the Component Methodology ensures that the loading history used to test the proposed component be at least as damaging (quantified in terms of accumulated deformation imposed on the specimen) as the loading history used to test the reference component.

Monotonic-load test data are required in addition to cyclic-load test data to help distinguish between different characteristics of component strength deterioration, such as cyclic versus in-cycle degradation that can influence system collapse behavior.

1.5.3 Development of Probabilistic Acceptance Criteria

Acceptance criteria were developed to ensure that a seismic-force-resisting system containing full or partial replacement of proposed components would have equivalent (or better) resistance to seismic-induced collapse as the same system containing reference components alone. Specifically, these criteria require that the ground shaking intensity large enough to cause a 10% probability of collapse in the seismic-force-resisting system would be equivalent in both cases. This requirement is consistent with the probabilistic concepts of the FEMA P-695 Methodology.

While based on probabilistic equations and results from numerical collapse sensitivity studies documented in Appendix C, the resulting criteria are deliberately simple. The principal acceptance criterion is that the factored median deformation capacity of the proposed component must be as large as, or larger than, the median deformation capacity of the reference component. The required margin between the proposed and reference component median deformation capacities is defined by two factors that account for uncertainties associated with component test data and design requirements, and differences in strength. The factors are unity when the uncertainties and differences in strength are relatively small.

Additional acceptance criteria are provided to ensure that the proposed and reference components have comparable values of initial stiffness when implemented in the reference SFRS, and that the effective ductility of the proposed component is at least 50 percent of the effective ductility of the reference component.

1.6 Content and Organization

This report is written and organized to facilitate use and potential adoption (with some modification) by seismic codes and standards development committees and product evaluation services with an interest in evaluating component equivalency.

Chapter 1 explains the background, objectives, and approach used in developing the recommended Component Methodology.

Chapter 2 presents the complete requirements of the Component Methodology. It defines the scope, terminology, and applicability, and provides step-by step requirements and acceptance criteria for evaluating component equivalency. The intent of Chapter 2 is to describe the Component Methodology in a stand-alone form that could readily be used as the basis for code, standard, or acceptance criteria requirements for component equivalency. The text, however, is not written in mandatory language and would require modification before adoption into a code, standard, or acceptance criteria document.

Chapter 3 is closely related to Chapter 2, providing section-by-section commentary on the requirements of the Component Methodology.

Chapter 4 provides an example application of the Component Methodology applied to the proposed substitution of a hypothetical new product in place of nailed wood structural panels in wood light-frame construction.

Chapter 5 concludes the main body of the report, providing a summary of the work and recommendations for future related research.

The appendices provide background information on the technical development of the Component Methodology and summarize test applications on additional components and reference seismic-force-resisting systems. Appendix A documents the analytical studies used to identify the key performance parameters related to collapse resistance of a seismic-force-resisting system.

Appendix B provides supporting material related to requirements for cyclic and monotonic testing of proposed and reference components.

Appendix C describes development of probabilistic-based acceptance criteria that are consistent with the collapse performance objectives of the FEMA P-695 Methodology.

Appendix D describes a test application of the Component Methodology for the substitution of stapled wood shear walls for nailed wood shear walls in a wood light-framed seismic-force-resisting system.

Appendix E describes a test application of the Component Methodology for the substitution of buckling-restrained braces for conventional braces in a special steel concentrically braced frame system.

Appendix F describes a test application of the Component Methodology for the substitution of a pre-fabricated wall product for nailed wood shear walls in a wood light-framed seismic-force-resisting system.

Chapter 2

Component Equivalency Methodology

2.1 Introduction

2.1.1 Scope

The Component Equivalency Methodology (referred to as the Component Methodology) is a statistically based procedure for evaluating and comparing test data to determine the seismic performance equivalency of new components (*proposed components*) that are proposed as substitutes for selected components (*reference components*) in a current code-approved seismic-force-resisting system (*reference SFRS*). The Component Methodology is intended for evaluation of proposed components whose inelastic deformation behavior is a central aspect to the performance of the reference SFRS. The Component Methodology is intended for evaluation of proposed components whose inelastic deformation behavior is a central aspect to the performance of the reference SFRS.

The Component Methodology is derived from the general methodology contained in FEMA P-695 *Quantification of Building Seismic Performance Factors* (FEMA, 2009b). Similar to the general methodology in FEMA P-695, the intent of the Component Methodology is to ensure that code-designed buildings have adequate resistance to earthquake-induced collapse. In the case of component equivalency, this intent implies equivalent safety against collapse when proposed components are substituted for reference components in a reference SFRS.

Proposed components found to be equivalent by the Component Methodology can be substituted for components of the reference SFRS, subject to design requirements and seismic design category restrictions on the use of the reference SFRS. Reference seismic-force-resisting systems include those systems contained in ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010) and other building code standards.

The Component Methodology is applicable to proposed components that meet the criteria of Section 2.3. For components that do not meet the criteria of Section 2.3, the more general procedures of the FEMA P-695

Methodology should be used to demonstrate seismic performance equivalency.

2.1.2 General Approach

The Component Methodology evaluates equivalency between proposed components and reference components based on criteria that compare statistical values of key performance parameters determined through cyclic-load and monotonic-load testing.

The primary criterion is based on deformation capacities determined from cyclic-load and monotonic-load testing. Additional parameters related to component strength, stiffness, and effective ductility are used to establish limits on acceptable differences between the properties of proposed and reference components.

Component test data are evaluated by comparing statistics of test data that are classified into *component performance groups*, comprised of component configurations that share common features. Multiple performance groups are required for components that have significantly different behavioral characteristics associated with distinctly different design features. For example, a component would require separate performance groups if behavior changed significantly as a function of component geometry (e.g., wood panel performance as function of wood panel aspect ratio). For each performance group, equivalency between proposed and reference components is evaluated by comparing median values of performance parameters for components in that group.

Each performance group contains a set of component configurations, which are defined by the component geometry, design strength, and other defining attributes. A large number of component configurations are required to statistically evaluate performance for the range of intended application of the proposed component. At least two *component test specimens* are required for each proposed component configuration. In addition to a basic check on the median deformation capacity of the entire performance group, the deformation capacity of each proposed component configuration is checked to ensure that no configuration has a deformation capacity that is substantially lower than the range of values within the performance group.

2.1.3 Description of Process

Figure 2-1 provides a flowchart illustrating the steps of the Component Methodology. The process is based on test data developed for the proposed and reference components in accordance with Section 2.2. It includes an evaluation of the applicability of the Component Methodology to the proposed component and the reference SFRS of interest (Section 2.3). Criteria for evaluating the applicability of the Component Methodology are based (in part) on the quality ratings of the test data and component design requirements defined in Section 2.7. Application of the Component Methodology is limited to reference SFRSs with test data and design requirements of reasonable quality. In cases where the Component Methodology is not applicable, the FEMA P-695 Methodology should be used to evaluate seismic performance equivalency.

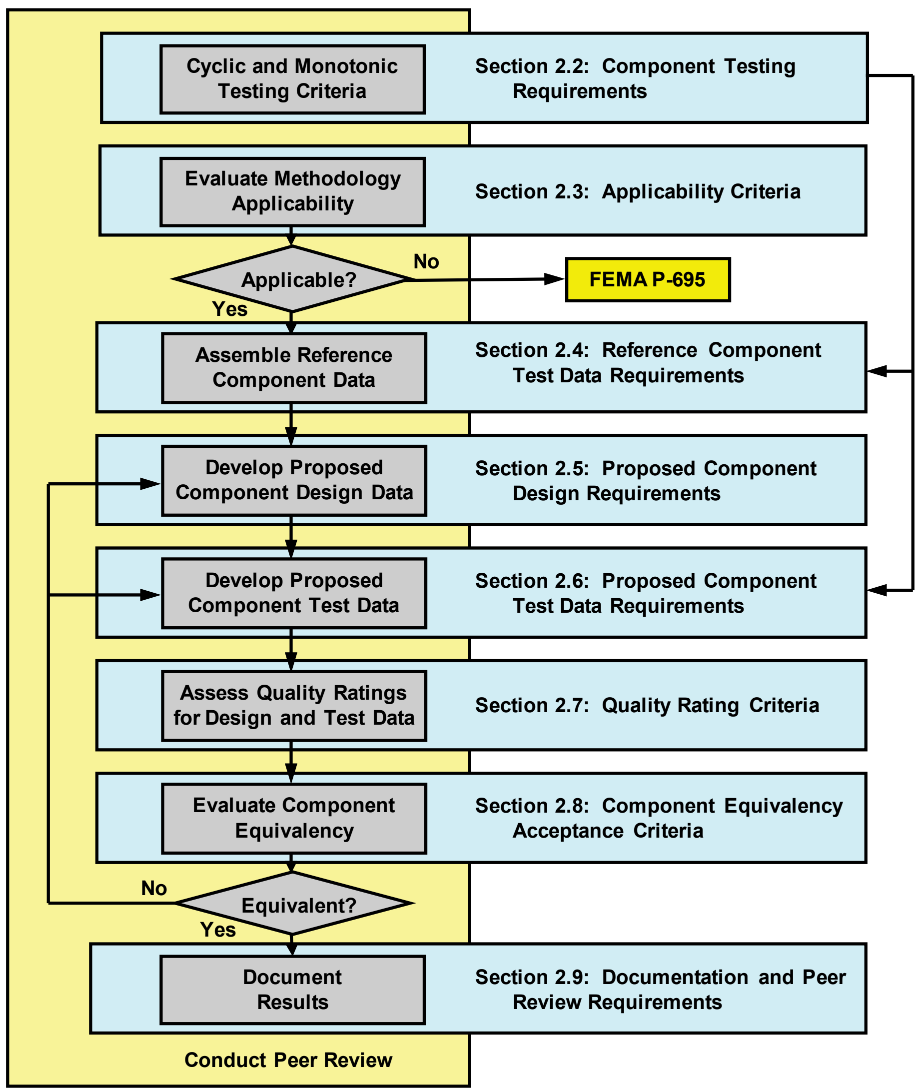


Figure 2-1 Process for establishing and documenting component equivalency.

Reference component test data and design requirements for the reference SFRS must be assembled (Section 2.4). Component performance groups are identified and test data are collected for different component configurations in each performance group. Statistical values of performance parameters are calculated from test data of the reference component. Although it is expected that reference component test data will be collected from available sources of information, some testing of reference components may be necessary for adequate comparison with proposed component test data.

Design requirements and test data are developed for the proposed component (Section 2.5 and Section 2.6, respectively). Where significant reference component data already exist, testing plans for the proposed component can be tailored to coincide with data and meet the needs of the Component Methodology. Component performance groups and component configurations for proposed and reference components should be compatible, and should represent the full range of the intended application of the proposed component. Statistical measures of performance parameters are developed from cyclic-load and monotonic-load test data. Although monotonic-load test data are optional for reference components, they are required for proposed components.

Quality of test data and design requirements are assessed using the criteria in Section 2.7. Quality ratings for the reference component are used to ensure that the seismic-force-resisting system of interest is suitable for use as a reference SFRS. Quality ratings for the proposed component are used to establish the acceptance criteria for equivalency. More stringent criteria are imposed for proposed components with lower quality test data and design requirements.

Component equivalency is evaluated by comparing statistical values of the reference and proposed component performance parameters in accordance with Section 2.8. Proposed components that meet the equivalency criteria can be used to replace components in the reference SFRS, subject to other design criteria and restrictions on use. For proposed components that do not meet the equivalency criteria, design requirements could be revised (e.g., use could be restricted to those configurations with acceptable performance), or test data could be augmented to improve the quality rating, and the proposed component could be re-evaluated with revised data.

Finally, use of the Component Methodology to determine equivalency should be fully documented and peer reviewed (Section 2.9).

2.1.4 Terminology

Component: A structural element, connection, or subassembly of the seismic-force-resisting system, or combination thereof, within the component boundary.

Component Boundary: Defined boundary between the component and the balance of the reference SFRS.

Component Configuration: Specific combination of component properties including component geometry, detailing, and the manner in which the component connects to the rest of the seismic-force-resisting system.

Component Design Space: Range of possible configurations in which the structural component can be used.

Component Performance Group: A subset of the component design space containing a group of component configurations that share a common set of features or behavioral characteristics.

Component Test Specimen: Fabricated structural element, connection, or subassembly representing a component configuration.

Cyclic-Load Testing: Physical testing that is based on complete reversals in the direction of loading. Example requirements related to cyclic-load testing can be found in ASTM E2126-09 *Standard Test Method for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Load Force Resisting System for Buildings* (ASTM, 2009).

Cyclic Envelope Curve: Envelope of cyclic-load test data for a particular component, used to evaluate maximum load, Q_M, initial stiffness, K_I, effective yield deformation, $\Delta_{Y,eff}$, and ultimate deformation, Δ_U, parameters, as illustrated in Figure 2-2.

Load History: The amplitude and sequence of deformations applied to the test specimen during cyclic-load testing.

Loading Protocol: The procedures governing cyclic-load and monotonic-load testing of components.

Monotonic-Load Testing: Physical testing that is based on loading the test specimen in a single direction, with no reversals in the direction of loading.

Monotonic Curve: Plot of monotonic-load test data used to evaluate the monotonic maximum load, Q_{MM}, and ultimate deformation, Δ_{UM}, parameters, as illustrated in Figure 2-3.

Proposed Component: Structural component proposed for substitution in the designated reference SFRS.

Quality Rating: Judgment-based measure of the quality of component test data and design requirements, based on criteria provided in Section 2.7.

Reference Component: Structural component of the reference SFRS used to evaluate equivalency of the proposed component.

Reference SFRS: Seismic-force-resisting system in which proposed components would be substituted for reference components.

2.1.5 Notation

K_D	=	Design stiffness of proposed or reference component configuration, as derived from, or specified in, design requirements documentation.
K_I	=	Effective value of initial stiffness of the component test specimen through the secant at $0.4Q_M$, based on positive and negative cycles of loading, as shown in Figure 2-2.
P_Q	=	Penalty factor due to the difference in the maximum load (strength) ratios of proposed and reference components, as defined in Table 2-4.
P_U	=	Penalty factor due to the collective uncertainty associated with proposed component design requirements, β_{DR}, and test data, β_{TD}, as defined in Table 2-3.
Q	=	Generic load (i.e., force, moment) on proposed or reference component test specimen, or load specified for design.
Q_D	=	Load corresponding to the specified design strength of a component configuration, as derived from, or specified in, design requirements documentation.
Q_M	=	Maximum load applied to a component during cyclic-load testing, based on positive and negative cycles of loading, as shown in Figure 2-2.
Q_{MM}	=	Maximum load applied to a component during monotonic testing, as shown in Figure 2-3.
R_K	=	Ratio of initial stiffness, K_I, to design stiffness, K_D, for a component test specimen.
R_Q	=	Ratio of the maximum cyclic load, Q_M, to the design load, Q_D, of a component test specimen.
β_{DR}	=	Uncertainty associated with the design requirements of the proposed or reference component, as specified in Section 2.7.2.
β_{TD}	=	Uncertainty associated with the test data of the proposed or reference component, as specified in Section 2.7.1.
Δ	=	Generic deformation (e.g., displacement, rotation) of a proposed or reference component test specimen.
Δ_U	=	Ultimate deformation of a component test specimen at $0.8Q_M$ based on positive and negative cycles of loading during cyclic-load testing, as shown in Figure 2-2.

Δ_{UM} = Ultimate deformation of a component test specimen at $0.8Q_{MM}$ based on monotonic-load testing, as shown in Figure 2-3.

$\Delta_{Y,eff}$ = Effective yield deformation of a component test specimen during cyclic-load testing based on positive and negative cycles of loading, defined by the ratio, Q_M / K_I, as shown in Figure 2-2.

μ_{eff} = Effective ductility capacity of a component test specimen, defined as the ultimate deformation, Δ_U, divided by the effective yield deformation, $\Delta_{Y,\,eff}$.

2.1.6 Statistical Notation

Note: The following median and lognormal standard deviation parameters are calculated from the associated data set assuming that the variable of interest has a lognormal probability distribution.

$\tilde{D}_C$ = Median cumulative damage factor based on the cyclic-load testing protocol, as defined in Section 2.8.5.

$\tilde{R}_{K,PC}$ = Median value of the initial stiffness ratio, R_K, for a proposed component (PC) performance group.

$\tilde{R}_{K,RC}$ = Median value of the initial stiffness ratio, R_K, for a reference component (RC) performance group.

$\tilde{R}_{Q,PC}$ = Median value of the maximum load (strength) ratio, R_Q, for a proposed component (PC) performance group.

$\tilde{R}_{Q,RC}$ = Median value of the maximum load (strength) ratio, R_Q, for a reference component (RC) performance group.

$\tilde{\Delta}_{U,PC}$ = Median value of ultimate deformation based on cyclic-load testing for a proposed component (PC) performance group.

$\tilde{\Delta}_{Uj,PC}$ = Median value of ultimate deformation based on cyclic-load testing for a proposed component (PC) of configuration j.

$\tilde{\Delta}_{UM,PC}$ = Median value of ultimate deformation based on monotonic-load testing for a proposed component (PC) performance group.

$\tilde{\Delta}_{U,RC}$ = Median value of ultimate deformation based on cyclic-load testing for a reference component (RC) performance group.

$\tilde{\Delta}_{UM,RC}$ = Median value of ultimate deformation based on monotonic-load testing for a reference component (RC) performance group.

$\tilde{\Delta}_{Y,eff,PC}$ = Median value of the effective yield deformation for a proposed component (PC) performance group.

$\tilde{\Delta}_{Y,eff,RC}$ = Median value of the effective yield deformation for a reference component (RC) performance group.

$\tilde{\mu}_{eff,PC}$ = Median value of effective ductility capacity for a proposed component (PC) performance group.

$\tilde{\mu}_{eff,RC}$ = Median value of effective ductility capacity for a reference component (RC) performance group.

$\sigma_{RK,PC}$ = Lognormal standard deviation (variability) of the initial stiffness ratio, R_K, of a proposed component (PC) performance group.

$\sigma_{RK,RC}$ = Lognormal standard deviation (variability) of the initial stiffness ratio, R_K, of a reference component (PC) performance group.

$\sigma_{RQ,PC}$ = Lognormal standard deviation (variability) of the maximum load (strength) ratio, R_Q, of a proposed component (PC) performance group.

$\sigma_{RQ,RC}$ = Lognormal standard deviation (variability) of the maximum load (strength) ratio, R_Q, of a reference component (PC) performance group.

$\sigma_{\Delta U,PC}$ = Lognormal standard deviation (variability) of the ultimate cyclic-load testing deformation for a proposed component (PC) performance group.

$\sigma_{\Delta U,RC}$ = Lognormal standard deviation (variability) of the ultimate cyclic-load testing deformation for a reference component (RC) performance group.

$\sigma_{\Delta I,PC}$ = Lognormal standard deviation (variability) of the initial stiffness deformation, Δ_I, for a proposed component (PC) performance group.

$\sigma_{\Delta I,RC}$ = Lognormal standard deviation (variability) of the initial stiffness deformation, Δ_I, for a reference component (RC) performance group.

2.2 Component Testing Requirements

This section includes general requirements for component testing. It describes test procedures and test data specific to cyclic-load testing and monotonic-load testing of components. These requirements are general in nature and apply to testing of both reference components and proposed components. Guidance on selecting specific component configurations for testing is provided in Section 2.4 (reference components) and Section 2.6 (proposed components). Criteria for rating the quality of test data are given in Section 2.7. Test data are used directly or with other specified data, in order to calculate statistical values of performance parameters required for component equivalency evaluation.

When interpreting test data, the generalized load, Q, and the generalized deformation, Δ, can be expressed in different ways, depending on the properties of the component. The choice of the appropriate load quantity (e.g., shear, moment) and the appropriate deformation quantity (e.g., displacement, drift, rotation) will depend on the component under consideration, but the quantities should be consistent between the proposed and reference component performance groups.

2.2.1 General Requirements for Component Testing

The following criteria should be considered when developing a program for component testing (e.g., testing of proposed component specimens), or when evaluating the quality of existing data from a previous testing program (e.g., assembling reference component test data).

- **Size effects.** Tests should be performed on full-size components unless it can be shown by theory or experimentation that testing of reduced-scale specimens will not significantly affect behavior.
- **Boundary conditions.** The boundary conditions of component tests should be: (1) representative of constraints that a component would experience in a typical structural system; and (2) sufficiently general so that the results can be applied to boundary conditions that might be experienced in other system configurations (within the bounds of those considered in Section 2.6.1). Boundary conditions should not impose beneficial effects on seismic behavior that would not exist in common system configurations.
- **Load application.** Loads should be applied to test specimens in a manner that replicates the transfer of load to the component as it would occur in common system configurations, and tests should generally be conducted using displacement control unless the component under investigation requires load-control testing (e.g., anchorage devices in a wood light-frame system).

  For components that resist vertical loads (gravity and overturning loads), test loading should include these loads, unless it can be shown that they do not significantly influence component performance.

  Exception. Reference component test data without applied vertical loads can be utilized, provided that the inclusion of vertical loads would not increase the reference component ultimate deformation ($\Delta_{U,RC}$ and $\Delta_{UM,RC}$).
- **Test specimen construction.** Specimens should be constructed in a setting that simulates commonly encountered field conditions. For example, if field conditions necessitate a particular construction technique (e.g., overhead welding) then the same techniques should be used in the construction of the test specimen.

- **Quality of test specimen construction.** The component should be of a construction quality that is equivalent to what will be commonly implemented in the field. Special construction techniques or quality control measures should not be employed, unless they are a required part of the design requirements for the component (see Section 2.5).
- **Testing of materials.** Material testing should be conducted when such data are needed to develop properties for component design requirements.
- **Laboratory accreditation.** Testing laboratories used to conduct an experimental investigation program should generally comply with national or international accreditation criteria, such as the ISO/IEC 17025 *General Requirements for the Competence of Testing and Calibration Laboratories* (ISO, 2005).

  Exception. Testing laboratories that are not accredited may be used for the experimental investigation program provided that the same information required by ISO/IEC 17025 is provided to the peer review panel, and the panel verifies acceptability of the laboratory.
- **Instrumentation.** Instrumentation should be installed to permit reliable measurement of all required strength, stiffness, and deformation quantities. Where necessary, deformation measurements should be corrected to remove rigid body displacement effects, inertial effects, or deformations due to the flexibility of the test apparatus.

2.2.2 Cyclic-Load Testing

Procedures used for cyclic-load testing of reference and proposed components should, as a minimum, comply with the requirements of this section.

Cyclic-Load Testing Protocol

Cyclic-load testing should be performed in accordance with the following protocol:

- Proposed components and reference components should be tested with load histories that are equivalently damaging, quantified in terms of accumulated deformation imposed on the test specimen.
- The number of cycles should be sufficient to measure possible degradation of strength, stiffness, or energy dissipation capacity of the component under repeated cycles of loading.
- The deformation history should be described in terms of a well-defined quantity (e.g., displacement, story drift, rotation) and should consist of

essentially symmetric deformation cycles of step-wise increasing amplitude. Cycles of smaller amplitudes between cycles of increasing amplitudes (trailing cycles) should only be included in the deformation history if they affect the cyclic response of the component.

- Proposed and reference component specimens should be tested to deformations large enough to achieve a 20% reduction in applied load, and therefore reach the ultimate deformation, Δ_U, in at least one direction of loading.

  Exception. Proposed component specimens need only be tested to deformations large enough to ensure compliance with the minimum equivalency criteria.

Number of Cyclic Test Specimens

Cyclic test specimens for all component configurations in each performance group should meet the following minimum requirements:

- The number of component configurations should be sufficiently large in number to characterize the range of design parameters and component behavior for the performance group (with the actual number requiring approval by the peer review). As a minimum, four component configurations should be included in each performance group.

  Exception. The number of component configurations in a given performance group need not exceed the number of possible configurations of the proposed component, provided that the performance group includes at least eight component test specimens.

- A minimum of two component test specimens should be included for each component configuration. A minimum of three tests should be included if any of the following occur: (1) if rapid and unpredicted deterioration occurs (such as that caused by brittle fracture); (2) if the strength, Q_M, varies by more than 15% between the two tests; or (3) if the ultimate deformation capacity, Δ_U, varies by more than 20% between the two tests.

  Exception. For the reference component data set only one specimen per configuration is permitted, provided that a minimum of 8 test specimens are included in the performance group, rapid and unpredictable deterioration is not observed, and the ultimate deformation capacity, Δ_U, does not vary by more than 20% as compared to specimens of similar configuration.

Cyclic-Load Test Data

- The following parameters should be determined using the cyclic envelope curve from the cyclic-load test data of each component test specimen:
  - Ultimate load, Q_M
  - Ultimate deformation, Δ_U (deformation at $0.8Q_M$)
  - Initial stiffness, K_I (based on force and deformation at $0.4Q_M$)
  - Effective yield deformation, $\Delta_{Y,eff}$ ($\Delta_{Y,eff} = Q_M / K_I$)
  - Effective ductility capacity, μ_{eff} ($\mu_{eff} = \Delta_U / \Delta_{Y,eff}$)
- Values of each parameter should be measured from both positive and negative portions of the envelope curve, as illustrated in Figure 2-2.

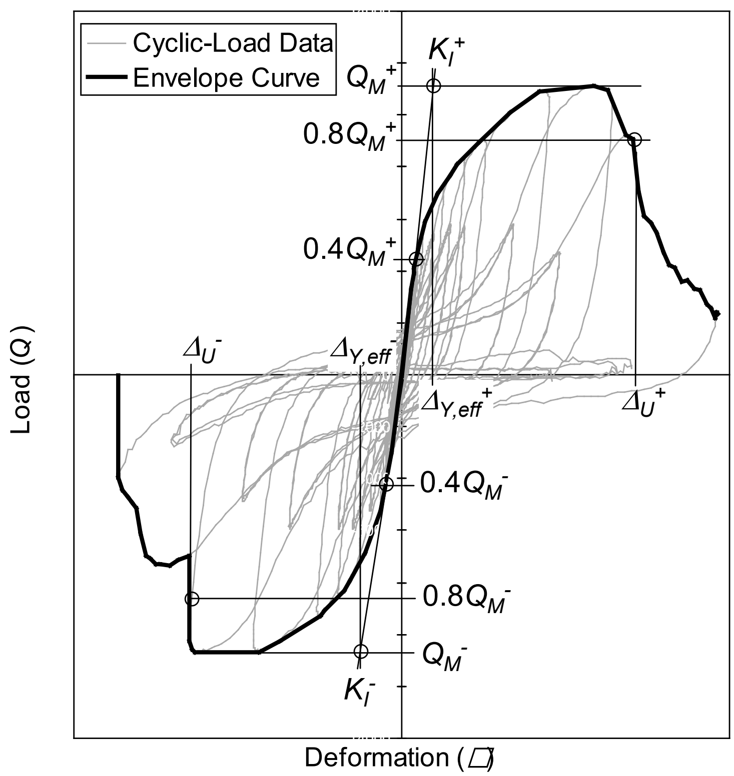


Figure 2-2 Illustration of cyclic-load test data, envelope curve, and maximum load, Q_M, effective yield deformation, $\Delta_{Y,eff}$, ultimate deformation, Δ_U, and initial stiffness, K_I, parameters for a component test specimen.

- The value of the ultimate deformation, Δ_U, is taken as the deformation corresponding to 80 percent of the maximum load, Q_M.

Exception. If the vertical load-carrying capacity of the component is compromised at a deformation less than that corresponding to $0.8Q_M$, then the ultimate deformation capacity, Δ_U, should be calculated as the deformation corresponding to loss of vertical-load carrying capacity.

- For components with reasonably symmetric behavior, values of ultimate deformation, Δ_U, initial stiffness, K_I, effective yield deformation, $\Delta_{Y,eff}$, and the effective ductility, μ_{eff}, should be calculated as the average of their respective values determined from the positive and negative portions of the envelope curve.
- For components with significant asymmetric behavior, positive and negative values of ultimate deformation, Δ_U, initial stiffness, K_I, effective yield deformation, $\Delta_{Y,eff}$, and effective ductility capacity, μ_{eff}, should be calculated and evaluated separately for each loading direction.

Construction of the Cyclic Envelope Curve

The cyclic envelope curve should be constructed in accordance with the following requirements:

- The curve for should be constructed separately for positive and negative directions of loading.
- At each level of deformation, up to the peak load experienced in the test, the load value of the cyclic envelope curve should be taken as the greater of: (1) the maximum value of load for all cycles at that level of deformation; or (2) the value of load described by a series of straight lines that connect points of peak load at subsequent deformation amplitudes.
- After the peak load has been reached, the envelope curve should be defined using only (1) above for the following two cases:
  - If there is more than 20% difference in peak loads at subsequent deformation amplitudes.
  - If the cyclic response curve has a negative stiffness (i.e., strength is lost in a single cycle of loading).
- The value of the cyclic envelope curve should drop to zero load at the maximum deformation executed in the test.

2.2.3 Monotonic-Load Testing

Procedures used for monotonic-load testing of reference and proposed components should, as a minimum, comply with the requirements of this section.

Monotonic-Load Testing Protocol

Monotonic-load testing should be performed in accordance with the following protocol:

- Component test specimens should be tested in both directions for components that have significant asymmetric behavior.
- Component test specimens should be tested to deformations large enough to achieve a 20% reduction in applied load, and therefore reach the ultimate deformation, Δ_{UM}.

  Exception. Proposed component specimens need only be tested to deformations large enough to ensure compliance with the minimum equivalency criteria.

Number of Monotonic-Load Test Specimens

Monotonic-load test specimens should meet the following minimum requirements:

- For each performance group, the set of component configurations used for monotonic-load testing should be representative of the larger set of configurations used for cyclic-load testing.
- A minimum of two component configurations should be included for each component performance group.
- A minimum of two component test specimens should be included for each selected component configuration. A minimum of three tests should be included if any of the following occur: (1) rapid and unpredicted deterioration (such as that caused by brittle fracture); (2) the strength, Q_{MM}, varies by more than 15% between the two tests; or (3) the ultimate deformation capacity, Δ_{UM}, varies by more than 30% between the two tests.

  Exception. Monotonic-load test data are not required for reference components, provided that the ultimate deformation capacity of the proposed component is shown to comply with Equation 2-6.

Monotonic-Load Test Data

The following parameters should be determined from the monotonic-load test data of each component test specimen:

- Ultimate load, Q_{MM}
- Ultimate deformation, Δ_{UM} (deformation at 0.8 Q_{MM})

Values of these parameters should be measured from the monotonic curve, as illustrated in Figure 2-3:

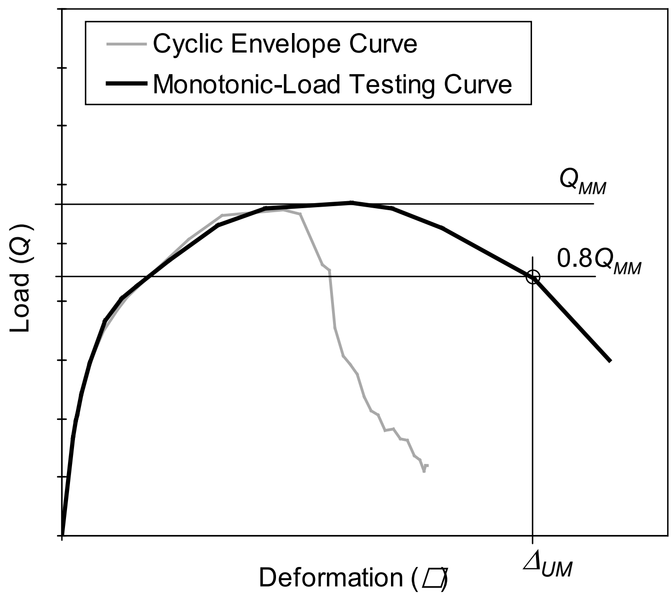


Figure 2-3 Illustration of a monotonic curve and determination of maximum load, Q_{MM}, and ultimate deformation, Δ_{UM}, parameters for a component test specimen.

The value of the ultimate deformation, Δ_{UM}, is taken as the deformation corresponding to 80 percent of the maximum load, Q_{MM}.

Exception. If the vertical load-carrying capacity of the component is compromised at a deformation less than that corresponding to $0.8Q_{MM}$, then the ultimate deformation capacity, Δ_{UM}, should be calculated as the deformation corresponding to loss of vertical-load carrying capacity.

For components whose cyclic response is essentially symmetric, the value of the ultimate deformation, Δ_{UM}, may be calculated for a single direction of monotonic-load testing. For components with asymmetric behavior, separate test specimens are required for testing in each direction of deformation, and values of the maximum load, Q_{MM}, and the ultimate deformation, Δ_{UM}, should be calculated separately for each direction (i.e., both positive and negative values are required for component equivalency evaluation).

2.3 Applicability Criteria

The Component Methodology may be used to evaluate a proposed component that complies with the criteria of this section. Applicability criteria address the suitability of the reference SFRS, the adequacy of reference component design criteria and test data, the adequacy of proposed component design criteria and test data, and the characteristics of a proposed component that would permit the use of the Component Methodology.

2.3.1 Required Information and Data

Design requirement information and test data should be collected or developed for proposed and reference components in order to:

- Determine applicability of the Component Methodology
- Determine values of all parameters required by the Component Methodology

2.3.2 Reference Seismic-Force-Resisting System: Collapse Performance Criteria

The reference SFRS should comply with the collapse performance criteria contained in the FEMA P-695 Methodology. For purposes of the Component Methodology, it is assumed that currently approved seismic-force-resisting systems contained within ASCE/SEI 7-10 comply with these criteria.

2.3.3 Quality Rating Criteria

Quality of test data and design requirements of the reference and proposed components should be rated in accordance with the requirements of Sections 2.7.1 and 2.7.2, respectively, and should comply with the following criteria:

- For reference components, the quality rating of design requirements and test data should be Good or Superior.
- For proposed components, the quality rating of design requirements and test data should be Fair, Good, or Superior.

2.3.4 General Criteria

The proposed component, reference component, and reference SFRS should comply with the following general criteria:

- **Component boundary.** The boundary between the components and the balance of the reference SFRS should be defined, such that:
    - Component test specimens and their connections to the reference SFRS are clearly defined.
    - Testing boundary conditions are clearly established and realistically represent the interaction between the component and the reference SFRS.
    - The component boundary is the same for the proposed and reference components and the transfer of forces across the component

boundary is essentially the same for proposed and reference components.

Exception. When proposed and reference component configurations are substantially different and necessitate different component boundaries, then different component boundaries are permitted provided that the transfer of forces across the proposed component boundary does not adversely affect the vertical-load carrying capacity of the proposed component and performance of the balance of the structure.

- **Balance of structure.** The balance of the reference SFRS and the distribution of forces and deformations beyond the component boundary are essentially unchanged by replacement of reference components with proposed components.
- **Component properties.** Load-deformation properties of proposed and reference components are substantially independent of the rate of loading (e.g., components are not velocity dependent).
- **Component testing.** Nonlinear (inelastic) response of components can be reliably measured by cyclic-load and monotonic-load testing.
- **Component similarity.** Proposed and reference components have a comparable range of seismic load resistance and capacity to support vertical loads (for components that support vertical loads).
- **Vertical-load carrying capacity.** For components that support vertical loads, the vertical-load carrying capacity of the proposed component is sufficient to resist vertical loads in combination with earthquake-induced deformations up to the ultimate deformation, Δ_U.
- **Seismic isolators and dampers.** Proposed and reference components are not intended for use as either an isolator unit, as defined by Chapter 17 of ASCE/SEI 7-10, or a damping device, as defined by Chapter 18 of ASCE/SEI 7-10.

2.4 Reference Component Test Data Requirements

2.4.1 Define Reference Component Design Space

The reference component design space should be established based on component configurations derived from established practice for the reference SFRS and the intended substitution with proposed components. The reference component design space should be developed in parallel with the proposed component design space (Section 2.6.1), and there should be consistency between the two.

The range of component configurations should include variations in system geometry, component section and material properties, and detailing requirements that are used in practice. Configurations should reflect any fundamental differences in how the reference component is used or in how it performs for different permissible configurations.

2.4.2 Compile or Generate Reference Component Test Data

The reference component test data should be compiled for component configurations and performance groups that reflect current use of the reference component. Data should be compiled from published sources or obtained from tests that are conducted specifically for the equivalency evaluations. Cyclic-load and monotonic-load test data should comply with the minimum requirements of Section 2.2.

2.4.3 Interpret Reference Component Test Results

Performance parameters should be determined from results of cyclic-load and monotonic-load testing for each test specimen, in accordance with Section 2.2. In addition to the measured quantities, Q_M, K_I, Δ_U, $\Delta_{Y,eff}$, μ_{eff}, Q_{MM}, and Δ_{UM}, the following quantities should be computed using the design strength and stiffness values, Q_D and K_D, as determined by the reference component design requirements:

- Ratio of measured maximum strength to design strength ($R_Q = Q_M / Q_D$)
- Ratio of measured initial stiffness to design stiffness ($R_K = K_I / K_D$)

2.4.4 Define Reference Component Performance Groups

The reference component data set should be placed into a single performance group unless there are fundamental differences in behavior among reference component data.

If there are fundamental differences in behavior among reference component data, the reference component data set should be separated into two or more performance groups that reflect major divisions, or changes in behavior, within the set of data that can be associated with different component geometries, detailing requirements, or loading conditions. Separation of reference component test data into performance groups should be based on the following considerations:

- A fundamental change in component design requirements
- A systematic difference in failure mode among different configurations

- A fundamental geometric difference that tangibly affects the ratio of the inelastic deformation capacity to the effective yield deformation of the component (e.g., a short versus long cantilever column)
- A substantial difference between loading histories used in testing
- A difference in the load or deformation quantity used to quantify component performance
- Any other difference that causes a clear change in the behavior or performance of the component

When separation into performance groups is necessary, reference component performance groups should be established in parallel with proposed component performance groups (Section 2.6.5), and there should be consistency in the manner in which the data are organized for both reference components and proposed components.

2.4.5 Compute Summary Statistics

Reference component summary statistics (i.e., median and lognormal standard deviation values) should be computed for each of the component parameters listed in Sections 2.2 and 2.4.3. Median and lognormal standard deviation values should be computed separately for each parameter in each component performance group, assuming that the component parameters have a lognormal probability distribution.

2.5 Proposed Component Design Requirements

A comprehensive set of design requirements should be developed for the proposed component.

2.5.1 Component Design Strength and Stiffness

Design requirements should include methods to predict both the design strength and design stiffness (effective stiffness at the design strength, Q_D) of the component. Methods for computing design strength and stiffness should identify the adopted design procedure (e.g., Load and Resistance Factor Design (LRFD) or Allowable Stress Design (ASD)). Where applicable, design requirements should include parameters to determine the expected strength of components that might be necessary for design of other elements in the reference SFRS (e.g., R_y factors for steel structures).

Certain acceptance criteria in Section 2.8 are related to restrictions on proposed components that are imposed within the design requirements. Proposed component design requirements should, therefore, be developed consistent with these restrictions. In addition, design strength requirements should be appropriate for other non-seismic concerns, such as elastic serviceability checks, design for gravity and wind loading, and fire resistance.

2.5.2 Component Detailing Requirements

Design requirements should include comprehensive provisions for seismic detailing of proposed components. Detailing requirements should cover the range of allowable configurations defined in Section 2.6.1.

2.5.3 Component Connection Requirements

Design requirements should include provisions for attachment of the proposed component to the balance of the reference SFRS. These requirements should ensure that the connection is strong enough to develop the full ultimate strength of the component, such that inelastic behavior occurs in the element, connection, or subassembly that is defined as the proposed component, and not at the boundary between the proposed component and the balance of the reference SFRS.

2.5.4 Limitations on Component Applicability and Use

Design requirements should include provisions that reasonably restrict the proposed component usage to the range of allowable configurations defined in Section 2.6.1. They should consider the vertical-load carrying capacity of the proposed component (for components that support vertical load) and ensure that the vertical-load carrying capacity of the proposed component is equal to, or greater than, that of the reference component.

For components tested without vertical loads, design provisions should restrict usage to ensure that the component cannot be substituted in a system where the level of vertical loads would tangibly affect the strength, deformation capacity, or stability of the component.

2.5.5 Component Construction, Inspection, and Maintenance Requirements

Design and quality assurance requirements should include provisions to ensure that the proposed component and the connection between the proposed component and the balance of the reference SFRS are properly constructed. These requirements should ensure that the as-built performance of the component will be consistent with the laboratory test data used as the basis for demonstrating equivalency.

Design and quality assurance requirements should include provisions related to inspection and maintenance that are necessary to ensure that the seismic performance of the proposed component and connection to the balance of the seismic-force-resisting system do not measurably deteriorate over the intended life of the structure.

2.6 Proposed Component Test Data Requirements

2.6.1 Define Proposed Component Design Space

The range of allowable configurations of the proposed component should be defined and documented. These configurations comprise the proposed component design space, and allowable configurations should adhere to the same requirements as described in Section 2.4.1 for reference components. The proposed component design space should be established in parallel with the reference component design space (Section 2.4.1), and there should be consistency between the two.

Exception. The proposed component design space may be more restrictive than the reference component design space, provided the design requirements similarly restrict the applicability and use of the proposed component.

2.6.2 Select Proposed Component Configurations for Testing

Proposed component configurations should be selected for testing based on the proposed component design space, which encompasses the envisioned use of the proposed component. The selected set of configurations should be sufficiently broad to capture the full range of the component design space. Selected configurations should represent all failure modes that are possible within the component design space. In addition, this selection process should consider those configurations expected to be most frequently used in practice.

2.6.3 Perform Cyclic-Load and Monotonic-Load Tests

The selected set of proposed component configurations should be tested in accordance with the requirements of Section 2.2.

2.6.4 Interpret Proposed Component Test Results

Performance parameters should be determined from results of cyclic-load and monotonic-load testing for each test specimen, in accordance with the requirements of Section 2.2. In addition to the measured quantities, Q_M, K_I, Δ_U, $\Delta_{Y,eff}$, Q_{MM}, μ_{eff}, and Δ_{UM}, the following quantities should be computed using the design strength and stiffness values, Q_D and K_D, as determined by the proposed component design requirements:

- Ratio of measured maximum strength to design strength ($R_Q = Q_M / Q_D$)

- Ratio of measured initial stiffness to design stiffness ($R_K = K_I / K_D$)

2.6.5 Define Proposed Component Performance Groups

The proposed component data set should be placed into a single performance group unless there are fundamental differences in behavior among proposed component data. If there are fundamental differences in behavior among the proposed component data, the data should be separated into two or more performance groups following the same requirements specified for reference components in Section 2.4.4.

When two or more performance groups are necessary, proposed component performance groups should be established in parallel with the reference component performance groups (Section 2.4.4). There should be consistency in the manner in which the data are organized into performance groups, to the extent that the divisions used for the reference component are also applicable to the proposed component. Each proposed component performance group should be clearly associated with a reference component performance group in which the proposed components are intended to be substituted for reference components.

2.6.6 Compute Summary Statistics

Proposed component summary statistics (i.e., median and lognormal standard deviation values) should be computed for each of the component parameters listed in Sections 2.2 and 2.6.4. Median and lognormal standard deviation values should be computed separately for each parameter in each component performance group, assuming that the component parameters have a lognormal probability distribution.

2.7 Quality Rating Criteria

2.7.1 Quality Rating of Test Data

Test data for the reference component and the proposed component should be rated in accordance with the criteria in Table 2-1. The quality rating of test data should be based on the completeness and robustness of the overall testing program, and confidence in the test results. Minimum quality ratings for applicability of the Component Methodology are defined in Section 2.3.3.

Completeness and Robustness of Tests

Evaluation of the completeness and robustness of the testing program should be based on the following considerations:

- The degree to which relevant testing issues have been considered in the development of the testing program.
- The extent to which the testing program and other documented experimental evidence quantify the necessary material, component, and connection properties (connection between the component and the rest of the structural system) and important behavior and failure modes.

Table 2-1 Quality Rating of Test Data

Completeness and Robustness of Tests	Confidence in Test Results		
	High	Medium	Low
High. Material, component, and connection behavior well understood and accounted for. All, or nearly all, important testing issues addressed.	**Superior**	**Good**	**Fair**
Medium. Material, component, and connection behavior generally understood and accounted for. Most important testing issues addressed.	**Good**	**Fair**	**Not Permitted**
Low. Material, component, and connection behavior fairly understood and accounted for. Several important testing issues not addressed.	**Fair**	**Not Permitted**	**Not Permitted**

Confidence in Test Results

Confidence in test results is related to the reliability and repeatability of the results obtained from the testing program, corroboration with available results from other relevant testing programs, and the extent to which the chosen (or generated) test data is representative of the typical usage of reference components in practice and the future usage of proposed components. Evaluation of the confidence in test results should be based on the following considerations:

- Consideration as to whether or not the experimental test data exhibit consistent behavior, performance, and failure modes.
- The number of tests completed.
- The number of independent labs and researchers that have completed the tests (i.e., independent investigators obtaining similar results leads to increased confidence).
- The extent to which the test results are supported by numerical or analytical modeling based on the basic principles of mechanics.
- The extent to which the data set represents the range of possible component configurations allowed within the component design space.

- The computed standard deviations of key component parameters from proposed component test data as compared with the same quantities in the reference component data set. If the standard deviation values are significantly larger in the proposed component data set, indicating the parameters of the proposed component are more uncertain, the quality rating should reflect a lower confidence in proposed component test results. This consideration only applies to the quality rating of the proposed component data set.

2.7.2 Quality Rating of Design Requirements

Design requirements for the reference component should be rated in accordance with the criteria in Table 2-2. The quality rating of design requirements should be based on the completeness and robustness of the requirements and on the confidence in the underlying methods and design data. Minimum quality ratings for applicability of the Component Methodology are defined in Section 2.3.3.

Table 2-2 Quality Rating of Design Requirements

Completeness and Robustness of Design Requirements	Confidence in Basis of Design Requirements		
	High	Medium	Low
High. Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	**Superior**	**Good**	**Fair**
Medium. Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	**Good**	**Fair**	**Not Allowed**
Low. Questionable safeguards against unanticipated failure modes. Many important design and quality assurance issues are not addressed.	**Fair**	**Not Allowed**	**Not Allowed**

Completeness and Robustness of Design Requirements

Completeness and robustness should be evaluated based on how well the design requirements address issues that could potentially lead to unanticipated failure modes, as well as quality assurance measures to ensure proper implementation of design through fabrication, erection, and final construction. Additionally, the rating should be based on how the design requirements also address important behavioral issues, such that the use of the design requirements results in highly reliable component behavior.

Confidence in Design Requirements

Confidence should be evaluated based on the degree to which the prescribed material properties, strength criteria, stiffness parameters, and design equations are representative of actual behavior, and that the component will perform as intended. High confidence requires evidence to substantiate that the design requirements reflect actual behavior (e.g., experimental data, supporting analytical studies, history of use, observations from previous seismic events, or demonstrated similarity with other components).

2.8 Component Equivalency Acceptance Criteria

2.8.1 Overall Approach to Establishing Equivalency

Acceptance criteria evaluate component equivalency using summary statistics of proposed and reference component performance parameters in each component performance group. Summary statistics of each proposed component performance group are compared with those of the associated reference component performance group. Proposed components are deemed equivalent to reference components when comparisons of summary statistics comply with the acceptance criteria of this section across all performance groups.

For cyclic-load testing parameters, summary statistics on proposed and reference component data sets of all component configurations in the respective performance groups are compared to evaluate equivalency. Separately, potential outliers are checked by comparing data for each individual proposed component configuration with the summary statistics of the associated reference component performance group.

For monotonic-load testing, summary statistics on subsets of proposed and reference component configurations in the respective performance group are compared to evaluate equivalency. The ultimate deformation capacity of the reference component determined from cyclic-load testing (factored to account for differences in the test protocol) may be used in lieu of the ultimate deformation capacity of the reference component determined from monotonic-load testing.

2.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation Capacity

Requirements for Component Performance Groups

For each proposed component performance group, the median value of ultimate deformation, $\tilde{\Delta}_{U,PC}$, should be compared with the median value of ultimate deformation capacity for the associated reference component performance group, $\tilde{\Delta}_{U,RC}$, using Equation 2-1:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q \qquad (2\text{-}1)$$

The uncertainty penalty factor, P_U, is obtained from Table 2-3, using the quality rating of the proposed component test data and the relative quality ratings of the proposed and reference component design requirements. Combinations of quality ratings that fall outside of those given in Table 2-3 are not permitted.

Table 2-3 Penalty Factor to Account for Uncertainty

Penalty Factor for Uncertainty (P_U)			
Quality Rating of Proposed Component Test Data[1]	Quality Rating of Proposed Component Design Requirements Relative to Reference Component Design Requirements[1]		
	Higher[2]	Same[3]	Lower[4]
Superior	0.95	1.00	1.15
Good	1.00	1.05	1.25
Fair	1.15	1.25	1.40

1. Quality ratings are computed in accordance with Sections 2.7.1 and 2.7.2.
2. Higher: Quality ratings of proposed component design requirements are higher than reference component design requirements (e.g., Superior versus Good).
3. Same: Quality ratings of proposed and reference component design requirements are the same.
4. Lower: Quality ratings of proposed component design requirements are lower than reference component design requirements (e.g., Good versus Superior, or Fair versus Good).

The load penalty factor, P_Q, is obtained from Table 2-4 using the strength ratio $\tilde{R}_{Q,PC} / \tilde{R}_{Q,RC}$. Values of $\tilde{R}_{Q,PC} / \tilde{R}_{Q,RC}$ less than 0.5, or greater than 1.2, are not permitted.

Exception. Values of $\tilde{R}_{Q,PC} / \tilde{R}_{Q,RC}$ greater than 1.2, but not greater than 2.0 are permitted, provided that the force-controlled and capacity-designed elements of the reference SFRS are designed for the expected strength of the component (i.e., proposed component design strength scaled up by a factor of $\tilde{R}_{Q,PC}$).

The value of the load penalty factor, P_Q, may be taken as equal to 1.2, for values of $\tilde{R}_{Q,PC} / \tilde{R}_{Q,RC}$ greater than 1.2, provided that: (1) the design requirements of the proposed components require full replacement of all reference components with proposed components (i.e., prohibit mixing of reference and proposed components in a building); and (2) the force-controlled and capacity-designed components of the reference SFRS are designed for the expected strength of the component (i.e., the component design strength scaled up by a factor of $\tilde{R}_{Q,PC}$).

Table 2-4 Penalty Factor to Account for Differences in Load (Strength)

Penalty Factor for Differences in Strength (P_Q)[1]			
$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q	$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q
0.50	1.88	1.10	1.00
0.60	1.55	1.20	1.00
0.70	1.31	1.30	1.04
0.80	1.14	1.40	1.09
0.90	1.00	1.50	1.13
1.00	1.00	1.80	1.24
1.10	1.00	2.00	1.32

1. Linear interpolation is permissible between tabulated values.

Requirements for Individual Component Configurations

For each configuration j of the proposed component, the median value of the ultimate deformation, $\tilde{\Delta}_{Uj,PC}$, should be compared with the median value of ultimate deformation capacity for the associated reference component performance group, $\tilde{\Delta}_{U,RC}$, using Equation 2-2:

$$\tilde{\Delta}_{Uj,PC} \geq \left(1 - 1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \ P_Q \tag{2-2}$$

If the value of $\sigma_{\Delta_U,RC}$ exceeds 0.3, then 0.3 should be used in Equation 2-2.

2.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

The median value of the proposed component initial stiffness ratio, $\tilde{R}_{K,PC}$, should be within the range specified in Equation 2-3:

$$0.75 \leq \frac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33 \tag{2-3}$$

Exception. Proposed components need not comply with Equation 2-3, provided that at each floor level, proposed components do not resist more than 25% of the design seismic force on a given line of framing in the direction of interest.

2.8.4 Requirements Based on Cyclic-Load Test Data: Effective Ductility Capacity

The median value of the effective ductility capacity of the proposed component should not be less than 50% of the median value of the effective ductility capacity of the reference component, in accordance with Equation 2-4.

$$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC} \tag{2-4}$$

Exception. Proposed components need not comply with Equation 2-4, provided that at each floor level, proposed components do not resist more than 25% of the design seismic force on a given line of framing in the direction of interest.

2.8.5 Requirements Based on Monotonic-Load Test Data: Ultimate Deformation

For each component performance group, the median value of the proposed component ultimate deformation, $\tilde{\Delta}_{UM,PC}$, should meet the requirements of either Equation 2-5 or Equation 2-6:

$$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC}\ P_U\ P_Q \tag{2-5}$$

$$\tilde{\Delta}_{UM,PC} \geq 1.2\,\tilde{D}_C\ \tilde{\Delta}_{U,RC}\ P_U\ P_Q \tag{2-6}$$

In Equation 2-6, the cyclic-load ultimate deformation, $\tilde{\Delta}_{U,PC}$, may be used in lieu of the monotonic-load ultimate deformation, $\tilde{\Delta}_{UM,PC}$.

In Equation 2-6, the value of the median cumulative damage factor, $\tilde{D}_C$, should be computed for the reference component performance group of interest and should be based on the median number of inelastic deformation cycles that occur prior to reaching the ultimate deformation, Δ_U, of the reference component. An inelastic deformation cycle is defined as a cycle in which the deformation exceeds the effective yield deformation, $\Delta_{Y,\,eff}$. The $\tilde{D}_C$ value should be taken as 1.0 for reference component cyclic-load testing protocols that have not more than 10 cumulative cycles of such inelastic deformation, and 1.5 for cyclic-load testing protocols that have 30 or more such cumulative cycles of inelastic deformation. Linear interpolation should be used to determine the value of the damage factor, $\tilde{D}_C$, for cyclic-load testing protocols that have between 10 and 30 such cumulative cycles of inelastic deformation.

2.9 Documentation and Peer Review Requirements

2.9.1 Documentation

The results of the Component Methodology should be thoroughly documented at each step of the process for review and approval by both the peer review panel and the authority having jurisdiction over its eventual use.

Documentation of component equivalency should include, as a minimum, the following information:

- Clear and complete description of the proposed component design requirements, including but not limited to methods and criteria for determining proposed component design stiffness and design strength.
- Proposed component detailing requirements.
- Description of limitations on proposed component usage within the reference SFRS.
- Description of component configurations and component performance groups for both the proposed and reference component data.
- Summary of experimental test data for cyclic-load and monotonic-load testing of reference and proposed components.
- Summary statistics of key component parameters for monotonic-load and cyclic-load testing data of each proposed and reference component performance group.
- Summary of the comparisons completed based on the acceptance criteria of Section 2.8, including all summary statistics of proposed component and reference component data sets, as well as quality ratings, penalty factors, and results for each acceptance criterion.

2.9.2 Documentation of Test Data

The documentation of each proposed component experiment should be comprehensive. For reference component test data, all available experimental information should be collected, but complete information may not always be available. Documentation of test data should include the following:

- Date of the test, test sponsor or agency, test facility name and location, and specimen identification number.
- Geometry and configuration of each test specimen.
- Important details of each test specimen, including construction process and fabrication details.

- Details of the test setup, including description of the connections to the proposed component test specimen.
- Type, location, and calibration of instruments used for measurement of important response parameters.
- Pertinent material test data for the construction material used in the test specimen.
- A record of all important events prior to and during the test, including documentation of the ability of the specimen to maintain any applied vertical loads.
- Important visual observations.
- A set of digital experimental data generated for component performance evaluation.
- Plots of load versus deformation data from all tests, including the cyclic envelope curves of cyclic-load testing.
- Values of the key component parameters, including the values for each direction of loading, as well as any computed average values.

2.9.3 Peer Review Panel Requirements

It is recommended that a peer review panel consisting of knowledgeable experts be retained for this purpose. The peer review panel should be familiar with the procedures of this Component Methodology, should have sufficient knowledge to render an informed opinion on the developmental process, and should include expertise in the following areas:

- Component testing
- Engineering design and construction

Members of the peer review panel must be qualified to critically evaluate the development of proposed and reference component test data.

2.9.4 Peer Review Panel Selection

It is envisioned that the cost of the peer review panel will be borne by the proposed component sponsor. As such, it is expected that members of the peer review panel will be selected by the proposed component sponsor. It is intended, however, that the peer review panel be an independent set of reviewers who will advise and guide the development team at each step in the process. It is recommended that other stakeholders, including authorities with jurisdiction over the eventual use of the proposed component in design and construction, be consulted in the selection of peer review panel members.

2.9.5 Peer Review Panel Responsibilities

The peer review panel is responsible for reviewing and commenting on the following:

- Test data for compliance with the component testing requirements of Section 2.2, including verification that the deformation history used for cyclic-load testing of the proposed component is equivalently damaging (or more damaging) as the deformation history used for cyclic-load testing of the reference component.
- Applicability of the Component Methodology for equivalency evaluation of the proposed component (Section 2.3).
- Values of monotonic-load and cyclic-load testing data obtained from individual test specimens of reference and proposed components, including envelope curves of cyclic-load testing (Sections 2.4 and 2.6).
- Adequacy of the design requirements for the proposed component, i.e., design methods and criteria related to component design stiffness and design strength (Section 2.5), and documentation of these requirements for use in design of proposed components for use in the reference SFRS.
- Quality ratings assigned to reference component design requirements and test data, and quality ratings assigned to proposed component design requirements and test data (Section 2.7).
- Statistical values of performance parameters (Sections 2.4 and 2.6), and their use in the component equivalency evaluation (Section 2.8).
- Documentation of the Component Methodology process (Section 2.9).

Chapter 3
Commentary on the Component Equivalency Methodology

3.1 Introduction

The Component Equivalency Methodology (Component Methodology) is an adaptation of the FEMA P-695 Methodology, which is a procedural methodology for quantifying collapse behavior and establishing seismic performance factors for newly proposed structural systems. Application of the FEMA P-695 Methodology requires significant effort, involving testing of materials, components, and connections, design of a large set of prototype buildings, creation of nonlinear models for each building, and running of nonlinear dynamic analyses. It evaluates an absolute level of collapse safety based on the response of the entire seismic-force-resisting system.

In contrast, the Component Methodology evaluates the seismic performance equivalency of structural components that are proposed as substitutes for selected components in an established seismic-force-resisting system. It evaluates a relative level of safety based on comparisons between the tested behavior of proposed and reference components. The Component Methodology is less comprehensive in scope and requires less effort to implement in the specific case of judging component equivalency.

The Component Methodology is intended for use on deformation-controlled elements in a structural system. In such elements, the behavior and controlling limit states in the overall system can be related to the inelastic deformation and energy dissipation characteristics of the elements. The Component Methodology relies on statistical comparisons of data from cyclic-load and monotonic-load tests on proposed and reference components. Statistical comparisons are based on probabilistic assessments of collapse risk, but the final acceptance criteria are deliberately simple.

The Component Methodology builds on previous methodologies that have been developed to establish seismic equivalency, such as AC322 *Acceptance Criteria for Prefabricated, Cold-Formed, Steel Lateral-Force-Resisting Vertical Assemblies* (ICC-ES, 2007). AC322 evaluates the equivalency of possible replacement components for the specific case of nailed wood shear walls in light-frame construction. The Component Methodology is similar, but is intended to apply to a wider range of components and reference seismic-force-resisting systems. It is supported by rigorous analytical studies conducted within a probabilistic framework.

3.2 Component Testing Requirements

A comprehensive experimental program is necessary to quantify the performance of reference and proposed components. Test results from previous research can be used to supplement an experimental investigation program, but such data should come from reliable sources and be well documented.

The Component Methodology requires both cyclic-load and monotonic-load testing. The data are interpreted using the following two curves, as illustrated in Figure 3-1. These curves are defined as follows:

- *Cyclic Envelope Curve*: The force-displacement curve formed by enveloping the cyclic-load test data of a specimen. The rules for constructing the cyclic envelope curve are defined in Section 2.2.2.
- *Monotonic Curve*: The force-displacement curve obtained from a monotonic-load test of a specimen.

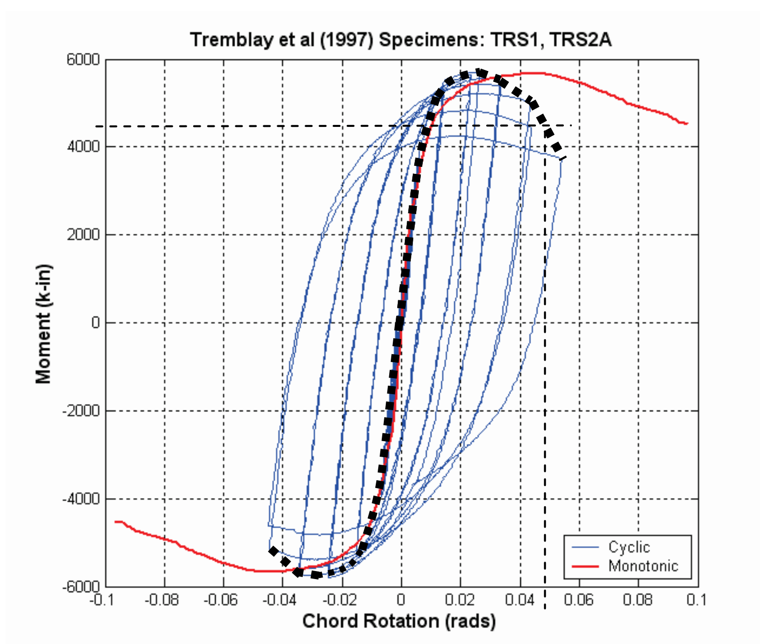


Figure 3-1 Monotonic and cyclic responses of identical steel specimens, and cyclic envelope curve fit to cyclic response. Data from Tremblay et al. (1997), figure from PEER/ATC-72-1 (2010).

Practical considerations limit the scope of an experimental testing program, but the number and quality of the component experimental tests will affect how well the collapse resistance of the seismic-force-resisting system can be characterized. Accordingly, the quality of test data, rated as described in Section 2.7.1, is a key aspect of the Component Methodology.

In applying the Component Methodology, the load and deformation quantities used to assess the proposed and reference component data sets must be comparable, and the relationship between component load-deformation properties and the system-level response must be well understood. It is expected that load-deformation quantities will often be the component shear force (load quantity) versus story drift (deformation quantity), but other load and deformation quantities might be appropriate for some components.

Where story drift is not the primary deformation quantity of interest, the relationship between component deformation and story drift in the seismic-force-resisting system must be similar for the proposed and reference components. Provided that the forces and deformations in the balance of the seismic-force-resisting system are comparable, slight differences between story drift and component deformation relations can be accommodated through kinematic adjustment factors in the deformation quantities for the proposed and reference components.

3.2.1 General Requirements for Component Testing

The experimental program is expected to focus on component testing, including testing of the proposed component, compiling existing test data for the reference component, and testing of the reference component (as needed to fill in gaps in existing data). In addition to the guidance provided in Section 2.2.1, other suggested sources for information on testing include ACI T1.1-01 *Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary* (ACI, 2001), ANSI/AISC 341-10 *Seismic Provisions for Structural Steel Buildings* (AISC, 2011), ASTM E2126-09 *Standard Test Method for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Load Force Resisting System for Buildings* (ASTM, 2009), ATC-24 *Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (ATC, 1992), Clark et al. (1997), FEMA 461 *Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components* (FEMA, 2007), as well as AC322.

Load Application (Vertical Loads)

For components that resist vertical loads (gravity load and overturning effects) in the seismic-force-resisting system, test loading should include these loads unless they do not significantly affect component performance. For components tested without vertical loads, it must be shown by analysis or experimentation that vertical loads expected to be present in buildings would not cause an appreciable change in the parameters used to establish equivalency.

Testing of Materials

Material testing will often be an important part of the experimental program for a proposed component, because new design requirements are often based on material properties. The need for materials testing will depend on the composition of the proposed component, so specific material testing requirements are not included in the Component Methodology. A proposed component constructed from a new type of material, for example, will likely require more materials testing than one constructed from an established material with behavior that is well understood. The extent of the materials testing program should be considered when determining the quality rating of the test data (Section 2.7.1).

Testing of the Connection between the Proposed Component and the Balance of the Seismic-Force-Resisting System

Properties of the connection between the proposed component and the rest of the seismic-force-resisting system are an important part of the proposed component design requirements (Section 2.5.3). It should be shown by analysis or experimental evidence that these connections remain essentially elastic when the proposed component is tested up to the ultimate deformation, Δ_U. If the connection does not remain essentially elastic, then the connection must be included as part of the tested component configuration.

3.2.2 Cyclic-Load Testing

Cyclic-Load Testing Protocol

Equivalently Damaging Load Histories. Section 2.2.2 requires that the cyclic-load history used for the proposed component is at least as damaging as that used for the reference component. Quantitative methods to fulfill this requirement should be approved by the peer review panel. One such method is provided in Section B.2.3 of Appendix B.

The method in Appendix B suggests that loading histories for the proposed and reference components are acceptable if the median normalized cumulative deformation imposed by the proposed component loading history at Δ_U is at least 0.75 the of comparable value for the reference component loading history (Equation B-3). This check is to be completed for each of the associated pairs of proposed and reference component performance groups. Per Section B.2.4, the method assumes that the requirements of Section 2.2.2 are automatically satisfied under the following conditions: (1) the same loading protocol is used for all reference and proposed components; and (2) the control variable used in the loading protocol is based on a measure of ultimate or maximum displacement. Additionally, Equations B-1 and B-2 provide requirements to ensure reasonable similarity between the normalized cumulative deformations of the loading histories of the various test specimens in each performance group.

Number of Cycles. Section 2.2.2 requires that the loading history have a sufficient minimum number of cycles to measure degradation of the component under repeated cycles of loading. The method of Appendix B states that all loading histories should have a normalized cumulative deformation equal to or greater than 6.0. Alternate quantitative methods are also allowable, but all methods should be approved by the peer review panel.

Appendix B also considers the possibility of specifying an upper limit on the number of cycles in the cyclic load protocol (or, more quantitatively, an upper limit on the normalized cumulative deformation of the protocol). The rationale for such an upper limit would be to avoid the use of an overly energetic cyclic load protocol that is not compatible with expected earthquake-induced motions, resulting in unrealistic failure modes and masking the true expected failure modes of the component. However, such an upper limit was not imposed on the Component Methodology because monotonic-load data are also used as an integral part of the acceptance criteria. It is expected that the acceptance criteria based on monotonic-load data will catch most problems introduced by the use of an overly energetic cyclic-load protocol. It is desirable that the loading protocol produce damage and failure mechanisms that are consistent with what is expected (or has been observed) in earthquakes.

Description of Deformation History. It is envisioned that loading histories will be displacement-controlled because structural components are expected to yield and lose strength throughout the loading history, and these effects cannot be captured using load-controlled loading histories.

Reduction in Applied Load. Section 2.2.2 requires that cyclic-load testing continue to deformations large enough to achieve at least 20% reduction in applied load for reference component data. A similar requirement is included in Section 2.2.3 for monotonic-load testing. This is necessary to prevent Δ_U from being underestimated for the reference component data set. If the reference component ultimate deformation is underestimated, the Component Methodology acceptance criteria would be non-conservative (i.e., easier for the proposed component to pass).

To provide more flexibility in the Component Methodology, this requirement need only be met in one direction of loading (for cyclic-load testing). This is done to permit the use of a greater proportion of the reference component data that might be available in the literature. If a 20% reduction in applied load is achieved in one direction only, the ultimate deformation should be averaged with the maximum deformation in the opposite direction. Based on test application data in Appendix D, this approximation was found to provide a reasonable representation of the ultimate deformation.

Number of Cyclic Test Specimens

The number of component configurations to be tested depends on the range of configurations that are allowed in the proposed component design space. The use of at least four component configurations per performance group (and two or three specimens per configuration) is intended to ensure that a sufficiently large number of tests are obtained to reliably estimate the median of the component performance parameters.

Two exceptions are allowed regarding the minimum number of configurations and the minimum number of test specimens for each configuration. First, if fewer than four configurations are possible for the proposed component (e.g., if there is no possible way to use the proposed component in four different configurations), then it is permissible to test fewer than four configurations. Second, when creating the reference component data set, it is permissible (within some constraints) to ignore the requirement of having multiple test specimens for each component configuration. This second exception was created to provide more flexibility when using existing test data to create the reference component data set.

The minimum requirements of Section 2.2.2 acknowledge that the costs of testing a large number of specimens for many performance groups may be prohibitive for some types of proposed components. These minimum requirements are generally comparable with requirements of other recent codes and standards such as ASTM E2126-09 and Section 1.3.1.3.2 of

ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010).

Beyond these minimum requirements, it is expected that four component configurations will often be insufficient to span the range of design parameters and component behavior for the performance group. A larger number of tests should be conducted for performance groups when warranted by a large range of possible configurations or large variability in response.

The values of 15% and 20% difference that trigger the need for a third component test for a given configuration were informed by similar criteria in ASTM E2126-09 and ASCE/SEI 7-10 Section 1.3.1.3.2.

Cyclic-Load Test Data

The cyclic-load data are interpreted through the use of a cyclic envelope curve. Key component performance parameters are then measured from the cyclic envelope curve and used in the later acceptance criteria. The short list of key component performance parameters was determined primarily from the sensitivity studies in Appendix A.

Construction of the Cyclic Envelope Curve

Section 2.2.2 contains the rules for constructing the cyclic envelope curve, which are based on the response in the first loading cycle under increasing deformation amplitudes, consistent with ASTM E2126-09 and PEER/ATC-72-1 *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings* (PEER/ATC, 2010). The first loading cycle is used because it de-emphasizes the effects of cyclic strength deterioration (i.e., the loss of strength between each subsequent cycle of loading at a given level of deformation). Cyclic strength deterioration is intentionally de-emphasized due to the findings of Appendix A.

Figures 3-2 and 3-3 illustrate the fitting of an envelope curve to cyclic-load test data along with identification of the resulting component properties. Figure 3-3 indicates values for positive and negative loading directions as "+" and "–," respectively. Since this particular component is a symmetric shear panel, the final values would be the average of these values for each direction of loading.

Figures 3-4 through 3-8 provide additional examples of cyclic-load test data and the associated envelope curves for nailed wood light-frame shear wall data. These figures are meant to illustrate how to construct the envelope curves under various circumstances. For example, Figure 3-4 shows a case where there are two subsequent cycles with negative stiffness. Figure 3-6 and Figure 3-8 show cases with negative stiffness in both directions of loading.

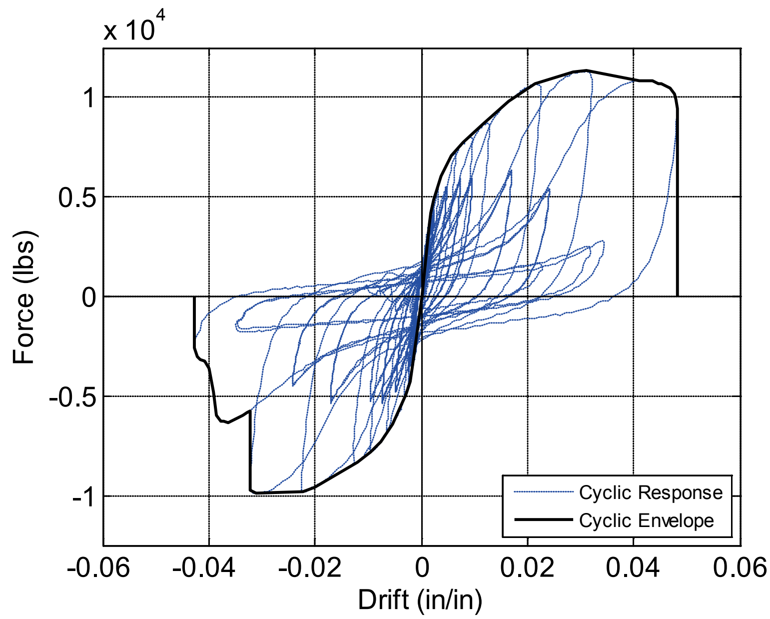


Figure 3-2 Cyclic-load test data and cyclic envelope curve for a nailed wood light-frame shear wall specimen (data from Rosowsky et al., 2004).

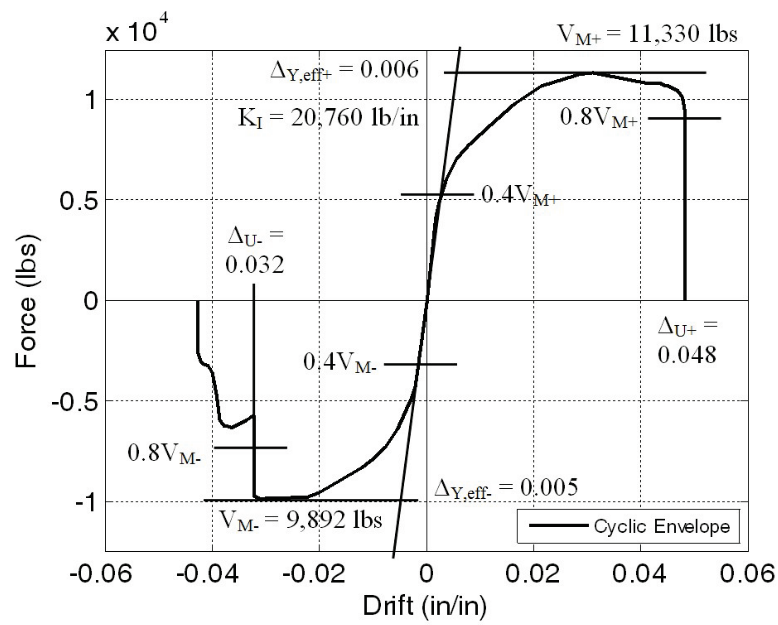


Figure 3-3 Illustration of cyclic envelope curve and key component performance parameters obtained from positive and negative loading directions based on Figure 3-2.

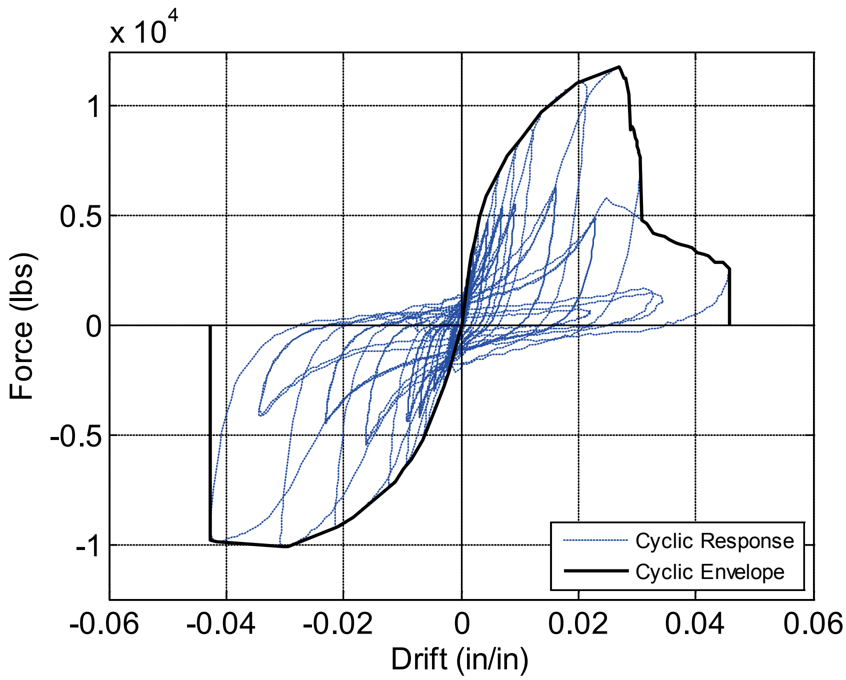


Figure 3-4 Cyclic-load test data and cyclic envelope curve for a nailed wood shear wall specimen (data from Rosowsky et al., 2004).

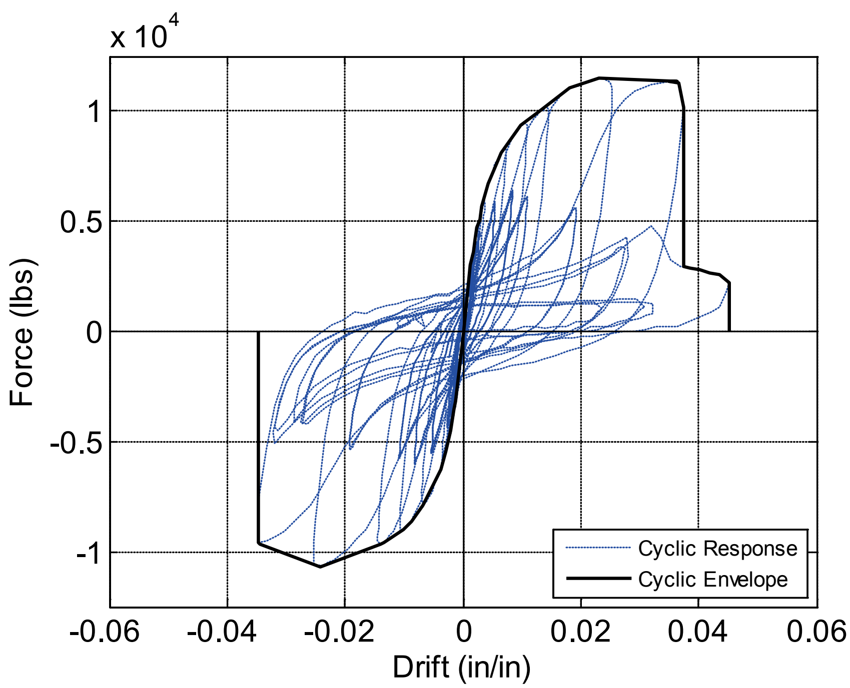


Figure 3-5 Cyclic-load test data and cyclic envelope curve for a nailed wood shear wall specimen (data from Rosowsky et al., 2004).

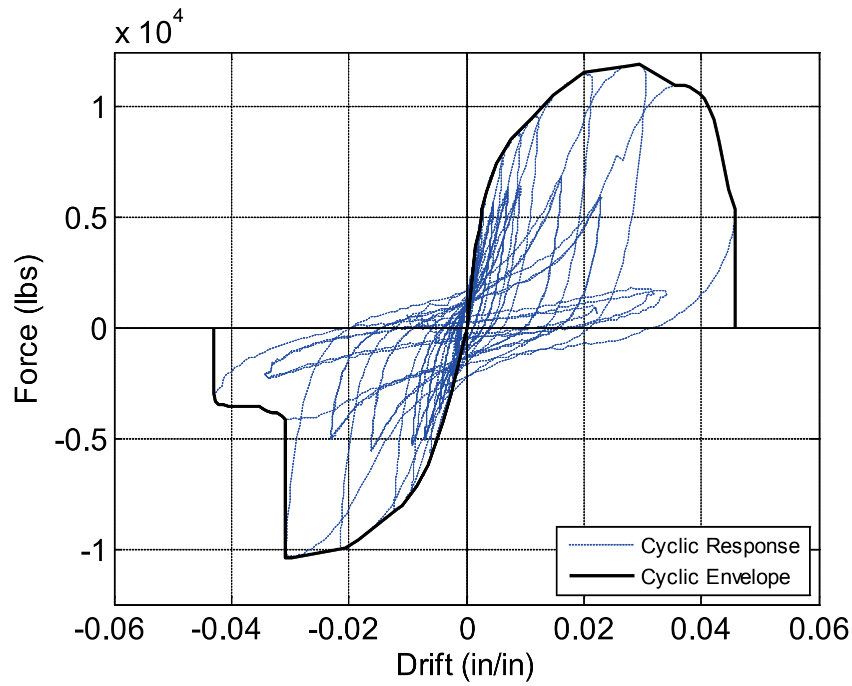


Figure 3-6 Cyclic-load test data and cyclic envelope curve for a nailed wood shear wall specimen (data from Rosowsky et al., 2004).

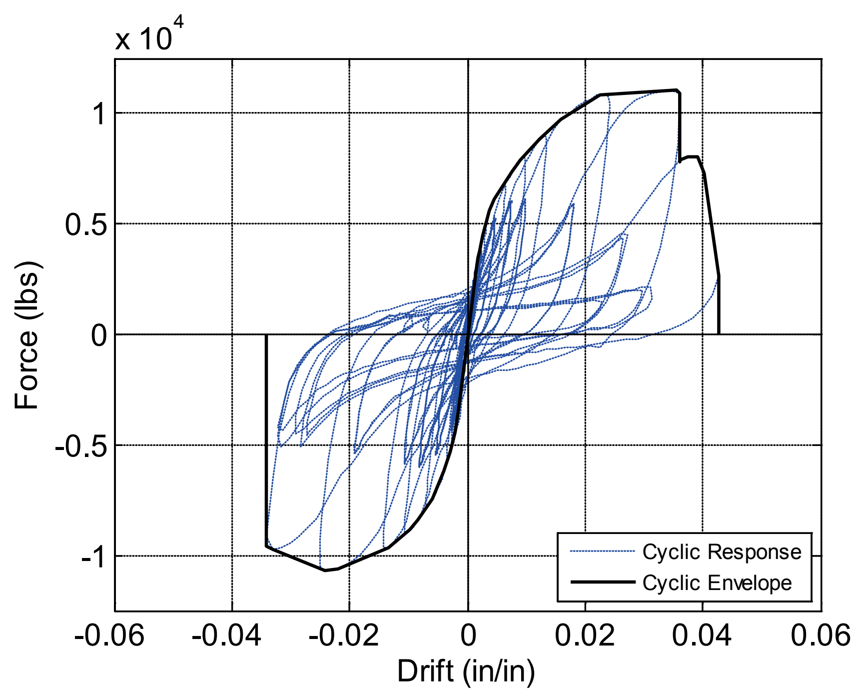


Figure 3-7 Cyclic-load test data and cyclic envelope curve for a nailed wood shear wall specimen (data from Rosowsky et al., 2004).

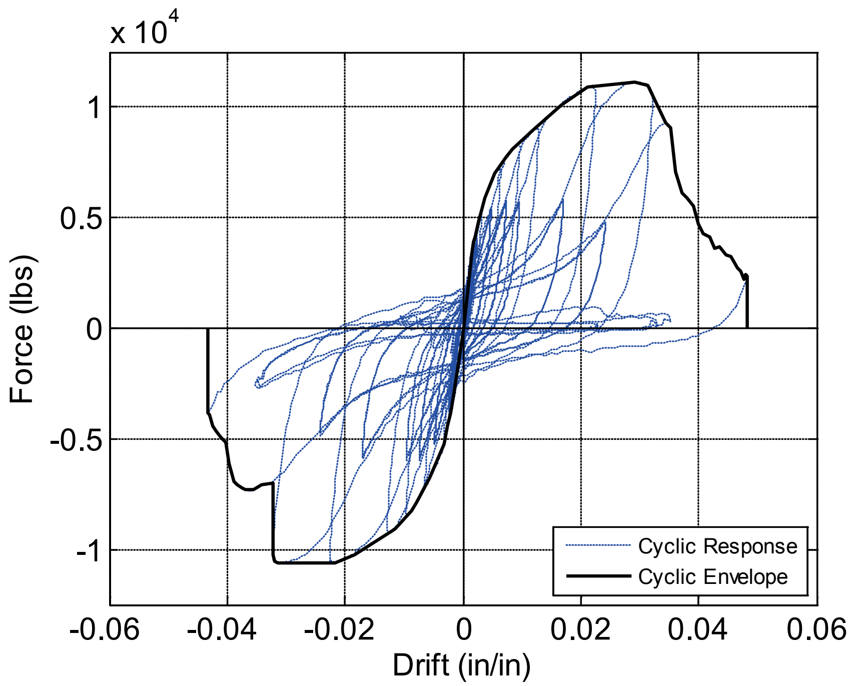


Figure 3-8 Cyclic-load test data and cyclic envelope curve for a nailed wood shear wall specimen (data from Rosowsky et al., 2004).

Figures 3-9 and 3-10 provide additional examples for steel brace components. These figures illustrate the fitting of cyclic envelope curves for behaviors that are drastically different from wood light-frame shear wall examples. Additionally, these figures show cases in which the initial stiffness estimate would be based on a single direction of loading.

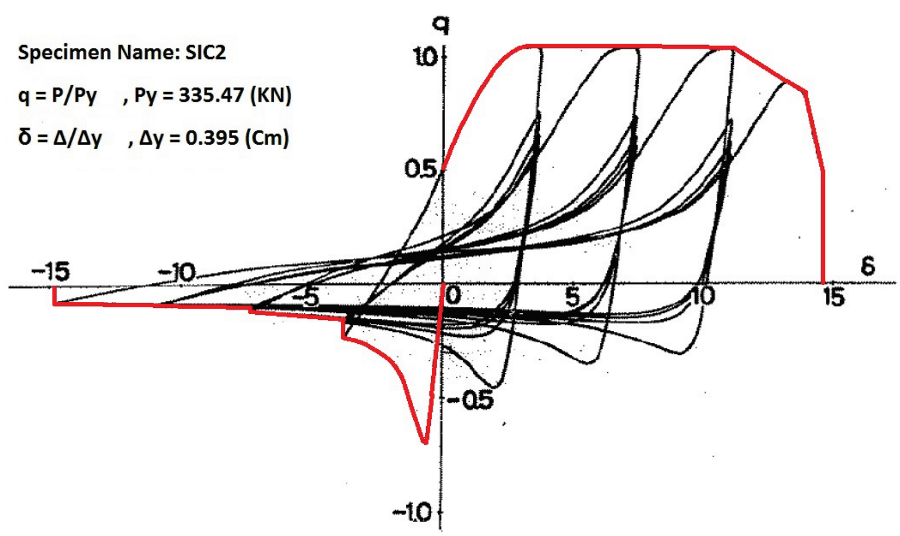


Figure 3-9 Cyclic response and cyclic envelope curve of a single HSS-section brace (data from Wakabayashi et al., 1979). In this case the initial stiffness should be determined from the negative quadrant only.

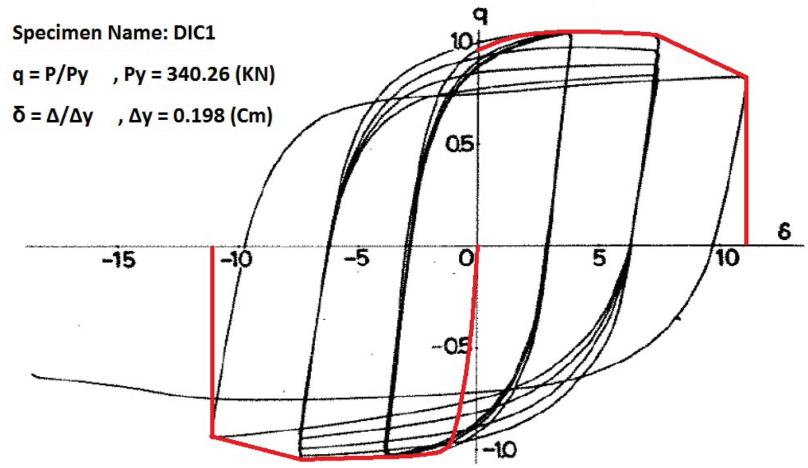


Figure 3-10 Cyclic response and cyclic envelope curve of an X-brace configuration (data from Wakabayashi et al., 1979). In this case the initial stiffness should be determined from the negative quadrant only.

The value of ultimate deformation, Δ_U, is typically computed from the cyclic envelope curve as the deformation at $0.8Q_M$. However, Section 2.2.2 provides guidance regarding how the value of ultimate deformation, Δ_U, should not be based on the cyclic envelope curve when the vertical-load carrying ability of the component is compromised before $0.8Q_M$ is reached (note that this modification is not illustrated in Figure 3-3 through Figure 3-10). The approach taken in Section 2.2.2 for dealing with vertical-load carrying ability is similar to the approach taken in AC322.

3.2.3 Monotonic-Load Testing

Monotonic-load test data are required because cyclic-load test data alone are not sufficient for establishing equivalency in seismic collapse resistance (more detail is provided in Appendix B).

Monotonic-load test data are required for the proposed component and are desirable for the reference component. When monotonic-load data are not available for the reference component, the ultimate deformation capacities (i.e., Δ_U and Δ_{UM}) of the proposed component must satisfy Equation 2-6. The rationale behind this check is discussed in more detail in Section 3.8.5.

3.3 Applicability Criteria

3.3.1 Required Information and Data

No commentary for this section.

3.3.2 Reference Seismic-Force-Resisting System: Collapse Performance Criteria

Seismic-force-resisting systems used as a reference system in the Component Methodology should comply with collapse performance criteria contained in the FEMA P-695 Methodology. This requirement is intended to ensure that equivalency is judged against a system with an appropriate minimum absolute level of safety, and that a seismic-force-resisting system consisting of proposed components will have adequate collapse safety.

Several seismic-force-resisting systems identified in ASCE/SEI 7 have been evaluated using the FEMA P-695 Methodology (FEMA, 2009b and NIST, 2010). These evaluations have shown that current code-approved systems, in general, appear to meet these criteria. Therefore, as a practical matter, and for consistency with current building code requirements, the Component Methodology allows the use of any currently approved system in ASCE/SEI 7-10 as a reference seismic-force-resisting system.

3.3.3 Quality Rating Criteria

Minimum quality ratings for reference and proposed component test data and design requirements are intended to ensure that available information is robust enough for application of the Component Methodology. The requirement of a Good quality rating for reference component test data reflects the concept that the Component Methodology should not be used for equivalency comparisons unless the reference component behavior is well understood. Proposed component test data are allowed to be rated as low as Fair, provided that the component is still able to pass the acceptance criteria of Sections 2.8.2 and 2.8.5 with the associated increase in the penalty factor, P_U.

3.3.4 General Criteria

Proposed and reference components must be unambiguously defined with a clear definition of the component boundary, or the Component Methodology is not applicable. Proposed and reference components can consist of a single element, a connection, or a sub-assemblage. In the Component Methodology, "connection" refers to whatever connects the proposed or reference component to the balance of the seismic-force-resisting system. It is possible that the proposed component could include elements that are typically thought of as a "connection."

To illustrate the definitions for proposed component, component boundary, and connection, several examples are discussed below. Figure 3-11 shows an example for a steel concentric brace. In this example, the proposed component includes the brace and the gusset plate, because the gusset plate will likely also experience inelastic deformation when the brace buckles under compressive loading. The connection consists of bolts and welds that connect the gusset plate to the adjacent beams and columns. Section 2.5.3 requires that these connections be strong enough to develop the full ultimate strength of the component, such that the inelastic deformation occurs in the component and not in the connection region.

Figure 3-11 Illustration of proposed component and connection definitions for a steel concentrically braced seismic-force-resisting system.

The intent is that the primary failure mechanism, which drives the level of collapse safety of the building, is contained within the component boundary. When this is the case, components being tested using the Component Methodology determine the performance of the system. This is difficult in some cases because there can be multiple components in a building that contribute to seismic behavior and collapse safety. Because of this, there are no requirements prohibiting inelastic behavior in the seismic-force-resisting system outside of the component boundary. To provide some control, Section 2.3.4 requires the balance of the reference SFRS to be essentially unchanged by replacement of reference components with proposed components. This is to ensure consistency in the portion of the seismic-force-resisting system that is outside of the component boundary.

Figure 3-12 shows another example of a steel concentrically braced frame system. Here, the geometry of the brace-to-gusset plate connection is different, so when the brace buckles, it is possible that fracture or tearing could occur in the weld region between the gusset plate and the beams and columns. Accordingly, these welded connections should be included within the component boundary in this case.

Figure 3-12 Illustration of two alternative definitions for the proposed component boundary in a steel concentrically braced seismic-force-resisting system (Engelhardt, 2007).

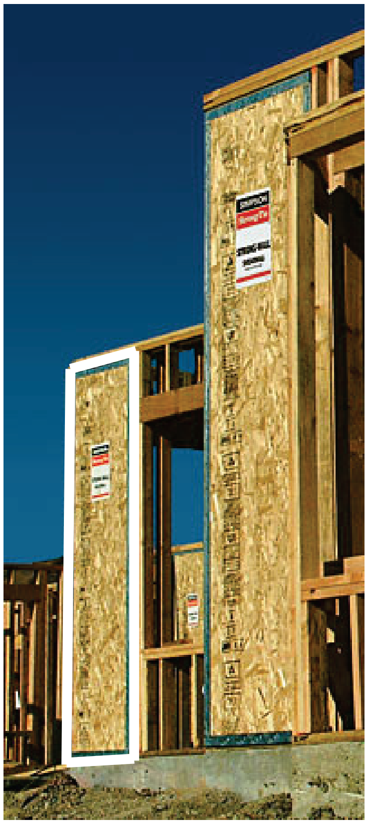
Two possible definitions of the component boundary are shown in Figure 3-12. A single brace could be defined as the proposed component, provided that strong interaction between adjacent braces does not occur. Alternatively, an assembly of two braces could also be defined as the proposed component. In this case, the pair of braces would need to be tested together in the assembly shown, and the proposed component design requirements would need to require that pairs of braces always be used in the configuration shown.

Figure 3-13 shows an example component boundary for a buckling-restrained brace element. Assuming that the gusset plates and connections are designed to remain elastic, ensuring that yielding only occurs in the braces themselves, the proposed component can consist of the brace alone.

Figure 3-13 Illustration of the proposed component boundary for a buckling-restrained brace seismic-force-resisting system (Engelhardt, 2007).

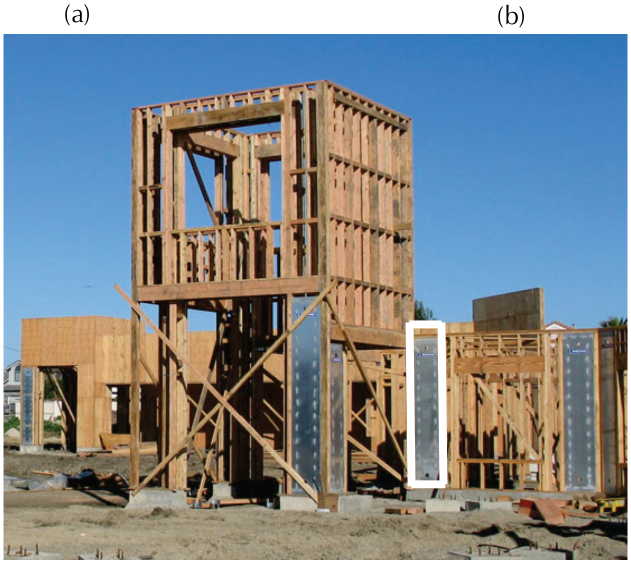
Similar examples are provided in Figure 3-14 to illustrate the definition of proposed component and component boundary for various types of proprietary wood light-frame shear walls, showing that these elements can be successfully isolated from the balance of the seismic-force-resisting system.

Figure 3-15 shows an example for a steel eccentrically braced frame, where the fuse is the proposed component. Consistent with the intended behavior of steel eccentrically braced frames, this definition assumes that a capacity design philosophy has been used to ensure that all of the inelastic behavior occurs in the fuse element and not in the beams or braces.



(a)

(b)



(c)

Figure 3-14 Illustration of proposed component boundaries for various wood light-frame shear walls, including (a) Simpson Strong-Tie (2009), (b) Simpson Steel Strong-Wall (Photo courtesy of Tools of the Trade Magazine, 2006), and (c) Hardy frame (Photo courtesy of SBE Builders).

Figure 3-15 Illustration of proposed component boundary for the fuse element in a steel eccentrically braced frame seismic-force-resisting system (Engelhardt, 2007).

Another example component boundary is illustrated in Figure 3-16, for a steel plate shear wall. This boundary assumes that the connections between the shear panel and the adjacent columns are capacity designed, such that all of the inelastic behavior occurs in the steel panel.

There are some cases where it might not be possible to clearly separate the component from the balance of the seismic-force-resisting system. One such example is a multistory reinforced concrete shear wall shown in Figure 3-17. In this case, the Component Methodology is not applicable, and the full FEMA P-695 Methodology should be used to assess the seismic performance equivalency of the proposed system.

Section 2.3.4 requires that the component boundary be the same for the proposed and reference components, unless their configurations are fundamentally different. One such example is the substitution of a 2'x 8' prefabricated wall component in place of an 8'x 8' site-built wood shear wall component. In this case, the component boundaries are not identical (i.e., one boundary is 2'x 8' and the other is 8'x 8'). Differences like this are permitted if the loads are transferred across the proposed component boundary in an acceptably similar manner. In this specific case, possible differences in load effects (such as overturning) should be directly investigated in the testing program or shown to be accommodated without detrimental effects on the proposed component or the balance of the reference SFRS up to the ultimate deformation, Δ_U.

Figure 3-16 Illustration of a proposed component boundary for a steel plate shear wall in a seismic-force-resisting system (Sabelli, 2007).

Figure 3-17 Illustration of the difficulty in identifying an isolated component boundary in a multistory reinforced concrete shear wall seismic-force-resisting system.

Section 2.3.4 requires that the vertical-load carrying capacity of components that support vertical loads must be maintained up to the ultimate deformation, Δ_U. However, if the vertical-load carrying capacity is compromised before Δ_U is reached, it is permissible to define a reduced Δ_U value corresponding to the deformation at which the vertical-load carrying capacity of the component was compromised.

3.4 Reference Component Test Data Requirements

Section 2.4 describes the data required for the reference component, and the approach taken in the Component Methodology when assembling this data set. The first step is to define the reference component design space (i.e., the range of possible component configurations for the reference component), and establish how often each configuration would be used in design practice. The data set should then be assembled to represent the design space, as illustrated in Figure 3-18.

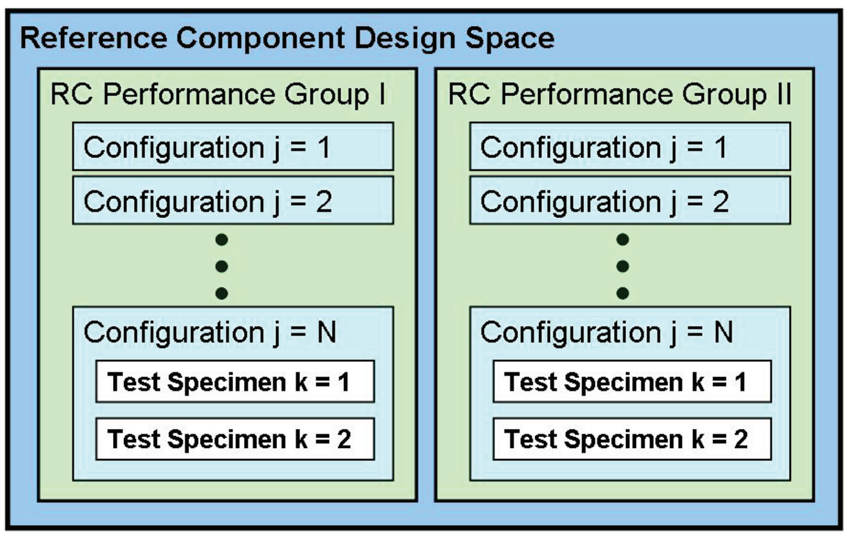


Figure 3-18 Overview of test data used in the reference component data set.

Within the design space, reference component configurations are typically placed into a single performance group. In some cases, they are divided into multiple performance groups (two groups in this example), that reflect significant differences in component failure modes or physical characteristics. Multiple component configurations populate each performance group.

The data set will typically include both cyclic-load and monotonic-load test data. At a minimum, two test specimens are needed for each configuration. Most reference component data are expected to be assembled from previous research and published information, but may need to be supplemented with additional testing.

Reference component data serve as the basis against which the proposed component data are compared, so compiling the reference component data set is a critical aspect of the Component Methodology. The process of compiling the reference component data, including the definition of the design space, identification of representative component configurations and selection of test results, requires judgment and experience and should be subject to close peer review.

3.4.1 Define Reference Component Design Space

The reference component design space should be established in parallel with the proposed component design space, in order to ensure consistency between the two. The definition of the reference component design space should consider typical component configurations and characteristics of the reference component, including:

- Component geometry (e.g., global aspect ratio, local geometries such as flange thicknesses for steel)
- Component composition (e.g., connector sizes for wood, sheathing type for wood, rebar diameters in reinforced concrete)
- Detailing (e.g., connector spacing for wood, stirrup spacing for reinforced concrete, connection details)
- Expected axial loading (based on expected building height, expected location of component within the full building system)
- System geometry (e.g., relation of component to overall structural system)

The relative importance of these characteristics, and others not listed here, depend on the nature of the reference component. When identifying the representative configurations for the reference component, care should be taken to identify any fundamental differences in how the component is used, or how the component performs for the various possible configurations. For example, in the case of wood light-frame construction, the aspect ratio of the wall component triggers a change in the required design strength. In other cases, some reference component configurations can result in a brittle failure while others result in ductile failure. Such differences in design requirements and failure modes should be clearly identified when compiling the reference component data set.

3.4.2 Compile or Generate Reference Component Test Data

Section 2.4.2 provides general guidance for compiling reference component test data, recognizing the diversity of these data and their sources. When there is a choice of reference component (e.g., where more than one reference component is allowed within the reference SFRS) or associated sources of test data, the user is expected to choose the reference component and associated data that best represent current use of the reference SFRS. When there is a choice of data sets, the user is expected to use all applicable data, or to select a data set that represents better performance of the reference component. As a possible exception, the user may select the data set corresponding to the "original benchmark" of the reference SFRS.

For many components, it is expected that much of the reference component data will be obtained from previous testing programs and available published literature. Even so, supplementary testing of the reference component may be necessary to complete the reference component data set, especially when monotonic-load data are needed. The data set should be compiled to include the representative configurations and the relative frequency with which these configurations are expected to be used in practice.

Many different cyclic-load histories are permitted in the test data that comprise the reference component data set. It is important to keep in mind, however, that the cyclic-load histories used in the reference component data set dictate the cyclic-load histories that are permitted in creating the proposed component data set.

It is expected that the sources of monotonic-load data from previous testing programs will be more limited than cyclic-loading data. For this reason, fewer monotonic-load data are required. When no suitable monotonic-load data are available for the reference component, and the proposed component data can be shown to comply with Equation 2-6, it is not necessary to generate monotonic-load test data for the reference component.

3.4.3 Interpret Reference Component Test Results

No commentary for this section.

3.4.4 Define Reference Component Performance Groups

The default approach to defining performance groups is to place all of the reference component data into a single performance group. Data are only separated into multiple performance groups if there are fundamental differences in behavior between various test specimens that make it inappropriate to average the results of the various tests. When multiple performance groups are created, the acceptance criteria of Section 2.8 are applied separately for each associated pair of proposed and reference component performance groups.

If some component configurations have significantly different design requirements, geometry, failure modes, or behavior (as described in Section 2.4.4), then they should be separated into multiple performance groups. In such cases, separate performance groups are needed to ensure that the statistical assessment (averaging) does not mask a potential problem with the performance of certain component configurations. For example, consider a set of 15 reference component tests that have distinct component geometries, such that 13 of them clearly and predictably fail in flexure, while two clearly and predictably fail in shear. Averaging these 15 test results (as required to compute $\tilde{\Delta}_{U,RC}$) would mask the potentially inferior performance of the shear-critical configurations.

Configurations should be separated into different performance groups based on fundamental changes related to component design requirements. For example, if component design requirements include a trigger that requires additional detailing for components subject to high levels of axial load, then two performance groups should be defined: one populated with component configurations that include the additional detailing and the other populated with component configurations that do not include this detailing.

Configurations should be separated into different performance groups based on systematic differences in failure mode. This is intended to ensure that: (1) dissimilar data are not grouped together; and (2) failure modes are closely scrutinized in the equivalency process. For example, in a steel eccentrically braced frame system, link beam fuses differ significantly in height or length such that some configurations fail in shear (web yielding) while some configurations fail in bending (flange yielding). If a new fuse component is proposed for steel eccentrically braced frame systems, the configurations should be separated into different performance groups based on failure mode.

Random differences in failure modes for tests with very similar component configurations, however, do not warrant separation into different performance groups. For example, in the case of wood light-frame shear walls, most tests exhibit fastener failures, but some tests experience sill-plate splitting failures. Since these different failures occurred randomly, rather than systematically, it would not be considered necessary to separate these data into performance groups by failure mode.

Configurations should be separated into different performance groups based on fundamental differences in geometry that tangibly affect the ratio of the inelastic deformation to the elastic (pseudo-yield) deformation of the component. Since the acceptance criteria of Section 2.8 are computed from ultimate deformation of the component, including both elastic and inelastic deformations, it is undesirable to group configurations together which have fundamental differences in the ratio of the inelastic to elastic deformations. Of course, all tests are expected to have slight differences in inelastic and elastic deformations, so separation of data into different performance groups is only warranted when fundamental differences are observed in the component behaviors.

Configurations should be separated into different performance groups based on substantial differences in loading histories used in testing. The recommended method for establishing the equivalence of loading histories is described in Section B.2.3 (Equations B-1 and B-2). Alternative quantitative methods are permitted provided they meet the intent of Section 2.4.4 and are approved by the peer review panel.

Reference component performance groups should be established in parallel with the proposed component performance groups, and there should be consistency in how the data are organized into performance groups. Each proposed component performance group must then be clearly associated with a reference component performance group (Section 2.6.5).

Determining such associations can be complicated. For example, consider a well-behaved proposed component that fails in flexure and cannot fail in shear. Comparing such a component to reference component configurations that only fail in flexure would overlook the potential benefits of the improved resistance to shear failure. In this case, the proposed component (which cannot fail in shear) should be compared to a reference component performance group that includes the full range of configurations for which it is a suitable replacement (which would include configurations that exhibit both shear and flexural failures).

Table 3-1 illustrates a sample reference component data set in tabular form, showing the separation of data into performance groups and the required number of cyclic-load and monotonic-load tests for each component performance group and component configuration.

Table 3-1 Sample Reference Component Data Set

Reference Component (RC) Performance Group	Component Configuration	Number of Cyclic-Load Tests	Number of Monotonic-Load Tests
RC-A	Config. A-1	≥ 2[1]	≥ 0
	Config. A-2	≥ 2[1]	
	Config. A-3	≥ 2[1]	
	Config. A-4	≥ 2[1]	
	…..	≥ 2[1]	
RC-B	Config. B-1	≥ 2[1]	≥ 0
	Config. B-2	≥ 2[1]	
	Config. B-3	≥ 2[1]	
	Config. B-4	≥ 2[1]	
	…..	≥ 2[1]	

1. Minimum of three cyclic load tests are required if any of the following occur: (1) rapid deterioration; (2) if the strength varies more than 15%; or (3) if the ultimate deformation capacity varies more than 20% between two tests.

3.4.5 *Compute Summary Statistics*

Summary statistics are computed for each performance group of the reference component data set, in accordance with the guidelines of Section 2.4.5.

From cyclic-load test data, the following statistics are required:

- Median of each parameter for each performance group ($\tilde{\Delta}_{U,RC}, \tilde{\Delta}_{Y,eff,RC}, \tilde{\mu}_{eff,RC}$, and $\tilde{R}_{K,RC}$)
- Lognormal standard deviation of each parameter for each performance group ($\sigma_{\Delta_U,RC}$, etc.)

From monotonic-load test data, the following statistics are required (if monotonic data are utilized for the reference component, per the requirements of Section 2.8.5):

- Median ultimate deformation for each performance group (only $\tilde{\Delta}_{UM,RC}$)

3.5 Proposed Component Design Requirements

A comprehensive set of design and quality assurance requirements should be created for the proposed component. In many cases, this process will be iterative, such that the test data and the acceptance criteria of Section 2.8 serve as mechanisms to inform necessary revisions to the design requirements.

3.5.1 Component Design Strength and Stiffness

Design requirements should include methods to predict both the design strength and design stiffness of the proposed component. Design strength is more precisely characterized as the ratio of ultimate load to design load. Initial stiffness is a fundamental property used in the ASCE/SEI 7-10 design processes related to: (1) story drift limits; (2) stability coefficient limits; (3) the distribution of force demands to components within a statically indeterminate structural system; and (4) other checks such as horizontal and vertical stiffness irregularities.

Requirements are intended to ensure that the proposed component and reference component strength and stiffness are sufficiently similar to avoid potential problems due to overstrength or stiffness irregularities when proposed components are substituted for reference components in the seismic-force-resisting system.

3.5.2 Component Detailing Requirements

When necessary, design requirements for the proposed component should include comprehensive provisions for the detailing of the component.

3.5.3 Component Connection Requirements

In the Component Methodology, the "connection" is defined as the interface between the proposed component and the balance of the seismic-force-resisting system (see Figure 3-11 through Figure 3-15). Design requirements should include provisions related to connections. Specifically, they should ensure that inelastic behavior occurs in the component, connection, or subassembly that is defined as the proposed component, and not in the connection between the proposed component and the balance of the seismic-force-resisting system.

It is expected that connection design requirements will be based on a capacity-design philosophy. It should be shown by analysis or experimental evidence that the connections remain essentially elastic when the proposed component is tested up to the ultimate deformation, Δ_U. Some limited yielding of the connections may be permissible at the discretion of the peer review panel, provided that such yielding will not result in additional failure modes not observed in component testing.

The intent of this requirement is to ensure that the proposed component (as defined and tested) captures the critical failure modes governing the collapse performance of the seismic-force-resisting system. If the connection between the proposed component and the balance of the seismic-force-resisting system is the critical weak link, then tests of the proposed component excluding the connection would provide insufficient information about the collapse resistance of the seismic-force-resisting system.

3.5.4 Limitations on Component Applicability and Use

Design requirements should include provisions that reasonably restrict use of the proposed component to the range of configurations that were considered in testing and in the development of the component design space. Clear limitations are needed to ensure that the proposed component is used, in practice, in a manner that is consistent with the configurations that were determined to be equivalent.

For components tested without vertical loads, design provisions should restrict the allowable range of use (e.g., height limitations or other limitations that control vertical load demands) to ensure that the component cannot be used in a configuration where vertical loads might appreciably affect component performance. This limitation ensures that a proposed component tested without vertical loads cannot be constructed in a system where vertical loads can trigger a different failure mode or other important changes in performance or behavior that were not observed as part of the proposed component testing program. This issue is also discussed in Section 3.2.1.

3.5.5 Component Construction, Inspection, and Maintenance Requirements

No commentary for this section.

3.6 Proposed Component Test Data Requirements

Section 2.6 describes the test data required for the proposed component, and the approach of the Component Methodology in creating this data set. The first step is to define the proposed component design space (i.e., the range of permissible configurations for the proposed component). Specific proposed component configurations, selected to represent the overall range of the permissible configurations, are then tested experimentally and separated into component performance groups of tests sharing common features and behavioral characteristics. Summary statistics are then computed for each performance group and compared with those of the reference component data in the equivalency process.

3.6.1 Define Proposed Component Design Space

No commentary for this section.

3.6.2 Select Proposed Component Configurations for Testing

A subset of configurations should be selected for testing based on the proposed component design space. Selected configurations should represent all possible failure modes and should represent those configurations expected to be used most frequently in practice. The required number of configurations and test specimens are described in Sections 2.2.2 and 2.2.3.

3.6.3 Perform Cyclic-Load and Monotonic-Load Tests

The selected set of proposed component configurations should be tested in accordance with the requirements of Section 2.2, with related commentary provided in Section 3.2.

3.6.4 Interpret Proposed Component Test Results

Proposed component tests are used to determine cyclic-load test parameters for each test specimen. These parameters are related to component strength, stiffness, ultimate deformation, and effective ductility capacity, and are listed in Section 2.6.4. These parameters should be determined by constructing a cyclic envelope curve for each test, as described in Section 2.2.2 (and illustrated in Section 3.2.2).

From monotonic-load testing, the monotonic-load ultimate deformation, $\Delta_{UM,PC}$, is to be determined for each test specimen. Monotonic data are used in the acceptance criteria of Section 2.8.5 to ensure that there is adequate monotonic deformation capacity of the proposed components.

3.6.5 Define Proposed Component Performance Groups

Consistent with the approach used in defining performance groups for reference component test data (Section 3.4.4), the default approach for proposed components is to place all test data into a single performance group. Proposed component test data are only separated into multiple performance groups if there are fundamental differences in behavior between various test specimens.

When multiple performance groups are necessary, similar divisions should be used to separate proposed and reference component data (Section 2.4.4), and each proposed component performance group should be clearly associated with a reference component performance group (Section 2.6.5).

Table 3-2 illustrates a sample proposed component data set in tabular form, showing the separation of data into performance groups and the required number of cyclic-load and monotonic-load tests for each component performance group and component configuration.

Table 3-2 Sample Proposed Component Data Set

Proposed Component (PC) Performance Group	Component Configuration	Number of Cyclic-Load Tests	Number of Monotonic-Load Tests
PC-A	Config. A-1	≥ 2[1]	≥ 2 configurations (≥ 4 tests)
	Config. A-2	≥ 2[1]	
	Config. A-3	≥ 2[1]	
	Config. A-4	≥ 2[1]	
	…..	≥ 2[1]	
PC-B	Config. B-1	≥ 2[1]	≥ 2 configurations (≥ 4 tests)
	Config. B-2	≥ 2[1]	
	Config. B-3	≥ 2[1]	
	Config. B-4	≥ 2[1]	
	…..	≥ 2[1]	

1. Minimum of three cyclic load tests are required if any of the following occur: (1) rapid deterioration; (2) if the strength varies more than 15%; or (3) if the ultimate deformation capacity varies more than 20% between two tests.

3.6.6 Compute Summary Statistics

Summary statistics are computed for each performance group of the proposed component data set, in accordance with the guidelines of Section 2.6.6.

From cyclic-load test data, the following statistics are required:

- Median of each parameter for each component performance group ($\tilde{\Delta}_{U,PC}$, $\tilde{\Delta}_{Y,eff,PC}$, $\tilde{\mu}_{eff,PC}$, $\tilde{R}_{Q,PC}$, and $\tilde{R}_{K,PC}$)
- Lognormal standard deviation of each parameter for each component performance group ($\sigma_{\Delta_U,PC}$, etc.)
- Median ultimate deformation for an individual configuration ($\tilde{\Delta}_{Uj,PC}$).

From monotonic load test data, the following statistics are required:

- Median deformation for each component performance group ($\tilde{\Delta}_{UM,PC}$)

3.7 Quality Rating Criteria

3.7.1 Quality Rating of Test Data

Quality ratings are assigned because uncertainty in the collapse capacity of a structural system is affected by the uncertainty (quality) of the test data on which the collapse prediction is based. The quality of the test data is rated in accordance with the requirements of Section 2.7.1. The data are rated as Superior, Good, or Fair, based on the completeness and robustness of the testing program and resulting data. The quality rating approach for test data was adopted from the FEMA P-695 Methodology, with some modification.

The quality rating should be completed separately for the reference component test data set and the proposed component test data set. Section 2.3.3 requires that the rating should be a minimum of Good when applied to the reference component data set, and a minimum of Fair when applied to the proposed component data set. The quality of the proposed and reference component data affect the acceptance criteria of Section 2.8.

Completeness and Robustness of Tests

Completeness and robustness characteristics of test data are rated from high to low, based on the following information:

- **High.** All, or nearly all, important general testing issues of Section 2.2 are addressed comprehensively in the testing program and other supporting evidence. Experimental evidence is sufficient so that all, or nearly all, important behavior aspects at all levels (material, component, connections to rest of structure) are well understood, and the results can be used to quantify all important component parameters that affect the design requirements or that are used in the acceptance criteria. There is confidence that the manner of testing accurately reflects the demands that the component will experience as part of the full structural system.
- **Medium.** Most of the important general testing issues of Section 2.2 are addressed adequately in the testing program and other supporting evidence. Experimental evidence is sufficient so that all, or nearly all, important behavior aspects at all levels are generally understood, and the results can be used to quantify or deduce the important component parameters that significantly affect the design requirements or that are used in the acceptance criteria. The behavior of the full structural system is acceptably well understood, and there is moderate confidence that the manner of testing accurately reflects the demands that the component will experience as part of the full structural system.
- **Low.** Several important general testing issues of Section 2.2 are not addressed adequately in the testing program and other supporting evidence. Experimental evidence is sufficient so that the most important behavior aspects at all levels are fairly well understood, but the results are not adequate to quantify or deduce, with high confidence, some of the important component parameters that significantly affect the design requirements or that are used in the acceptance criteria. The behavior of the full structural system is not well understood, such that there is questionable confidence that the manner of testing accurately reflects the demands that the component will experience as part of the full structural system.

Confidence in Test Results

The confidence in test results is rated from high to low based on the following information:

- **High.** Reliable experimental information is produced on all important parameters. Test results are fully supported by basic principles of mechanics. The data set includes all component configurations needed to fully represent the expected populations.

  For use only when rating the proposed component data: The standard deviations of the proposed component parameters are not appreciably larger than the standard deviations of associated parameters in the reference component data.

- **Medium.** Moderately reliable experimental information is produced on all important parameters. Test results are supported by basic principles of mechanics. The reference and proposed component data sets are reasonably robust and include nearly all component configurations needed to represent the expected populations.

  For use only when rating the proposed component data: Most of the standard deviations for the proposed component parameters are not appreciably larger than the standard deviations of associated parameters in the reference component data.

- **Low.** Experimental information produced on many of the important parameters is of limited reliability. Basic principles of mechanics do not support some of the results of the testing program. The robustness of the data set is questionable; the set possibly still includes many component configurations, but the included component configurations do not adequately represent the expected populations.

3.7.2 Quality Rating of Design Requirements

The quality of the design requirements affects the uncertainty in the overall collapse performance of the structural system. The selection of a quality rating for design requirements considers the completeness and robustness of the requirements, and confidence in the basis for the design equations. The quality rating approach for design requirements was adopted from the FEMA P-695 Methodology, with some modification.

The highest rating of Superior applies to a comprehensive set of design requirements that provides safeguards against unanticipated failure modes. For a Superior rating, there should be a high level of confidence that the design requirements produce the anticipated structural performance.

Existing code requirements for components of special concrete moment frames, for example, have been vetted with detailed experimental results and real-world earthquake performance. Design and detailing provisions include capacity design requirements to safeguard against unanticipated behaviors. Such a set of requirements would be rated Superior.

The lowest ratings (e.g., "Not Allowed" in Table 2-2) apply to design requirements that have minimal safeguards against unanticipated failure modes, do not ensure a hierarchy of yielding and failure, and would generally be associated with components that exhibit behavior that is difficult to predict.

The quality rating approach outlined in Section 2.7.2 should be applied separately for the reference component design requirements and the proposed component design requirements. Section 2.3.3 requires that the rating should be a minimum of Good when applied to the reference component design requirements, and be at least Fair when applied to the proposed component design requirements. The quality of the proposed component design requirements affects the acceptance criteria of Section 2.8.

Completeness and Robustness of Design Requirements

Completeness and robustness characteristics of design requirements are rated from high to low based on the following information:

- **High.** Design requirements are extensive, well-vetted and provide extensive safeguards against unanticipated failure modes. All important behavioral issues have been addressed, resulting in a high reliability in the behavior of the component. Through mature construction practices and tightly specified quality assurance requirements, there is a high likelihood that the design provisions will be well executed through fabrication, erection, and final construction.
- **Medium.** Design requirements are reasonably extensive and provide reasonable safeguards against unanticipated failure modes, leaving some limited potential for the occurrence of such modes. While most important behavioral issues have been addressed, some have not, which somewhat reduces the reliability of the component. Quality assurance requirements are specified but may not fully address all the important aspects of fabrication, erection and final construction.
- **Low.** Design requirements provide questionable safeguards against unanticipated failure modes. Design requirements do not address all important behaviors, resulting in marginally reliable behavior of the

component. Quality assurance is lacking, written guidance is not provided, and construction practices are not well-developed for the type of system and materials.

Simplified, but conservative, design requirements by themselves are not a reason for a low Completeness and Robustness rating. However, if the conservatism is being used to compensate for the lack of safeguards against unanticipated failure modes, then a lower quality rating may be warranted.

Confidence in Design Requirements

Confidence in design requirements is rated from high to low based on the following information:

- **High.** There is substantiating evidence (experimental data, history of use, similarity with other components) that results in a high level of confidence that the properties, criteria, and equations provided in the design requirements will result in components that perform as intended.
- **Medium.** There is some substantiating evidence that results in a moderate level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended.
- **Low.** There is little substantiating evidence (little experimental data, no history of use, no similarity with other systems) that results in a low level of confidence that the properties, criteria, and equations provided in the design requirements will result in component designs that perform as intended.

3.8 Component Equivalency Acceptance Criteria

3.8.1 Overall Approach to Establishing Equivalency

Consistent with the FEMA P-695 Methodology, the intent of the Component Methodology is to ensure that code-designed buildings have adequate resistance to earthquake-induced collapse. Specifically, the Component Methodology ensures equivalent system-level collapse performance between seismic-force-resisting systems utilizing: (1) the reference component; (2) the proposed component; and (3) when permitted by the design provisions, any combination of the reference and proposed components.

The Component Methodology evaluates equivalency based on comparison of statistical values of performance parameters from proposed and reference components, calculated from results of both cyclic-load and monotonic-load tests. The primary component performance parameter is ultimate deformation of component test specimens, obtained from monotonic-load or cyclic-load testing, i.e., Δ_U or Δ_{UM}. Additionally, other acceptance criteria are based upon the component strength, stiffness, and effective ductility capacity.

This short-list of component parameters is necessarily and intentionally a simplification of the many parameters that could be extracted from the component test data, and could have an impact on collapse behavior. For example, an additional deformation parameter was considered at 50% strength loss (as compared with Δ_U and Δ_{UM} being defined at 20% strength loss), in order to capture the post-capping deformation capacity behavior, but it was decided that the added complication outweighed the potential benefit. Additionally, other parameters were also discussed, such as a specific measure of cyclic deterioration behavior and a measure of residual strength. These and other parameters were ultimately not included in the acceptance criteria of the Component Methodology for reasons of simplicity.

When component configurations are separated into groups with different behavioral characteristics, reference and proposed component test data are evaluated by performance group. The acceptance criteria to establish equivalency compares key performance parameters from each proposed component performance group to the associated reference component performance group. Each performance group should contain test data representing different component configurations. The number of different configurations should be sufficiently large to characterize behavior for the range of possible applications of the component of interest.

The overall process of establishing equivalency will often be iterative. If the proposed component does not meet the acceptance criteria, the user may introduce additional test data, thereby improving the quality rating of the test data and changing the acceptance criteria requirements. If particular performance groups or individual component configurations do not meet the acceptance criteria, the proposed component design requirements may be altered to prohibit such configurations, thus allowing the improved proposed component to pass the equivalency criteria.

3.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation

Requirements for Component Performance Groups

For each proposed component performance group, the median value of ultimate deformation, $\tilde{\Delta}_{U,PC}$, should be compared with the median value of ultimate deformation for the associated reference component performance group, $\tilde{\Delta}_{U,RC}$, according to Equation 2-1. It is notable that this criterion is a median-to-median comparison and not a criterion based on a high level of statistical confidence. A high statistical confidence approach was attempted and studied at length. It was determined that such an approach, by definition, would lead to acceptance criteria requiring proposed component parameters to greatly exceed reference component parameters (e.g., much larger ultimate deformation capacities). It was decided that such an approach would appear to unfairly penalize proposed components, so a median-to-median approach was adopted as the philosophical basis for the criterion in Equation 2-1.

In Equation 2-1, the reference component deformation is increased by penalty factors that account for uncertainties associated with component test data and design requirements, and differences in strength between the proposed and reference components. The derivation of these factors is described in Appendix C.

The penalty factor for strength, P_Q, accounts for a reduction in structural collapse resistance associated with a decrease in strength of its constituent components. If the proposed components are stronger than the reference components, there may also be a detrimental impact on structural collapse resistance if proposed and reference components are used together, because the stronger proposed components may cause damage localization in the weaker reference component or other elements in the structure.

Consider for example, a wood light-frame shear wall seismic-force-resisting system, in which the reference component is used in the first story and the stronger proposed component is used for upper stories (as shown in Appendix A). The strengthened upper stories may worsen the collapse performance, creating a weak first story in the structure. A penalty factor for strength, P_Q, greater than 1.0 is therefore applied if the normalized strength of the proposed component, quantified by $R_{Q,PC}$, is significantly greater or significantly smaller than the normalized strength of the reference component, quantified by $R_{Q,RC}$. The value of P_Q is obtained from Table 2-4 and depends on the ratio of $R_{Q,PC}$ to $R_{Q,RC}$.

P_Q may be taken as 1.0 if: (a) the design requirements do not allow mixing of the proposed and reference components in the same structure; and (b) the proposed component is stronger than the reference component (subject to a maximum additional strength ratio of 1.2 in some cases, per Section 2.8.2). This exception is permitted because a stronger proposed component is not detrimental to collapse safety when the proposed component is used consistently throughout the seismic-force-resisting system.

The value of $\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$ is limited to 1.2. This requirement is intended to protect capacity-designed and force-controlled components from additional overstrength that was not initially expected during the creation of the reference component design provisions. An exception to this requirement permits values of $\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$ greater than 1.2 to be used, provided that the design of such capacity-designed and force-controlled components is properly accounted for in the design provisions. If the typical Ω_O approach is used for designing such capacity-designed and force-controlled components, $\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$ values greater than 1.2 could be accounted for by increasing the seismic-force-resisting system design Ω_O value by a factor of $\left(\tilde{R}_{Q,PC}\right)/\left(1.2\tilde{R}_{Q,RC}\right)$, to account for the additional overstrength from the proposed component.

Requirements for Individual Component Configurations

Additional acceptance criteria related to individual component configurations (Equation 2-2) ensure that unsafe configurations are not approved because poor-performing configurations are averaged with better-performing configurations. This criterion also discourages the use of proposed components that have a standard deviation of Δ_U that is substantially larger than the reference component (i.e., more uncertain behavior).

3.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

Requirements for initial stiffness of the proposed component in Section 2.8.3 ensure that the stiffness of the component, and by extension the seismic-force-resisting system, is acceptably similar when the proposed component is fully substituted or partially substituted for the reference component. These requirements lead to equivalently stiff seismic-force-resisting systems, regardless of whether the design is controlled by strength requirements or drift limits.

In most cases, the collapse performance of a seismic-force-resisting system is less dependent on component initial stiffness than parameters related to component deformation capacity and strength (see Appendix A). However, component stiffness is identified as a key parameter in the Component Methodology because it is fundamental to ASCE/SEI 7-10 design procedures (see Section 3.5.1).

The requirement of Equation 2-3 specifies that $\tilde{R}_{K,PC}$ must be acceptably similar to $\tilde{R}_{K,RC}$ (recall that $\tilde{R}_K = \tilde{K}_I/\tilde{K}_D$). This equation ensures that the proposed component design requirements for predicting initial stiffness are compatible with the related design requirements for the reference component. If design of the seismic-force-resisting system is controlled by drift, this requirement ensures that the stiffness of the overall structure is comparable using either proposed components, reference components, or a mixture of both.

In the unusual case that a reference component does not have a method to predict initial stiffness, the proposed component initial stiffness prediction should be calibrated such that $\tilde{R}_{K,PC}$ is approximately equal to unity.

To add flexibility to this requirement, Section 2.8.3 provides an exception that proposed components need not comply with Equation 2-3 if they resist only a limited amount of lateral force along any framing line (i.e., less than or equal to 25% of the design force).

3.8.4 Requirements Based on Cyclic-Load Test Data: Effective Ductility Capacity

Section 2.8.4 (Equation 2-4) requires that the proposed component effective ductility capacity should be acceptably similar to that of the reference component. This requirement is enforced because studies in Appendix A showed that large changes in the energy dissipation capacity of a component can lead to important changes in the collapse capacity of a seismic-force-resisting system. Equation 2-4 is calibrated to catch possible extreme cases in the reduction of energy dissipation capacity, related to a large (more than 50%) reduction in post-yield deformation capacity.

Section 2.8.4 provides an exception that proposed components need not comply with Equation 2-4 if they resist only a limited amount of lateral force along any framing line (i.e., less than or equal to 25% of the design force).

3.8.5 Requirements Based on Monotonic-Load Test Data: Ultimate Deformation

For each performance group, the median value of proposed component ultimate deformation from monotonic-load testing, $\tilde{\Delta}_{UM,PC}$, should meet the requirements of either Equation 2-5 or Equation 2-6. Equation 2-5 is similar to the criterion for ultimate deformation from cyclic-load testing (Equation 2-1). Equation 2-6 is different, and was specifically designed for the case where monotonic-load test data are not readily available for the reference component. In this case, it must be shown that the median ultimate deformation from monotonic-load testing of the proposed component, $\tilde{\Delta}_{UM,PC}$, is acceptably larger than the median ultimate deformation of the reference component determined from cyclic-load testing, $\tilde{\Delta}_{U,RC}$. The intent of Equation 2-6 is to identify and prohibit proposed components that have only a small difference between the monotonic curve and the required cyclic envelope curve. This is necessary because the seismic-force-resisting system containing these components might have a smaller collapse resistance than would be expected based on cyclic-load test data alone (see Section B.3 of Appendix B).

The ratio between the monotonic-load and cyclic-load ultimate deformations, ($\tilde{\Delta}_{UM} / \tilde{\Delta}_U$), depends on the cyclic-load history used in testing. Past research listed below shows that there is significant variability in the ratio of (Δ_{UM} / Δ_U).

- Δ_{UM} / Δ_U = 1.0-2.0 for wood light frame shear walls subjected to a variety of cyclic-load histories (Gatto and Uang, 2002)
- Δ_{UM} / Δ_U = 1.7-1.8 for reinforced concrete columns (Panagiotakos and Fardis, 2001; Fardis and Biskini, 2003)
- Δ_{UM} / Δ_U = 1.5-1.6 based on data from both reinforced concrete and steel elements (PEER/ATC-72-1).

To differentiate between cyclic-load testing protocols that have either a small number or a large number of cycles, the 1.2 $\tilde{D}_C$ term is a reasonable representation of the ratio between the monotonic and cyclic values of ultimate deformation, (Δ_{UM} / Δ_U). The median cumulative damage factor, $\tilde{D}_C$, ranges from 1.0 to 1.5 and approximately accounts for the number of cycles in the cyclic-load testing protocol. This factor reflects the fact that there is naturally a larger difference between Δ_{UM} and Δ_U for cyclic-load testing protocols that have a larger number of cycles before reaching Δ_U.

The $\tilde{D}_C$ value of 1.0 (resulting in an approximate target of Δ_{UM} / Δ_U= 1.2) is based on observations for cyclic-load testing protocols that have a relatively small number of cycles (e.g., the CUREE protocol). The $\tilde{D}_C$ value of 1.5 (resulting in an approximate target of Δ_{UM} / Δ_U = 1.8) is based on observations for cyclic-load testing protocols that have a relatively large number of cycles (but still with fewer cycles than protocols like SPD). As a point of comparison, the ratio of ($\tilde{\Delta}_{U,CUREE} / \tilde{\Delta}_{U,SPD}$) is 0.035/0.0210 = 1.7, based on comparison of the wood light-frame shear wall data sets in Chapter 4 and Appendix D. This would suggest a $\tilde{D}_C$ value of 1.7 for loading protocols with an extremely large number of cycles (like SPD), but the $\tilde{D}_C$ value is capped at a maximum of 1.5 for simplicity.

3.9 Documentation and Peer Review Requirements

Use of the Component Methodology to determine equivalency should be fully documented and peer reviewed. Peer review encompasses the entire process and is most effective when implemented at the outset. The Component Methodology was developed to be generally applicable to a broad base of component types, but the peer review process will likely be needed to interpret the intent of the requirements for special circumstances related to proposed components, reference components, or the reference seismic-force-resisting system.

Chapter 4
Example Application

4.1 Introduction

This chapter illustrates the application of the Component Methodology to a possible substitution for nailed wood shear wall reference components in a wood light-frame seismic-force-resisting system. The objective is to determine if a hypothetical proposed component is equivalent to line A.15 in Table 12.2-1of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), which reads “Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets.” An actual component with complete set of test data fulfilling all of the requirements of the Component Methodology was not readily available, so a hypothetical proposed component with fictitious data was utilized in order to provide a full illustration of the Component Methodology.

In addition to illustrating the application of the Component Methodology, this example also presents the development of a robust and representative data set containing cyclic-load test results for nailed wood shear wall components available in the literature. The data set is based on Line et al. (2008), but has been modified to comply with the reference component design space used in this example. The data set consists of wall components with aspect ratios ranging from 2:1 to 1:1 that were tested using the CUREE loading protocol, which is a loading protocol that was developed as part of the CUREE-Caltech Woodframe Project (Krawinkler et al., 2000).

The steps in this example follow directly from the requirements of the Component Methodology defined in Sections 2.2 through 2.8.

4.2 Component Testing Requirements

The component testing requirements of Section 2.2 are fulfilled by the reference component data given in Section 4.4 and proposed component data created for the purpose of this example. Gravity loads were not included in either testing program, so a limitation on the building height was enforced for the proposed component (see Section 4.5.4). This was deemed acceptable because a moderate level of gravity load was not expected to have a substantial effect on the lateral load resistance of the shear wall components.

To fulfill the requirement of equivalency between cyclic loading protocols, both the reference component and proposed component data sets utilized data from tests that employed only the CUREE loading protocol.

4.3 Evaluation of Applicability Criteria

Based on the criteria of Section 2.3, including minimum acceptable quality ratings of test data and design requirements, the Component Methodology was deemed applicable for evaluating equivalency between the proposed component (hypothetical shear walls) and the reference component (nailed wood shear walls). In this example, quality ratings for test data are developed in Section 4.7.1, and quality ratings for design requirements are developed in Section 4.7.2. Based on the reported quality ratings, both the reference and proposed components meet the minimum applicability criteria for test data and design requirements. Additionally, both the reference and proposed shear wall components can be isolated from the remainder of the seismic-force-resisting system by a clear component boundary.

4.4 Reference Component Test Data

4.4.1 Define Reference Component Design Space

To clearly define the range of possible reference components, the reference component was defined as a nailed wood shear wall with the following representative configurations:

- Wall dimensions (height and length): 8'x4' to 8'x8'
- Aspect ratio (height/length): 2:1 to 1:1 (aspect ratios above 2:1 are not included in the design space)
- Sheathing: OSB and STR, 3/8" to 19/32" thickness
- Nails: 6d to 10d common nails
- Nail spacing (on-center): Edge spacing: 2" to 6"; field spacing: 6" to 12"
- Openings: No openings are considered. While openings are likely to occur in any wood light-frame building, the proposed component design requirements will be based on a segmented shear wall design approach, such that only full height piers (without openings) will be considered as part of the seismic-force-resisting system. Accordingly, the compiled reference component test data do not consider openings.

4.4.2 Compile or Generate Reference Component Test Data

The set of reference component test data were compiled based on the requirements of Sections 2.4.1, 2.4.2, and 2.2.

Line et al. (2008) compiled CUREE cyclic-load data from a total of 80 test specimens. This data set includes results from 48 wall tests that were compiled as part of the International Code Council Evaluation Services Acceptance Criteria AC322 effort (ICC-ES, 2007), as well as results from 32 additional tests. To create the reference component data set for nailed wood shear walls used in this example, data were removed for walls with openings, stapled walls, and walls with box nails, leaving a total of 65 tests in the final data set. This set is considered to be a robust and representative data set for nailed wood shear walls with aspect ratios ranging from 2:1 to 1:1, tested using the CUREE loading protocol.

4.4.3 Interpret the Reference Component Test Results

Test data were interpreted in accordance with the requirements of Section 2.4.3. The force quantity was taken as the horizontal shear force in the wall, and the displacement quantity was taken as the lateral drift in the wall (lateral displacement divided by height).

Figures 4-1 through 4-3 show the cyclic response and construction of the cyclic envelope curve for a nailed wood shear wall specimen from the above data set (Specimen E-A2, test index 17, which is an 8’x8’ wall with 7/16” OSB sheathing nailed with 8d common nails at 3” edge spacing and 12” field spacing).

Figure 4-2 shows the cyclic response and the cyclic envelope curves superimposed for the same wall specimen. The cyclic envelope curve was constructed using the requirements of Section 2.2.2, which differ slightly from the rules utilized by Line et al. (2008), so the values of ultimate deformation also differ slightly from those reported in Line et al. (2008).

Figure 4-3 shows the envelope curve with the parameters that are utilized in the acceptance criteria of the Component Methodology. The maximum strengths in the positive and negative directions are 11,330 lbs. and 9,892 lbs., respectively, with an average V_M = 10,611 lbs. Using the shear wall strength table, Table 2306.4.1, in the 2006 *International Building Code* (ICC, 2006), the allowable stress design strength, V_D, of this shear wall is 3,920 lbs. (490 lb/ft), so V_M / V_D = 2.7. This value is denoted as R_Q in the Component Methodology.

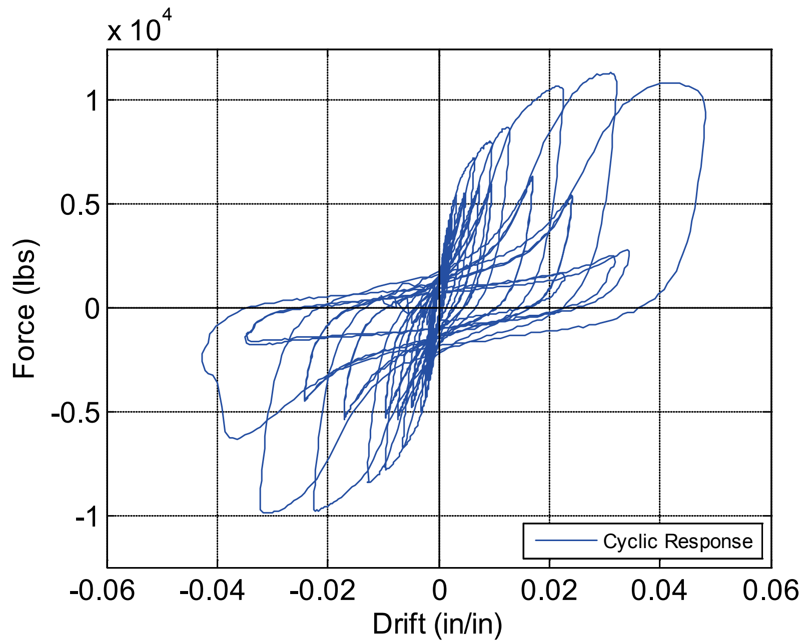


Figure 4-1 Illustration of cyclic response of a nailed wood shear wall, data from Line et al. (2008) and Rosowsky et al. (2004).

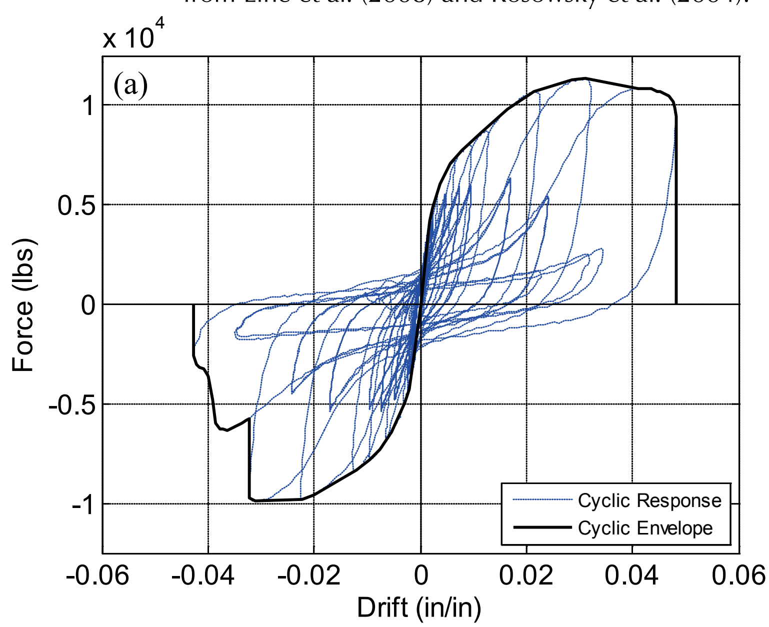


Figure 4-2 Illustration of cyclic response and envelope curve for a nailed wood shear wall, data from Line et al. (2008) and Rosowsky et al. (2004).

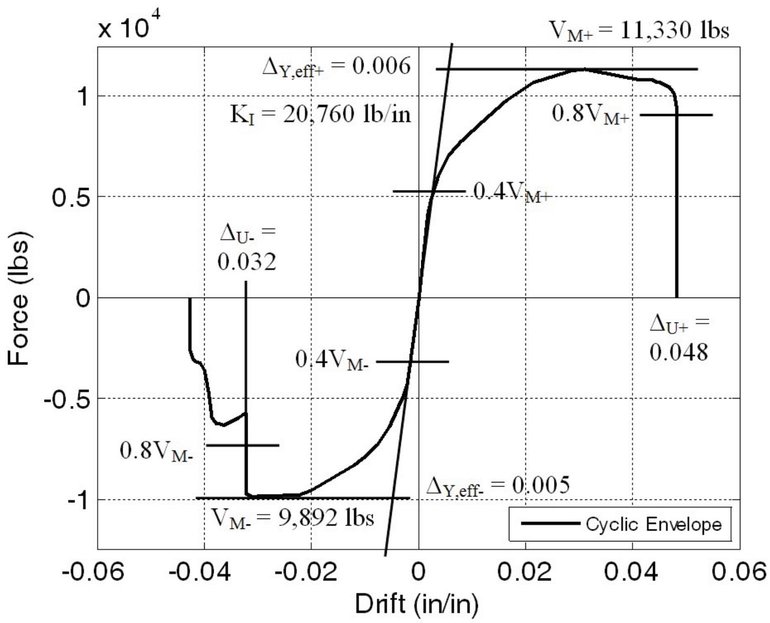


Figure 4-3 Illustration of cyclic envelope curve and calculation of component response quantities for a nailed wood shear wall, data from Line et al. (2008) and Rosowsky et al. (2004).

The initial stiffness was computed as the secant stiffness at 40% of the peak strength. The average of the positive and negative loading directions is K_I = 20,755 lb/in. Using the allowable stress design (ASD) load of 3,920 lbs. and the displacement at the ASD design load (computed according to IBC Equation 23-2), the design initial stiffness is K_D = 19,311 lb/in. Therefore, for this test specimen, $K_I / K_D = 1.1$. This value is denoted as R_K in the Component Methodology.

Computing the effective ductility capacity, μ_{eff}, requires calculation of the effective yield displacement, $\Delta_{Y,eff}$. The values of $\Delta_{Y,\,eff}$ were computed separately in the positive and negative directions, and the average value was $\Delta_{Y,eff}$ = 0.53% drift for this test specimen.

The ultimate deformation, Δ_U, is 0.032 in/in. in the negative loading direction. In the positive loading direction, the specimen does not experience a 20% loss in the applied maximum load, so the deformation capacity was limited to the maximum displacement reached in the positive direction, which corresponds to a drift of 0.048 in/in. The final average ultimate deformation is Δ_U = 0.040 in/in. for this specimen. Per Section 2.2.2, the value of Δ_U could be affected if the vertical load-carrying ability of the specimen were compromised. In this example, such a modification was not necessary. Using the final value of Δ_U and the previously computed value of $\Delta_{Y,eff}$, the effective ductility capacity was calculated to be $\mu_{eff} = 7.5$.

These calculations were completed for each of the 65 sets of data included in the reference component data set. Table 4-1 summarizes the configuration of each test specimen and the failure modes observed during testing.

Table 4-2 summarizes the values of important response quantities (e.g., strength, stiffness, ultimate deformation) for each specimen of the reference component data set. Per the requirements of Section 2.2.2, values of the ultimate deformation, Δ_U, were not reported for test specimens in which the loading was not continued to 20% strength loss.

Table 4-1 Summary of Nailed Wood Shear Wall Configurations in the Reference Component Data Set

Test Index	Wall Dimensions (HxL)	OSB Sheathing	Fastener	Fastener Spacing (edge/field)	Failure Mode
1	8.5' x 4.5'	15/32" Str. 1	10d com	2"/12"	Not Reported
2	8.5' x 4.5'	15/32" Str. 1	10d com	2"/12"	Not Reported
3	8' x 8'	7/16"	8d com	4"/6"	Not Reported
4	8' x 8'	7/16"	8d com	4"/6"	Not Reported
5	8' x 8'	7/16"	8d com	4"/6"	Not Reported
6	8' x 8'	19/32"	10d com	2"/12"	Not Reported
7	8' x 8'	19/32"	10d com	2"/12"	End Post Tension
8	8' x 8'	19/32"	10d com	2"/12"	Not Reported
9	8' x 8'	19/32"	10d com	2"/12"	Not Reported
10	8' x 8'	7/16"	8d com	4"/6"	Not Reported
11	8' x 8'	7/16"	8d com	4"/6"	Not Reported
12	8' x 8'	7/16"	8d com	4"/6"	Not Reported
13	8' x 8'	7/16"	8d com	4"/6"	Not Reported
14	8' x 8'	7/16"	8d com	4"/6"	Not Reported
15	8' x 8'	7/16"	8d com	4"/6"	Not Reported
16	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
17	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
18	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
19	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
20	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
21	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
22	8' x 8'	7/16"	8d com	3"/12"	Sill Plate Splitting
23	8' x 8'	7/16"	8d com	3"/12"	Sill Plate Splitting
24	8' x 8'	7/16"	8d com	3"/12"	Sill Plate Splitting
25	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
26	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
27	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
28	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
29	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
30	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure

Table 4-1 Summary of Nailed Wood Shear Wall Configurations in the Reference Component Data Set (continued)

Test Index	Wall Dimensions (HxL)	OSB Sheathing	Fastener	Fastener Spacing (edge/field)	Failure Mode
31	8' x 8'	7/16"	8d com	3"/12"	Sill Plate Splitting
32	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
33	8' x 8'	7/16"	8d com	3"/12"	Fastener Failure
34	8' x 8'	3/8" Str.1	6d com	6"/6"	Fastener Failure
35	8' x 8'	3/8" Str.1	6d com	6"/6"	Fastener Failure
36	8' x 4'	3/8" Str.1	6d com	6"/6"	Fastener Failure
37	8' x 4'	3/8" Str.1	6d com	6"/6"	Fastener Failure
38	8' x 8'	3/8" Str.1	6d com	2"/6"	Fastener Failure
39	8' x 8'	3/8" Str.1	6d com	2"/6"	Fastener Failure
40	8' x 4'	3/8" Str.1	6d com	2"/6"	Fastener Failure
41	8' x 4'	3/8" Str.1	6d com	2"/6"	Fastener Failure
42	8' x 8'	7/16"	8d com	6"/6"	Fastener Failure
43	8' x 8'	7/16"	8d com	6"/6"	Fastener Failure
44	8' x 4'	7/16"	8d com	6"/6"	Fastener Failure
45	8' x 4'	7/16"	8d com	6"/6"	Fastener Failure
46	8' x 8'	7/16"	8d com	2"/6"	Fastener Failure
47	8' x 8'	7/16"	8d com	2"/6"	Fastener Failure
48	8' x 4'	7/16"	8d com	2"/6"	End Post Failure
49	8' x 4'	7/16"	8d com	2"/6"	Fastener Failure
50	8' x 8'	19/32"	10d com	6"/12"	Fastener Failure
51	8' x 8'	19/32"	10d com	6"/12"	Fastener Failure
52	8' x 4'	19/32"	10d com	6"/12"	Fastener Failure
53	8' x 4'	19/32"	10d com	6"/12"	Fastener Failure
54	8' x 8'	19/32"	10d com	2"/12"	Fastener Failure
55	8' x 8'	19/32"	10d com	2"/12"	Fastener Failure
56	8' x 4'	19/32"	10d com	2"/12"	Fast./Post Failure
57	8' x 4'	19/32"	10d com	2"/12"	Fastener Failure
58	8' x 8'	3/8" Str.1	8d com	6"/6"	Fastener Failure
59	8' x 8'	3/8" Str.1	8d com	6"/6"	Fastener Failure
60	8' x 4'	3/8" Str.1	8d com	6"/6"	Fastener Failure
61	8' x 4'	3/8" Str.1	8d com	6"/6"	Fastener Failure
62	8' x 8'	3/8" Str.1	8d com	2"/6"	Fastener Failure
63	8' x 8'	3/8" Str.1	8d com	2"/6"	Fastener Failure
64	8' x 4'	3/8" Str.1	8d com	2"/6"	Fastener Failure
65	8' x 4'	3/8" Str.1	8d com	2"/6"	Fastener Failure

Table 4-2 Summary of Important Component Parameters for the Reference Component Data Set

Test Index	Strength			Stiffness			Ductility		Deformation Capacity
	V_M (lb)	V_D (lb)	R_Q	K_I (lb/in)	K_D (lb/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)
1	10,239	3,915	2.6	7,594	14,961	0.5	0.013	n/a	n/a
2	10,461	3,915	2.7	7,520	14,961	0.5	0.014	2.7	0.037
3	7,019	2,800	2.5	21,195	15,043	1.4	0.003	7.6	0.026
4	6,965	2,800	2.5	18,475	15,043	1.2	0.004	10.2	0.040
5	6,701	2,800	2.4	19,496	15,043	1.3	0.004	11.1	0.040
6	15,947	6,960	2.3	30,989	27,981	1.1	0.005	7.0	0.038
7	15,953	6,960	2.3	26,755	27,981	1.0	0.006	6.0	0.037
8	17,856	6,960	2.6	24,219	27,981	0.9	0.008	6.2	0.048
9	17,421	6,960	2.5	24,903	27,981	0.9	0.007	6.6	0.048
10	7,184	2,800	2.6	7,007	15,043	0.5	0.011	2.8	0.030
11	7,454	2,800	2.7	6,431	15,043	0.4	0.012	2.6	0.031
12	6,906	2,800	2.5	12,020	15,043	0.8	0.006	5.5	0.033
13	7,136	2,800	2.5	11,516	15,043	0.8	0.006	5.0	0.032
14	7,631	2,800	2.7	28,419	15,043	1.9	0.003	9.3	0.026
15	7,478	2,800	2.7	28,346	15,043	1.9	0.003	9.5	0.026
16	10,208	3,920	2.6	12,721	19,311	0.7	0.008	n/a	n/a
17	10,611	3,920	2.7	20,755	19,311	1.1	0.005	7.5	0.040
18	11,151	3,920	2.8	15,990	19,311	0.8	0.007	5.7	0.042
19	10,877	3,920	2.8	18,639	19,311	1.0	0.006	5.8	0.035
20	11,554	3,920	2.9	22,241	19,311	1.2	0.005	7.9	0.043
21	11,396	3,920	2.9	21,876	19,311	1.1	0.005	7.7	0.042
22	10,952	3,920	2.8	11,337	19,311	0.6	0.010	3.5	0.036
23	10,574	3,920	2.7	15,910	19,311	0.8	0.007	6.0	0.041
24	11,156	3,920	2.8	14,604	19,311	0.8	0.008	4.8	0.038
25	10,863	3,920	2.8	20,521	19,311	1.1	0.006	n/a	n/a
26	10,771	3,920	2.7	21,449	19,311	1.1	0.005	n/a	n/a
27	10,265	3,920	2.6	13,381	19,311	0.7	0.008	n/a	n/a
28	10,794	3,920	2.8	19,110	19,311	1.0	0.006	6.2	0.036
29	10,923	3,920	2.8	19,181	19,311	1.0	0.006	6.4	0.038
30	10,920	3,920	2.8	20,288	19,311	1.1	0.006	6.1	0.034
31	10,113	3,920	2.6	18,624	19,311	1.0	0.006	5.8	0.033
32	10,831	3,920	2.8	20,115	19,311	1.0	0.006	6.3	0.035
33	11,079	3,920	2.8	19,328	19,311	1.0	0.006	6.1	0.036
34	4,573	1,780	2.6	16,020	7,517	2.1	0.003	9.5	0.028
35	4,280	1,780	2.4	16,406	7,517	2.2	0.003	10.3	0.028
36	2,502	890	2.8	7,929	3,674	2.2	0.003	11.1	0.036
37	2,467	890	2.8	7,076	3,674	1.9	0.004	11.7	0.042
38	12,753	4,530	2.8	27,980	18,253	1.5	0.005	7.6	0.036
39	12,805	4,530	2.8	16,722	18,253	0.9	0.008	4.2	0.034
40	6,746	2,265	3.0	7,076	8,642	0.8	0.010	4.4	0.043
41	5,888	2,265	2.6	10,985	8,642	1.3	0.006	8.1	0.045
42	5,750	1,920	3.0	21,464	11,105	1.9	0.003	11.1	0.031
43	6,273	1,920	3.3	22,698	11,105	2.0	0.003	10.0	0.029
44	2,889	960	3.0	5,554	5,369	1.0	0.005	7.4	0.040
45	3,305	960	3.4	6,840	5,369	1.3	0.005	8.0	0.040

Table 4-2 Summary of Important Component Parameters for the Reference Component Data Set (continued)

Test Index	Strength			Stiffness			Ductility		Deformation Capacity
	V_M (lb)	V_D (lb)	R_Q	K_I (lb/in)	K_D (lb/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)
46	16,058	4,680	3.4	23,492	32,858	0.7	0.007	4.2	0.030
47	16,345	4,680	3.5	19,476	32,858	0.6	0.009	3.6	0.032
48	7,584	2,340	3.2	7,444	14,922	0.5	0.011	3.4	0.036
49	6,590	2,340	2.8	7,602	14,922	0.5	0.009	3.6	0.033
50	6,577	2,720	2.4	18,481	12,311	1.5	0.004	7.9	0.029
51	6,851	2,720	2.5	18,897	12,311	1.5	0.004	9.8	0.037
52	3,345	1,360	2.5	6,574	5,931	1.1	0.005	6.4	0.034
53	3,296	1,360	2.4	6,474	5,931	1.1	0.005	7.4	0.039
54	16,826	6,960	2.4	28,283	24,443	1.2	0.006	5.9	0.036
55	16,992	6,960	2.4	27,607	24,443	1.1	0.006	5.6	0.036
56	7,504	3,480	2.2	8,626	11,368	0.8	0.009	4.5	0.041
57	7,363	3,480	2.1	9,443	11,368	0.8	0.008	5.3	0.043
58	5,614	1,840	3.1	22,288	12,458	1.8	0.003	10.8	0.028
59	5,646	1,840	3.1	20,629	12,458	1.7	0.003	8.4	0.024
60	2,757	920	3.0	8,469	6,000	1.4	0.003	8.6	0.029
61	2,849	920	3.1	7,570	6,000	1.3	0.004	8.8	0.034
62	15,695	4,880	3.2	20,870	36,571	0.6	0.008	4.0	0.031
63	14,804	4,880	3.0	23,317	36,571	0.6	0.007	5.3	0.035
64	6,838	2,440	2.8	8,909	16,438	0.5	0.008	4.6	0.036
65	7,216	2,440	3.0	9,510	16,438	0.6	0.008	4.7	0.038
Median:	$\tilde{R}_Q = 2.7$			$\tilde{R}_K = 1.00$			$\tilde{\mu}_{eff} = 6.3$		$\tilde{\Delta}_U = 0.035$
Variability:	$\sigma_{RQ} = 0.11$			$\sigma_{RK} = 0.42$			$\sigma_{\mu,eff} = 0.38$		$\sigma_{\Delta U} = 0.16$

4.4.4 Define Reference Component Performance Groups

Based on the requirements of Section 2.4.4, the reference component data were placed into a single performance group.

4.4.5 Compute Summary Statistics

The summary statistics for each component parameter were computed in accordance with Section 2.4.5, assuming an underlying lognormal distribution of the test data. Table 4-3 presents the summary statistics for the data set, which is information repeated from the bottom of Table 4-2. Example calculations for summary statistics are illustrated in Section 4.6.6 using the proposed component test data.

Table 4-3 Summary Statistics for Reference Component Parameters

Summary Statistic	R_Q ($=V_M / V_D$)	R_K ($=K_I / K_D$)	μ_{eff}	Δ_U (in/in)
Median	$\tilde{R}_Q = 2.7$	$\tilde{R}_K = 1.00$	$\tilde{\mu}_{eff} = 6.3$	$\tilde{\Delta}_U = 0.035$
Variability	$\sigma_{RQ} = 0.11$	$\sigma_{RK} = 0.42$	$\sigma_{\mu,eff} = 0.38$	$\sigma_{\Delta U} = 0.16$

4.5 Proposed Component Design Requirements

In accordance with Section 2.5, a comprehensive set of design requirements is needed for the proposed component. This section presents the hypothetical design requirements for the proposed component.

4.5.1 Component Design Strength and Stiffness

To meet the acceptance criteria of Section 2.8, the ASD strengths of the proposed component were roughly calibrated to the experimental test data. Table 4-4 provides the design strengths in the traditional format for each hypothetical combination of sheathing type and connector spacing.

Table 4-4 Proposed Component Design Strengths for Hypothetical Combinations of Sheathing Type and Connector Spacing

Proposed Component Allowable Stress Design Strengths (lbs/ft)			
Sheathing Type / Thickness	Connector Spacing Pattern		
	1	2	3
1	225	450	540
2	250	500	600
3	280	570	680

Design requirements are also needed for the prediction of initial stiffness. Initial stiffness is an important aspect of design for various code requirements (e.g., checking design drift limits) as well as for proper modeling to distribute system-level forces to individual components.

In some wood light-frame components, the design stiffness is included implicitly within the published design strength values. In this example, however, the initial stiffness values were considered separately. Various approaches could be used for computing the design stiffness. For example, Equation 23-2 in the 2006 IBC could be used for computing the wall displacement at the design load, and this equation is repeated below. Another viable option would be to consider the use of Equation 4.3-1 of

ANSI/AF&PA SDPWS-2008 *Special Design Provisions for Wind and Seismic with Commentary* (AF&PA, 2009).

To compute these values, parameters in the equation, such as the connector deformation value, e_n, were calibrated based on available test data, with the goal of making the stiffness ratio, $\tilde{R}_{K,PC}/\tilde{R}_{K,RC}$, approximately equal to unity.

$$\Delta = \frac{8vh^3}{Eab} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$

where:

a = area of boundary element (in^2)

b = wall width (feet)

d_a = vertical elongation of overturning anchorage at the design shear load (v) (inches)

E = elastic modulus of boundary element (pounds/inch/inch)

e_n = connector deformation (inches)

Gt = panel rigidity through the thickness (pounds/inch)

h = wall height (feet)

v = maximum shear due to design loads, applied at the top of the wall (pounds/foot)

The design stiffness values in this example were computed for each hypothetical combination of sheathing, connector, and panel geometry, and summarized in Table 4-5. It should be noted that the values in Table 4-5 do not include any effect of anchorage deflection. Anchorage deflection would need to be considered separately if determined to be important to the overall as-built component stiffness.

4.5.2 Component Detailing Requirements

A proposed component must have a complete set of design detailing requirements. It is not possible, however, to illustrate the development of these requirements in a meaningful way for a hypothetical proposed component. The specific test applications presented in the Appendices of this report provide additional illustration of this important step in the process.

Table 4-5 Proposed Component Design Stiffness for Each Panel Geometry

Proposed Component Design Stiffness (lb/in) - For 8'x4' Panels			
Sheathing Type / Thickness	Connector Spacing Pattern		
	1	2	3
1	3,300	5,400	9,000
2	3,500	5,800	9,600
3	3,700	9,600	10,000

Proposed Component Design Stiffness (lb/in) - For 8'x8' Panels			
Sheathing Type / Thickness	Connector Spacing Pattern		
	1	2	3
1	7,200	12,000	19,800
2	7,700	12,800	21,200
3	8,000	13,300	22,000

Proposed Component Design Stiffness (lb/in) - For 8'x12' Panels			
Sheathing Type / Thickness	Connector Spacing Pattern		
	1	2	3
1	12,600	20,400	33,000
2	13,500	21,800	35,300
3	14,000	22,600	36,600

4.5.3 Component Connection Requirements

To meet the requirements of Section 2.5.3, connection requirements for the proposed component were developed to specify that, in addition to vertical loads, the framing members of the proposed component shear wall must be designed to resist induced seismic forces per the design methods listed in Section 2.1.2 of ANSI/AFPA SDPWS-2008. The connections between the wall and the rest of the structural system must also be designed to resist the induced seismic forces per the design methods listed in ANSI/AFPA SDPWS-2008, and the boundary elements of the proposed component shear wall (i.e., the end studs) must be designed to transfer the design tension and compression forces to the rest of the structural system.

Though not explicitly demonstrated in this example, it is assumed that these connection requirements ensure that the inelastic behavior occurs in the proposed shear wall components and not in the connections between the wall and the balance of the seismic-force-resisting system.

4.5.4 Limitations on Component Applicability and Use

The Component Methodology requires that the use of the proposed component must be reasonably restricted to the range of possible configurations that were considered in the testing program (i.e., use must be restricted to the "component design space"). Accordingly, the hypothetical proposed component should only be used within the range of configurations outlined in Section 4.6.1 (e.g., aspect ratios less than 2:1).

Additionally, since the tests were completed without applied vertical loads, the proposed component design provisions include a restriction to the use of the component to ensure that applied vertical loads would not tangibly affect the behavior and performance of the component. In this example, the current height limits in ASCE/SEI 7-10 (i.e., 65' in high seismic regions) were deemed sufficient to meet the requirements of Section 2.5.4.

4.5.5 Component Construction, Inspection, and Maintenance Requirements

The proposed component was assumed to be a pre-fabricated component that would have quality control standards pertaining to the quality and consistency of the as-built pre-fabricated shear wall components. Additionally, it was assumed that the proposed component would also have requirements for on-site installation, inspection, and maintenance.

4.6 Proposed Component Test Data

4.6.1 Define Proposed Component Design Space

The proposed component design space includes wall components with the following representative configurations:

- Wall height and length: 8'x 4' to 8'x12'
- Aspect ratio (height/length): 2:1 to 0.67:1. This range is broader than the reference component data set, but such differences are permitted when the transfer of forces across the component boundary is not significantly affected.
- Sheathing type/thickness: Hypothetical sheathing types 1, 2, and 3 identified in Table 4-4.
- Connectors: Hypothetical connectors with spacing patterns 1, 2, and 3 identified in Table 4-4.

- Openings: No openings are included in the test specimens. Since openings are likely to occur in wood light-frame building applications, a segmented shear wall design approach will be used, such that only full height piers will be considered as part of the seismic-force-resisting system.

4.6.2 Select Proposed Component Configurations for Testing

Table 4-6 summarizes the proposed component configurations that were selected for cyclic-load testing in this example.

Table 4-6 Summary of Proposed Component Wall Configurations for Cyclic-Load Testing

Test Index	Wall Dimensions (HxL)	Sheathing Type/ Thickness	Connector Spacing Pattern	Connector Type
1	8' x 4'	1	2	1
2	8' x 4'	1	2	1
3	8' x 4'	2	2	1
4	8' x 4'	2	2	1
5	8' x 4'	3	2	1
6	8' x 4'	3	2	1
7	8' x 4'	2	1	1
8	8' x 4'	2	1	1
9	8' x 4'	2	3	1
10	8' x 4'	2	3	1
11	8' x 8'	1	2	1
12	8' x 8'	1	2	1
13	8' x 8'	2	2	1
14	8' x 8'	2	2	1
15	8' x 8'	3	2	1
16	8' x 8'	3	2	1
17	8' x 8'	2	1	1
18	8' x 8'	2	1	1
19	8' x 8'	2	3	1
20	8' x 8'	2	3	1
21	8' x 12'	1	2	1
22	8' x 12'	1	2	1
23	8' x 12'	1	2	1
24	8' x 12'	3	2	1
25	8' x 12'	3	2	1
26	8' x 12'	2	1	1
27	8' x 12'	2	1	1

Component configurations used for cyclic-load testing should span the component design space. The selected configurations fulfill the minimum test data requirements of four configurations per performance group with at least two specimens per configuration. These configurations do not cover all possible combinations of sheathing, connector spacing, and component geometry. Even so, it is assumed that the peer review panel would agree that this test matrix was sufficiently extensive because it spanned the design space in a reasonably complete manner, and no problematic trends were observed in the chosen configurations, so there is no expectation that the behavior would be systematically different in the configurations that were not tested.

Table 4-7 summarizes the proposed component configurations chosen for monotonic testing in this example. Configurations selected for monotonic-load testing are more limited, but they fulfill the minimum test data requirements of Section 2.2.3, which specifies a minimum of two configurations per performance group with at least two specimens per configuration. The selected configurations were chosen because they were considered the most representative of the configurations used in the full cyclic data set.

Table 4-7 Summary of Proposed Component Wall Configurations for Monotonic-Load Testing

Test Index	Wall Dimensions (HxL)	Sheathing Type/ Thickness	Connector Spacing Pattern	Connector Type
3	8' x 4'	2	2	1
4	8' x 4'	2	2	1
13	8' x 8'	2	2	1
14	8' x 8'	2	2	1

4.6.3 Perform Cyclic-Load and Monotonic-Load Tests

Cyclic-load testing would need to be completed for the component configurations listed in Table 4-6. Because the reference component data set utilized the CUREE loading protocol, the proposed component specimens were tested with the CUREE loading protocol to satisfy requirements for equivalency of loading history.

Similarly, monotonic-load testing would also need to be completed for the component configurations listed in Table 4-7. The hypothetical specimens were considered to have nominally symmetric behavior, and were therefore pushed monotonically in only one direction, in accordance with the requirements of Section 2.2.3.

4.6.4 Interpret Proposed Component Test Results

Test data were compiled for each of the 27 proposed component shear wall cyclic-load tests and summarized in Table 4-8.

Table 4-8 Summary of Important Component Parameters from the Proposed Component Cyclic-Load Data Set

Test Index	Strength			Stiffness			Ductility		Deformation Capacity	
	V_M (lb)	V_D (lb)	R_Q	K_I (lb/in)	K_D (lb/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)	
1	5,645	1,800	3.1	3,897	5,400	0.7	0.015	3.4	0.052	0.051
2	5,786	1,800	3.2	4,125	5,400	0.8	0.015	3.4	0.050	
3	5,485	2,000	2.7	4,456	5,800	0.8	0.013	3.3	0.042	0.044
4	6,125	2,000	3.1	4,658	5,800	0.8	0.014	3.4	0.046	
5	6,030	2,280	2.6	7,035	9,600	0.7	0.009	6.5	0.058	0.057
6	5,895	2,280	2.6	7,212	9,600	0.8	0.009	6.6	0.056	
7	2,889	1,000	2.9	3,325	3,500	1.0	0.009	4.0	0.036	0.038
8	3,158	1,000	3.2	4,100	3,500	1.2	0.008	5.0	0.040	
9	7,056	2,400	2.9	4,987	9,600	0.5	0.015	3.3	0.048	0.050
10	6,590	2,400	2.7	5,123	9,600	0.5	0.013	3.9	0.052	
11	10,564	3,600	2.9	10,245	12,000	0.9	0.011	4.5	0.048	0.046
12	10,250	3,600	2.8	12,560	12,000	1.0	0.009	5.3	0.045	
13	11,890	4,000	3.0	9,594	12,800	0.7	0.013	3.3	0.043	0.042
14	12,236	4,000	3.1	11,256	12,800	0.9	0.011	3.6	0.041	
15	14,592	4,560	3.2	10,589	13,300	0.8	0.014	3.6	0.051	0.050
16	15,624	4,560	3.4	11,305	13,300	0.9	0.014	3.4	0.049	
17	5,750	2,000	2.9	13,251	7,700	1.7	0.005	8.0	0.036	0.035
18	6,273	2,000	3.1	9,856	7,700	1.3	0.007	5.3	0.035	
19	16,058	4,800	3.3	14,095	21,200	0.7	0.012	3.8	0.045	0.046
20	15,987	4,800	3.3	11,686	21,200	0.6	0.014	3.4	0.048	
21	18,956	5,400	3.5	21,897	20,400	1.1	0.009	4.7	0.042	0.035
22	21,560	6,000	3.6	22,589	21,800	1.0	0.010	2.8	0.028	
23	19,258	5,400	3.6	20,156	20,400	1.0	0.010	3.8	0.038	
24	22,561	6,840	3.3	27,895	22,600	1.2	0.008	5.3	0.045	0.042
25	23,520	6,840	3.4	28,945	22,600	1.3	0.008	4.7	0.040	
26	10,678	3,000	3.6	10,057	13,500	0.7	0.011	3.4	0.038	0.037
27	10,253	3,000	3.4	9,762	13,500	0.7	0.011	3.3	0.036	
Median:	$\tilde{R}_Q = 3.1$			$\tilde{R}_K = 0.86$			$\tilde{\mu}_{eff} = 4.1$		$\tilde{\Delta}_U = 0.043$	
Variability:	$\sigma_{RQ} = 0.10$			$\sigma_{RK} = 0.28$			$\sigma_{\mu,eff} = 0.26$		$\sigma_{\Delta U} = 0.17$	

The test data were also compiled for each of the four proposed component shear wall monotonic-load tests and summarized in Table 4-9.

Table 4-9 Summary of Important Component Parameters from the Proposed Component Monotonic-Load Data Set

Test Index	V_{MM} (lb)	V_D (lb)	V_{MM} / V_D	Δ_{UM} (in/in)
3	5,645	2,000	2.8	0.070
4	5,786	2,000	2.9	0.064
13	5,485	4,000	1.4	0.056
14	6,125	4,000	1.5	0.061
		Median:	**2.0**	**0.062**

4.6.5 Define Proposed Component Performance Groups

The proposed component data were placed in a single performance group, consistent with the requirements of Section 2.6.5, and also consistent with the approach taken for the reference component data set.

Although strength and stiffness values differ between various wall configurations, these differences were considered to be minor so that the test data need not be separated into multiple performance groups. Additionally, there were no strong trends in component behavior (i.e., failure modes) that would necessitate breaking the data set into multiple performance groups.

For use in the acceptance criteria evaluation, the following statistical parameters were computed for the proposed component data set. Similar parameters are also needed for the reference component (unless otherwise noted below).

Performance Parameters based on Cyclic-Load Testing:

- Median value of ultimate deformation, $\tilde{\Delta}_{U,PC}$
- Lognormal standard deviation of ultimate deformation, $\sigma_{\Delta U,PC}$
- Individual configuration median ultimate deformation, $\tilde{\Delta}_{Uj,PC}$ (not needed for the reference component)
- Median value of the stiffness ratio, $\tilde{R}_{K,PC}$
- Median value of the load (strength) ratio, $\tilde{R}_{Q,PC}$
- Median value of the effective ductility capacity, $\tilde{\mu}_{eff,PC}$

Performance Parameters based on Monotonic-Load Testing:

- Median value of ultimate deformation, $\tilde{\Delta}_{UM,PC}$ (only needed for the reference component if Equation 2-6 cannot be fulfilled by the proposed component monotonic data)

4.6.6 Compute Summary Statistics

In accordance with the requirements of Sections 2.4.5 and 2.6.6, summary statistics were computed using the assumption of a lognormal distribution of the data. Using $\tilde{\Delta}_{U,PC}$ as an example, summary statistics can be calculated from the following steps (for a given performance group):

1. Compute the natural logarithm of $\Delta_{U,PC}$, i.e., LN[$\Delta_{U,PC}$], for each test.
2. Compute the average of the LN[$\Delta_{U,PC}$] values.
3. Compute the exponent of the result from step (2), i.e., $e^{Mean(LN[\Delta U,PC])}$. This value is the fitted median value, $\tilde{\Delta}_{U,PC}$.
4. Compute the standard deviation of the LN[$\Delta_{U,PC}$] values. This is the fitted logarithmic standard deviation value, $\sigma_{\Delta U,PC}$.

The above process (steps 1-3) can also be used to compute the median ultimate deformation value for each individual component configuration, $\tilde{\Delta}_{Uj,PC}$, as well as other statistical parameters. Proposed component summary statistics for cyclic-load test data were presented at the bottom of Table 4-8 and are summarized again in Table 4-10.

Table 4-10 Summary Statistics for Proposed Component Parameters from Cyclic-Load Test Data

Summary Statistic	R_Q $(=V_M/V_D)$	R_K $(=K_I/K_D)$	μ_{eff}	Δ_U (in/in)
Median	$\tilde{R}_Q = 3.1$	$\tilde{R}_K = 0.86$	$\tilde{\mu}_{eff} = 4.1$	$\tilde{\Delta}_U = 0.043$
Variability	$\sigma_{RQ} = 0.10$	$\sigma_{RK} = 0.28$	$\sigma_{\mu,eff} = 0.26$	$\sigma_{\Delta U} = 0.17$

For the monotonic-load test data, the same approach is used to compute summary statistics. The only monotonic-load test summary statistic used in the acceptance criteria is the median ultimate deformation capacity value, $\tilde{\Delta}_{UM,PC}$, which was computed to be 0.062 in this example.

4.7 Evaluate Quality Ratings

4.7.1 Quality Rating of Test Data

The reference component test data were rated as Superior. This comes from a high rating for completeness and robustness and a high rating for confidence in the test results. The high rating for completeness and robustness was based on all of the important testing issues being addressed and the expectation that all important failure modes were uncovered in the testing. The high rating for confidence in the test results was based on the large number of test data, as well as multiple researchers and test laboratories being involved in completing the testing.

The proposed component test data were rated as Good. This rating comes from a medium rating for completeness and robustness and a high rating for confidence in the test results. The medium rating for completeness and robustness was based on the hypothetical panel component being made of a new type of material for which the material testing was deemed to be questionable by the peer review panel. The high confidence rating was based on the coherent and acceptably complete set of tested component configurations, as well as the fact that multiple researchers and labs were involved in completing the tests.

4.7.2 Quality Rating of Design Requirements

The reference component design requirements were rated as Good. This rating comes from a medium rating for completeness and robustness and a high rating for confidence in the design requirements. The medium rating for completeness and robustness was based on the fact that even though there is a great deal of experience with the design and construction of nailed wood shear panel components, they are still site-built, leaving more room for error that could result in unanticipated failure modes. The high confidence rating was based on the existence of a long history of use, and a good understanding of component behavior.

The proposed component design requirements were rated as Good. This comes from a high rating for completeness and robustness and a medium rating for confidence. The high rating for completeness and robustness was based on the proposed component being pre-fabricated in a controlled lab environment with careful quality control, having well-developed connection design requirements, and having clear construction requirements. The medium rating for confidence was based on substantial experience and mature construction practices for similar components, but also based on questionable testing practices presumed to exist for the new material used in the hypothetical proposed component.

4.8 Evaluate Component Equivalency

4.8.1 Overview

This section evaluates the possible equivalency between the proposed and reference components. Summary data were provided in Table 4-3 for the reference component and Table 4-10 for the proposed component.

4.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation

The median ultimate deformation of each proposed component performance group must satisfy Equation 2-1:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q$$

The median ultimate deformation from the reference component data set, $\tilde{\Delta}_{U,RC}$, is 0.035. The penalty factor for strength is based on the median proposed and reference component strength ratios, which is:

$$\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} = \frac{3.1}{2.7} = 1.15$$

Table 4-11 shows how the uncertainty penalty factor, $P_U = 1.05$, is retrieved from Table 2-3, based on the quality ratings discussed in Section 4.7. Similarly, Table 4-12 shows how the strength penalty factor, $P_Q = 1.00$, is retrieved from Table 2-4.

Table 4-11 Penalty Factor to Account for Uncertainty (from Table 2-3)

Penalty Factor for Uncertainty (P_U)			
Quality Rating of Proposed Component Test Data	Quality Rating of Proposed Component Design Requirements Relative to Reference Component Design Requirements		
	Higher	Same	Lower
Superior	0.95	1.00	1.15
Good	1.00	1.05	1.25
Fair	1.15	1.25	1.40

Table 4-12 Penalty Factor to Account for Difference in Component Strengths (from Table 2-4)

Penalty Factor for Differences in Strength (P_Q)			
$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q	$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q
0.50	1.88	1.10	1.00
0.60	1.55	1.20	1.00
0.70	1.31	1.30	1.04
0.80	1.14	1.40	1.09
0.90	1.00	1.50	1.13
1.00	1.00	1.80	1.24
1.10	1.00	2.00	1.32

Incorporating these values leads to the following check, showing that the median ultimate deformation value of 0.043 meets the criterion of Equation 2-1.

$$0.043 \geq (0.035)(1.05)(1.00) = 0.037 \qquad \underline{\underline{OK}}$$

Because $\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$ is less than 1.2, the exception of Section 2.8.2 does not need to be invoked.

In addition, the median ultimate deformation value of each geometric configuration, $\tilde{\Delta}_{Uj,PC}$, must also meet the requirement of Equation 2-2, as shown below. According to Table 4-3, the variability in ultimate deformation, $\sigma_{\Delta U,RC}$, is 0.16 for the reference component data.

$$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right)\cdot P_U \cdot P_Q$$

$$\tilde{\Delta}_{Uj,PC} \geq \left[1-1.5(0.16)\right](0.035)\cdot(1.05)(1.00) = 0.028$$

Table 4-13 summarizes the above acceptance criteria checks as applied to the full performance group and to each individual geometric configuration. The proposed component shear walls pass the basic ultimate deformation acceptance criterion for the full performance group (i.e., 0.043 versus 0.037), and also pass the individual configuration criteria for each of the panel configurations (i.e., the 0.028 criteria).

Table 4-13 Evaluation of Equivalency Acceptance Criteria

Test Index	Deformation Capacity		Acceptance Check	
	Δ_U	Median Δ_U And $\tilde{\Delta}_{Uj}$	Acceptance Criteria	Pass/ Fail
Perf. Group I:		$\tilde{\Delta}_U = 0.043$	**0.037**	**Pass**
1	0.058	0.051	0.028	Pass
2	0.056			
3	0.042	0.044	0.028	Pass
4	0.046			
5	0.061	0.057	0.028	Pass
6	0.058			
7	0.037	0.038	0.028	Pass
8	0.040			
9	0.048	0.050	0.028	Pass
10	0.052			
11	0.048	0.046	0.028	Pass
12	0.045			
13	0.047	0.042	0.028	Pass
14	0.043			
15	0.051	0.050	0.028	Pass
16	0.049			
17	0.036	0.035	0.028	Pass
18	0.035			
19	0.045	0.046	0.028	Pass
20	0.048			
21	0.042	0.035	0.028	Pass
22	0.028			
23	0.040			
24	0.045	0.042	0.028	Pass
25	0.041			
26	0.038	0.037	0.028	Pass
27	0.036			

4.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

To fulfill the requirements of Section 2.8.3 (Equation 2-3), design provisions for initial stiffness were roughly calibrated based on the observed stiffness values of the proposed and reference components. In this example, this was done by adjusting the e_n value for connector deformation in the design displacement equation. The other parameters of the equation, such as panel shear rigidity values, could have also been further calibrated, but the calculation below shows that the resulting ratio of 0.86 already falls within the range of 0.75 to 1.33 allowed by Equation 2-3. Therefore, the design stiffness provisions fulfill the Equation 2-3 acceptance criterion without need for further calibration.

$$0.75 \leq \frac{0.86}{1.00} \leq 1.33 \qquad \underline{\underline{OK}}$$

4.8.4 Requirements Based on Cyclic-Load Test Data: Effective Ductility Capacity

In order to fulfill the requirements of Section 2.8.4 (Equation 2-4) and to ensure approximate parity between the post-yield deformation capacities (and energy dissipation capacities) of the proposed and reference components, the median ductility capacity of the proposed components, $\tilde{\mu}_{eff,PC}$, must be at least half as large as the median ductility capacity of the reference components, $\tilde{\mu}_{eff,RC}$. In this example, $\tilde{\mu}_{eff,PC} = 4.1$ and $\tilde{\mu}_{eff,RC} =$ 6.3, so this requirement is fulfilled. Note that this difference in ductility capacity stems from the fact that the proposed components are systematically more flexible than the reference components.

4.8.5 Requirements Based on Monotonic-Load Test Data: Ultimate Deformation

The results of monotonic-load testing presented in Table 4-9 showed that the median ultimate monotonic deformation capacity for the proposed component, $\tilde{\Delta}_{UM,PC}$, is 0.062. The first step is to determine if the cyclic deformation capacities are large enough to use Equation 2-6 and avoid the need for monotonic-load testing of the proposed or reference components. Based on the number of cycles in the CUREE loading protocol, the $\tilde{D}_C$ value is 1.0 (see Section 2.8.5). The median ultimate cyclic deformation capacity for the reference component, $\tilde{\Delta}_{U,RC}$, is 0.035 (from Table 4-3). The calculation below shows that that $\tilde{\Delta}_{UM,PC}$ must be greater than 0.044:

$$\tilde{\Delta}_{UM,PC} \geq 1.2 \cdot (1.0) \cdot (0.035) \cdot (1.05) \cdot (1.00) = 0.044$$

Note that this requirement cannot be fulfilled by using the cyclic-load test data for the proposed component as a proxy for monotonic data (i.e., $\tilde{\Delta}_{U,PC}$ is 0.043, which is less than 0.044). This requirement, however, is fulfilled by the monotonic-load test results for the proposed component data , $\tilde{\Delta}_{UM,PC} =$ 0.062, which is greater than 0.44, so monotonic-load testing of the reference components is not necessary.

4.9 Summary of Example Component Equivalency Evaluation

Possible equivalency between hypothetical proposed component shear walls and reference component nailed wood shear walls was evaluated based on the requirements of the Component Methodology. This example showed that the hypothetical proposed component passed all of the acceptance criteria, as summarized below in Table 4-14, and that the proposed shear wall components would be considered equivalent to the reference nailed wood shear wall components according to the criteria of the Component Methodology.

Table 4-14 Summary of Acceptance Criteria Evaluation for Proposed Component Shear Walls

Acceptance Criteria		Equation Reference	Pass/Fail
Requirements Based on Cyclic-Load Test Data			
Ultimate Deformation Capacity (performance group)	$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} \cdot P_U \cdot P_Q$	2-1	Pass
Ultimate Deformation Capacity (individual configurations)	$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\,\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \; P_Q$	2-2	Pass All
Initial Stiffness Ratio	$0.75 \leq \dfrac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33$	2-3	Pass
Effective Ductility Capacity	$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC}$	2-4	Pass
Requirements Based on Monotonic-Load Test Data			
Ultimate Deformation Capacity (Option 1)	$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC} \; P_U \; P_Q$	2-5	Not Used
Ultimate Deformation Capacity (Option 2)	$\tilde{\Delta}_{UM,PC} \geq 1.2\,\tilde{D}_C \; \tilde{\Delta}_{U,RC} \; P_U \; P_Q$	2-6	Pass

Chapter 5

Conclusions and Recommendations

5.1 Introduction

The recommended Component Equivalency Methodology (Component Methodology) provides a rational basis for evaluating the seismic performance equivalency of new components that are proposed as substitutes for selected components in a currently approved seismic-force-resisting system. Proposed components found to be equivalent using the Component Methodology can be substituted for components of a reference seismic-force-resisting system (reference SFRS), but are subject to design requirements and seismic design category restrictions on the use of the reference system.

This chapter summarizes findings from supporting studies that provided the technical basis for the development of the Component Methodology, observations from test applications, and recommendations for possible further study.

5.2 Findings from Supporting Studies

In the development of the Component Methodology, key performance parameters were identified (Appendix A), cyclic-load and monotonic-load testing requirements were investigated (Appendix B), and probabilistic acceptance criteria were developed (Appendix C). This section summarizes major findings from the supporting studies conducted on these topics.

5.2.1 Key Performance Parameters

Studies identifying key component parameters that are important to system collapse performance are presented in Appendix A. Also included are the results of a literature search for related studies, which was used to identify an initial list of potential parameters. This list was systematically evaluated using two- and three-dimensional analytical models of archetype buildings. Importance was based on relative changes to the system-level probability of collapse related to changes in the value of the component parameters. System-level probability of collapse was evaluated using the analysis methods of FEMA-P-695. The resulting component parameters and their relative importance to system collapse behavior are summarized in Table 5-1.

Table 5-1 Relative Importance of Component Parameters

Importance	Parameter
Consistently of High Importance	• Inelastic deformation capacity • Yield strength (maximum load)
Consistently of Moderate Importance	• Cyclic deterioration capacity (which controls cyclic strength degradation) • Post-capping (post-maximum load) deformation capacity • Post-yield strength gain • Residual strength
Periodically of Moderate Importance	• Initial stiffness
Consistently of Low Importance	• Details of the initial stiffness branch • Hysteretic behavior (low, moderate, and high levels of pinching)

Based on these results, the Component Methodology included deformation capacity and strength as explicit measures for equivalency comparisons. Parameters identified to be consistently of moderate importance or periodically of high importance were considered indirectly, as discussed below.

Deformation Capacity

The primary acceptance criterion of the Component Methodology is the ultimate deformation capacity of the component, as determined by cyclic-load testing. This criterion is based on total deformation rather than inelastic deformation. However, since the elastic component of deformation is similar for the proposed and reference components (due to initial stiffness similarity requirements), the total deformations are fairly representative of the inelastic deformations. The total deformation is used to avoid having to define a yield deformation, because there is no clear yield deformation for many types of components.

Ultimate deformation capacity is defined as the deformation at which the component loses 20% of its strength. As such, the ultimate deformation metric reflects some degree of "post-capping" behavior (i.e., degradation after the peak strength has been achieved). While the effects of post-capping deformation are not fully accounted for at 20% strength loss, additional deformation levels (e.g., 50% strength loss) were not considered to be essential for equivalency evaluation, and were not included in the resulting Component Methodology to avoid unnecessary complexity.

Ultimate deformation capacity is based on cyclic-load testing for a required minimum number of cycles, and addresses, to some degree, cyclic deterioration capacity (strength degradation). That is, the ultimate deformation capacity decreases with cyclic loss of strength, which is roughly associated with the energy dissipated per cycle. While the effects of cyclic deterioration capacity are not fully accounted for by the ultimate deformation, studies indicate that additional comparisons of energy-related parameters would not appreciably change the results.

In addition, the Component Methodology also included acceptance criteria based on monotonic-load test data. Criteria based on monotonic-load testing were necessary because the ultimate deformation under cyclic-load testing mixes the effects of component deformation capacity and cyclic strength deterioration behavior. In contrast, monotonic-load test data show a component deformation capacity that is independent of cyclic loading history. It was decided that this measure of deformation capacity should be quantified as part of the equivalency comparisons.

Strength

The Component Methodology effectively requires the ratios of tested maximum strength and design maximum strength to be similar for proposed and reference components, and the acceptance criteria impose a penalty factor when the strength ratios differ. Maximum strength combines yield strength and post-yield strength gain parameters. Maximum strength was used rather than yield strength for simplicity because yield strength is ambiguous for many types of components. This avoided the need to provide a definition for the calculation of yield strength.

Although found to be of lesser importance to collapse, the Component Methodology included initial stiffness and effective ductility capacity in equivalency comparisons. Similarity of these parameters was considered necessary to avoid potential design issues related to: (1) drift-based design; (2) distribution of force demand based on relative component stiffness; and (3) potential horizontal and vertical stiffness irregularities. The requirement for similarity of initial stiffness also helps to ensure some parity between total and inelastic deformation so that it is not necessary to explicitly check both.

Absent from the list of equivalency parameters is an explicit check of hysteretic loop equivalency. Consistent with past findings in the literature, parametric studies found that pinching of the hysteresis loop (due to repeated cycles of load) was of little importance to collapse performance, unless pinching was extreme. This finding reflects the benefits of systems that stiffen (strengthen) at large amplitudes of cyclic deformation. These systems might exhibit different response characteristics and damage at lower amplitudes of cyclic deformation, but in general, have no significantly greater likelihood of reaching deformation demands associated with collapse.

An explicit check of hysteretic loop equivalency was not included in the Component Methodology for the following reasons. First, consistent with the FEMA P-695 Methodology, the Component Methodology evaluates equivalency solely in terms of collapse performance. Second, equivalency of other parameters, namely ultimate deformation capacity, initial stiffness and effective ductility capacity, is believed to preclude radical differences in component damage (i.e., at other levels of deformation). Finally, the Component Methodology is not intended for use in evaluating equivalency of components whose performance depends directly on hysteretic behavior. The applicability criteria specifically exclude the use of the Component Methodology for evaluating equivalency of isolator units or dampers whose design properties are based directly on hysteretic energy dissipation.

5.2.2 Cyclic-Load and Monotonic-Load Test Data Requirements

Requirements for cyclic-load and monotonic-load testing of components are described in Appendix B. Also included are background information on various types of cyclic-load testing protocols, a recommended method a for judging the equivalency of load histories, and a discussion of the importance of monotonic-load testing.

Component properties are highly dependent on the deformation history used to conduct cyclic-load tests. Deformation history affects the measurement of component ultimate deformation capacity, strength, and failure mode, so experimental investigations of proposed and reference components are required to be the same, or similar in terms of the normalized cumulative deformation (i.e., effective number of cycles to ultimate deformation capacity). The importance of the deformation history should not be underestimated, since the value of the ultimate deformation capacity can vary by a factor of two (or more) for cyclic-load testing with different deformation histories.

Monotonic-load test data are required in addition to cyclic-load test data because monotonic-load testing (single direction of push) provides collapse displacement patterns that are more representative of actual earthquake collapse response. This is in contrast to a symmetric-cyclic deformation history containing many cycles. When subjected to an earthquake large enough to cause collapse, story drift and component responses depend on failure mode, frequency characteristics, and duration of strong motion in the seismic input. Component responses might be close to symmetric-cyclic until severe deterioration occurs, at which point they can become one-sided and closer to monotonic than symmetric-cyclic.

In general, ultimate deformation measured from monotonic-load testing will be larger than that measured from cyclic-load testing. In some cases it can be as much as 1.5 to 2.0 times as large. Systems with components that do not exhibit this behavior are more prone to sidesway collapse. Thus, components with the same value of cyclic-load ultimate deformation may not always be equivalent in terms of collapse performance, so a comparison of reference and proposed component ultimate deformations from monotonic-load testing was included as part of equivalency comparisons.

5.2.3 Probabilistic Acceptance Criteria

Development of probabilistic-based acceptance criteria for evaluating component equivalency is described in Appendix C. The probabilistic underpinnings of the criteria are based on the collapse safety goals contained within the FEMA P-695 Methodology, explicitly considering related sources of uncertainty.

Median values of ultimate deformation are calculated using simple statistics for each performance group, where each performance group is assumed to represent a statistically homogeneous set of component configurations. Proposed components are deemed equivalent when the median value of ultimate deformation of the proposed component is equal to, or greater than, the factored median value of the reference component for the performance group of interest. In this sense, equivalency implies equal or better collapse performance.

Median values of the reference component are adjusted by penalty factors that account for uncertainties associated with design and data uncertainty, and potential differences in proposed and reference component strength characteristics. Penalty factors are set equal to 1.0 (i.e., no penalty) for proposed components with high quality data and strength characteristics similar to those of the reference component.

In cases where proposed and reference component medians are equal, acceptance criteria for components with high quality data and similar strength characteristics essentially provide 50% confidence that the median value of the ultimate deformation capacity of the proposed component is equal to, or greater than, that of the reference component. Subject to the assumptions in variability implied by the quality ratings, the penalty factors maintain this 50% confidence where the medians differ.

Although effectively more conservative than the acceptance criteria of other equivalency methods, such as AC322 *Acceptance Criteria for Prefabricated, Cold-Formed, Steel Lateral-Force-Resisting Vertical Assemblies* (ICC-ES, 2007), the statistical approach underlying the Component Methodology was considered to be an appropriate and rational approach for evaluating component equivalency that explicitly evaluates the probabilistic distribution and potential uncertainty of tested parameters. Penalty factors were included to encourage the use of high quality data and comparisons between components with similar strength characteristics. Use of lower quality data and components with appreciably different strength characteristics, however, is also permitted, and can result in equivalence for proposed components with median properties that are significantly better than the reference component of interest.

5.3 Findings of Test Applications

The Component Methodology was tested in applications evaluating equivalency for substitution of: (1) stapled wood shear wall components for nailed wood shear wall components; (2) buckling-restrained braces for special steel concentric braces; and (3) pre-fabricated shear wall products for use in wood light-frame construction. This section discusses the findings from each of these test applications.

5.3.1 General Findings

The Component Methodology was found to provide a practical and rational process for evaluating component equivalency.

Performance Groups

As a default, proposed and reference component data are lumped into a single performance group. Sorting of proposed and reference components into separate performance groups and associated test data sets is required, when necessary, to properly consider behavior associated with difference component configurations and failure modes. While grouping is somewhat subjective and, therefore, poses a challenge, it also permits proponents to separate test data into sets that provide a more appropriate basis for comparing proposed component data with reference component data.

Some flexibility is permitted in choosing how to group reference and proposed component data. In the case of proposed components, the trade off is between using a larger number of performance groups with inherently less scatter in the data, and the alternative of using fewer performance groups, which would presumably require less component testing.

Simple statistical calculations of median (and lognormal standard deviation) values are used as a practical approach for summarizing test data for proposed and reference components. From a statistical perspective, some data sets will have too few data points to properly compute medians and lognormal standard deviations with high levels of confidence, but this was considered an acceptable compromise considering the large expense involved in generating large statistically robust sets of test data for each performance group.

Available Test Data

Test data of reasonable quality are required for reference components and proposed components. It is expected that reference component data will be compiled mostly from past research and existing sources of published literature. In the case of wood light-frame shear wall components, substantial data was found to be available. These data were considered suitable for use with the Component Methodology, but similar data for other seismic-force-resisting systems listed in ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010) may not be as readily available.

Proposed component data are expected to be generated by the developer, who will have control over what data are produced. Much of these data will likely be produced during product development or through preliminary research studies. If not specifically developed for use in the Component Methodology, however, it is likely that such data will not meet the necessary requirements, and that supplementary testing will be necessary for the purpose of evaluating equivalency. One such example is monotonic-load testing, which is often not routinely performed, but is a requirement of the Component Methodology. Additional cyclic-load testing may also be needed to complete component performance groups.

Cyclic-Load Testing Protocol

The cyclic-load testing protocol was found to significantly affect component parameters determined from cyclic-load testing. This had a direct effect on equivalency comparisons. In general, the same cyclic-load testing protocol should be used in generating the proposed component and reference component test data to avoid biasing results.

Iterative Process for Component Equivalency

The use of the Component Methodology was found to be iterative. Possible iterative approaches included improving the number and quality of test data, refining design requirements, and restricting use of the proposed component to configurations that met the acceptance criteria. In many cases, such iterations served as a mechanism for improving the relative performance characteristics of the proposed component.

5.3.2 Specific Findings: Stapled-Wood Shear Wall Components

Overall, based on the limited set of stapled wood shear wall data used in the test application, it was found that stapled wood shear walls have higher component overstrength and slightly lower median deformation capacity than nailed wood shear walls. Application of the Component Methodology to these data sets did not find stapled wood shear walls to be equivalent to nailed wood shear walls.

In order for stapled wood shear walls to be found equivalent, changes would need to be made to the design requirements, or additional test data would need to be generated. Possible changes included improvements to the detailing requirements to increase the deformation capacity, modifications to the strength design requirements to make the normalized strength ratios similar between the stapled and nailed wood shear walls, or generation of more stapled wood shear wall data to improve the quality rating of the test data.

5.3.3 Specific Findings: Buckling-Restrained Brace Components

Overall, the test application comparing buckling-restrained braces to special steel concentric braces found that buckling-restrained braces have a larger median deformation capacity and easily satisfied the criteria of the Component Methodology. Although buckling-restrained braces have higher overstrength than conventional braces, connections are designed using a capacity design philosophy that satisfied Component Methodology requirements for strength. Requirements based on stiffness and effective ductility capacity were also satisfied. As a result, application of the Component Methodology found that buckling-restrained braces are equivalent to special steel concentric braces.

It should be noted, however, that available data for buckling-restrained braces and conventional braces lack monotonic-load test data, and did not include two component test specimens per brace configuration. This test application was based on the best data available at the time of this comparison and may not reflect the true variation in section and connection properties for steel braces. However, based on observed trends in available test results, it is expected that data from additional test specimens and monotonic-load testing would further confirm the above conclusions.

5.3.4 Specific Findings: Pre-Fabricated Wall Components

In this case, pre-fabricated wall products are relatively strong, short (slender) wall products intended for substitution in wood light-frame systems, representative of similar proprietary products from a number of manufacturers. The test application on pre-fabricated wall products, although limited to the evaluation of just three cyclic-load test specimens of a single configuration, provided useful information regarding the relative stiffness of proposed and reference components (specifically, the case in which the proposed component is inherently more flexible than the reference component).

Overall, the test application found that pre-fabricated wall components are equivalent to nailed wood shear wall components, subject to restrictions on indiscriminate substitution of the product within a system, due to significant differences in the relative stiffness of proposed and reference components. This finding is consistent with findings of the International Code Council Evaluation Services (ICC ES) for similar products using the acceptance criteria of AC322.

5.3.5 Specific Findings: Nailed Wood Shear Wall Reference Component Data Set

A robust and representative data set containing cyclic-load test results for nailed wood shear wall components was developed from information available in the literature. Line et al. (2008) compiled CUREE cyclic-load data from a total of 80 test specimens. This data set included results from 48 wall tests that were compiled as part of the International Code Council Evaluation Services Acceptance Criteria AC322 effort (ICC-ES, 2007), as well as results from 32 additional tests. To create the reference component data set for nailed wood shear walls, data were removed for walls with openings, stapled walls, and walls with box nails, leaving a total of 65 tests in the final data set.

The resulting reference component data set is considered to be a robust and representative data set for nailed wood shear wall configurations with aspect ratios ranging from 2:1 to 1:1, and tested using the CUREE loading protocol. Such a set is considered suitable for equivalency comparisons with appropriately similar components in applications of the Component Methodology.

5.4 Recommendations for Further Study

The following recommendations are provided for possible future studies that would help to further improve and refine equivalency methods or to broaden the potential applications of the Component Methodology.

5.4.1 Compilation of Available Reference System Benchmark Data

A potential hindrance to the use of the Component Methodology is a lack of sufficient quality test data for reference components (i.e., "benchmark" test data on components in current code-approved systems). A very real need exists to identify and compile component test data for as many systems in ASCE/SEI 7-10 as possible. This effort would need to address configuration issues, failure modes, and other characteristics, as necessary to develop appropriate performance groups and associated statistics.

The product of this effort would be median values of ultimate deformation and other parameters required for equivalency comparison using the Component Methodology. While this effort would likely generate benchmark test data for only a limited number of reference components, the contribution would still be very useful. Product developers would have access to reference component test data that they would otherwise be required to compile and process, and regulatory agencies would have confidence that these data were developed in an appropriate and unbiased manner.

5.4.2 Development of Additional Reference System Benchmark Data

It may be possible to expand the applicability of available test data (discussed above) to other systems recognizing possible trends in the relationship between the system R factor (and other related performance parameters) and the ultimate deformation capacity of reference components. Systems with larger values of R, in general, should have larger values of ultimate deformation capacity. Supplemented with appropriate FEMA P-695 analytical studies, the compiled sets of reference component test data could be used to develop a complete set of benchmark data for use as surrogate reference component test data.

As envisioned, this surrogate benchmark test data would define, for example, the required minimum value of ultimate deformation capacity of proposed components (which would necessarily be based on an assumption regarding the cyclic-deformation history used in testing) as a function of the R factor of the reference system of interest, and possibly the initial (elastic) stiffness of the component of interest. This could be done for a large number of current code-approved seismic-force-resisting systems, and would remove the need for a proponent to develop a set of reference component data when using the Component Methodology. This concept is similar to existing AISC seismic provisions for steel moment frame systems that specify testing requirements for beam-column connections and acceptable values of deformation capacity (e.g., 4% story drift). While ambitious, this project, if successful, would be a major contribution to the development and approval of new component products.

5.4.3 Development of Standard Cyclic-Load Testing Methods

There is very clear need for standard test methods, similar to ASTM E2126-09 (ASTM, 2009), for cyclic-load testing of components. A broader range of component applications and greater consideration of the testing protocol and its relationship to earthquake ground motions and component performance (e.g., collapse resistance) in terms of the appropriate number of damaging cycles of load are needed. Appendix B provides a starting point for such an effort.

5.4.4 Implications for Design Requirements Related to Overstrength

Development of the Component Methodology highlights areas where design standards could be improved to facilitate the safe adoption of alternative components. One such area is related to the issue of overstrength. Whereas typical building code provisions often rely on a single system overstrength factor, Ω_O, for the design of force-controlled components, this single factor is a crude measure of the forces that can be developed in force-controlled components.

In the development of the FEMA P-695 Methodology, system overstrength was shown to vary considerably for a given seismic-force-resisting system, depending on the configuration and proportions of the systems. Development of the Component Methodology has further highlighted situations where it may be necessary to revise design requirements for component connections, wall anchors, and other force-controlled components, where deformation-controlled replacement components have higher overstrength than the selected reference components.

The Component Methodology addresses this by placing an upper-limit on the relative overstrength in replacement and reference components, but this assumes that overstrength is accurately accounted for in the existing design provisions for the reference seismic-force-resisting system. It may still be possible in certain cases to have “mismatches” within the system design. Moreover, unintentional overstrength may occur and not be detected by existing design requirements for the reference component system of interest.

Appendix A
Identification of Component Parameters Important for Equivalency

A.1 Introduction

This appendix identifies key component parameters that are important contributors to system-level collapse performance and are therefore considered in the component equivalency evaluation.

To identify which component parameters tangibly affect system-level collapse performance, a representative component model was first developed. Related available research considering collapse behavior and non-collapse response was collected and reviewed to identify an initial list of potential parameters.

To build on the findings of previous research, and further identify which component parameters were most important to system-level collapse performance, a collapse sensitivity study was performed utilizing multiple-degree-of-freedom models of two-dimensional and three-dimensional wood light-frame buildings, and two-dimensional reinforced concrete moment frame structures.

A.2 Representative Component Behavior

This section presents the representative component model considered in the literature review discussion and sensitivity studies. This model is capable of capturing monotonic and cyclic behaviors of most types of common structural components.

Figure A-1 illustrates the monotonic behavior of the component model which is consistent with recent research, such as Vamvatsikos and Cornell (2002), Ibarra et al. (2005), Zareian (2006), Haselton et al. (2008), and with the model used in the FEMA P-440A report, *Effects of Strength and Stiffness Degradation on Seismic Response* (FEMA, 2009a). The primary difference between this model and similar models commonly used is the inclusion of bilinear initial stiffness, which allows better representation of wood shear wall behavior, and also allows testing the importance of these initial stiffness assumptions in the sensitivity study.

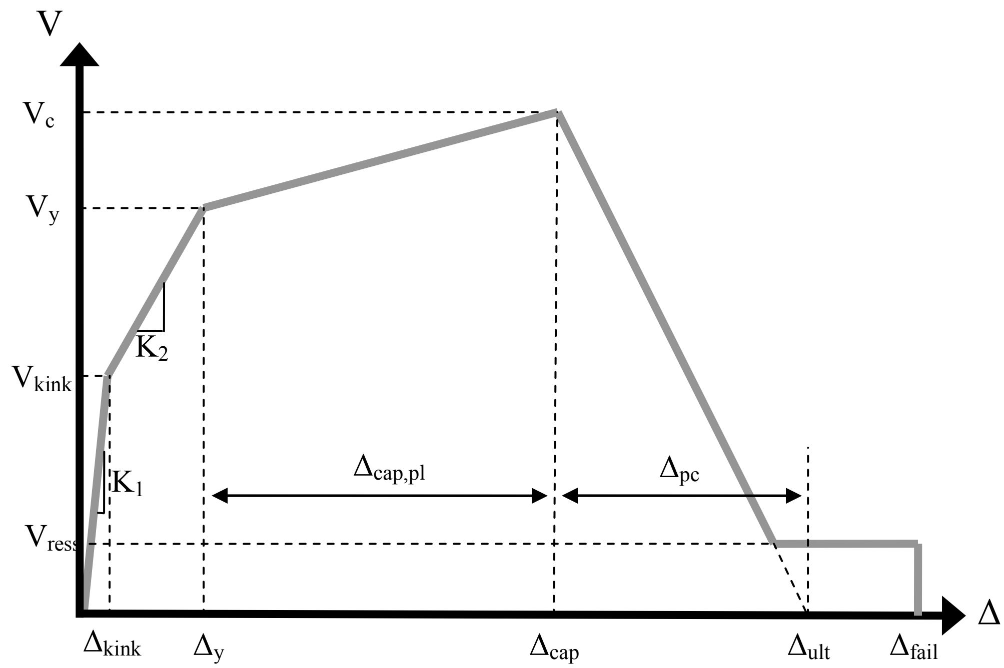


Figure A-1 Representative monotonic component behavior.

The following parameters are illustrated in Figure A-1 and studied in this appendix:

K_1 = initial stiffness of component a low levels of load

K_2 = reduced pre-yield stiffness of component a higher levels of load (due to cracking or other nonlinearities before primary yielding)

V_c = shear force at capping (maximum shear capacity of the component)

V_{kink} = shear at the transition between the initial stiffness (K_1) and the reduced pre-yield stiffness (K_2)

V_{res} = residual shear force capacity of component

V_y = yield shear force of component

Δ_{cap} = deformation of component at capping (at maximum force level)

$\Delta_{cap,pl}$ = deformation of component between yielding and capping

Δ_{fail} = deformation of component at failure (defined by zero strength)

Δ_{kink} = deformation of component and V_{kink}

Δ_{ult} = ultimate deformation of the component

Δ_y = deformation of component at yielding

Δ_{pc} = post-capping deformation of component

In addition to the above illustrated monotonic component behavior, the component model is also able to capture important cyclic behaviors such as strength deterioration and various cyclic unloading and reloading behaviors. This component model was developed by Ibarra (2003) and Ibarra et al. (2005). The specific component model option used in this sensitivity study is based on a peak-oriented reloading behavior, has cyclic strength deterioration that is based on the energy dissipated throughout the loading history as a ratio to the energy dissipation capacity of the component (Ibarra et al., 2005), and has a pinching response where the degree of pinching can be varied by the user. The level of strength deterioration, which is based on the energy dissipation capacity of the component, and the level of pinching are parameters that are varied in the later sensitivity study. Figure A-2 illustrates cyclic behaviors of the component model, for a few various components and loading histories. While all four figures illustrate cyclic strength deterioration, Figure A-2a illustrates a clear negative stiffness and strength loss within a single cycle of loading, and Figure A-2d illustrates pinching behavior.

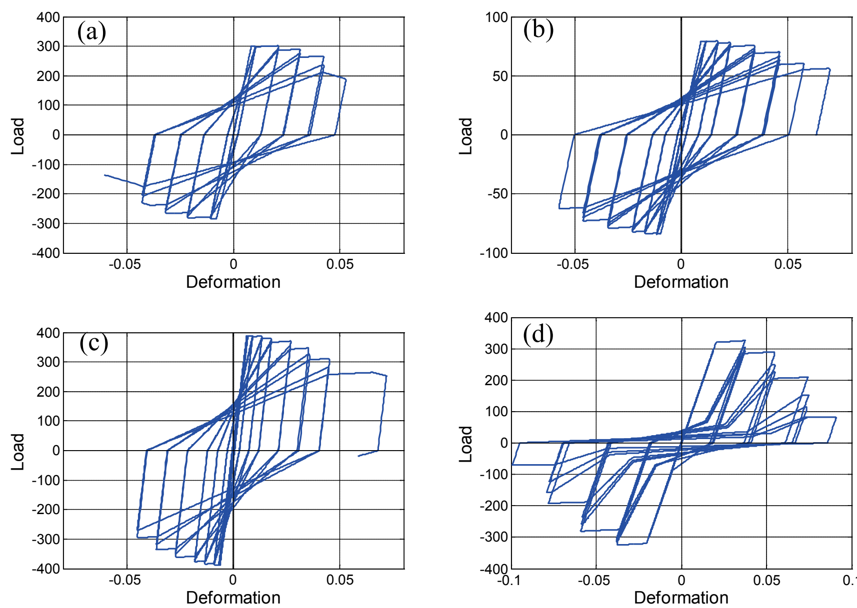


Figure A-2 Illustration of the cyclic behavior of the component model. Test data from the PEER Structural Performance Database (Berry et al., 2004) for test index numbers 8, 48, 154, and 212.

A.3 Literature Review

The expressed goal of "equivalency" is equal performance with respect to system-level collapse safety. The purpose of this section is to identify component-level parameters that tangibly affect the system-level collapse performance. Accordingly, this literature review is primarily based on analytical studies that are focused on collapse prediction; this is included in Section A.3.1. However, the number of such collapse sensitivity studies is limited, and there have been many other efforts that have been focused on displacement or ductility demands in structures rather than focusing specifically on collapse. Therefore, some of these non-collapse studies are also reviewed in Section A.3.2 for comparison to the findings of the collapse studies.

A.3.1 Collapse Studies

Table A-1 summarizes the results of the literature review for recent analytical studies that focused on collapse capacity prediction. The remainder of this section discusses the results of each study in more detail. Note that the number of such studies is limited because most research efforts are focused on prediction of other structural responses (such as displacement or ductility demands) rather than collapse response.

Table A-1 first outlines the nature of the study by identifying the materials considered in the study and the types of analytical models considered (single-degree-of-freedom, (SDOF) or multiple-degree-of-freedom (MDOF)). The remainder of the table ranks each component parameter with respect to the impact that each parameter has on the collapse prediction. The parameters are ranked as H for high importance (e.g., strength and inelastic deformation capacity), M for medium importance, or L for low importance. A blank field indicates that the component parameter was not considered in the study described by that row in the table.

Note that the ranking of importance as high, medium, or low is not always simple because some parameters are important in some cases and not in important in other cases (e.g., changes to the rate of cyclic deterioration are very important for rapid deterioration, but not important for slow deterioration). Even so, such qualitative rankings are important to help identify the importance of each parameter in a relatively simple manner that is tractable. To be more specific about the definitions of the importance levels, H is used when the parameter is important in most cases, M is used to describe two different situations (consistently of moderate importance, or sometimes highly important and sometime of low importance), and L is used for cases where the parameter is not important in most cases.

Table A-1 Summary of Component Parameter Studies in Literature with Focus on Collapse Capacity

	Study Info.		Component Parameters									
Study	Material(s)	SDOF or MDOF	Δ_y	V_y	V_c/V_y	$\Delta_{cap,pl}$	Δ_{pc} / $\Delta_{cap,pl}$	V_{res}/V_y or d_{fail}	Hysteretic Model (*B*, *P*, or *PO*)	Cyclic Strength Deterioration	Cyclic Stiffness Deterioration	Level of *P-Δ*
Vamvatsikos and Cornell (2002)	Gen	SDOF		H	H	H	H	H				
Ibarra et al. (2005): SDOF	Gen	SDOF		H		H	M		L	M-H	L-M	H
Ibarra et al. (2005): MDOF	Gen	MDOF		H		H	M		L	M	L-M	H
Zareian (2006)	Gen	MDOF		H		H	L			L	L	
Haselton and Deierlein (2007)	RC-F	MDOF	L	H	L	H	L			M		H
FEMA P-440A (2009a)	Gen	SDOF		H		H	H	H		L-M		
		Summary:	L	H	L-H	H	L-M	H	L	M	L-M	H

Hysteretic Model: B = bilinear, P = pinched, and PO = peak-oriented.
Importance: H is high importance, M is medium importance (or H/L), L = low importance, blank = not considered.
Materials: Gen = general, RC = concrete (-F = flexure).

The research by Ibarra, Medina, and Krawinkler is the most complete and rigorous study available that is specifically aimed at investigating the relationship between component parameters and system-level collapse performance. The study includes investigation of SDOF systems with periods from 0.1 to 4.0 seconds, as well as single bay MDOF frame systems from one to eighteen stories in height which have fundamental periods from 0.3 to 3.6 seconds.

The findings of their research that are most relevant to the Component Methodology are summarized as follows (note that the terminology and notation is adapted to be consistent with this current report):

- The component deformation capacity, $\Delta_{cap,pl}$, has a significant effect on the collapse capacity for both SDOF systems and MDOF frame systems.
- The slope of the post-capping branch, (Δ_{pc} / $\Delta_{cap,pl}$), has a significant effect on the collapse capacity when Δ_{pc} is small, but has a small effect when Δ_{pc} is large. This effect is reduced when P-delta effects are included, especially for ductile systems.

- The strength of the system is important to collapse capacity. This point is not explicitly addressed in the Ibarra et al. study, but rather is an implicit conclusion by how the collapse capacity was defined in his study.
- The effects of cyclic deterioration (specifically strength deterioration) have an important, but not dominant, effect on collapse capacity.
- The type of hysteretic model used (bilinear, peak-oriented, or pinched) has small impact on collapse capacity.
- P-delta effects have a large effect on collapse capacity, especially for flexible long-period systems that are ductile.

A.3.2 Non-Collapse Studies

To supplement the findings of the previous section, this section summarizes previous research that examines the effects of component modeling parameters on structural responses other than collapse, such as displacement response and ductility demands.

The FEMA P-440A report contains a comprehensive literature review on this topic, covering the past 40 years of research. This includes: Clough and Johnston (1966), Iwan (1973, 1977, 1978), Riddell and Newmark (1979a and 1979b), Mahin and Bertero (1981), Otani (1981), Bernal (1992, 1998), Rahnama and Krawinkler (1993), Foutch and Shi (1998), Gupta and Krawinkler (1998, 1999), Pincheira, Dotiwala and D'Souza (1999), Song and Pincheira (2000), Vian and Bruneau (2002, 2003), Miranda and Akkar (2003), Lee et al. (1999), Dolsek and Fajfar (2004), Medina and Krawinkler (2004), and Ruiz-Garcia and Miranda (2005).

Table A-2 summarizes the focus of each study listed in the above report. Similarly to Table A-1, this table outlines the nature of each study, such as materials, model type, and the focus of the study. These studies focus on strength demands (such as required *R* factor), displacement demands, or dynamic instability (which is included in one study, along with discussion of strength demands). The remainder of the table ranks each component parameter with respect to the impact that each parameter has on the collapse prediction.

Table A-2 Summary of Component Parameter Studies in Literature with a Focus Different than Collapse Capacity

Study	Study Information			Component Parameters								
	Material(s)	SDOF or MDOF	Response Parameter	Δ_y	V_y	V_c/V_y	$\Delta_{cap,pl}$ and/or Δ_{pc}	V_{res}/V_y or d_{fail}	Hysteretic Model (*B, P,* or *PO*)	Cyclic Strength Deterioration	Cyclic Stiffness Deterioration	Level of *P-Δ*
Lee et al. (1999)	Gen	SDOF	S						L	L	L	
Ruiz-Garcia and Miranda (2005)	Gen	SDOF	D			L-M				L-M	L-M	
Foutch and Shi (1998)	Gen	SDOF/MDOF	D				M		L			
Dolsek and Fajfar (2004)	RC	SDOF	S				M-H					
Pincheira, Dotiwala, D'Souza (1999)	RC	SDOF	D							H	H	
Song, Pincheira (2000)	Gen	SDOF	D			H				M	M	
Bernal (1992, 1998)	Gen	SDOF/MDOF	S									H
Miranda and Akkar (2003)	Gen	SDOF	S/C	M			H					
Vian and Bruneau (2002, 2003)	Gen	SDOF	C									H
Clough and Johnston (1966)	Gen	SDOF	D							L	L	
Otani (1981)	Gen	SDOF	D						L			
Iwan (1973, 1977, 1978)	Gen	SDOF	S						L	L	L	
Riddell and Newmark (1979a and 1979b)	Gen	SDOF	S								L	
Mahin and Bertero (1981)	Gen	SDOF	D								L-M	
Rahnama and Krawinkler (1993)	Gen	SDOF	S							L	L	
Gupta and Krawinkler (1998, 1999)	Gen	SDOF	D				H		M			
Medina and Krawinkler (2004)	Gen	MDOF	D								M-H	
Summary:				M	--	M	M-H	--	L	M	M	H

Hysteretic Model: B = bilinear, P = pinched, and PO = peak-oriented.
Materials: Gen = general, RC = concrete.
Response parameter of interest: S = lateral strength demand, D = displacement, C = lateral strength associated with dynamic instability.
Importance: H = high importance in most cases, M = medium importance (or H/L), L= low importance, blank = not considered.

The parameters are ranked as H for high importance, M for medium importance, or L for low importance. A blank field indicates that the component parameter was not considered in the study described by that row in the table.

In addition to the rough descriptions of importance that are summarized in Table A-2, several general conclusions from the above research are summarized as follows:

- The hysteretic model is not highly important, meaning that the final structural response parameters are not sensitive to the exact features of the load-deformation path, as shown in Lee et al. (1999), Foutch and Shi (1998), Clough and Johnston (1966), Otani (1981), Iwan (1973, 1977, 1978), Riddell and Newmark (1979a and 1979b), and Mahin and Bertero (1981).
- To be more specific regarding the hysteretic model, pinching characteristics do not have a large influence on global response, as shown in Gupta and Krawinkler (1998 and 1999) and Lee et al. (1999).
- The rules for cyclic deterioration of strength and stiffness are not highly important in all cases. However, in certain specific cases, cyclic deterioration of strength and stiffness are important. For example, they are important for short-period structures, for structures on soft soils and for relatively weak structures, as shown in Ruiz-Garcia and Miranda (2005), Foutch and Shi (1998), Song and Pincheira (2000), and Mahin and Bertero (1981). Medina and Krawinkler (2004) suggest that stiffness degradation may also be somewhat more important for MDOF structures.
- In general, response is not sensitive to post-yield hardening stiffness (V_c/V_y), though short-period structures are more sensitive to this parameter than long period structures. Even so, within a reasonable range of post-yield stiffness, it is still only moderately important for short-period structures, as shown in Ruiz-Garcia and Miranda (2005).
- There is general agreement that capping behavior and the post-capping slope can have a significant influence on structural response. Post-capping slope seems to be more significant if the residual strength of the system is low, as shown in Dolsek and Fajfar (2004), Miranda and Akkar (2003), Rahnama and Krawinkler (1993) and Gupta and Krawinkler (1998, 1999).

A.4 Wood Light-Frame Building Collapse Sensitivity Studies

This section presents a sensitivity study that employs eight wood light-frame buildings, ranging from one to five stories in height. The purpose of this sensitivity study is threefold:

- Supplement past research by considering component parameters that have not been thoroughly investigated (e.g., K_1 / K_2 and V_c / V_y in Figure A-1).
- Supplement past research by considering the possibility of mixing-and-matching proposed components with reference components. Accordingly, this sensitivity study looks both at full replacement (where all parameters of all components are varied simultaneously) and mixing-and-matching (where parameters of some components are varied, while parameters of other components are held at their baseline values).
- Create a set of sensitivity data that will be utilized later when developing quantitative Component Methodology acceptance criteria (see Appendix C).

First, the eight wood light-frame buildings used for sensitivity analyses are described, and the component parameter values that are assumed as the baseline of the sensitivity study are documented. Then the results of the full-replacement sensitivity study are presented, followed by the two-dimensional mixing-and-matching sensitivity study, and the three-dimensional mixing-and-matching sensitivity study. Results from all wood light-frame sensitivity studies are summarized.

A.4.1 Building Models and Baseline Component Parameter Values

This sensitivity study is based on eight of the wood light-frame buildings that were utilized in the FEMA P-695 project[1] (FEMA, 2009b). All eight buildings were designed for a high seismic site (SDC D_{max}), ranging from one to five stories in height, with story heights of 10 feet for all buildings and evaluated using the Far-Field record set provided in FEMA P-695. Table A-3 summarizes the designs showing properties such as height, fundamental period, design base shear, (V / W), and maximum considered earthquake (MCE) spectral acceleration demand for the site used for design, S_{MT}. The first subset of buildings range in height from one to three stories, and utilize

[1] For this study, preliminary models were used. Models used in the final FEMA P-695 report differ slightly.

low aspect ratio walls. The second subset of buildings range in height from one to five stories, and utilize high aspect ratio walls.

Table A-3 Wood Light-Frame Structural Design Properties (after FEMA P-695 Table 9-20)

Model No.	No. of Stories	Building Configuration	Wall Aspect Ratio	Period *T* (sec)	*V/W*	S_{MT} (g)
SDC Dmax – Low Aspect Ratio Walls						
1	1	Commercial	Low	0.25	0.167	1.50
5	2	Commercial	Low	0.25	0.167	1.50
9	3	Commercial	Low	0.36	0.167	1.50
SDC Dmax – High Aspect Ratio Walls						
2	1	1&2 Family	High	0.25	0.167	1.50
6	2	1&2 Family	High	0.26	0.167	1.50
10	3	Multi-Family	High	0.36	0.167	1.50
13	4	Multi-Family	High	0.45	0.167	1.50
15	5	Multi-Family	High	0.53	0.167	1.50

In the original FEMA P-695 study, the wood light-frame models were created using the SAWS software program (Seismic Analysis of Woodframe Structures) developed within the CUREE-Caltech Woodframe Project (Folz and Filiatrault, 2004a, 2004b). For the purposes of this current sensitivity study, the general quadralinear model described in Section A.2.2 is calibrated to the SAWS model, an example of which is shown in Figure A-3. Using such a fit for each story, the OpenSees computer platform (OpenSees, 2009) is then used to create the MDOF model of the building. The structural model is then simplified slightly for purposes of the sensitivity study. The differences in strength and yield displacement are maintained between the different stories of the building, but all other parameters, such as story deformation capacity, are made uniform over the height of the building.

The MDOF OpenSees model consists of a single shear spring per story (describing the shear force versus interstory displacement behavior), masses and weights lumped at each floor level, and a leaning column element to account for P-delta effects. Information regarding building masses and loading can be found in Appendix C of FEMA P-695.

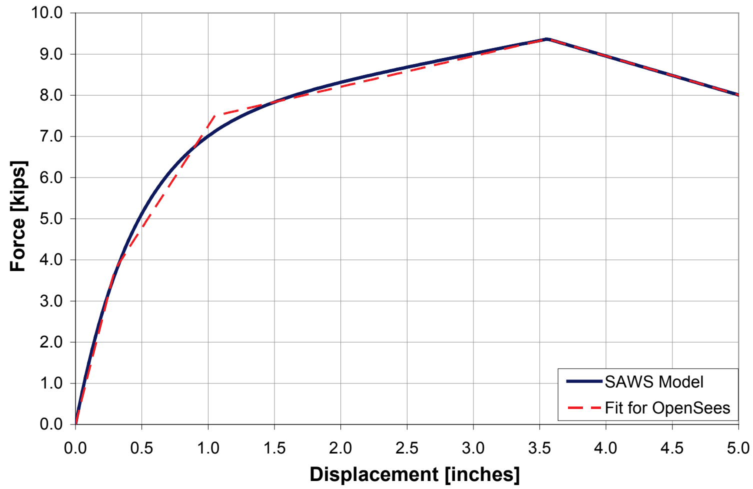


Figure A-3 Example fitting of backbone curve to SAWS model, for the one-story high aspect ratio building No. 2.

Table A-4 documents the baseline calibrated values of V_y and Δ_y for each of the eight wood light-frame buildings. Table A-5 documents baseline and ranges of values used in the sensitivity study. Note that when the V_y and Δ_y values are varied in the sensitivity study, a multiplication factor is applied to all elements in the building (except for the mix-and-match cases, where only a portion of the building is modified).

Table A-4 Baseline Component Strength and Stiffness Properties for Wood Light-Frame Building Models

Model No.	No. of Stories	Building Configuration	Baseline Strength and Stiffness Component Properties									
			Story 1		Story 2		Story 3		Story 4		Story 5	
			V_y [k]	Δ_y/120"	V_y [k]	Δ_y/120"	V_y [k]	Δ_y/120"	V_y [k]	Δ_y/120"	V_y [k]	Δ_y/120"
Low Aspect Ratio Walls												
1	1	Commercial	7.4	0.0086	--	--	--	--	--	--	--	--
5	2	Commercial	30.2	0.0093	14.5	0.0086	--	--	--	--	--	--
9	3	Commercial	53.5	0.0090	50.0	0.0098	18.0	0.0078	--	--	--	--
High Aspect Ratio Walls												
2	1	1&2 Family	7.5	0.0088	--	--	--	--	--	--	--	--
6	2	1&2 Family	17.5	0.0095	9.3	0.0086	--	--	--	--	--	--
10	3	Multi-Family	31.8	0.0096	29.9	0.0108	11.2	0.0086	--	--	--	--
13	4	Multi-Family	36.0	0.0096	36.0	0.0096	34.0	0.0108	12.0	0.0083	--	--
15	5	Multi-Family	40.0	0.0089	40.0	0.0089	38.2	0.0102	26.5	0.0096	13.6	0.0081

Table A-5 Values of Component Parameters for Wood Light-Frame Building Models

Component Parameter	Baseline Parameter Value	Range of Values Considered in Sensitivity Study
V_y / $V_{y,\mathrm{baseline}}$	1.00	0.3 - 3.0
Δ_y / $\Delta_{y,\mathrm{baseline}}$	1.00	0.3 - 3.0
K_1/K_2	2.50	1.0 - 5.0
V_{kink}/V_c	0.40	0.2 - 0.8
V_c / V_y	1.25	1.01 - 1.50
$\Delta_{cap,pl}$ / story height	0.025	0.005 - 0.10
Δ_{pc} / $\Delta_{cap,pl}$	4.00	1.0 - 10.0
V_{res} / V_y	0.001	0.001 - 0.50
Δ_{fail}	Infinite	n/a
Cyclic Deterioration Capacity (λ')	Infinite	5.0 - 100.0
Level of Pinching	Moderate(2)	None(1) - High(3)

A.4.2 Sensitivity Study Results for Three-Story Building: Full Replacement

This section presents the results of a sensitivity study looking at fully replacing all of the structural components in a three-story structural model (building No. 10 in FEMA P-695). Figure A-4 shows the results for the two parameters shown to be highly important to collapse response, namely strength, V_y, and plastic deformation capacity, $\Delta_{cap,pl}$. Consistent with use of the FEMA P-695 Methodology, these figures present the collapse margin ratio (CMR), which is the ratio of the median collapse capacity and the MCE demand, as well as the adjusted collapse margin ratio (ACMR) which also accounts for the effects of spectral shape. The ACMR value is computed using the requirements of the FEMA P-695 Methodology, with the exception that β_1 is computed directly from regression analyses of the collapse results rather than using Equation B-3 of FEMA P-695. This is done to avoid completing a pushover analysis at every step of the sensitivity study. This simplification can cause the ACMR line to be slightly more "bumpy" in some cases. Note that the ACMR value is used when assessing collapse performance.

The changes in strength, V_y, are shown to be consistently important for all levels of strength. The changes to plastic deformation capacity, $\Delta_{cap,pl}$, are also shown to be important for all values, but are shown to be especially important for low levels of deformation capacity (for $\Delta_{cap,pl}$ / story height <

0.03). These findings are consistent with previous research (Ibarra and Krawinkler 2005, Haselton and Deierlein 2007).

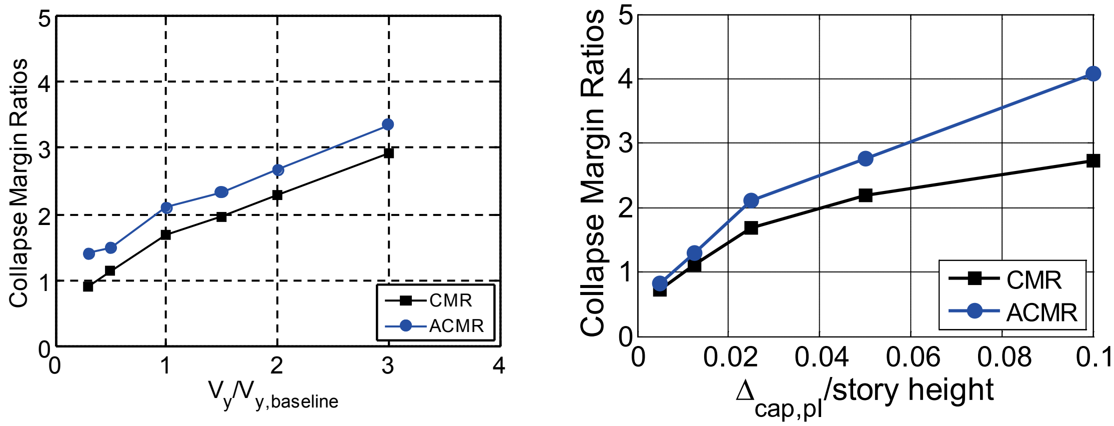


Figure A-4 Component parameters that are important for collapse response of three-story building.

Figure A-5 shows the results for four parameters shown to be moderately important to collapse response, namely post-capping deformation capacity, (Δ_{pc} / $\Delta_{cap,pl}$), the cyclic deterioration capacity, λ', the level of pinching, and the post-yield strength gain, (V_c / V_y). The post-capping deformation capacity is moderately important for all parameter values. In contrast, changes to λ' are unimportant when λ' is large (and does not dominate collapse response) but are highly important when λ' is small (and the cyclic deterioration behavior dominates the collapse response). This observation is consistent with findings of previous work (FEMA P-440A, Haselton and Deierlein, 2007, and Ibarra, 2003). The level of pinching only becomes important when the response is highly pinched (pinching level #3), but this level of pinching is a relatively extreme case. This unimportance of pinching, when predicting collapse capacity, is consistent with the observations from the literature review (Ibarra et al., 2005). The level of post-yield strength gain is only slightly important when the ratio of (V_c / V_y) is near unity.

Figure A-6 shows the results for the remaining four parameters that are not important to the collapse response of this building. These unimportant parameters are the initial stiffness, Δ_Y, the related details of the initial stiffness branch (V_{kink} / V_c and K_1 / K_2), and the residual strength.

The unimportance of residual strength seemingly disagrees with the previous findings of FEMA P-440A and work by Vamvatsikos and Cornell (2002), but this is not the case entirely. This wood light-frame building (and the other seven used in this sensitivity study) is relatively ductile, with ($\Delta_{cap,pl}$ / story height) = 0.025, and (Δ_{pc} / $\Delta_{cap,pl}$) = 4.0. This results in a case where the P-delta effect (even though small) is likely large enough to cause collapse before the residual strength is reached.

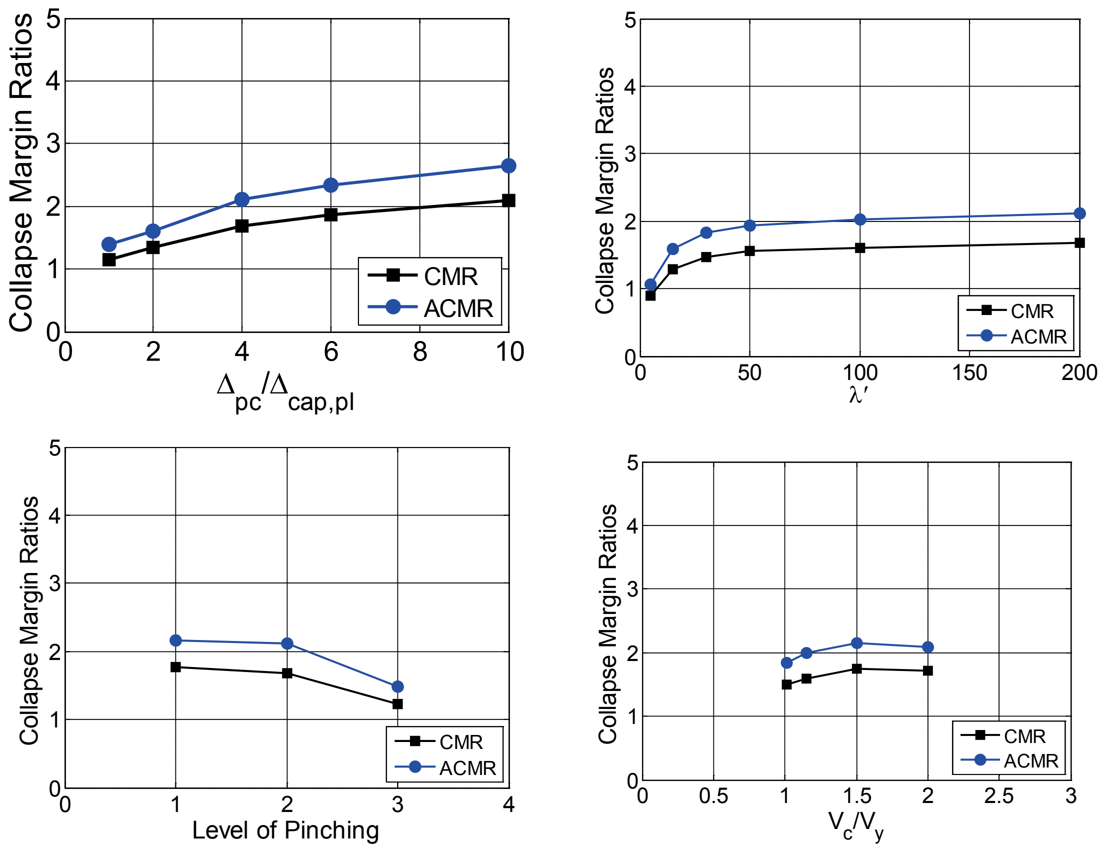


Figure A-5 Component parameters that are moderately important for collapse response of three-story building.

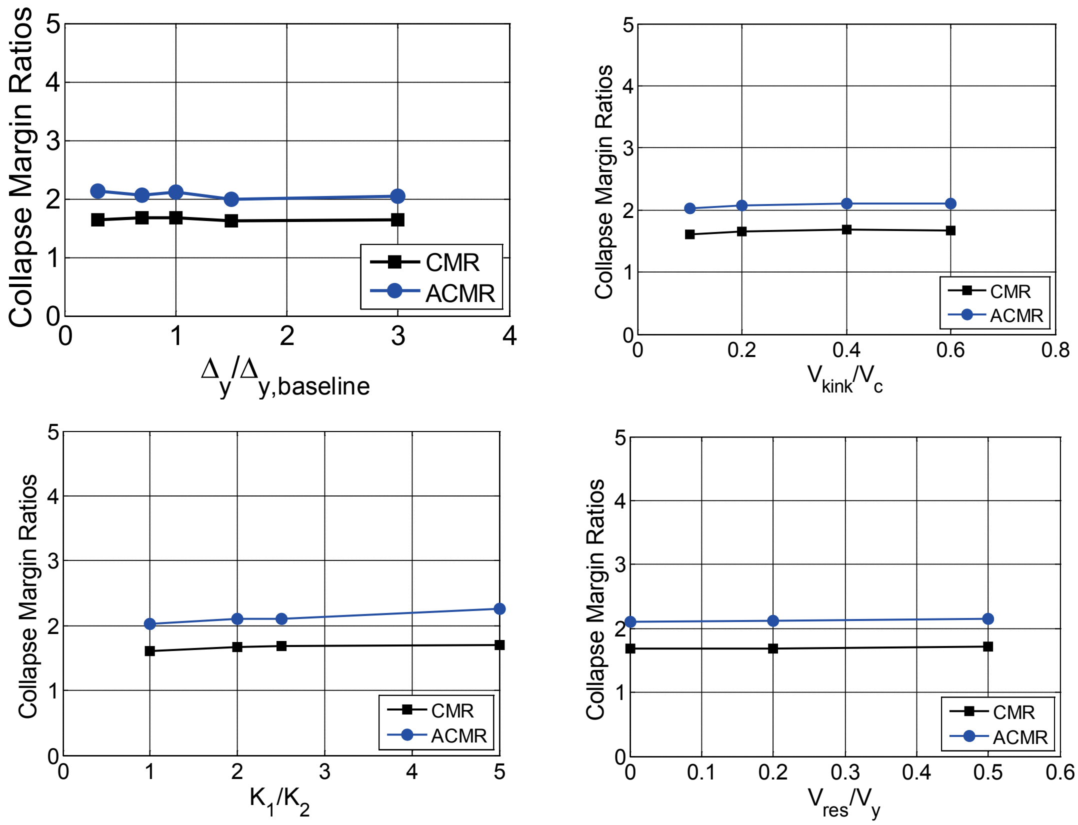


Figure A-6 Component parameters that are not important the collapse response of three-story building.

In summary, the following component properties were found to be highly important to the collapse response of the three-story building studied: (1) yield strength, V_y; and (2) inelastic deformation capacity, $\Delta_{cap,pl}$. In addition, the following parameters were found to be only moderately important: (1) post-capping deformation capacity, (Δ_{pc} / $\Delta_{cap,pl}$); (2) cyclic deterioration capacity, λ'; (3) hysteretic behavior (low, moderate, and high levels of pinching); and (4) post-yield strength gain, (V_c / V_y). The following parameters were not important to the collapse response: (1) initial stiffness, Δ_y; (2) details of the initial stiffness branch (V_{kink} / V_c and K_1 / K_2); and (3) residual strength (V_{res} / V_y).

A.4.3 Sensitivity Study Results for Three-Story Planar Model: Mixing-and-Matching Over the Height of Building

This section investigates the effects of the same component parameters, but looks at the case of mixing-and-matching where some structural components are modified and some components are left to have their baseline properties. This section uses a two-dimensional structural model to investigate the effects of mixing-and-matching over the height of the building.

This mixing-and-matching sensitivity study is important because some seemingly "good" changes to component properties may in fact cause detrimental changes in system-level collapse safety. For example, increased strength is typically thought of as beneficial, but if the strengths of only the upper stories are increased, the damage will localize more in the first story causing a reduction in the system-level deformation capacity and a reduction in the resulting collapse capacity. In the development of the Component Methodology, this concept is important because it will likely lead to an upper bound on the expected strength of the proposed component.

Figure A-7 shows a schematic of the various cases that are considered for the three-story building mixing-and-matching sensitivity study. The figure on the left represents the situation previously investigated in Section A.4.2, and the remaining figures represent the two situations considered in this section.

For the three-story building, the largest difference between mixing-and-matching and full replacement comes with changes to component strength. This observation is expected, because strength discontinuities over height can result in more damage localization, and can alter the collapse mechanism. Figure A-8 shows the results of modifying component strength for the mix-and-match case of replacing only the first story walls. This shows that decreasing the first-story strength decreases collapse capacity, which is expected. These results also show that increasing the strength by 50% causes a 15% decrease in the collapse capacity, due to the damage localizing more over the height of the building.

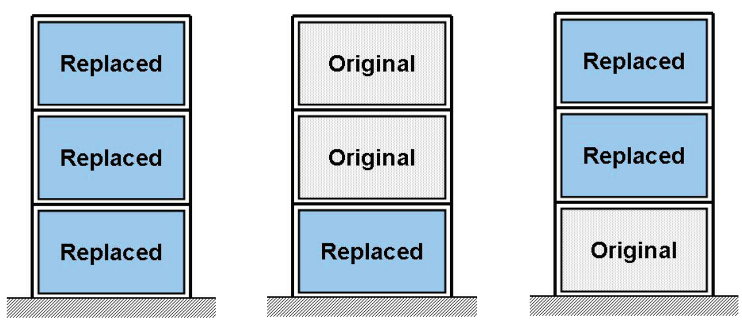


Figure A-7 Schematic diagram of various cases considered for the sensitivity study. Possible mixing-and-matching over the height of the building was considered.

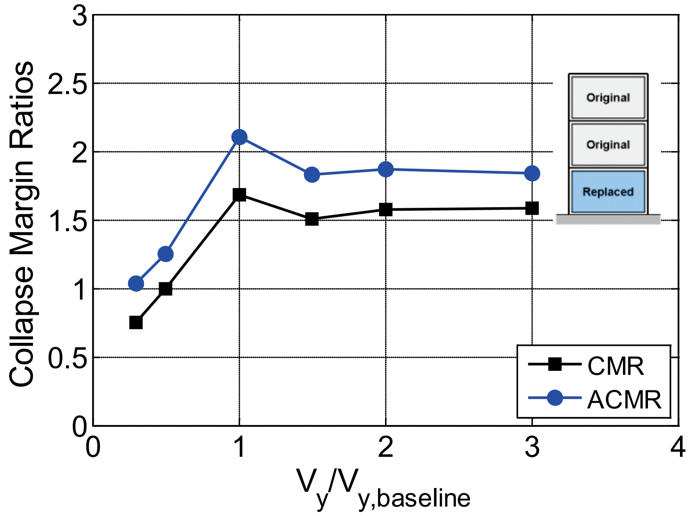


Figure A-8 Effects of strength on collapse capacity for the mix-and-match case of only story one being replaced.

Figure A-9 shows the results for replacing the upper stories of the building. Similarly to Figure A-8, this shows that both increases and decreases in upper story strength result in a decreased collapse capacity. This occurs because an increase in upper story strength exacerbates the damage concentration in the first story, and causes the building to have a lower system-level deformation capacity. Alternatively, a decrease in upper-story strength changes the collapse mechanism from the first story to the weakened upper-stories, thereby also decreasing the collapse capacity.

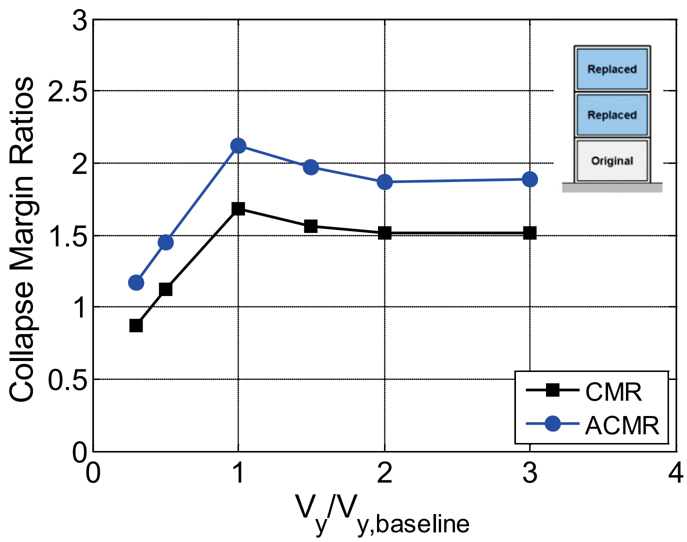


Figure A-9 Effects of strength on collapse capacity of three-story building for the mix-and-match case of stories two and three being replaced.

For more complete documentation, Figure A-10 provides the sensitivity study results for the case where the first story walls are replaced. Similarly, Figure A-11 provides the sensitivity study results for the case where the upper story walls (stories 2-3) are replaced.

These figures show the effects of plastic deformation capacity, $\Delta_{cap,pl}$. Trends are similar between the case of full replacement and the mix-and-match cases where only the first story is modified. In the case where the plastic deformation capacity of upper stories is modified, this only affects the response when a decreased deformation capacity causes the collapse mechanism to more fully involve the upper stories.

The effect of post-capping deformation capacity, (Δ_{pc} / $\Delta_{cap,pl}$), is generally consistent for the full replacement case as well as both of the mix-and-match cases. The one exception is that an increased post-capping deformation capacity in the upper stories does not affect the collapse response.

The remainder of the results shown in Figure A-10 and Figure A-11 are relatively consistent between the full replacement and the two mixing-and-matching cases.

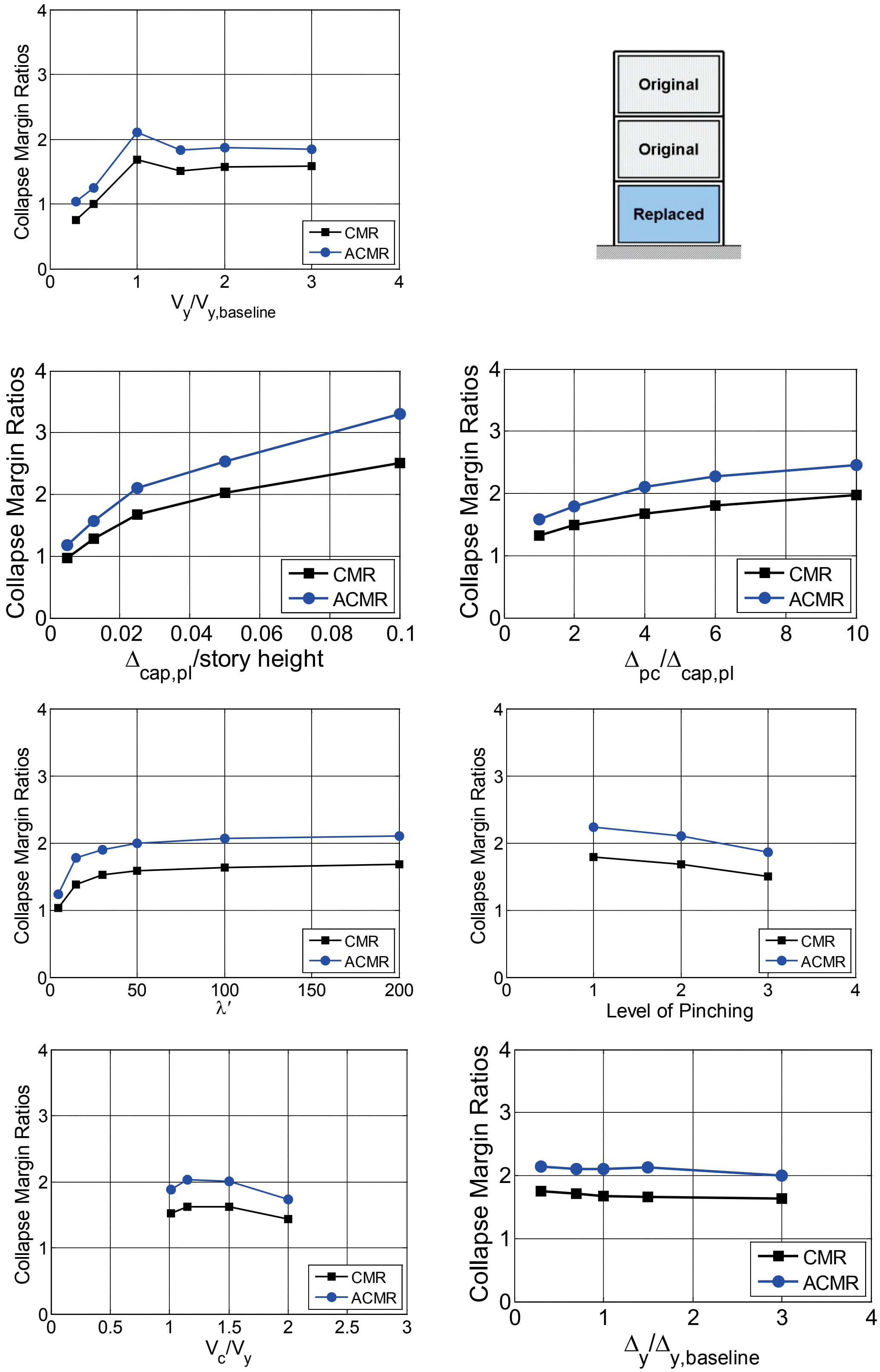


Figure A-10 Sensitivity study results for three-story building for the mix-and-match case with the bottom story walls replaced.

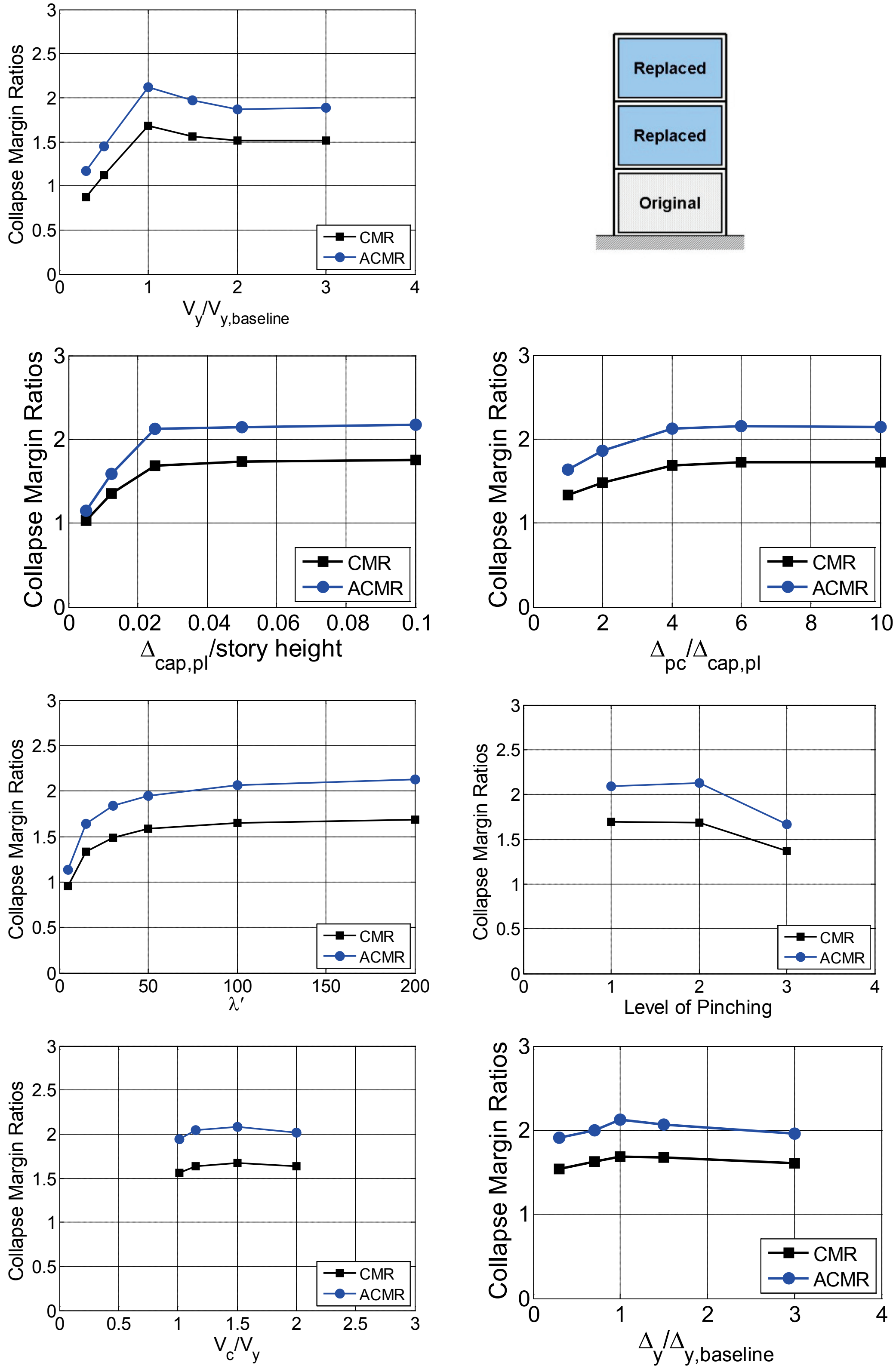


Figure A-11 Sensitivity study results for three-story building, for the mix-and-match case with the upper story (stories 2-3) walls replaced.

A.4.4 *Sensitivity Study Results for Three-Story Three-Dimensional Model: Mixing-and-Matching of Walls in Plan and Over Height*

In order to investigate possible mixing-and-matching cases that may occur in a building in more depth, this section utilizes a three-dimensional structural model to investigate other mixing-and-matching configurations. Figure A-12 shows three additional mixing-and-matching cases considered for the three-story building. There are three variations where walls are replaced (marked with "R") on the South and East sides of the building. The North and West walls are left with their original properties. The replacement is done either: (1) over the full height of the building; (2) only for the first story of the building; or (3) only for the upper stories (stories 2-3) of the building.

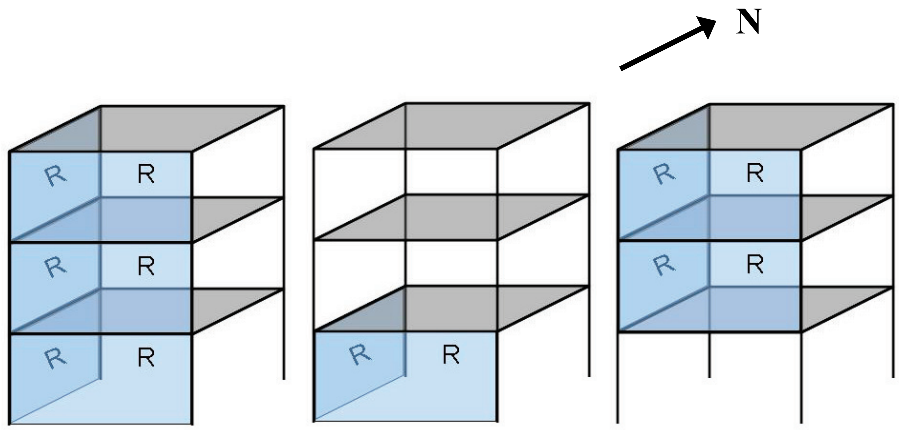


Figure A-12 Schematic diagram of the various mix-and-match cases considered in the sensitivity study. Mixing-and-matching both in plan and over the height of the building are considered.

Figure A-13 shows the sensitivity study results for the case where walls are replaced over the full height, Figure A-14 shows the results for the case where the walls are replaced only in the first story, and Figure A-15 shows the results for the case where the walls are replaced in only the upper stories. These results are similar to the results of the last section, which were based on mixing-and-matching only over the building height.

The primary additional finding of this section is that the mixing-and-matching within a single story of the building (i.e., in the plan view) is suggested to be less detrimental to collapse safety as compared with mixing-and-matching over the building height. This is based on the observation from Figure A-13 which shows that an increased strength is beneficial to the collapse capacity even though the proposed and reference components were mixed-and-matched. For the cases of mixing-and-matching over the building height (Section A.4.3), there was no case where increased strength led to increased collapse capacity. However, this is an observation based on only a single building, and additional study would be needed to confirm this for additional cases, in order to generalize this observation.

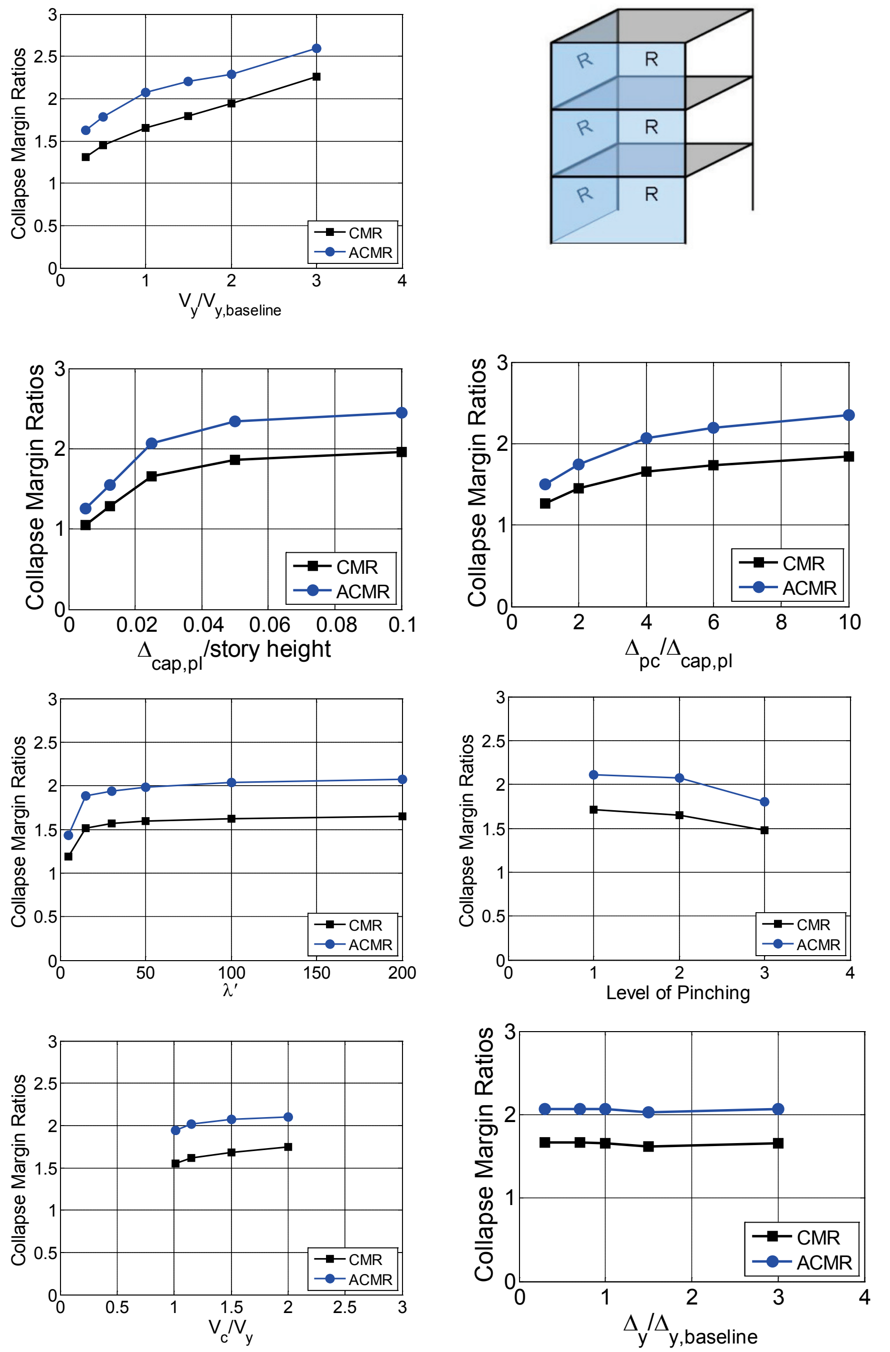


Figure A-13 Sensitivity study results for mix-and-match case where walls are only replaced on the South and West sides of the building.

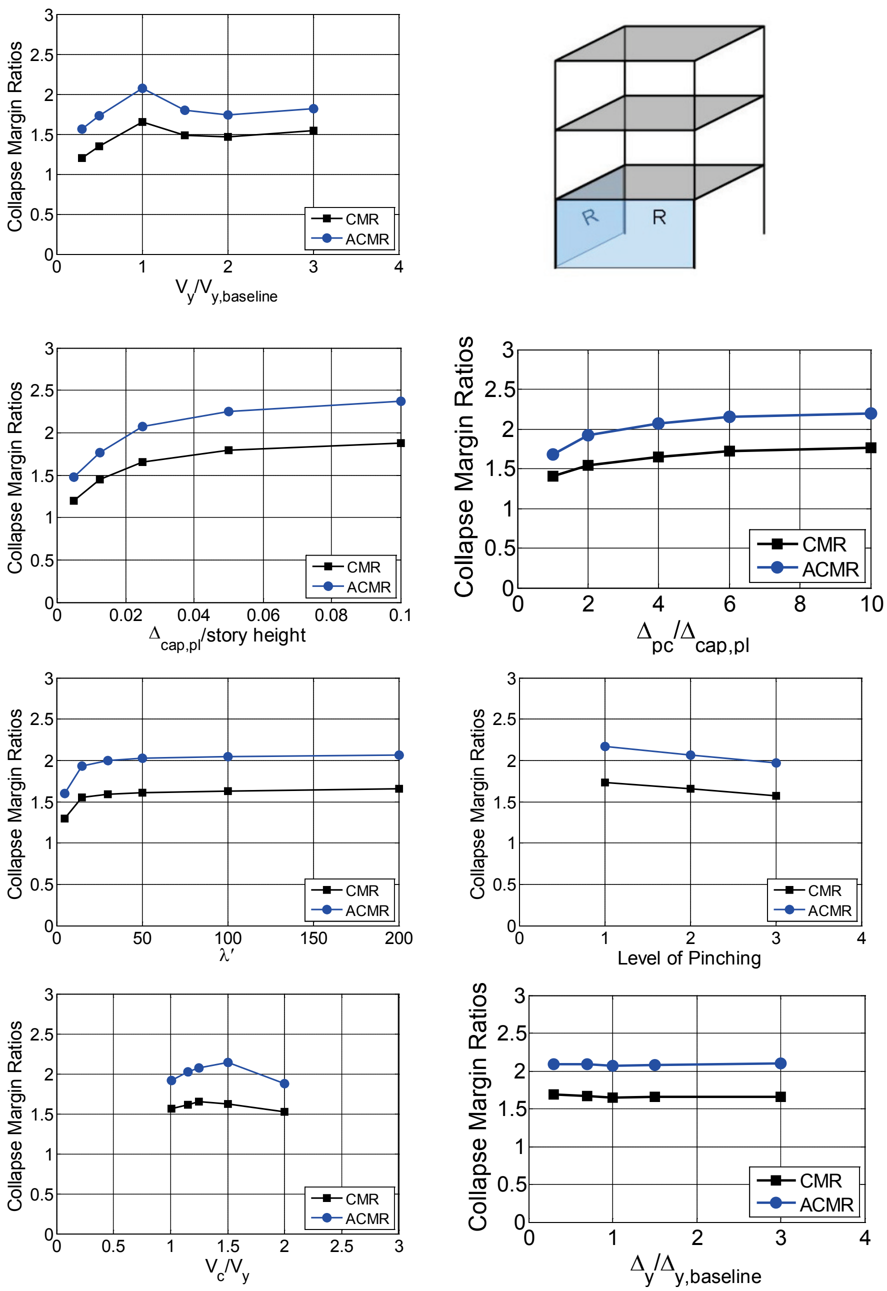


Figure A-14 Sensitivity study results for mix-and-match case where walls are only replaced on the first story of the South and West sides of the building.

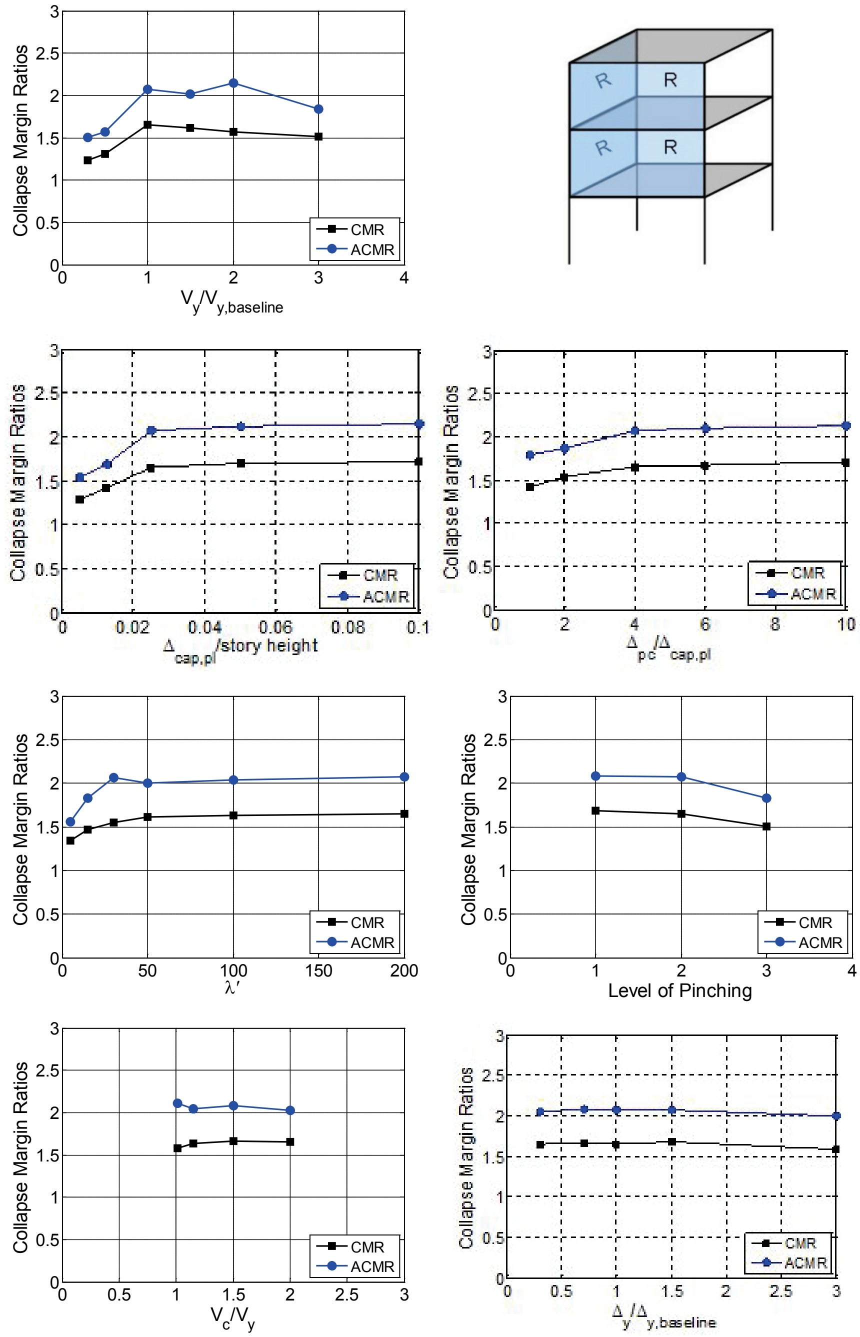


Figure A-15 Sensitivity study results for mix-and-match case where walls are only replaced on the second and third stories of the South and West sides of the building.

A.4.5 Summary of Parameter Importance for Wood Light-Frame Buildings

Table A-6 summarizes the importance of component properties as observed in the results of the sensitivity studies. These results for the full set of eight buildings confirm earlier observations made based on the study of one building.

Table A-6 Summary of the Level of Importance of Component Parameters for Eight Wood Light-Frame Buildings

Wood Light-Frame Building Model	Walls Modified In Sensitivity Study	Component Parameters: V_y	d_y	K_1/K_2	V_{kink}/V_c	V_c/V_y	$\Delta_{cap,pl}$	Δ_{pc} / $\Delta_{cap,pl}$	V_{res}/V_y	Cyclic Deterioration Capacity (I')	Level of Pinching
Full Replacement Sensitivity Study											
1-story, low aspect ratio (No. 1)	All	L-M	L	L	L	L	H	H	L-M	M	L
2-story, low aspect ratio (No. 5)	All	H	L-M	L	L	L	H	M	L	M	L-M
3-story, low aspect ratio (No. 9)	All	H	L-M	L	L	L-M	H	M	L	M	L-M
1-story, high aspect ratio (No. 2)	All	H	L	L	L	L-M	H	H	M	M	L-M
2-story, high aspect ratio (No. 6)	All	H	L	L	L	L-M	H	H	L	M	L-M
3-story, high aspect ratio (No. 10)	All	H	L	L	L	M	H	M	L	M	L-M
4-story, high aspect ratio (No. 13)	All	H	L-M	L	L	L	H	M	L	M	L-M
5-story, high aspect ratio (No. 15)	All	H	M	L	L	M	H	M	L	M	L
Mix-and-Match Sensitivity Study: 3-Story No. 10											
3-story, high aspect ratio (No. 10)	All stories, all sides	H	L	L	L	M	H	M	L	M	L-M
3-story, high aspect ratio (No. 10)	Story 1, all sides	M-H	L	L	L	L-M	H	M	L	M	M
3-story, high aspect ratio (No. 10)	Stories 2-3, all sides	M	L	L	L	L-M	M	M	L	M	L-M
3-story, high aspect ratio (No. 10)	All stories, two sides	H	L	L	L	L	H	M	L	M	L-M
3-story, high aspect ratio (No. 10)	Story 1, two sides	M	L	L	L	L-M	M	L-M	L	L-M	L
3-story, high aspect ratio (No. 10)	Stories 2-3, two sides	M	L	L	L	L	M	L-M	L	M	L
Mix-and-Match Sensitivity Study: 5-Story No. 15											
5-story, high aspect ratio (No. 15)	All stories, all sides	H	M	L	L	M	H	M	L	M	L
5-story, high aspect ratio (No. 15)	Story 1, all sides	M-H	L	L	L	M	H	M	L	M	L-M
5-story, high aspect ratio (No. 15)	Stories 1-2, all sides	M-H	L-M	L	L	M	H	M	L	M	L
5-story, high aspect ratio (No. 15)	Stories 2-5, all sides	L-M	M	L	L	L-M	M-H	L	L	L-M	L
5-story, high aspect ratio (No. 15)	Stories 3-5, all sides	M	L-M	L	L	L	M	L	L	M	L
Overall Summary											
Summary for all Buildings	--	H	L	L	L	L-M	H	M	L	M	L-M

Legend: H means highly important in most cases, M is medium importance (or H is some cases and L in some cases), L is low importance in most cases.

As summarized in Table A-6, the wood light-frame sensitivity study identified the following parameters to be consistently of high importance to collapse response: (1) inelastic deformation capacity, $\Delta_{cap,pl}$; and (2) yield strength, V_y. In addition, the following parameters were found to be consistently of moderate importance, or periodically of high importance: (1) cyclic deterioration capacity, λ'; (2) post-capping deformation capacity, (Δ_{pc} / $\Delta_{cap,pl}$); and (3) post-yield strength gain (V_c / V_y).

A.5 Reinforced Concrete Special Moment Frame Collapse Sensitivity Study

In order to generalize previous findings, this section presents a sensitivity study for a reinforced concrete special moment frame system. Table A-7 summarizes the design properties for the six special moment frame buildings used in this sensitivity study. This table shows that the selected set of buildings includes heights ranging from two to twenty stories, fundamental periods from 0.45 to 3.36 seconds, and includes both space- and perimeter-frames. The table also includes the design base shear coefficient (V / W) and the maximum considered earthquake (MCE) spectral acceleration demand for the site (S_{MT}). These building designs and models were developed as a part of the development of the FEMA P-695 Methodology and were evaluated using the Far-Field record set provided in FEMA P-695. All six buildings were designed for a high seismic site (SDC D_{max} of FEMA P-695).

Table A-7 Reinforced Concrete Special Moment Frame Structural Design Properties

Model No.	No. of Stories	Building Configuration	Bay Width	Period T (sec)	V/W	S_{MT} (g)
2064	2	Perimeter	20'	0.45	0.125	1.50
1003	4	Perimeter	20'	0.81	0.092	1.11
1010	4	Space	30'	1.03	0.092	1.03
5013	12	Perimeter	20'	2.13	0.035	0.42
2009	12	Space	20'	2.13	0.035	0.42
5020	20	Perimeter	20'	3.36	0.022	0.27

Table A-8 shows the component parameters and parameter value ranges typically used in the sensitivity study, though some of the buildings were tested for more extreme parameter values beyond these ranges (as will be shown in Figure A-16). The differences between the moment frame and wood light-frame shear wall sensitivity studies are as follows:

- The moment frame study focuses solely on the full replacement case (no mixing-and-matching considered).

- The force and deformation parameters are moment and element rotation, rather than shear force and lateral drift, where

  $\theta_{cap,pl}$ = deformation of component, defined in terms of rotation, between yielding and capping

  θ_{pc} = post-capping deformation of component, defined in terms rotation

- The moment frame study does not investigate the details of the component behavior prior to yielding, because the wood light-frame study showed them to be clearly unimportant.
- The cyclic deterioration (energy dissipation) capacity, λ, is defined slightly differently, as defined by $\lambda = \lambda' \times \theta_{cap,pl} / \theta_y$.
- The initial stiffness term is defined as a stiffness parameter, K_1, in terms of moment and rotation, rather than being defined as a yield deformation.

Table A-8 Values of Component Parameters for the Reinforced Concrete Moment Frame Models

Component Parameter	Baseline Parameter Value	Range of Values Considered in Sensitivity Study
$V_y / V_{y,baseline}$	1.00	1.0 - 4.0
$K_1 / K_{1,baseline}$	1.00	0.5 - 2.0
K_1 / K_2	1.00	--
V_{kink}/V_c	--	--
M_c / M_y	1.20	1.01 - 1.50
$\theta_{cap,pl}$	0.050	0.015 - 0.10
$\theta_{pc} / \theta_{cap,pl}$	1.50	0.5 - 5.0
V_{res}/V_y	0.00	--
Δ_{fail}	Infinite	--
λ	90.0	20 - Infinite
Level of Pinching	None	--

Figure A-16 provides sample sensitivity study results for the four-story building (ID1003), and Table A-9 summarizes the results for all moment frame buildings considered in this study. These findings are similar to those of the wood light-frame study, with the following differences:

- Both studies identify the strength and deformation capacity as the two most important parameters. However, changes to strength have greater impact for moment frame buildings and changes to deformation capacity have greater impact for shear wall buildings. This is discussed more quantitatively in Appendix C.

- The post-capping deformation capacity is less important for moment frame buildings.
- The cyclic deterioration capacity is more important for moment frame buildings.

Part of the reason that changes to deformation capacity have less impact for moment frame buildings is that when the deformation capacities are large, the P-delta effects drive the collapse response. This causes the benefits of increased deformation capacity to saturate. This saturation effect can even be seen for the four-story building results shown in Figure A-16. Additionally, this saturation effect becomes more predominant for taller buildings, causing increased deformation capacity to have less benefit to the collapse capacity; this is shown in the results of Table A-9.

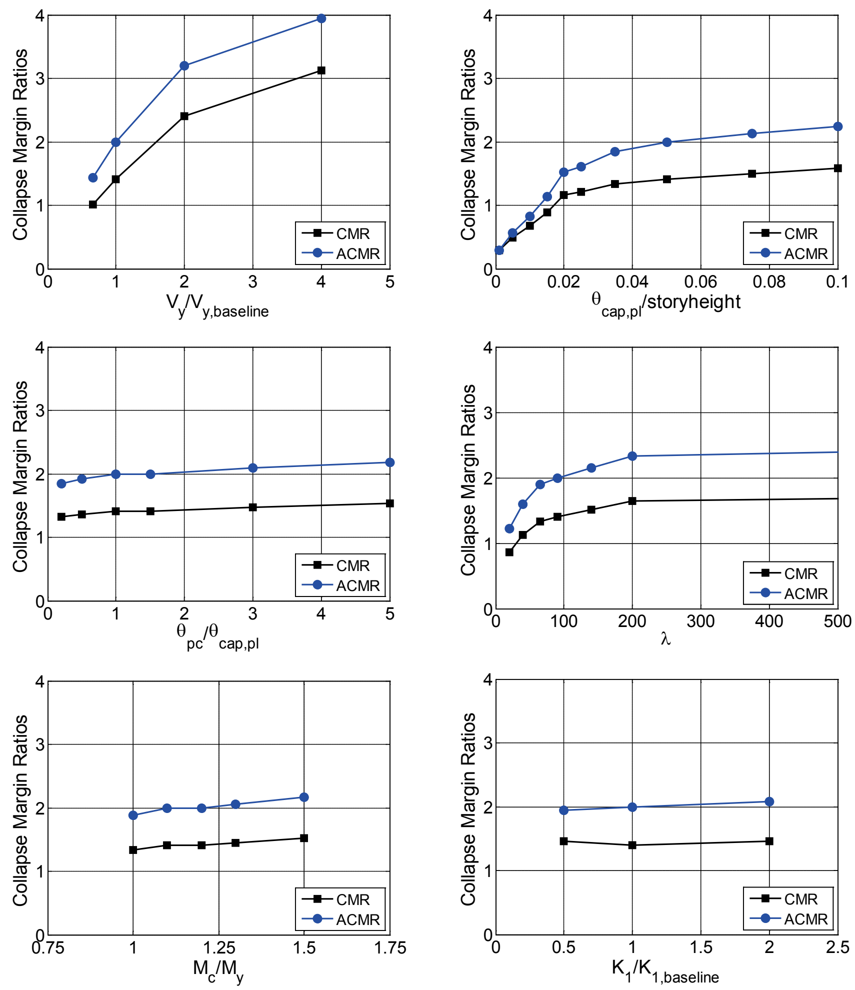


Figure A-16 Sensitivity study results for the 4-story building ID1003, for the case of full replacement.

A.6 Summary of Key Component Parameters

Based on the observations from the literature review, and the analytical collapse sensitivity studies, the following parameters are classified as highly important: (1) inelastic deformation capacity, $\Delta_{cap,pl}$ and $\theta_{cap,pl}$; and (2) yield strength, V_y. In addition, the following parameters were found to be consistently of moderate importance or periodically of high importance: (1) cyclic deterioration capacity, λ'; (2) post-capping deformation capacity, (Δ_{pc} / $\Delta_{cap,pl}$) and (θ_{pc} / $\theta_{cap,pl}$); (3) post-yield strength gain, (V_c / V_y); (4) residual strength, (V_{res} / V_y). Initial stiffness, Δ_y, was periodically of moderate importance. The following parameters were consistently of low or no importance: (1) details of the initial stiffness branch, (V_{kink} / V_c and K_1 / K_2); and (2) hysteretic behavior (low, moderate, and high levels of pinching).

Table A-9 Summary of the Level of Importance of Component Parameters for Six Reinforced Concrete Special Moment Frame Buildings

		Component Parameters									
Reinforced Concrete Moment-Frame Building Model	Elements Modified In Sensitivity Study	V_y	Initial Stiffness (K_1)	K_1 / K_2	V_{kink}/V_c	M_c/M_y	$\theta_{cap,pl}$	θ_{pc} / $\theta_{cap,pl}$	V_{res}/V_y	Cyclic Deterioration Capacity (l)	Level of Pinching
Full Replacement Sensitivity Study											
2-story, perimeter frame	All	H	L	--	--	L-M	H	M	--	H	--
4-story, space frame	All	M	L	--	--	L-M	H	M	--	H	--
4-story, perimeter frame	All	H	L	--	--	L-M	H	L-M	--	H	--
12-story, space frame	All	H	L	--	--	L-M	H	L-M	--	H	--
12-story, perimeter frame	All	H	L	--	--	L-M	M	L	--	M	--
20-story, perimeter frame	All	H	L-M	--	--	L-M	L-M	L	--	M-H	--
Overall Summary											
Summary for all Buildings	--	H	L	--	--	L-M	M-H	L-M	--	M-H	--

Legend: H means highly important in most cases, M is medium importance (or H is some cases and L in some cases), L is low importance in most cases.

The component parameters in the first two categories (highly or moderately important) are those that should be considered for inclusion in the Component Methodology. The following list summarizes the final acceptance criteria checks of the Component Methodology, and how the criteria are designed to target these important component parameters:

- **Deformation Capacity.** The primary acceptance criterion of the Component Methodology (Equation 2-1) is based on the ultimate deformation capacity of the component from cyclic-load testing. This ultimate deformation capacity is defined as the deformation at which the component loses 20% strength. This criterion is based on total deformation rather than inelastic deformation, but the elastic component of deformation should be similar for the proposed and reference components (based on the Equation 2-4 criterion), so these two approaches are judged to be roughly equivalent. The total deformation is used to avoid trying to define a yield deformation, because there is no clear yield deformation for many types of components.

  The post-capping deformation capacity is not accounted for in the Component Methodology. The use of additional deformation levels were considered (such as the deformation at 50% strength loss), but were intentionally excluded from the final acceptance criteria of the Component Methodology, in order to reduce the complexity of the Methodology. The fact that the primary acceptance criterion (Equation 2-1) is based on the deformation at 20% strength loss, means that the post-capping deformation capacity is somewhat, but far from fully, accounted for.

  In addition to the primary acceptance criterion based on cyclic-load test data (Equation 2-1), Section 2.8.4 also includes acceptance criteria based on monotonic-load test data. This is done because the ultimate deformation under cyclic-load testing mixes the effects of the component deformation capacity and the cyclic strength deterioration behavior. In contrast, the monotonic-load test data clearly show the component deformation capacity, without mixing in any effects of cyclic strength deterioration; the monotonic-load ultimate deformation is a clearer measure of the deformation capacity that should be quantified for purposes of demonstrating collapse equivalency. This is more fully discussed in Appendix B.

- **Strength.** The primary acceptance criterion of the Component Methodology (Equation 2-1) includes a strength penalty factor, P_Q, which is based on the maximum strength from cyclic-load testing (called Q_M in Chapter 2 and called V_C or M_C in this Appendix). This approach is based on the maximum strength alone, rather than also trying to quantify some measure of a yielding strength. This was done intentionally, for purposes of simplicity and to avoid trying to define the yield strength in a generic manner, because there is no clear yield force for many types of components. The weakness of this approach is that it does not differentiate between components with low yield strength and large post-yield strength gain, versus high yield strength and small post-yield strength gain. Even so, it is judged that the simplicity of using the maximum strength alone outweighs this potential weakness.

- **Cyclic Deterioration Capacity.** The acceptance criteria of the Component Methodology include ultimate deformation criteria under *cyclic*-load testing (Equation 2-1); this approach partially accounts for the cyclic deterioration capacity, λ' or λ, of the component because a component with low cyclic deterioration capacity will have smaller ultimate deformation when subjected to cyclic loading. To supplement this, the Component Methodology also imposes a requirement on the effective ductility capacity of the component (Equation 2-4). The sensitivity studies showed that small or moderate changes in the energy dissipation capacity of a component were often not critical, but that large changes could lead to meaningful reductions in the collapse capacity of the seismic-force-resisting system. Accordingly, the Equation 2-4 criterion is liberal and is set up to catch only these possibly extreme cases in the reduction of energy dissipation capacity, related to a large (more than 50%) reduction in post-yield deformation capacity.
- **Initial Stiffness.** The initial stiffness is also included in the acceptance criteria of the Component Methodology (Equation 2-3) but this was done for reasons separate from the discussion of this Appendix. This is discussed in detail in Section 3.8.3.
- **Residual Strength.** The residual strength is not considered in the Component Methodology, primarily because the residual strength is difficult to reliably quantify from test data.

Appendix B
Development of Requirements for Cyclic-Load and Monotonic-Load Testing

B.1 Introduction

This appendix describes the requirements for cyclic-load and monotonic-load testing of components for use in the Component Methodology. Since estimates of component strength and deformation capacity are highly dependent on the loading history used to conduct the tests, it is required that loading histories used in the experimental investigations for the proposed and reference components be similar. This appendix provides a method that may be used to show quantitatively that the cyclic-loading histories chosen meet the requirements of Section 2.2.2.

This appendix also describes why monotonic-load test results are also required for acceptance criteria. Requirements for monotonic-load testing ensure that the proposed and reference components are compared with a set of experimental results that are not dependent on the loading history. In addition, monotonic-load testing provides a lower bound on the number of cycles the building may experience as it nears sidesway collapse, which may be more representative for some cases than testing with many cycles.

B.2 Cyclic-Load Test Data Considerations

This section provides guidelines for the selection and comparison of cyclic-loading histories for use in testing proposed and reference component specimens. The guidelines ensure that the loading history used to test the proposed component is as damaging as that used in the reference component tests, such that component parameters can be directly compared in acceptance criteria.

It is anticipated that the proposed component loading protocols will be selected after reference component data have been assembled. In many cases, most or all reference component data will be obtained from past tests and the loading histories used in generating reference component parameters are known. These loading histories will vary and may or may not be based on an established loading protocol. Proposed and reference component tests performed as part of the equivalency evaluation should employ loading protocols in accordance with the criteria set forth in this appendix. The choice of loading protocols utilizes information about the past reference component tests and, if needed, preliminary tests of the proposed component.

Procedures in this section show how to determine what loading histories may be used for experimental investigations of the proposed and reference components to satisfy the requirements of Section 2.2.2. Section B.2.4 considers the case where all proposed and reference component tests are conducted using the same loading history and certain other conditions are met; in this case, loading history requirements may be assumed to be automatically satisfied. Section B.2.5 illustrates an evaluation of acceptability for different cyclic-loading histories.

Any alternative method used to demonstrate that cyclic-loading histories fulfill the requirements of Section 2.2.2 should be quantitative and approved by the peer review panel.

B.2.1 Importance of Cyclic-Loading History

It is well documented that the cyclic-loading history applied in a structural component test can impact the results and conclusions of experimental investigations. Specifically, the applied loading history has been shown to influence estimates of the following component parameters:

Component Deformation Capacity. In general, the larger the number of inelastic cycles, the smaller the deformation capacity that will be measured during the experiment. This observation has been confirmed for a variety of different materials and subassemblage systems in PEER/ATC-72-1, (PEER/ATC, 2010), Krawinkler (2009) and Gatto and Uang (2002). The concept is illustrated in Figure B-1 and Figure B-2.

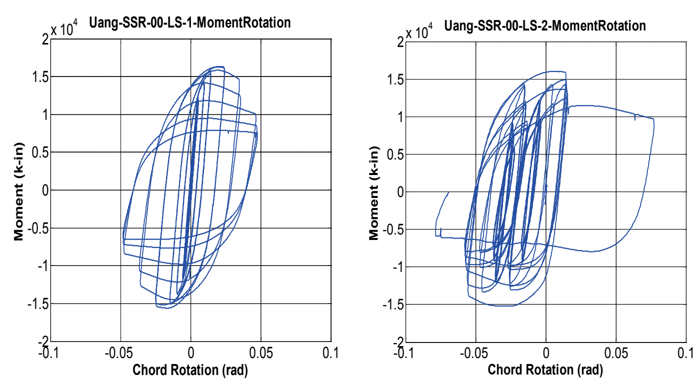


Figure B-1 Identical steel specimens tested at the University of California at San Diego under different loading histories (Figure from PEER/ATC-72-1, data from Uang et al., 2000).

Component Strength. Component strength measured may differ by as much as 20 to 30% under different loading histories, as shown in Krawinkler

(2009) and Gatto and Uang (2002), with components subjected to a larger number of inelastic cycles exhibiting smaller strength. Cyclic-loading histories generally lead to estimates of component strength that are less than or equal to those observed from monotonic-load tests of identical specimens.

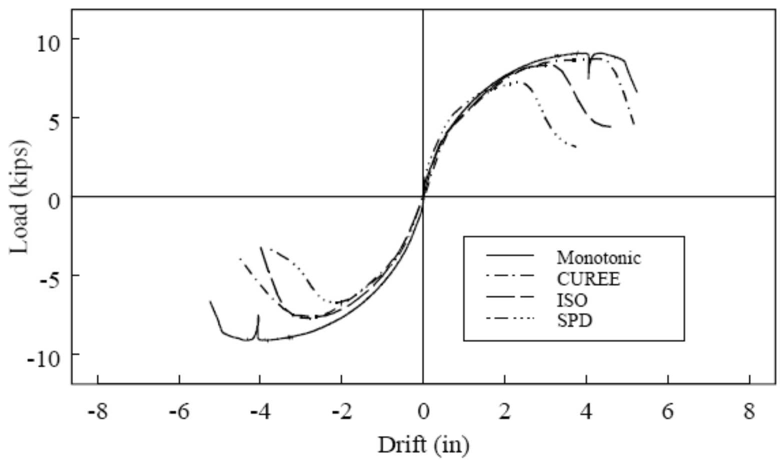


Figure B-2 Comparison of cyclic envelope curves obtained for identical specimens under different loading protocols (from Gatto and Uang 2002), illustrating the effects of loading history choice on strength and deformation capacity.

Failure Mode. The executed loading history may have a significant effect on the failure mode observed for the component and/or system. Loading histories with very large number of cycles may produce failure modes that are not representative of those expected in an earthquake (Krawinkler, 2009; Gatto and Uang, 2002). The applied loading history has not been shown to have an important influence on estimates of initial stiffness (Gatto and Uang, 2002).

Loading protocols and histories with different characteristics, therefore, may lead to substantially different estimates of key component performance parameters. The number and relative amplitude of cycles in a loading history, particularly the number of inelastic cycles before failure, is critical. The protocol control reference value used to define the loading history may also be important because it affects the number of cycles before the ultimate deformation, Δ_U, is reached.

B.2.2 Overview of Commonly Used Loading Protocols

Loading protocols are defined by a number of different characteristics, which have been shown to affect observed values of component deformation capacity and strength. These include:

- Number and relative amplitudes of cycles, particularly the number of inelastic cycles when close to failure.
- Test control parameter which is a physical quantity used to control the loading applied to the test specimen. Specific reference values of the test control parameter are used to define the number and amplitudes of cycles applied to the specimen. Some reference values, such as the yield deformation, may be difficult to define, so that even with the same loading protocol two identical specimens may give different results depending on the definition of the reference value (Krawinkler et al., 2000).
- It is generally observed that specimens loaded more rapidly have higher strength (Uang et al., 2000).

A variety of different loading protocols have been used in past experimental studies and some of these are summarized in Table B-1. The Component Methodology does not limit the application of loading protocols (histories) to those shown here. However, the cyclic-loading history must meet the requirements of Section 2.2.2, and it is desirable that testing follow an established protocol. These protocols apply to quasi-static cyclic (reverse) testing of elements of the seismic force resisting system. All are deformation (rather than load) controlled in inelastic cycles.

Table B-1 reports important characteristics of various loading protocols, including the component type it was developed for and the test control reference values, denoted here as $\Delta_{control}$. This reference value is commonly related to some measure of the "yield" or "maximum" ("ultimate") deformation. As Table B-1 shows, the ISO, CEN-long, CUREE and FEMA 461 protocols are based on control reference values related to various definitions of ultimate/maximum deformation. The New Zealand, ATC-24, FCC, and SPD loading histories are based on control reference values related to an estimate of yield deformation. The SAC protocol implicitly assumes a yield story-drift ratio of 0.01, which provides the control reference value. Definitions of loading protocol and history are included in the references listed in the table and in Filiatrault et al. (2008), Krawinkler (2009) and Gatto and Uang (2002).

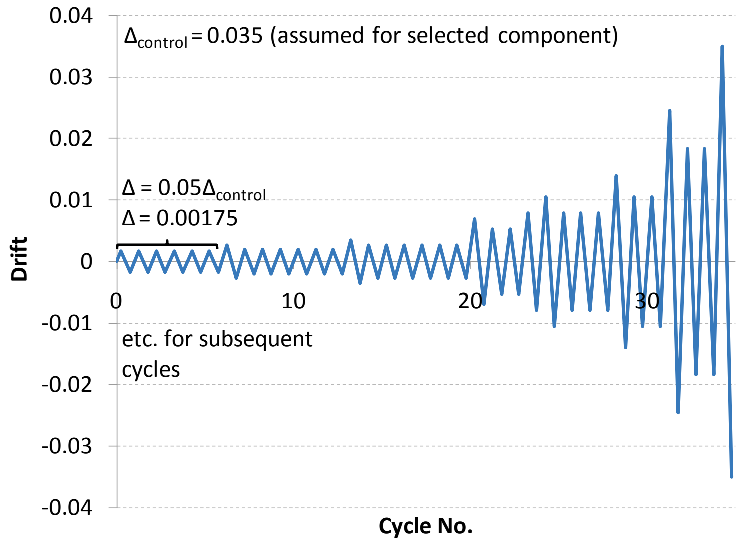
Table B-1 Key Features of Selected Loading Protocols

Loading Protocol/History	Component of Interest	Test Control Reference Value
FEMA 461 (FEMA, 2007)	Primarily for nonstructural components	Test control defined by Δ_o (smallest deformation amplitude of interest), Δ_m (target maximum deformation amplitude), n (number of steps in loading history).
ISO (ISO, 2003)	Wood connections/ walls	$\Delta_{control} = \Delta_{max}$ defined as the deformation at maximum load (obtained from an initial monotonic-load test)
CUREE (Krawinkler et al., 2000)	Wood	$\Delta_{control} = \Delta$ defined as the maximum deformation the test specimen is expected to sustain before it triggers a prescribed failure criterion (e.g., 20% strength loss)
FCC (Karacabeyli, 1998)	Wood	$\Delta_{control} = \Delta_y$ (yield deformation)
SAC/AISC (Clark et al., 1997)	Steel	$\Delta_{control}$ = Interstory drift of 0.01
CEN – short CEN – long (CEN, 1995)	Wood connections	$\Delta_{control} = \Delta_y$ multiplied by an assumed ductility (CEN – short) $\Delta_{control} = \Delta_{max}$ defined as the deformation at maximum load (obtained from an initial monotonic-load test) (CEN- long)
ATC-24 (ATC,1992)	Steel	$\Delta_{control} = \Delta_y$ (yield deformation) estimated from initial load control cycles
New Zealand (Cheung et al., 1991)	Concrete	$\Delta_{control} = \Delta_y$ (yield deformation) estimated by a theoretical estimate of component strength and extrapolation of displacement of specimen during early, load-controlled cycles.
SPD (Porter, 1987)	Masonry (Wood)	$\Delta_{control}$ = deformation at FME (first major event; usually taken as event causing "yielding")

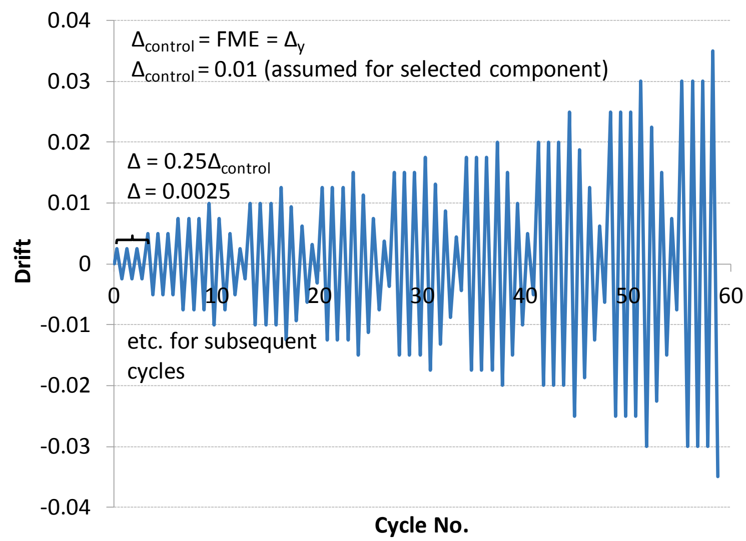
Two commonly used loading histories are shown in Figure B-3, to illustrate key terms and features. Figure B-3a shows the CUREE protocol, assuming $\Delta_{control}$ = 0.035, where 0.035 is the deformation at 20% strength loss, i.e., Δ_u. Figure B-3b shows SPD protocol, executed to the same maximum drift of 0.035. The control reference value is the deformation (drift) at the first major event, usually taken as the yield displacement. In this example, first major event (FME) is defined as a drift of 0.01 (though in practice this value may be much smaller). Tests may be continued beyond the deformation amplitude to obtain all necessary component response parameters for the Component Methodology.

B.2.3 Selection of Acceptable Loading Histories and Protocols

This section suggests a method for showing that loading histories and protocols are acceptable according to Section 2.2.2. Requirements for acceptable cyclic-loading histories for the proposed and reference component data sets are described below. In each case, general guidelines are provided, followed by a specific criterion.



(a)



(b)

Figure B-3 Illustration of two cyclic-loading protocols for an example component test specimen: (a) CUREE; and (b) SPD.

In the criteria that follow, normalized cumulative deformation demand is employed as a measure of how damaging the loading history is to the component of interest. The normalized cumulative deformation parameter is used to define criteria for identifying acceptable loading histories for testing. More damaging protocols impose higher normalized cumulative deformation demands. Cumulative deformation is defined here as the sum of the deformation amplitudes of all relevant excursions (a cycle consists of two excursions, one in the positive direction and one in the positive direction). The measured deformation at 20% strength loss from the cyclic envelope curve, $\Delta_{U,}$ is employed to normalize the cumulative deformation demand. Examples showing how to compute the normalized cumulative deformation demand for a given loading history and component are given in Section B.2.5.

For the purpose of comparing specimens and loading histories, the cumulative deformation demand imposed during a specific test *i* before reaching the deformation amplitude of Δ_U, normalized by $\Delta_{U,}$ is denoted normalized cumulative deformation demand, NCD_i. The value of the normalized cumulative deformation demand depends on the loading history applied and the deformation capacity of the particular specimen. To show loading history acceptability, NCD_i must be computed for every proposed and reference component test.

Loading histories applied to proposed and reference components should be shown to satisfy the following guidelines, in order to meet the requirements of Section 2.2.2 and 2.4.4.

Experimental tests should be conducted using a cyclic symmetric deformation history of step-wise increasing amplitude. Loading histories should be deformation controlled in inelastic cycles, rather than load controlled. The features of the loading history, including the number of cycles per step and the amplitude of individual cycles in a step, may vary, as, for instance, in the CUREE protocol and the SPD protocol, as illustrated in Figure B-3.

The normalized cumulative deformation demand of each test (NCD_i) is used to ensure that components tested with similarly damaging loading histories are grouped together in performance groups.

Limits of Normalized Cumulative Deformation Demand

The normalized cumulative deformation demand from each test, NCDi, should fall within the range defined by Equations B-1 or B-2, such that

$$0.5N\tilde{C}D_{RC} \leq NCD_{RC,i} \leq 2N\tilde{C}D_{RC} \tag{B-1}$$

and

$$0.5N\tilde{C}D_{PC} \leq NCD_{PC,i} \leq 2N\tilde{C}D_{PC} \tag{B-2}$$

where $N\tilde{C}D_{RC}$ and $N\tilde{C}D_{PC}$ are the median normalized cumulative deformation for each performance group for reference and proposed components, respectively.

This comparison between the median and each individual test should be carried out for each performance group, ensuring that data for a given component performance group is sufficiently similar to be combined in generating statistics on key component parameters. If the component is grouped into several performance groups, multiple values of the median value will be computed. Tests falling outside this range are deemed substantially different and should be separated into a different performance group. The range defined in Equations B-1 and B-2 is based on judgment and review of experimental data.

No tests with $NCD_i < 6.0$ are permitted. Tests with $NCD_i < 6.0$ were executed with a loading history that is not damaging enough to be used in the Component Methodology. The lower bound *NCD* of 6.0 is based on examination of data in Figure B-5 and falls significantly below the values of normalized cumulative deformation calculated for commonly used protocols. This lower bound is intended to ensure that most of the available protocols can be used with a variety of different types of components even those with characteristics different from what was assumed in Figure B-5. Under only an isolated number of cases, therefore, a test may have to be discarded for imposing too small a normalized cumulative deformation.

The proposed component shall be tested under a loading protocol(s) that imposes approximately the same, or more, normalized cumulative deformation on the component, compared to the loading histories used to establish the reference component data set. Proposed components may not be tested using protocols that impose significantly less normalized cumulative deformation, because these protocols are less damaging and will lead to overestimates of component strength and deformation capacity relative to the reference component.

Selection Criteria for Proposed Component Loading History

Loading histories for proposed component tests are considered to be acceptable provided that the median normalized cumulative deformation at the deformation amplitude of Δ_U for the proposed component, $N\tilde{C}D_{PC}$ is at least 75% of the median normalized cumulative deformation for the reference component, $N\tilde{C}D_{RC}$. Thus,

$$N\tilde{C}D_{PC} \geq 0.75 N\tilde{C}D_{RC} \tag{B-3}$$

This requirement should be met by comparing the $N\tilde{C}D_{PC}$ for each proposed component performance group to the $N\tilde{C}D_{RC}$ of the associated reference component performance group.

There is no strict upper limit placed on $N\tilde{C}D_{PC}$, compared to $N\tilde{C}D_{RC}$ However, it is recommended that the proposed component not be tested using protocols that impose significantly more normalized cumulative deformation than is measured in the reference component tests. In this case, the proposed component protocol may have too many cycles to permit a realistic comparison between proposed and reference component parameters for strength and deformation. In addition, a large number of cycles may lead to an underestimation of the component strength ratio. Estimations of strength are generally conservative, but not in the case where proposed and reference components will be used together in a structural system, such that disparities in component strength may lead to damage localization. The use of a proposed component protocol that has a normalized cumulative deformation demand significantly greater than the measured reference component demand is therefore discouraged.

Both the proposed and reference components should be tested under loading histories or protocols that are appropriate for the component under consideration, and it is desirable that the loading protocol produce damage and failure mechanisms that are consistent with what is expected (or has been observed) in earthquakes. Some of the loading protocols have built-in assumptions about behavior, according to the application for which the protocol was developed. For example, for moment frames designed for high seismic regions, the SAC protocol assumes a typical value of yield story drift of 0.01, which is appropriate for steel beam-column subassemblies but not necessarily other configurations. ASTM E2126 Standard describes several standard test methods and their applicability (ASTM, 2009).

When a generally accepted industry loading protocol exists for the material or component of interest, every effort should be made to use data obtained using this protocol to establish equivalency.

Note that, in general, a loading protocol for the proposed component needs to be selected before the proposed component tests are conducted. To make this selection, a reasonable approximation of $N\tilde{C}D_{PC}$ is needed for the different candidate protocols. If some information about proposed component performance is already available, $N\tilde{C}D_{PC}$ may be initially estimated from available data and the loading protocol of interest. If not, it may be necessary to test one or more proposed components with the loading protocols under consideration to determine if the selected protocol is likely to meet the requirements. However, the final assessment of loading history equivalency should be based on data obtained directly from proposed and reference component tests.

B.2.4 Special Case: Same Loading Protocol Used to Generate Proposed and Reference Component Data

Loading history requirements can be assumed to be automatically satisfied if the following conditions are met: (1) the same loading protocol is used for all proposed and reference component tests; and (2) the test control parameter and associated reference value are based on a measure of ultimate or maximum deformation.

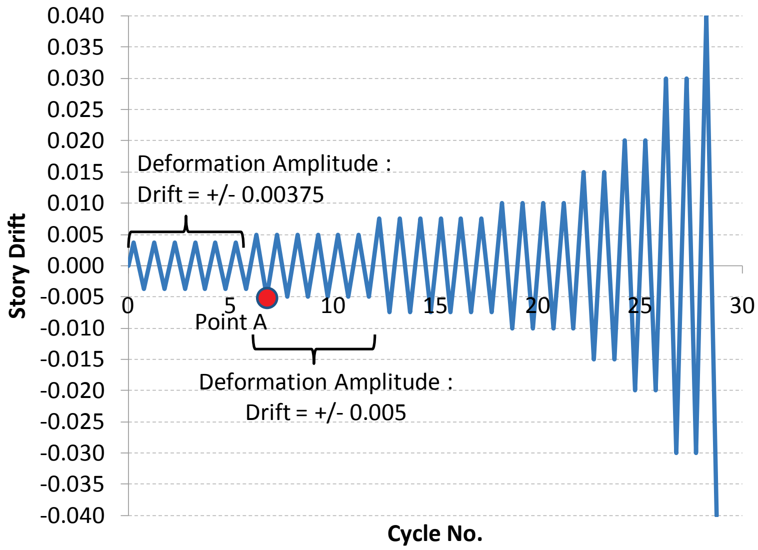
If both of the above conditions are satisfied, the selected loading protocol is approximately equivalently damaging to the proposed and reference components in the Component Methodology, satisfying the requirements of Section 2.2.2. This special case takes advantage of Component Methodology requirements in Chapter 2, which ensure that $\Delta_{U,PC}$ is approximately equal to $\Delta_{U,RC}$. Given that the ultimate deformation is known to be similar for the proposed and reference components, a loading protocol will impose similar deformations to the proposed and reference component specimens, provided that the test control parameter is a similar measure of ultimate deformation.

If these conditions are not satisfied, NCD_i should be calculated for each test and loading histories must be shown to meet the requirement of Section B.2.3. The same loading protocol, when applied to components with different properties, may be more or less damaging, providing, the rationale for the rules in this appendix.

B.2.5 Illustration: Comparison of Loading Histories

Computation of Normalized Cumulative Deformation

In this illustration, the story drift, Δ_U, is assumed as 0.035 for the hypothetical component under consideration. The SAC loading protocol, defined in terms of story drift, is shown in Figure B-4a. The first set of six cycles has a deformation (drift) amplitude of 0.00375, the second set of six cycles have a deformation amplitude of 0.005, and so on. The x-axis of the normalized cumulative deformation plot in Figure B-4b represents the maximum deformation amplitude the specimen has experienced. After the first set of positive and negative excursions to a deformation of 0.005 (one full cycle, point A), the maximum drift amplitude is 0.005. The deformation amplitude is normalized by Δ_U, therefore 0.005/0.035 = 0.14, which corresponds to the x-value of the first point shown in Figure B-4b. The normalized cumulative deformation, shown on the y-axis of Figure B-4b, accounts for the total deformation experienced by the specimen. Before reaching the deformation amplitude of 0.005, the SAC protocol (Figure B-4a) has pushed the specimen through 6 cycles +/- 0.00375 and one cycle of +/- 0.005. Therefore the cumulative deformation (accounting for both positive and negative excursions) is 0.00375 x 12+ 0.005 x 2, or a cumulative deformation of 0.055. For a normalized deformation amplitude of 0.14 (x-value on Figure B-4b), the normalized cumulative deformation is 0.055/0.035 = 1.57 (y-value on Figure B-4b). This process is repeated to obtain the relationship between normalized cumulative deformation and the normalized deformation amplitude over the entire deformation range imposed during the test. When computing the cumulative deformation, elastic or nearly elastic cycles should not be included; this example assumes there are no such cycles.



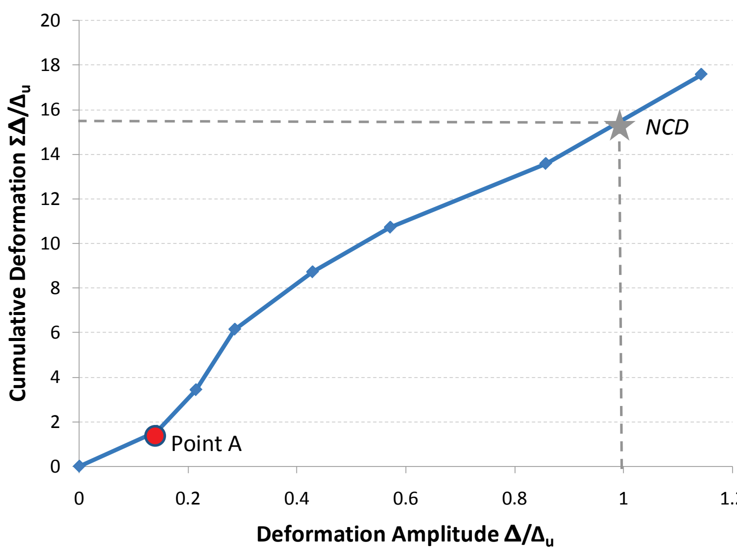


Figure B-4 Illustration of normalized cumulative deformation plot, showing (a) SAC loading protocol, (b) plot of normalized cumulative deformation vs. normalized deformation amplitude.

Equivalency between loading histories is judged based on the normalized cumulative deformation at the deformation amplitude corresponding to Δ_U.

This value is denoted *NCD*. Since Δ_U will rarely coincide with a deformation amplitude of the loading history, interpolation is needed to obtain the value of *NCD*, as illustrated in the figure. In Figure B-4b, the normalized cumulative deformation at the assumed Δ_U of 0.035 is 15.6.

Comparison of Loading Histories for Proposed and Reference Component Tests

The criteria of Section B.2.3 are illustrated in Figure B-5 for the case of a single reference component test and various options of loading protocols for proposed component tests.

In this example, it is assumed that the reference component test has been executed with the CUREE protocol and that the measured Δ_U is 0.035 and is equal to the target value $\Delta_{control}$ of the protocol. For this case, the normalized cumulative deformation plot is as shown in a bold line in Figure B-5, denoted RC (CUREE), and the NCD_{RC} is equal to 13.3. For simplicity of illustration, it assumed that the median value, $\tilde{NCD}_{RC}$, accounting for all reference component tests, is also 13.3. Note that in an actual case there will be several reference component tests and each may have a different Δ_U and a different *NCD*. According to the requirements of Equation B-3, these reference component test results impose a lower bound of 0.75 x 13.3 or 10.0 on the $\tilde{NCD}_{PC}$.

For the purposes of this illustration, suppose that a preliminary proposed component test has been executed and Δ_U is equal to 0.035 in this case as well. In Figure B-5 several candidate protocols are considered; these plots are generated assuming control variables of Δ_{max} = 0.035 or Δ_Y = 0.01 (for illustration only, recognizing that the actual values of Δ_{max} and Δ_Y may be quite different from the assumed ones). NCD_{PC} can then be compared to for the candidate proposed component protocols. Of the other protocols shown in Figure B-5, the CEN-short protocol would not be acceptable to test the proposed component because the *NCD* value in the example is below the lower bound of 10.0. The SPD protocol is shown to be significantly more damaging (i.e., leading to a much larger value of *NCD*) and it should be carefully considered whether this protocol is appropriate for testing the proposed component. All other protocols shown in the example are acceptable according to Equation B.3.

Once a candidate proposed component protocol is selected, all proposed components are tested and *NCD* is computed for each individual test. These test data are then used to determine whether the requirements of Section B.2.3 are met.

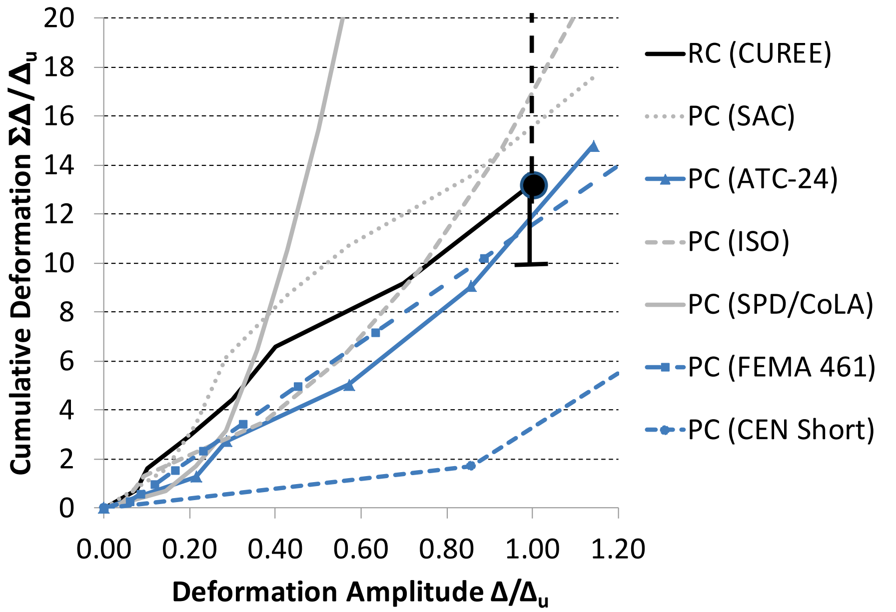


Figure B-5 Normalized cumulative deformation plots for an example reference component protocol (CUREE) and several candidate protocols. This plot is created based on specific assumptions about the proposed and reference component and is not generally applicable.

B.2.6 Additional Considerations for Cyclic-Load Testing

The Component Methodology does not stipulate specific requirements for testing to deformation levels beyond Δ_U. However, it is emphasized that testing should be performed to levels of deformations that permit full assessment of important force-deformation characteristics of component at all performance levels, including structural collapse. Although not required by the Component Methodology, tests can be continued to larger deformations at little or no additional expense. Users will find that these data may be very useful in the future, especially for efforts related to analytical modeling of component behavior.

B.3 Monotonic-Load Test Data Considerations

The Component Methodology requires monotonic-load test data, in addition to the cyclic-load test data already described. Requirements for monotonic-load test data are included for two reasons.

Firstly, the Component Methodology focuses on collapse resistance, so it is desirable that components are tested under displacement patterns representative of the response history experienced by a component in a seismic-force-resisting system that is near-collapse or collapsing during an earthquake. It has been shown that in many cases these displacement patterns may be more similar to a monotonic push than a symmetric cyclic-loading history. Experimental test results for both cyclic- and monotonic-loading patterns, as required in the Component Methodology, are likely to bracket the true response, which depends on the frequency content and duration of the earthquake ground motion.

Second, the cyclic envelope curve, created from cyclic-load test data, gives an incomplete representation of component collapse resistance because it mixes two different modes of strength deterioration. To more reliably quantify collapse resistance, monotonic-load test data are a necessary supplement to cyclic-load test data.

B.3.1 Importance of Monotonic-Load Test Data in Component Methodology

Cyclic-loading protocols generally prescribe a deformation history that is symmetric with respect to the undeformed structural or component configuration. For structures subjected to code design-level ground motions, story drift and component responses during earthquake shaking are mostly symmetric and cyclic-load testing provides a reasonable representation of loads and deformation in the structure (Krawinkler et al., 2001). When subjected to an earthquake large enough to cause collapse, however, patterns of story drift and component responses depend on the failure mode and on the frequency characteristics and strong motion duration of the seismic input. Component responses might be close to symmetric-cyclic until severe deterioration occurs, or they might be one-sided and more similar to monotonic response than symmetric-cyclic. In the latter case, for example, a structure may deform permanently in one direction because deformation reversals are not sufficiently large to return the building to its original position (Krawinkler and Lignos, 2009). This case deserves consideration in the context of component equivalency evaluation. The requirements for both monotonic-load and cyclic-load test data are therefore intended to ensure that components are tested with deformation histories that represent the range of deformation patterns that may be experienced by the component during an earthquake.

In addition, there are two distinctly different modes of strength deterioration that may occur in a structure or component, and each has substantially different impacts on the collapse resistance. The use of an envelope curve, based on cyclic-load test data, mixes these two modes of strength deterioration and gives an incomplete representation of component or system collapse resistance. Monotonic-load test data serve to separate these two modes of component strength deterioration, thereby improving understanding of collapse performance, as illustrated in this section.

The two different modes of strength deterioration that may be experienced by a component or structure are defined as follows and illustrated in Figure B-6.

- Cyclic strength deterioration: Strength loss occurs between two cycles of loading. In this mode of strength deterioration, the element response maintains a positive tangent stiffness.
- In-cycle strength deterioration: Strength loss occurs during a single cycle of loading. In this mode of strength deterioration, the element response exhibits a negative tangent stiffness.

For purposes of illustration, Figure B-6 shows two components, one of which is experiencing only cyclic strength deterioration and the other only in-cycle strength deterioration. In reality, most component behavior reflects a combination of the two.

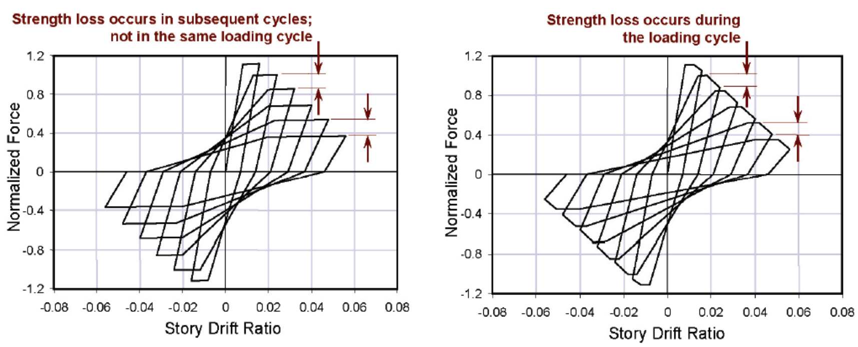


Figure B-6 Cyclic behavior of an element experiencing (a) only cyclic strength deterioration, and (b) only in-cycle strength deterioration, figures from FEMA P-440A (FEMA, 2009).

As a structure approaches collapse, components experience both strength deterioration modes. It has been shown that in-cycle strength deterioration is usually more important in bringing on global collapse than cyclic strength deterioration (Ibarra et al., 2005). The deformation at the onset of these important deterioration modes depends on the loading history applied to the component or structure. As a result, cyclic-load testing may not capture important differences in component response in a real earthquake history. The cyclic envelope curve used to summarize experimental results from cyclic-load testing combine the effects of in-cycle and between-cycle strength deterioration.

B.3.2 Illustration: Limitations of Using Only Cyclic-Load Test Data for Component Equivalency

This example illustrates the possible problems that arise if equivalency criteria did not utilize both monotonic load and cyclic load testing.

Figure B-7 shows the response of two components with different types of cyclic behavior. The cyclic envelope curves show that both components have approximately the same $\Delta_U \approx 0.06$ [story drift]. The reference component behavior in this example, Figure B-7a, exhibits a combination of cyclic strength deterioration (for cycles with peak story drift ≤ 0.06) and in-cycle strength deterioration (for cycles with peak story drift > 0.06). This behavior is representative of many types of structural components. In contrast, the proposed component behavior, Figure B-7b, exhibits no cyclic strength deterioration, and the behavior is controlled entirely by in-cycle strength deterioration.

Despite differences in strength deterioration and cyclic behavior of the proposed and reference components, equivalency comparisons based only on the cyclic-load test data (i.e., the cyclic envelope curve) would find the two components to be equivalent since the components have the same strength, ultimate deformation, Δ_U, and stiffness.

Figure B-7 also shows results of monotonic-load tests for the proposed and reference components. Due to the observed differences in strength deterioration, the behavior of these two components under monotonic-load tests is quite different, with the reference component test data showing significantly larger monotonic ultimate deformation than the proposed component test data. These distinct differences in monotonic-loading curves call into question whether these proposed and reference components are truly equivalent. The scope of the Component Methodology is to ensure equivalency in collapse performance, so these components should only be deemed equivalent if they result in equivalent collapse performance.

In order to determine if the example proposed and reference components have equivalent collapse performance, a single-degree-of-freedom (SDOF) model is created for each system. These models are subjected to an incremental dynamic analysis process (as described by Vamvatsikos and Cornell, 2002) to assess seismic collapse resistance, using a set of 20 far-field ground motions. The period of each SDOF oscillator is 1.0 second and the models have 5% damping, and are assigned a yield strength of 200kN (equivalent to a yield S_a(1.0s) of 0.25g).

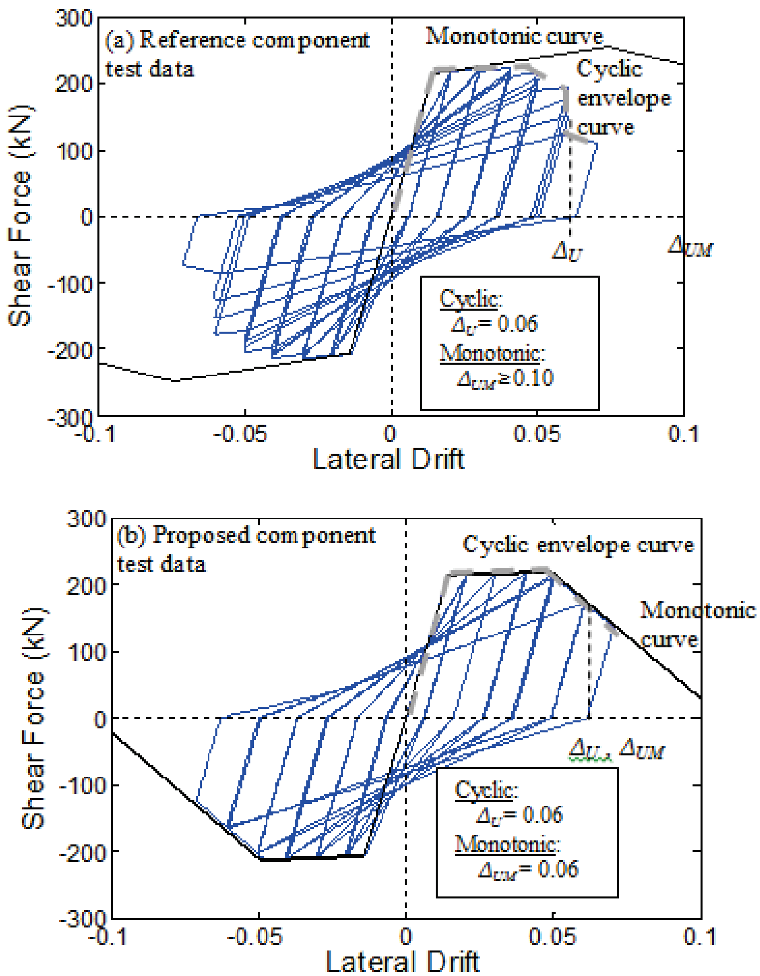


Figure B-7 Hypothetical cyclic-load and monotonic-load test data for reference component (top) and proposed component (bottom) illustrating the importance of considering both cyclic and monotonic behavior. Identical cyclic test parameters may obscure differences in monotonic behavior due to differences in cyclic and in-cycle strength deterioration.

Figure B-8 shows the time-history response for a single ground motion scaled to S_a(1.0s) = 1.0g. Under this ground motion, the SDOF calibrated to represent the reference component is substantially damaged (exhibiting permanent drift deformation of 0.05), but does not collapse. In contrast, the SDOF calibrated to represent the proposed component behavior collapses laterally, exhibiting very large interstory drifts.

Collapse fragility curves generated from incremental dynamic analysis of all 20 ground motions are shown for the proposed and reference component SDOF models in Figure B-9. The results in Figure B-9 show that the proposed component is substantially more prone to collapse than the reference component, with median collapse capacities of 1.6g and 2.9g for the proposed and reference component SDOF models, respectively. These results imply higher probabilities of collapse for the proposed component model for a specified level of ground motion intensity.

This example illustrates that equivalency between the cyclic envelope curves may not be sufficient to demonstrate equivalency in collapse safety. Equivalency in the monotonic-loading curves is helpful to discern the relative amounts of in-cycle deterioration present in the component behavior. An approach based on equivalency of cyclic envelope curve properties alone could lead to the approval of a proposed component that has much lower collapse capacity than the reference component.

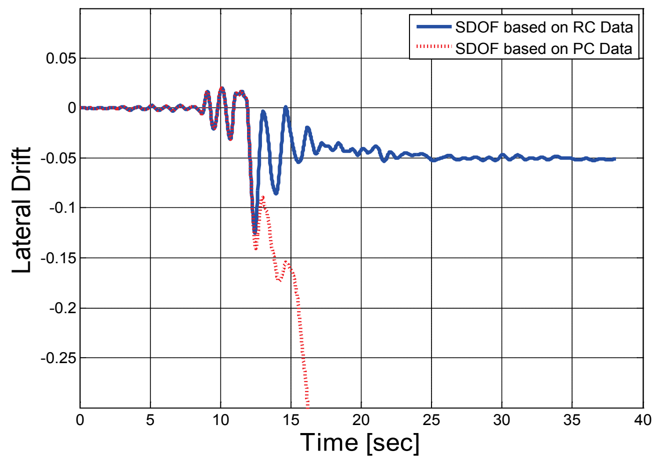


Figure B-8 Example time-history response of the proposed and reference component SDOF models subjected to a single ground motion scaled to $S_a(1.0s) = 1.0g$.

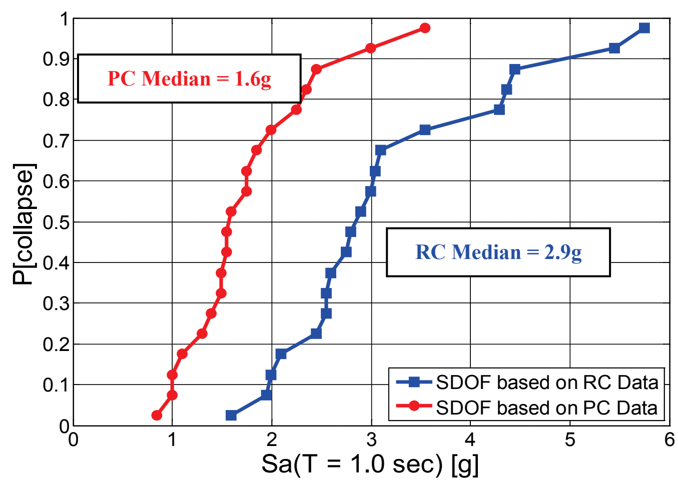


Figure B-9 Collapse fragility curves for the proposed and reference component SDOF models subjected to 20 ground motions.

Note that in developing this example, several simplifications were made including the use of idealized rather than actual test data and the representation of structural response using a SDOF system rather than a multiple-degree-of-freedom system. Most real components exhibit some combination of in-cycle and cyclic strength deterioration. The simplifications in component behavior made in this example exaggerate the differences in collapse capacity between the two components, but the conclusions hold for less extreme comparisons also. For any comparison in which the proposed component exhibits relatively more in-cycle strength deterioration (and relatively less cyclic strength deterioration) as compared with the reference component, establishing equivalency based only on the cyclic envelope curves would be non-conservative. Such an approach could lead to the approval of a proposed component that has much lower collapse capacity than the reference component.

B.3.3 Monotonic-Load Test Data Requirements

The Component Methodology requires both monotonic-load and cyclic-load test data for establishing equivalency of the proposed and reference components.

Specifically, the monotonic-load ultimate deformation of the proposed component should equal or exceed the monotonic load ultimate deformation of the reference component, increased by the uncertainty penalty factor. These comparisons should be conducted for a subset of the proposed and reference component configurations. Detailed requirements are given in Section 2.8.4.

Appendix C
Development of Probabilistic Acceptance Criteria

C.1 Introduction

This appendix describes the development of probabilistic-based acceptance criteria expressed in Equation 2-1 for evaluating component equivalency, and provides a detailed description of the development of Tables 2-3 and 2-4.

Quantitatively, equivalent collapse safety implies that the 10th percentile collapse capacities are equivalent for proposed components and reference components, and this concept has been incorporated into the acceptance criteria of the Component Methodology. Requirements for relative strength and ultimate deformation capacity of proposed and reference components have been set to achieve this equivalent collapse probability, and have been adjusted considering the uncertainties inherent in estimation of collapse capacity.

C.2 Collapse Capacity Fragilities and the Effects of Uncertainty

Figure C-1 illustrates two different collapse fragility curves relating ground motion intensity at collapse (normalized by the maximum considered earthquake (MCE) and denoted as the collapse margin ratio) to the probability of collapse. This appendix assumes that collapse capacities are lognormally distributed.

These two fragility curves have identical 10th percentile collapse capacities, i.e., in both cases a collapse margin ratio of 1.0 corresponds to a 10 percent probability of collapse. Even though the 10th percentile collapse capacities are identical, the median collapse capacity values corresponding to a 50 percent probability of collapse differ because of the differences in underlying uncertainty or variability, quantified by the standard deviation. While Collapse Distribution “A” has a logarithmic standard deviation, β_{TOT}, of 0.50 and a median collapse margin ratio of 2.0, Collapse Distribution “B” has a logarithmic standard deviation of 0.70, which leads to the larger required median value of 2.58.

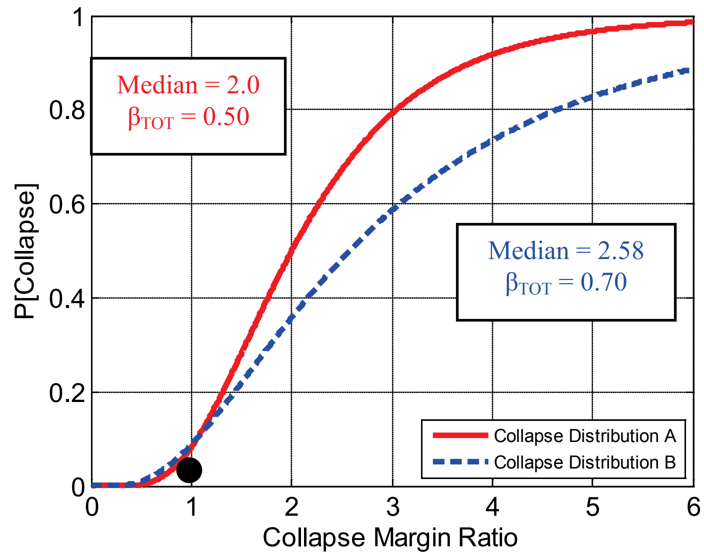


Figure C-1 Illustration of two collapse capacity fragilities, with the same 10th percentile collapse capacity but different variability and medians.

Thus, for a given 10th percentile collapse capacity, there is a relationship between the uncertainty in the fragility and the required median collapse capacity necessary to achieve the same 10th percentile collapse capacity. This relationship is an important component of the probabilistic acceptance criteria of the Component Methodology.

Following the framework used in FEMA P-695 *Quantification of Building Seismic Performance Factors* (FEMA, 2009b), the variability in the collapse capacity fragility, β_{TOT}, is assumed to come from four sources of uncertainty, as follows:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^{\ 2} + \beta_{DR}^{\ 2} + \beta_{TD}^{\ 2} + \beta_{MDL}^{\ 2}}$$

where β_{TOT} is the total variability in collapse fragility, β_{RTR} is the variability in collapse capacity fragility associated with record-to-record variability, β_{DR} is the uncertainty associated with imperfect design requirements, β_{TD} is the uncertainty associated with variation in test data, and β_{MDL} is the uncertainty associated with the imperfect numerical modeling of the structure. Further detail regarding the approach to assigning values for these parameters can be found in FEMA P-695.

The above equation for evaluating the total uncertainty in the collapse fragility represents the uncertainty in collapse capacity for a complete class of structural systems. Accordingly, not all terms are needed for the simpler Component Methodology. The equation can be simplified by assuming β_{RTR}

= 0.40 for record-to-record variability (consistent with FEMA P-695). In addition, since structural modeling is not a direct part of the Component Methodology, β_{MDL} = 0.20 is assumed for the structural modeling (consistent with "Good" modeling in FEMA P-695). After these substitutions, the design requirement and test data uncertainty terms (β_{DR} and β_{TD}) remain to be evaluated for the proposed component as part of the Component Methodology, as follows:

$$\beta_{TOT,PC} = \sqrt{(0.40)^2 + \beta_{DR}{}^2 + \beta_{TD}{}^2 + (0.20)^2}$$

The subscript PC implies that this total uncertainty reflects our uncertainty in the prediction of the collapse resistance of the seismic force resisting system consisting of the proposed component.

For purposes of equivalency, this $\beta_{TOT,PC}$ must have a basis to be compared against. This basis utilizes the same record-to-record and modeling uncertainty values used above, and assumes Good design requirement (β_{DR} = 0.20), which allows the user a beneficial effect if the design requirements are rated as Superior, and Superior test data (β_{TD} = 0.10). The numeric values of β_{DR} and β_{TD} are based on FEMA P-695. The basis value is calculated as follows:

$$\beta_{TOT,\text{Target}} = \sqrt{(0.40)^2 + (0.20)^2 + (0.10)^2 + (0.20)^2} = 0.50$$

The relationship between the variability in the collapse fragility and the required median collapse capacity, in order to achieve the same 10th percentile collapse capacity, is quantified below:

$$\mu_{C,PC} - \mu_{C,RC} = 1.28\left(\beta_{TOT,PC} - \beta_{TOT,\text{Target}}\right) \qquad \text{(C-1)}$$

where $\mu_{C,PC}$ is the logarithmic mean collapse capacity for the proposed component and $\mu_{C,RC}$ is the logarithmic mean collapse capacity for the reference component. In this equation the median is defined in logarithmic terms, i.e., $\mu_{C,PC}$ is the natural logarithm of the median collapse capacity of the seismic force resisting system of interest. The use of Equation C-1 is illustrated below for the example shown in Figure C-1:

$$\ln(2.58) - \ln(2.00) = 1.28(0.70 - 0.50)$$

Equation C-1 shows how to relate the collapse capacity uncertainty to the required median collapse capacity, but this is not enough to create a methodology for component equivalency. Such a methodology also requires an approximate relationship between the collapse capacity and the component parameter values that are observable from test data (e.g., strength and deformation capacity). The focus of the next two sections is to establish such relationships.

C.3 Effect of Changes in Deformation Capacity on the Collapse Fragility

In order to create a probabilistic component equivalency criterion, a quantitative link is needed between the ultimate deformation of a component type and the collapse capacity of the structural system containing that component. Admittedly, the relationship between component and system behavior depends on the type and characteristics of the structural system (e.g., building height, stiffness, irregularities), and it is impossible to provide a single relationship that is generally applicable to all cases. This section develops a single approximate relationship between the component deformation capacity and the system-level collapse capacity.

To create this quantitative relationship, the literature review and sensitivity studies of Appendix A are utilized. Figure C-2 shows an example from the sensitivity study of a three-story wood light-frame building (Building No. 10). As the ultimate deformation of the components increases, the collapse capacity of the structural system also increases. The significance of this trend depends on the gradient (slope) between the collapse capacity and the component parameter (i.e., deformation capacity in this case).

To quantify the relationship between component ultimate deformation and system-level collapse capacity, the gradient in Figure C-2b is utilized, denoted a_1. The gradient is the slope of a linear trend line relating the natural logarithm (LN) of the component deformation capacity to the natural logarithm of the collapse capacity. Here, the collapse capacity is expressed in terms of the adjusted collapse margin ratio (ACMR), as defined in FEMA P-695 and Appendix A. For the example shown in Figure C-2b, the value of a_1 is 0.70 for a decrease in deformation capacity from the baseline value of 0.025. Note that the a_1 decreases slightly for larger values of ultimate deformation, but this is neglected for simplicity.

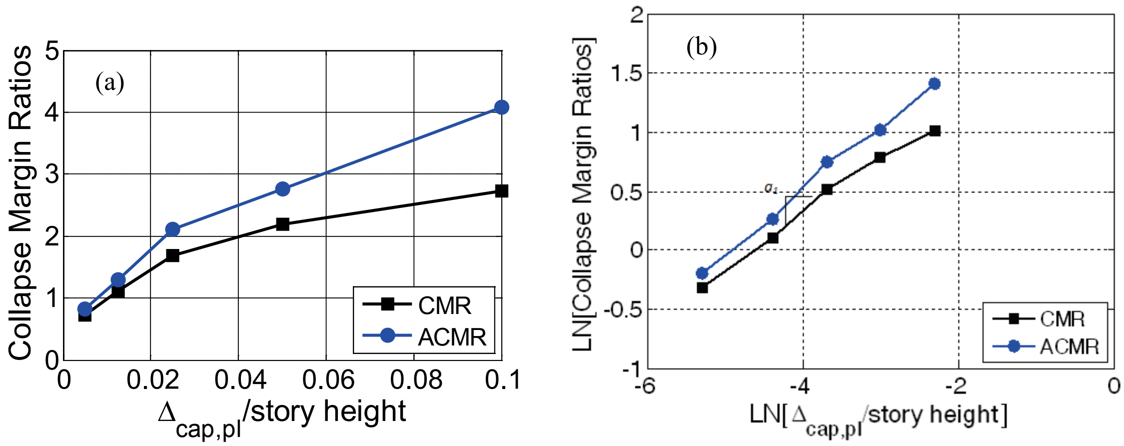


Figure C-2 Relationship between component deformation capacity and system-level collapse capacity, for a three-story wood light frame building (from Figure A-4 of Appendix A).

A similar sensitivity study was completed for several other wood light-frame buildings, including configurations where all components were modified (full replacement of reference components by proposed components) and configurations where only a subset of components were modified in the structural system (mix-and-match of proposed and reference components). These sensitivity studies are documented in Appendix A, and Table C-1 reports the a_1 values for each of the wood light-frame buildings. The median value of the gradient for the full replacement cases is 0.74 where the median gradient is lower for the mix-and-match cases, such that the overall median value is 0.64. There is significant scatter in the a_1 value across the various buildings.

Table C-2 presents similar results for the set of reinforced concrete special moment frame sensitivity studies completed in Appendix A. The moment frame results show a substantially lower median value of a_1 = 0.36.

A similar recent study by Zareian and Krawinkler (2010) found a_1 values of 0.66 on average for generic shear wall buildings failing in flexure. Additionally, a_1 values of 0.32 were found for generic moment resisting frames. The Zareian and Krawinkler study considered only full replacement cases.

Comparing the results above, there is a trend suggesting that component ultimate deformation capacity has a larger effect on collapse resistance of shear wall buildings (with a_1 = 0.66 to 0.74) as compared to moment frame buildings (with a_1 = 0.32 to 0.36).

Table C-1 Relationship Between Component Ultimate Deformation and System-Level Collapse Capacity for Wood Light-Frame Buildings

Wood Light-Frame Building Model	Stories Modified In Sensitivity Study	Gradient a_1 for Decrease in Component Deformation Capacity
Full Replacement Sensitivity Study		
1-story, low aspect ratio (No. 1)	All	0.43
1-story, high aspect ratio (No. 2)	All	0.78
2-story, low aspect ratio (No. 5)	All	0.74
2-story, high aspect ratio (No. 6)	All	0.71
3-story, low aspect ratio (No. 9)	All	1.15
3-story, high aspect ratio (No. 10)	All	0.70
4-story, high aspect ratio (No. 13)	All	1.05
5-story, high aspect ratio (No. 15)	All	0.73
	Median:	**0.74**
Mix-and-Match Sensitivity Study		
3-story, high aspect ratio (No. 10)	1	0.56
3-story, high aspect ratio (No. 10)	2-3	0.42
5-story, high aspect ratio (No. 15)	1	0.44
5-story, high aspect ratio (No. 15)	1-2	0.44
5-story, high aspect ratio (No. 15)	2-5	0.29
5-story, high aspect ratio (No. 15)	3-5	0.28
	Median (lower floors modified):	**0.44**
	Median (upper floors modified):	**0.29**

Table C-2 Relationship Between Component Ultimate Deformation and System-Level Collapse Capacity for Reinforced Concrete Special Moment Frame Buildings

Reinforced Concrete Special Moment Frame Building Model	Stories Modified In Sensitivity Study	Gradient a_1 for Decrease in Component Deformation Capacity
Full Replacement Sensitivity Study		
2-story ID2064 (Perimeter Frame)	All	0.42
4-story ID1003 (Perimeter Frame)	All	0.30
4-story ID1010 (Space Frame)	All	0.47
12-story ID5013 (Perimeter Frame)	All	0.19
12-story ID2009 (Space Frame)	All	0.49
20-story ID5020 (Perimeter Frame)	All	0.20
	Median:	**0.36**

For purposes of the component equivalency criterion, a single a_1 value is selected and applied to all types of structural systems. Therefore, in order to avoid underpredicting the effects of component ultimate deformation, the value of $a_1 = 0.65$ is selected. This value corresponds to the lower range value for shear wall buildings. It is consistent with the findings of the more extensive study by Zareian and Krawinkler, and approximately consistent with the findings for wood light-frame walls in Appendix A.

C.4 Effect of Changes in Strength on the Collapse Fragility

This section creates an approximate relationship between the strength of a component and the collapse capacity of the structural system. This relationship is analogous to that considered above, except component strength rather than component ultimate deformation is the underlying variable.

Figure C-3 shows an example sensitivity study of the same three-story wood light-frame building. In both full replacement and mix-and-match configurations, a decrease in component strength leads to a decrease in the system-level collapse capacity.

For increases in component strength, the collapse safety increases when all of the component strengths are increased uniformly (Figure C-3a). However, when the strengths are increased for only the upper story components (Figure C-3b), the damage localizes more in the first story, thus the increased strength is detrimental to the system-level collapse safety. Similarly to the approach used in the last section, these relationships are quantified by the gradient a_2, which is the slope between the natural logarithm of the ACMR and the natural logarithm of the component strength.

When the Component Methodology is employed, any approved proposed component can be freely mixed-and-matched with the component for which it is a replacement (the reference component). Therefore, the observation that increased component strength may have a detrimental impact on collapse safety when mixing-and-matching occurs (i.e., using the proposed component and reference component together in an individual building) must be accounted for in the Component Methodology. Accordingly a different a_2 value is used to represent the effects of component strength decrease versus component strength increase. For example, the a_2 values for Figure C-3b are 0.55 for a decrease in strength and -0.19 for an increase in strength.

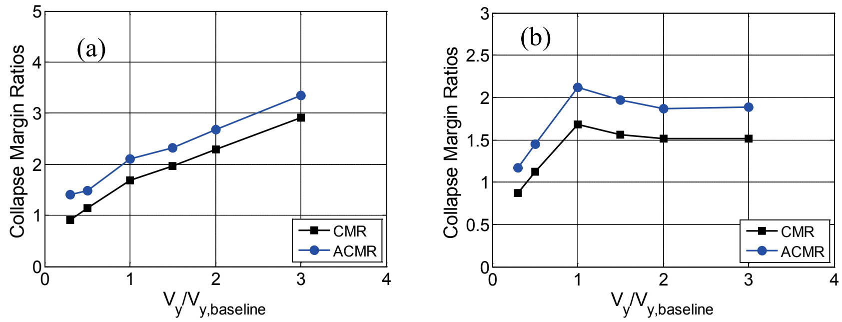


Figure C-3 Relationship between component strength and system-level collapse capacity, for a three-story wood light-frame building showing the results for: (a) full replacement; and (b) mixing-and-matching when only stories 2-3 are replaced with the stronger components.

Table C-3 summarizes the results of sensitivity study for the full set of wood light-frame buildings that were presented in Appendix A. For decreases in the component strength, the median a_2 value is 0.43 for the full replacement case, and the a_2 values are similar for mixing-and-matching cases. For an increase in component strength, we focus on the mixing-and-matching cases where the strength is increased in only the upper stories; for this small subset of cases, the median a_2 value is -0.40. The negative gradient suggests that increases in component strength can sometimes be just as detrimental as decreases in component strength. Even so, this is not always the case and these observations are based on only three buildings, so judgment must be applied when determining how this should impact the a_2 values used in the Component Methodology.

For cases of full replacement, Zareian and Krawinkler found that $a_2 = 0.33$ is appropriate for generic shear wall buildings failing in flexure and $a_2 = 0.73$ is appropriate for generic moment resisting frame buildings.

Based on the above results, there is again an observed difference between shear wall structures and frame structures. Reduced component strength has a smaller effect on the collapse capacity of shear wall buildings (with $a_2 =$ 0.33 to 0.41) than moment frame buildings (with $a_2 = 0.65$ to 0.73).

For purposes of the Component Methodology, a single set of a_2 values must be selected (one for decreased strength and one for increased strength) and used as the approximate value for all types of structural systems. For decreases in component strength, the larger value for moment-resisting frames is utilized ($a_2 = 0.70$), in order to avoid underpredicting the effects of component strength on system collapse performance.

Table C-3 Relationship Between Component Strength and System-Level Collapse Capacity for Wood Light-Frame Buildings

Wood Light-Frame Building Model	Stories Modified In Sensitivity Study	Gradient a_2 for Decrease in Component Strength	Gradient a_2 for Increase in Component Strength
Full Replacement Sensitivity Study			
1-story, low aspect ratio (No. 1)	All	--	--
1-story, high aspect ratio (No. 2)	All	0.35	--
2-story, low aspect ratio (No. 5)	All	0.62	--
2-story, high aspect ratio (No. 6)	All	0.36	--
3-story, low aspect ratio (No. 9)	All	0.41	--
3-story, high aspect ratio (No. 10)	All	0.51	--
4-story, high aspect ratio (No. 13)	All	0.44	--
5-story, high aspect ratio (No. 15)	All	0.43	--
	Median:	**0.43**	**--**
Mix-and-Match Sensitivity Study			
3-story, high aspect ratio (No. 10)	1	0.20	--
3-story, high aspect ratio (No. 10)	2-3	0.55	-0.19
5-story, high aspect ratio (No. 15)	1	0.84	--
5-story, high aspect ratio (No. 15)	1-2	0.57	--
5-story, high aspect ratio (No. 15)	2-5	0.44	-0.58
5-story, high aspect ratio (No. 15)	3-5	0.22	-0.40
	Median (lower floors modified):	**0.57**	**--**
	Median (upper floors modified):	**0.44**	**-0.40**

For increases in component strength, the effect on collapse safety can be either beneficial (for the full replacement case) or detrimental (for the mixing-and-matching case with stronger upper stories). Even though both effects are possible, a single value of a_2 is needed for developing the equivalency acceptance criterion. From the results of the wood light-frame sensitivity studies, it was suggested that increased component strength can sometimes be as detrimental as decreased strength implying that the use of a_2 = 0.70 could be appropriate for both strength increases and decreases. However, the use of such a large value would be unfairly based on a small subset of the most detrimental building configurations. Therefore, after evaluating the results from sensitivity studies and then applying judgment, a value of a_2 = -0.35 was utilized for strength increases (set as half of the value for decreased strength). This value accounts for the fact that increased component strength can be detrimental to the system-level collapse safety, but is not pessimistically based on only the worst-case building configurations.

C.5 Probabilistic Acceptance Criterion Used in Component Equivalency Methodology

C.5.1 Overall Approach

Establishing equivalency between proposed and reference components requires equivalency in the resulting 10th percentile system-level collapse capacities. Differences in these collapse capacities could arise from differences in one or more of the following:

- Differences in collapse capacity uncertainty ($\beta_{TOT,PC} \neq \beta_{TOT,Target}$)
- Differences in component ultimate deformation ($\tilde{\Delta}_{U,PC} \neq \tilde{\Delta}_{U,RC}$)
- Differences in component strength ($\tilde{R}_{Q,PC} \neq \tilde{R}_{Q,RC}$)

To account for each of the above differences between proposed and replacement components, each are related back to a requirement on the median deformation capacities of the proposed and reference components ($\tilde{\Delta}_{U,PC}$ and $\tilde{\Delta}_{U,RC}$, respectively) in Equation 2-1, repeated below:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} \cdot P_U \cdot P_Q$$

The penalty factor for uncertainty, P_U, is the ratio of $\tilde{\Delta}_{U,PC} / \tilde{\Delta}_{U,RC}$ that is required to offset the effects of any differences in collapse capacity uncertainty ($\beta_{TOT,PC} \neq \beta_{TOT,Target}$), in order to ensure uniform collapse safety between the proposed and reference components. Similarly, the penalty factor for strength, P_Q, is the ratio of $\tilde{\Delta}_{U,PC} / \tilde{\Delta}_{U,RC}$ that is required to offset the effects of any differences in strength ($\tilde{R}_{Q,PC} \neq \tilde{R}_{Q,RC}$) between the proposed and reference components.

C.5.2 Development of the Penalty Factor for Differences in Uncertainty

The penalty factor for uncertainty, P_U, accounts for the relationship between uncertainties in the collapse fragility and the required median collapse capacities.

Differences in median collapse capacity are related to differences in the median component ultimate deformation, following the guidance of Section C.3 which related the logarithmic mean of the collapse capacity to the logarithmic mean of the deformation capacity, which leads to the following:

$$\mu_{C,PC} - \mu_{C,RC} = a_1 \left[LN\left(\tilde{\Delta}_{U,PC}\right) - LN\left(\tilde{\Delta}_{U,RC}\right) \right]$$

Combining the above equation with $a_1 = 0.65$ with Equation C-1 leads to the following, effectively determining the improvement in component deformation capacity necessary to make up for differences in collapse fragility uncertainty such that the same 10th percentile collapse capacity is achieved regardless of whether proposed or replacement components are used:

$$a_1\left[LN\left(\tilde{\Delta}_{U,PC}\right)-LN\left(\tilde{\Delta}_{U,RC}\right)\right]=1.28\left(\beta_{TOT,PC}-\beta_{TOT,Target}\right)$$

which can then be algebraically manipulated into:

$$\left[LN\left(\tilde{\Delta}_{U,PC}\right)-LN\left(\tilde{\Delta}_{U,RC}\right)\right]=\left(\frac{1}{a_1}\right)1.28\left(\beta_{TOT,PC}-\beta_{TOT,Target}\right)$$

$$\frac{\tilde{\Delta}_{U,PC}}{\tilde{\Delta}_{U,RC}}=\exp\left[\left(\frac{1}{a_1}\right)1.28\left(\beta_{TOT,PC}-\beta_{TOT,Target}\right)\right]$$

Substituting the uncertainty values from Section C.2 results in the final equation defining the penalty factor for uncertainty, P_U. This penalty factor defines the required ratio of $\tilde{\Delta}_{U,PC}/\tilde{\Delta}_{U,RC}$ that is needed to offset any difference in uncertainty, in order to maintain equivalent collapse safety.

$$P_U=\frac{\tilde{\Delta}_{U,PC}}{\tilde{\Delta}_{U,RC}}=\exp\left[\left(\frac{1}{a_1}\right)1.28\left(\sqrt{(0.40)^2+\beta_{DR}{}^2+\beta_{TD}{}^2+(0.20)^2}-0.50\right)\right]$$

Table C-4 presents the resulting penalty factors for uncertainties, P_U, for various levels of uncertainty associated with the proposed component test data, β_{TD}, and proposed component design requirements, β_{DR}.

Table C-4 Penalty Factor to Account for Uncertainty

Penalty Factor for Uncertainty (P_U)			
Test Data Quality Rating (β_{TD})	Design Requirements Quality Rating (β_{DR})		
	Superior (0.10)	Good (0.20)	Fair (0.35)
Superior (0.10)	0.95	1.00	1.15
Good (0.20)	1.00	1.05	1.25
Fair (0.35)	1.15	1.25	1.40

In developing Table C-4, target values of 0.10 (Superior) and 0.20 (Good) were used for the uncertainties associated with reference component test data and design requirements, respectively. This allows the final penalty factor, P_U, to be evaluated from the table based only on the quality ratings associated with the proposed component. In order to better generalize how the design requirement ratings are handled, Table 2-3 of Chapter 2 uses a slightly different comparative rating (Better, Same, Worst) for the proposed versus reference component design requirements.

C.5.3 Development of the Penalty Factor for Differences in Strength

The penalty factor for strength, P_Q, is developed similarly to the penalty factor for uncertainty, P_U. Equations below show how collapse capacity is affected by differences in component ultimate deformations and differences in component strengths, respectively.

$$\mu_{C,PC} - \mu_{C,RC} = a_1 \left[LN\left(\tilde{\Delta}_{U,PC}\right) - LN\left(\tilde{\Delta}_{U,RC}\right) \right]$$

$$\mu_{C,PC} - \mu_{C,RC} = a_2 \left[LN\left(\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} \right) \right]$$

For equivalency in collapse safety, any reduction in collapse capacity caused by a difference in the component strengths must be offset by an increase in collapse capacity caused by a difference in component deformation capacities. This can be accounted for by setting the above equations equal to one another, as follows:

$$a_1 \left[LN\left(\tilde{\Delta}_{U,PC}\right) - LN\left(\tilde{\Delta}_{U,RC}\right) \right] = a_2 \left[LN\left(\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} \right) \right]$$

Rearranging leads to the theoretical penalty factor for strength, as follows:

$$\frac{\tilde{\Delta}_{U,PC}}{\tilde{\Delta}_{U,RC}} = \exp\left(\frac{a_2}{a_1} \left[LN\left(\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} \right) \right] \right)$$

However, if the load (strength) ratio of the proposed component is similar to the reference component, i.e., $\tilde{R}_{Q,PC} / \tilde{R}_{Q,RC}$ is close to 1.0, there is no penalty factor for component strength. It was determined that a range of $0.9 \leq \tilde{R}_{Q,PC} / \tilde{R}_{Q,RC} \leq 1.2$ should be allowable with no penalty, so the revised equations are as follows:

$$\text{If } \tilde{R}_{Q,PC} / \tilde{R}_{Q,RC} < 0.9 \text{, then } P_Q = \exp\left(\frac{a_2}{a_1} \left[LN\left(\frac{\tilde{R}_{Q,PC}}{0.9\tilde{R}_{Q,RC}} \right) \right] \right)$$

$$\text{If } \tilde{R}_{Q,PC} / \tilde{R}_{Q,RC} > 1.2 \text{, then } P_Q = \exp\left(\frac{a_2}{a_1} \left[LN\left(\frac{\tilde{R}_{Q,PC}}{1.2\tilde{R}_{Q,RC}} \right) \right] \right)$$

Otherwise, $P_Q = \exp\left(\frac{a_2}{a_1}\left[LN(1.0)\right]\right) = 1.0$

Table C-5 presents penalty factors for strength, P_Q, calculated with a_2= -0.35, and provides the basis for Table 2-4 in Chapter 2.

Table C-5 Penalty Factor to Account for Differences in Strength

Penalty Factor for Differences in Strength (P_Q)			
$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q	$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q
0.50	1.88	1.10	1.00
0.60	1.55	1.20	1.00
0.70	1.31	1.30	1.04
0.80	1.14	1.40	1.09
0.90	1.00	1.50	1.13
1.00	1.00	1.80	1.24
1.10	1.00	2.00	1.32

Appendix D
Test Application: Stapled Wood Shear Wall Components

D.1 Introduction

This appendix illustrates the application of the Component Methodology to the possible substitution of stapled wood shear wall components for nailed wood shear wall components in wood light-frame construction. The objective is to determine whether or not a stapled wood shear wall component is equivalent to line A.15 in Table 12.2-1 of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), which reads "Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance or steel sheets." This example assumes that the stapled wood shear wall component is not currently approved by the building code, and is being proposed as an alternative to the code-recognized wood shear wall seismic-force-resisting system listed in Table 2306.3 of the 2009 IBC *International Building Code* (ICC, 2009).

The steps in this test application follow directly from the requirements of the Component Methodology defined in Section 2.2 through Section 2.8

D.2 Description of Stapled Wood Shear Walls

The stapled wood shear walls are designed and detailed with the approach proposed by Talbot et al. (2009), which includes specific detailing requirements that are intended to address common weaknesses and failure modes in the stapled wood shear wall component. Additionally, the associated cyclic-load test data from Talbot et al. (2009) is utilized in this example; these data include four geometric configurations and a total of 13 component tests. Table D-1 summarizes the tested components, providing general information about dimensions, sheathing thickness, and staple spacing. More complete information about the test specimens, including specific detailing information, is presented in Section D.5.2.

Table D-1 shows that the dominant failure mode in the tested stapled wood shear walls is progressive staple withdrawal, leading to separation of the sheathing from the framing. Other failure modes were also observed, such as splitting of the sill plate and failure of the anchor rod, failure of the anchor rod due to pullout from concrete and staple shear failure at the blocking locations. The staple shear failure mode at the blocking was corrected in the latter tests (denoted with superscript 1 in Table D-1) by doubling the number of staples in the blocking region. Figure D-1 illustrates most of these failure modes.

Table D-1 Overview of Stapled Wood Shear Wall Configurations Tested (after Talbot et al. 2009)

Wall Number	Wall Dimensions (HxL)	OSB Sheathing Thickness	Staple Spacing (edge/field)	Failure Mode[2]
3-A#1	8' x 3'	7/16"	2"/4"	SW
3-A#2	8' x 3'	7/16"	2"/4"	SW
3-B	8' x 3'	7/16"	2"/4"	SW
3-C	8' x 3'	7/16"	2"/4"	SW
4-A	8' x 4'	7/16"	2"/4"	SW
4-B	8' x 4'	7/16"	2"/4"	AR
4-C	8' x 4'	7/16"	2"/4"	SW, EP
8-A	8' x 8'	7/16"	2"/4"	SSB
8-B	8' x 8'	7/16"	2"/4"	SSB
8-C[1]	8' x 8'	7/16"	2"/4"	AR, SPS
12-A[1]	8' x 12'	7/16"	2"/4"	SW, STP
12-B[1]	8' x 12'	7/16"	2"/4"	STR, EP
12-C[1]	8' x 12'	7/16"	2"/4"	SW

Note 1: Twice as many staples in the blocking region.
Note 2: Failure Modes: SW - staple withdrawal, STP - splitting of top plate, SSB - staple shear at blocking, EP - end-post, SPS - sill plate splitting, AR - anchor rod pull-out, STR - strap failure.

The breadth of the stapled wood shear wall data set is limited (one staple type/length, one stapling pattern, one sheathing type/thickness, and four geometric configurations). As such, if this example shows that stapled wood shear walls are equivalent to nailed wood shear walls, this conclusion would only apply to the configurations that were included in the stapled wood shear wall testing program. Accordingly, the stapled wood shear wall design requirements must be written to "reasonably restrict" the usage of the stapled wood shear walls to the range of configurations that were considered in testing (see Section 2.5.4); this is addressed later in Section D.5.4.

Figure D-1 Failure modes of stapled wood shear wall test specimens showing: (a) staple withdrawal for wall number 4-C; (b) staple and splitting of top plate for wall number 12-A; (c) anchor rod failure and splitting of sill plate for wall number 8-C; and (d) staple shear and blocking failure for wall number 8-B (from Talbot et al., 2009).

D.3 Evaluation of Applicability Criteria

The Component Methodology is applicable for evaluating the possible equivalency between stapled wood shear walls and nailed wood shear walls. This is based on the criteria of Section 2.3, including the acceptable quality ratings of the test data and design requirements. As described in Section D.7.1, the quality rating of the nailed wood shear wall test data is Superior and the stapled wood shear wall test data are rated as Good. As described in Section D.7.2, the nailed and stapled wood shear wall design requirements are rated as Good and Superior, respectively. Based on these quality ratings, both the reference component and the proposed component meet the minimum requirements for test data and design requirements. In addition, the stapled and nailed wood shear wall components can be isolated from the remainder of the seismic-force-resisting system by a clear component boundary.

D.4 Reference Component Test Data

D.4.1 Define the Reference Component Design Space

The reference component is a nailed wood shear wall with the following expected representative configurations:

- Wall height: 8’
- Aspect ratio (height/length): 1:1
- Sheathing:
  - OSB and STR1
  - 3/8” to 15/32”
- Nails: 8d and 10d hand driven common nails
- Nail spacing (on-center):
  - Edge spacing: 2” to 6”
  - Field spacing: 6” to 12”
- Openings: No openings are considered in the test data set. While openings are likely to occur in any wood light-frame building, the proposed component design requirements will require that only full height piers (without openings) be considered as part of the seismic force-resisting system. Accordingly, the proposed components will be tested without openings, so this reference component data set will be consistent.

D.4.2 Compile or Generate Reference Component Test Data

The reference component data set is established based on available published information. The reference component data set for this test application is based on work by Pardoen at the University of California at Irvine (COLA, 2001). This database includes 108 tests on 8’x8’ shear walls consisting of 36 configurations with three tests per configuration.

In this test application, tests that employ Sequential Phased Displacement (SPD) loading protocol (Porter, 1987) are used. This is done for consistency with the loading protocol used for testing of the proposed component. The final reference component data set includes a total of 63 tests (on 21 configurations) that represent the remaining data after removal of data on gypsum wallboard, screws, stucco, and sheathing on both sides of the panel.

The reference component data set generally requires sets of both cyclic-load and monotonic-load data (Section 2.2). In this example, the monotonic-load data set is not compiled because monotonic-load data are not available for the proposed component data set. As such, this example is incomplete. Even so, this example is continued with the assumption that all acceptance criteria associated with monotonic-load data are fulfilled (Section 2.8.5), and the quality rating of the test data (Section 2.7.1) is not degraded due to the exclusion of monotonic-load data.

D.4.3 Interpret Reference Component Test Results

The test data are interpreted in accordance with the requirements of Section 2.4.3 and Section 2.2 (with related illustrative examples in Section 3.2.2). The utilized force quantity is the horizontal shear force in the shear wall, and the displacement quantity is the lateral drift of the wall (lateral displacement divided by height) (Section 2.2). Per Section 2.2.2, the value of Δ_U would be affected if the vertical load carrying ability of the specimen were compromised. Modification of the value of Δ_U was not necessary in this example.

Table D-2 summarizes the configuration of each test specimen, as well as the observed failure modes. Table D-3 provides values of important response quantities, such as strength, stiffness, ductility, and deformation capacity, for each specimen of the reference component data set. According to the guidelines of Section 2.4.4, the data are placed into a single performance group.

Table D-2 Summary of Nailed Wood Shear Wall Configurations in the Reference Component Data Set

Test Index	Group	Wall Dimensions (HxL)	OSB Sheathing		Fastener	Fastener Spacing (edge/field)
1, 2, 3	1	8' by 8'	3/8"	STR I	8d hand driven common	6"/12"
4, 5, 6	2	8' by 8'	3/8"	STR I	8d hand driven common	4"/12"
7, 8, 9	3	8' by 8'	15/32"	STR I	10d hand driven common	6"/12"
10, 11, 12	4	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
13, 14, 15	5	8' by 8'	3/8"	STR I	8d hand driven common	4"/12"
16, 17, 18	9	8' by 8'	15/32"	STR I	10d hand driven common	2"/12"
19, 20, 21	10	8' by 8'	7/16"	OSB	8d hand driven common	4"/12"
22, 23, 24	11	8' by 8'	15/32"	OSB	10d hand driven common	6"/12"
25, 26, 27	12	8' by 8'	15/32"	OSB	10d hand driven common	4"/12"
28, 29, 30	13	8' by 8'	15/32"	OSB	10d hand driven common	2"/12"
31, 32, 33	22	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
34, 35, 36	23	8' by 8'	15/32 "	STR I	10d hand driven common	4"/12"
37, 38, 39	24	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
40, 41, 42	25	8' by 8'	3/8"	STR I	8d hand driven common	2"/12"

Table D-2 Summary of Nailed Wood Shear Wall Configurations in the Reference Component Data Set (continued)

Test Index	Group	Wall Dimensions (HxL)	OSB Sheathing		Fastener	Fastener Spacing (edge/field)
43 44 45	26	8' by 8'	15/32"	STR I	8d hand driven common	4"/12"
46 47 48	27	8' by 8'	3/8"	STR I	8d hand driven common	4"/6"
49 50 51	28	8' by 8'	3/8"	STR I	8d hand driven common	4"/6"
52 53 54	29	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
55 56 57	30	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
58 59 60	31	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"
61 62 63	32	8' by 8'	15/32"	STR I	10d hand driven common	4"/12"

Table D-3 Summary of Important Component Parameters for the Reference Component Data Set

Test Index	Strength			Stiffness			Ductility		Deformation Capacity
	V_M (lb)	V_D (lb)	$R_Q = V_M / V_D$	K_I (lb/in)	K_D (lb/in)	$R_K = K_I / K_D$	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)
1	5,299	2,240	2.4	15,019	10,481	1.4	0.004	5.4	0.020
2	5,481	2,240	2.4	11,849	10,481	1.1	0.005	4.0	0.019
3	5,632	2,240	2.5	14,649	10,481	1.4	0.004	5.2	0.021
4	7,433	2,240	3.3	13,879	23,903	0.6	0.006	4.2	0.024
5	7,214	2,240	3.2	15,161	23,903	0.6	0.005	5.1	0.025
6	8,144	2,240	3.6	17,002	23,903	0.7	0.005	4.5	0.022
7	6,567	2,720	2.4	18,020	14,246	1.3	0.004	6.8	0.026
8	6,509	2,720	2.4	16,558	14,246	1.2	0.004	6.2	0.025
9	7,136	2,720	2.6	16,599	14,246	1.2	0.004	5.9	0.026
10	9,770	4,080	2.4	18,276	19,300	0.9	0.006	n/a	n/a
11	9,331	4,080	2.3	21,438	19,300	1.1	0.005	5.0	0.022
12	8,337	4,080	2.0	16,020	19,300	0.8	0.005	5.1	0.028
13	6,417	3,440	1.9	15,202	14,842	1.0	0.004	n/a	n/a
14	6,879	3,440	2.0	17,444	14,842	1.2	0.004	4.4	0.018
15	6,774	3,440	2.0	16,150	14,842	1.1	0.004	5.2	0.023
16	15,450	6,960	2.2	25,806	30,969	0.8	0.006	4.5	0.028
17	14,718	6,960	2.1	20,649	30,969	0.7	0.007	n/a	n/a
18	15,362	6,960	2.2	26,146	30,969	0.8	0.006	4.7	0.029

Table D-3 Summary of Important Component Parameters for the Reference Component Data Set (continued)

Test Index	Strength			Stiffness			Ductility		Deformation Capacity
	V_M (lb)	V_D (lb)	$R_Q = V_M / V_D$	K_I (lb/in)	K_D (lb/in)	$R_K = K_I / K_D$	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)
19	6,784	3,040	2.2	26,079	13,466	1.9	0.003	6.7	0.018
20	5,727	3,040	1.9	29,593	13,466	2.2	0.002	6.9	0.014
21	5,735	3,040	1.9	28,749	13,466	2.1	0.002	6.5	0.014
22	4,234	2,480	1.7	23,651	13,677	1.7	0.002	8.4	0.016
23	4,583	2,480	1.8	22,915	13,677	1.7	0.002	7.2	0.015
24	4,567	2,480	1.8	22,256	13,677	1.6	0.002	7.7	0.016
25	8,371	3,680	2.3	31,967	18,457	1.7	0.003	8.2	0.023
26	8,515	3,680	2.3	29,687	18,457	1.6	0.003	6.9	0.021
27	8,635	3,680	2.3	34,538	18,457	1.9	0.003	7.7	0.020
28	13,986	6,160	2.3	37,323	30,692	1.2	0.004	n/a	n/a
29	15,273	6,160	2.5	32,303	30,692	1.1	0.005	5.4	0.027
30	15,275	6,160	2.5	30,689	30,692	1.0	0.005	4.4	0.023
31	8,855	4,080	2.2	22,834	19,300	1.2	0.004	5.6	0.022
32	8,560	4,080	2.1	22,096	19,300	1.1	0.004	5.5	0.022
33	9,096	4,080	2.2	24,271	19,300	1.3	0.004	6.0	0.023
34	8,836	4,080	2.2	22,196	19,300	1.2	0.004	6.1	0.025
35	9,391	4,080	2.3	25,056	19,300	1.3	0.004	5.9	0.023
36	8,642	4,080	2.1	20,970	19,300	1.1	0.004	4.3	0.018
37	9,807	4,080	2.4	21,793	19,300	1.1	0.005	n/a	n/a
38	10,512	4,080	2.6	25,254	19,300	1.3	0.004	4.5	0.019
39	10,012	4,080	2.5	22,286	19,300	1.2	0.005	5.1	0.024
40	14,037	5,840	2.4	20,816	28,092	0.7	0.007	3.1	0.022
41	13,776	5,840	2.4	21,833	28,092	0.8	0.007	3.2	0.021
42	14,815	5,840	2.5	23,255	28,092	0.8	0.007	3.2	0.021
43	7,797	3,440	2.3	19,812	20,027	1.0	0.004	n/a	n/a
44	7,832	3,440	2.3	17,440	20,027	0.9	0.005	n/a	n/a
45	7,389	3,440	2.1	22,905	20,027	1.1	0.003	n/a	n/a
46	7,099	3,440	2.1	20,075	20,027	1.0	0.004	4.6	0.017
47	6,626	3,440	1.9	21,281	20,027	1.1	0.003	5.2	0.017
48	7,605	3,440	2.2	20,972	20,027	1.0	0.004	4.4	0.016
49	7,771	3,440	2.3	17,268	20,027	0.9	0.005	3.5	0.016
50	8,206	3,440	2.4	20,194	20,027	1.0	0.004	3.8	0.016
51	8,245	3,440	2.4	21,987	20,027	1.1	0.004	4.5	0.018
52	8,513	4,080	2.1	21,283	19,300	1.1	0.004	5.3	0.022
53	8,498	4,080	2.1	20,926	19,300	1.1	0.004	n/a	n/a
54	8,511	4,080	2.1	18,654	19,300	1.0	0.005	n/a	n/a
55	9,157	4,080	2.2	22,891	19,300	1.2	0.004	n/a	n/a
56	9,177	4,080	2.2	20,699	19,300	1.1	0.005	n/a	n/a
57	9,768	4,080	2.4	20,650	19,300	1.1	0.005	n/a	n/a
58	8,981	4,080	2.2	11,579	19,300	0.6	0.008	2.1	0.017
59	9,137	4,080	2.2	13,279	19,300	0.7	0.007	3.1	0.022
60	8,746	4,080	2.1	7,547	19,300	0.4	0.012	1.9	0.023
61	8,744	4,080	2.1	9,742	19,300	0.5	0.009	2.5	0.023
62	9,012	4,080	2.2	6,755	19,300	0.4	0.014	n/a	n/a
63	8,720	4,080	2.1	12,803	19,300	0.7	0.007	3.3	0.023
Median:	$\tilde{R}_Q = 2.3$			$\tilde{R}_K = 1.04$			$\tilde{\mu}_{eff} = 4.8$		$\tilde{\Delta}_U = 0.021$
Variability:	$\sigma_{RQ} = 0.13$			$\sigma_{RK} = 0.36$			$\sigma_{\mu,eff} = 0.33$		$\sigma_{\Delta U} = 0.19$

D.4.4 Define Reference Component Performance Groups

In accordance with Section 2.4.4, the reference component data were all placed into a single performance group.

D.4.5 Compute Summary Statistics

The summary statistics for each component parameter are computed in accordance with Section 2.4.5, assuming an underlying lognormal distribution of the data. Table D-4 presents the summary statistics for the data set, which is information repeated from the bottom of Table D-3.

Table D-4 Summary Statistics for the Reference Component Parameters

Summary Statistic	R_Q $(=V_M/V_D)$	R_K $(=K_I/K_D)$	μ_{eff}	Δ_U (in/in)
Median	$\tilde{R}_Q = 2.3$	$\tilde{R}_K = 1.04$	$\tilde{\mu}_{eff} = 4.8$	$\tilde{\Delta}_U = 0.021$
Variability	$\sigma_{RQ} = 0.13$	$\sigma_{RK} = 0.36$	$\sigma_{\mu,eff} = 0.33$	$\sigma_{\Delta U} = 0.19$

D.5 Proposed Component Design Requirements

Design requirements for stapled wood shear walls are based on a combination of requirements in 2006 IBC *International Building Code* (ICC, 2006) and ANSI/AF&PA SDPWS-2005 *Special Design Provisions for Wind and Seismic Standard with Commentary* (AF&PA, 2005), with detailing requirements proposed by Talbot et al. (2009) and additional requirements pertaining to the connections, construction, inspection, and maintenance.

In an actual application of the Component Methodology, code requirements for the proposed component may not exist, and such requirements would need to be developed.

D.5.1 Component Design Strength and Stiffness

The proposed allowable stress design (ASD) strength of the stapled wood shear walls is taken from the provisions of the 2006 IBC. Based on IBC Table 2306.4.1, the ASD design shear strength is 395 lb/ft for a blocked shear wall with 1½" 16 gauge staples which are spaced at 2" on-center and have larger than 1" penetration. This 395 lb/ft design strength applies only to walls with a maximum aspect ratio of 2:1 (i.e., 4', 8', and 12' long walls). Walls 3 feet in length exceed the 2:1 aspect ratio limit, and have a lower design shear strength of 296 lb/ft, which accounts for the 25% strength reduction in Section 4.3.4 of ANSI/AF&PA SDPWS-2005.

Design requirements are also needed for the prediction of initial stiffness. For stapled wood shear walls, Equation 23-2 from 2006 IBC is used here for computing the wall displacement at the design load, as shown below. Another viable option would be to consider the use of Equation 4.3-1 of ANSI/AFPA SDPWS-2005.

$$\Delta = \frac{8vh^3}{Eab} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$

where:

a = area of boundary element (in^2)

b = wall width (feet)

d_a = vertical elongation of overturning anchorage at the design shear load (v) (inches)

E = elastic modulus of boundary element (pounds/inch/inch)

e_n = staple or nail deformation (inches)

Gt = panel rigidity through the thickness (pounds/inch)

h = wall height (feet)

v = maximum shear due to design loads, applied at the top of the wall (pounds/foot)

The IBC requirements only contain e_n values for 2-inch, 14 gauge staples (IBC Table 2305.2.2(1)). To fulfill the stiffness-related requirements of the acceptance criteria in Chapter 2, it is proposed that the e_n values for 2-inch, 16 gauge staples be a factor of 2.0 larger than the values in the IBC table for 14 gauge staples. This 2.0 factor is a calibrated value based on test data, such that the stiffness ratio $\tilde{R}_{K,PC}/\tilde{R}_{K,RC}$ is equal to unity.

D.5.2 Component Detailing Requirements

The stapled wood shear wall design requirements adopt many detailing requirements proposed by Talbot et al. (2009). The stapled wood shear wall test data utilized in Section D.6 are consistent with the use of these detailing requirements.

The detailing requirements for the stapled wood shear wall components are summarized as follows:

- Douglas fir-larch 2x4 studs spaced at 16" on-center (or 18" on-center for the 8'x3' wall).
- Top and sill plates consist of double studs.
- Two rows of staples along perimeter of double studs (4" on-center providing the equivalent 2" spacing).

- Simpson Strong-Tie RSP4 type 1 connectors (SST, 2009) used for stud to sill plate connection (see Figure D-2). This detail was used in the tests to simulate the effects of vertical loads (to prevent uplift of studs from sill plate) so this detail would not be required for site-built shear walls that are expected to carry vertical load.
- Simpson Strong-Tie PHD5 hold-downs (SST, 2009) (see Figure D-2).
- Backup anchor bolt (see Figure D-2).

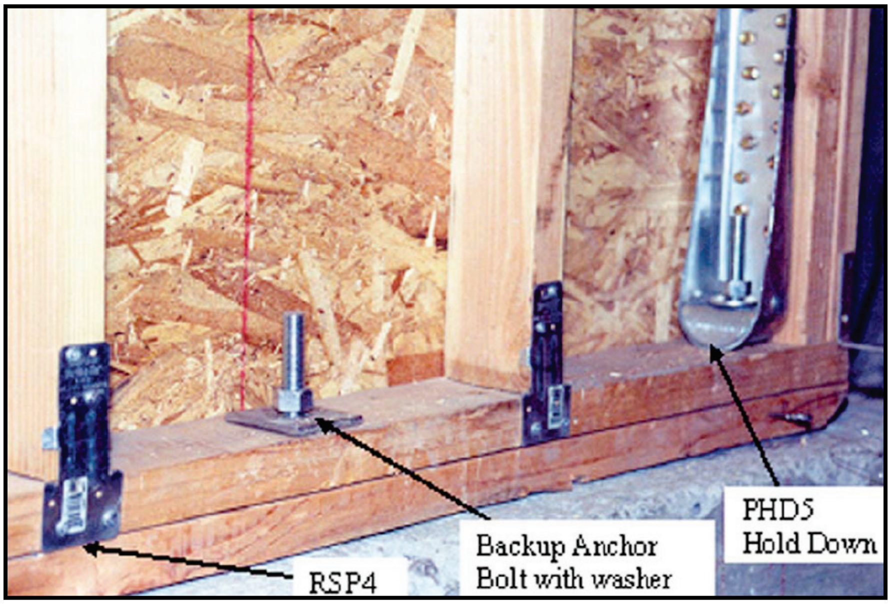


Figure D-2 Illustration of several stapled wood shear wall detailing requirements, including hold-down, stud-to-sill plate connector, and backup anchor bolt. Photo from Talbot et al. (2009).

- Vertical studs (3x3) at adjoining panel edges (these are only needed for the 8'x12' wall).
- End post detail to minimize bending of end post with Simpson Strong-Tie flat strap ST6224 connecting end post to bottom sill plate (see Figure D-1a):
  - 45-degree cut at end of the sill plate bottom stud, done to reduce slack in the strap and reduce crushing of the sill plate bottom stud (see Figure D-1a and the right side of Figure D-2).
  - Strap attached by anchor bolt (on bottom) and 16 to 18 Simpson Strong-Tie N10 nails (SST, 2009).
- Blocking details:
  - Douglas fir-larch 2x4 blocks cut 14" long and placed with the longer dimension against the sheathing joint (see Figure D-1d). Blocking is secured by two rows of staples (one on each panel) at 2" on-center.

This detailing requirement applies only to the 8’x3’, 8’x 4’, and 8’x8’ walls.

- Special provisions for the blocking region of the 8’x12’ wall, requiring four rows of staples (two on each panel) at 2” on-center. Note that this detailing requirement was changed in the middle of the testing program, and the new detail was only utilized for wall numbers 8C and 12A-12C. Talbot et al. (2009) propose this detailing requirement for all wall geometries.

D.5.3 Component Connection Requirements

Both the framing members of the stapled wood shear wall and the connections between the wall and the rest of the structural system are designed to resist induced seismic forces per the design methods listed in Section 2.1.2 of ANSI/AF&PA SDPWS-2005. Boundary elements of the wall, such as end studs, are also designed to transfer design tension and compression forces to the rest of the structural system.

Though not explicitly demonstrated in this example, it is assumed that these connection requirements ensure that the inelastic behavior occurs in the proposed shear wall components and not in the connections between the wall and the balance of the seismic-force-resisting system.

D.5.4 Limitations on Component Applicability and Use

The Component Methodology requires that the configurations of the proposed stapled wood shear wall component be reasonably restricted to the range of configurations that were considered in the testing program (i.e., usage must be restricted to the component design space). Accordingly, the stapled wood shear wall component can only be used within the range of configurations outlined in Section D.6.1.

Additionally, since the tests were completed without applied vertical loads, the proposed component design provisions include a restriction to the use of the component to ensure that applied vertical loads would not tangibly affect the behavior and performance of the component. In this example, the current height limits in ASCE/SEI 7-10 (i.e., 65’ in high seismic regions) were deemed sufficient to meet the requirements of Section 2.5.4.

D.5.5 Component Construction, Inspection, and Maintenance Requirements

The construction, inspection, and maintenance requirements for stapled wood shear walls are proposed to be consistent with the general requirements for nailed wood shear walls contained in Chapter 23 of the 2006 IBC.

D.6 Proposed Component Test Data

The work by Talbot et al. (2009) outlined in Section D.2 is used as the source of data for the proposed stapled wood shear wall component.

D.6.1 Define Proposed Component Design Space

The proposed component design space includes walls that conform to the detailing requirements of Section D.5.2 with the following range of configurations:

- Wall height and length: 8'x3' to 8'x12'
- Aspect ratio (height/length): 2.67:1 to 0.67:1. This range is broader than the reference component data set, but such differences are permitted when the transfer of forces across the component boundary is not significantly affected.
- Sheathing: 7/16" OSB, Exposure 1
- Staples: 2" long, 1/2" crown, 16 gauge galvanized
- Staple spacing (on-center): 2" edge, 4" field
- Openings: No openings are included in test specimens. While openings are expected in wood light-frame buildings, since only full height piers (without openings) were tested, only full height piers will be considered as part of the seismic-force-resisting system.

D.6.2 Select Component Configurations for Testing

The component configurations selected for testing should span the component design space, in accordance with Section 2.6.2. In this example, the configurations tested by Talbot et al. (2009) are utilized. Within these configurations, the only variable is wall length, which is varied to 3', 4', 8', and 12'. For a more complete evaluation of stapled wood shear walls, additional variables such as staple spacing, staple length, sheathing thickness, can be included.

D.6.3 Perform Cyclic-Load and Monotonic-Load Tests

The Component Methodology requires that both cyclic-load and monotonic-load data sets are provided for the proposed component. In this test example, only the cyclic-load data set is utilized, and monotonic-load data are not compiled because monotonic-load data are not available. This test application assumes that all acceptance criteria associated with monotonic-load data are fulfilled, and the quality rating of the test data has not been degraded due to the exclusion of monotonic-load data.

The Talbot et al. (2009) tests were conducted at a rate of one cycle per second (with a sampling rate of 50 samples/second) utilizing the SPD loading protocol. The nailed wood shear wall tests were done with lower strain rates than the stapled shear wall tests, but this is judged to be acceptable according to the requirements of Section 2.2. Figure D-3 shows an example of the cyclic response for specimen 8C, which is an 8'x8' wall. The stapled wood shear wall data was interpreted in the same manner as the nailed wood shear wall data.

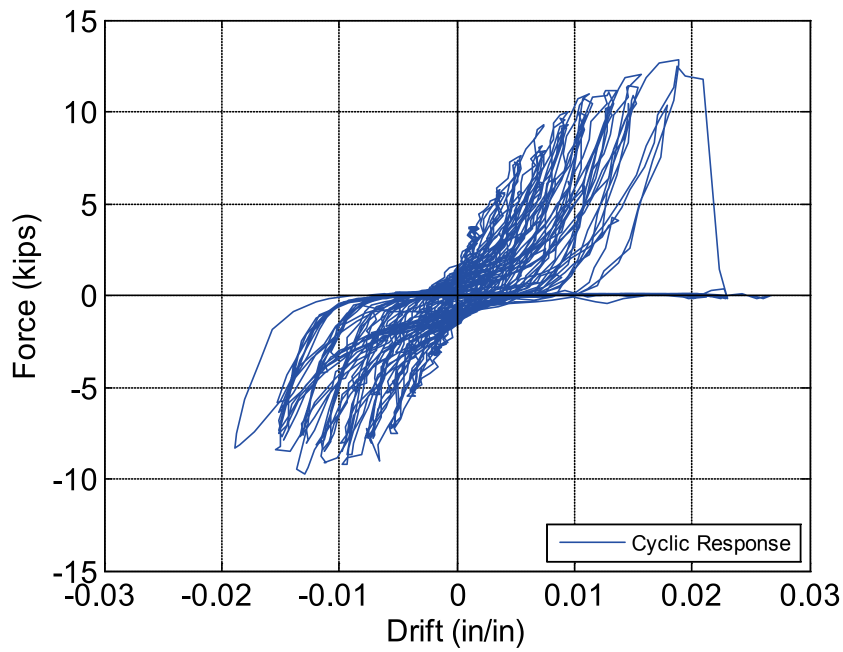


Figure D-3 Illustration of cyclic response of stapled wood shear wall specimen 8C (Talbot et al., 2009).

D.6.4 Interpret Proposed Component Test Results

Data were compiled for each of the 13 stapled wood shear wall tests and Table D-5 presents important component parameters for each of the tests.

The data in Table D-5 technically fulfill the minimum test data requirements of Section 2.2.2, which specifies that a minimum of four configurations (with at least two specimens per configuration) are needed for each component performance group. Even so, the proposed component design space that is defined in Section D.6.1 and sampled by these component configurations is extremely narrow. The component configurations adequately sample this narrow design space, but additional component configurations would still be desirable to provide more confidence that the test results accurately reflect the behavior of the design space for stapled wood shear walls.

Table D-5 Summary of Important Component Parameters for Proposed Component Data Set

Wall Number	Strength			Stiffness			Ductility		Deformation Capacity	
	V_M (lb)	V_D (lb)	$R_Q = V_M / V_D$	K_I (lb/in)	K_D (lb/in)	$R_K = K_I / K_D$	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)	$\tilde{\Delta}_{Uj}$
3-A#1	2,900	888	3.3	5,015	4,307	1.2	0.006	3.7	0.022	0.025
3-A#2	3,900	888	4.4	2,714	4,307	0.6	0.015	1.7	0.026	
3-B	3,900	888	4.4	4,603	4,307	1.1	0.009	2.8	0.025	
3-C	3,700	888	4.2	3,085	4,307	0.7	0.012	2.3	0.029	
4-A	5,500	1,580	3.5	4,461	6,463	0.7	0.013	1.8	0.024	0.021
4-B	5,200	1,580	3.3	4,384	6,463	0.7	0.012	1.6	0.020	
4-C	4,800	1,580	3.0	7,038	6,463	1.1	0.007	2.7	0.019	
8-A	8,700	3,160	2.8	16,734	13,461	1.2	0.005	2.4	0.013	0.013
8-B	10,100	3,160	3.2	20,694	13,461	1.5	0.005	2.6	0.013	
8-C[1]	11,300	3,160	3.6	21,917	13,461	1.6	0.005	2.9	0.016	--
12-A[1]	21,700	4,740	4.6	27,079	20,474	1.3	0.008	2.4	0.020	0.019
12-B[1]	19,900	4,740	4.2	21,239	20,474	1.0	0.010	1.7	0.017	
12-C[1]	22,400	4,740	4.7	26,786	20,474	1.3	0.009	2.2	0.019	
Median:	$\tilde{R}_Q = 3.7$			$\tilde{R}_K = 1.04$			$\tilde{\mu}_{eff} = 2.3$		$\tilde{\Delta}_U = 0.020$	
Variability:	$\sigma_{RQ} = 0.18$			$\sigma_{RK} = 0.32$			$\sigma_{\mu,eff} = 0.25$		$\sigma_{\Delta U} = 0.25$	

Note 1: Twice as many staples in the blocking region.

D.6.5 Define Proposed Component Performance Groups and Compute Summary Statistics

The proposed component data are all placed into a single performance group, consistent with the requirements of Section 2.6.5, and also consistent with the approach taken for the reference component data set. Table D-6 presents the summary statistics for the stapled wood shear wall test data.

It should be noted that the design requirements change slightly between various wall configurations in the data set, with the 3' wide wall having a slightly different design strength (due to an aspect ratio exceeding 2:1), and the 12' wall having an additional detailing requirement of twice the staples in the blocking region. Although strength and stiffness values differ between various wall configurations, these differences were considered to be minor so that the test data need not be separated into multiple performance groups.

Table D-6 Summary Statistics for Proposed Component Parameters

Summary Statistic	R_Q $(=V_M / V_D)$	R_K $(=K_I / K_D)$	μ_{eff}	Δ_U (in/in)
Median	$\tilde{R}_Q = 3.7$	$\tilde{R}_K = 1.04$	$\tilde{\mu}_{eff} = 2.3$	$\tilde{\Delta}_U = 0.020$
Variability	$\sigma_{RQ} = 0.18$	$\sigma_{RK} = 0.32$	$\sigma_{\mu,eff} = 0.25$	$\sigma_{\Delta U} = 0.25$

D.7 Evaluate Quality Ratings

D.7.1 Quality Rating of Test Data

According to the requirements of Section 2.7.1, the quality of the reference component test data is rated as Superior. This comes from a high rating for completeness and robustness and a high rating for confidence in the test results. The high rating for completeness and robustness is based on all of the important testing issues being addressed and the expectation that all important failure modes were uncovered in the testing. The high rating for confidence in the test results is based on the large number of test data, as well as multiple researchers and labs being involved in completing the testing.

The quality of the proposed component test data is rated as Good, based on a high rating for completeness and robustness, and a medium rating for confidence in the test results. The medium confidence rating is based on the limited number of tests – only four configurations and 13 specimens, and the fact that the same researcher and lab completed all of the tests used for evaluation. Note that data from Talbot et al. (2009) are limited with respect to the Component Methodology (e.g., numbers of tests) because the experimental program was developed for other purposes.

As noted previously, the data set does not include monotonic-load data, which is required for the proposed component. For the purpose of this test application, this shortcoming in available data has been overlooked, and was not considered in the quality rating of the test data.

D.7.2 Quality Rating of Design Requirements

According to the requirements of Section 2.7.2, the quality of the reference component design requirements is rated as Good. This slightly lower rating is due to nailed wood shear walls having less stringent detailing requirements, as compared with the extensive detailing requirements used in the design of stapled wood shear walls.

The quality of the proposed component design requirements is rated as Superior. This is based on high ratings for both completeness and robustness and confidence in the design requirements, based on the extensive detailing requirements being enforced for the stapled wood shear walls (see Section D.5.2), as well as the substantial amount of experience and mature construction practices that exist for nailed wood shear wall components.

D.8 Evaluate Component Equivalency

D.8.1 Overview

This section discusses the acceptance criteria of the Component Methodology and evaluates the possible equivalency between stapled and nailed wood shear walls.

D.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation

The median ultimate deformation of each proposed component performance group is required to fulfill Equation 2-1, as follows:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q$$

When evaluating the above equation, the median ultimate deformation, $\tilde{\Delta}_{U,RC}$, from the reference component data set is 0.021 (from Table D-4). The uncertainty penalty factor, P_U, is based on the quality ratings discussed in Section D.7.1. Incorporating these quality ratings into Table D-7 shows that the uncertainty penalty factor, P_U, equals 1.00.

The penalty factor for strength is based on the median proposed and reference component strength ratios, as follows:

$$\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} = \frac{3.7}{2.3} = 1.6$$

Linear interpolation based on Table D-8 results with strength penalty factor, P_Q, as 1.17. Incorporating these values leads to the following requirement check, showing that the median ultimate deformation value of 0.020 does not meet this requirement.

$$0.020 < (0.021)(1.00)(1.17) = 0.025 \qquad \underline{\underline{FAIL}}$$

Table D-7 Penalty Factor to Account for Uncertainty (from Table 2-3)

Penalty Factor for Uncertainties (P_U)			
Quality Rating of Proposed Component Test Data	Quality Rating of Proposed Component Design Requirements Relative to Reference Component Design Requirements		
	Higher	Same	Lower
Superior	0.95	1.00	1.15
Good	1.00	1.05	1.25
Fair	1.15	1.25	1.40

Table D-8 Penalty Factor to Account for Difference in Component Strengths (from Table 2-4)

Penalty Factor for Differences in Strength (P_Q)			
$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q	$\tilde{R}_{Q,PC}/\tilde{R}_{Q,RC}$	P_Q
0.50	1.88	1.10	1.00
0.60	1.55	1.20	1.00
0.70	1.31	1.30	1.04
0.80	1.14	1.40	1.09
0.90	1.00	1.50	1.13
1.00	1.00	1.80	1.24
1.10	1.00	2.00	1.32

The median value of ultimate deformation for each geometric configuration, $\tilde{\Delta}_{Uj,PC}$, must meet the requirement of Equation 2-2, as shown below. According to Table D-4, the variability in ultimate deformation, $\sigma_{\Delta U,RC}$, is 0.19 for the reference component data.

Since the ratio of the median and proposed reference component strengths is greater than 1.2 but not greater than 2.0, the exception of Section 2.8.2 would be invoked. Accordingly, provisions would need to be added to the proposed component design requirements stating that the force-controlled and capacity-designed components of the reference SFRS must be designed for the expected strength of the proposed components (i.e., the component design strength scaled up by a factor of $\tilde{R}_{Q,PC}$).

$$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right)P_U P_Q$$

$$\tilde{\Delta}_{Uj,PC} \geq \left[1-1.5(0.19)\right](0.021)\cdot(1.00)(1.17) = 0.018$$

Table D-9 summarizes the above acceptance criteria checks being applied to the full performance group and to each individual geometric configuration.

The stapled wood shear walls fail the basic ultimate deformation acceptance criterion for the full performance group; they also fail the individual configuration check for the 8'x8' panel configuration.

Table D-9 Evaluation of Equivalency Acceptance Criteria for Stapled Wood Shear Walls

Wall Number	Deformation Capacity		Acceptance Check	
	Δ_U	Median Δ_U And $\tilde{\Delta}_{Uj}$	Accept. Criteria	Pass/ Fail
Perf. Group I:		$\tilde{\Delta}_U = 0.020$	**0.025**	**FAIL**
3-A#1	0.022	0.025	0.018	**Pass**
3-A#2	0.026			
3-B	0.025			
3-C	0.029			
4-A	0.024	0.021	0.018	**Pass**
4-B	0.020			
4-C	0.019			
8-A	0.013	0.013	0.018	**FAIL**
8-B	0.013			
8-C	0.016	--	--	--
12-A	0.020	0.019	0.018	**Pass**
12-B	0.017			
12-C	0.019			

D.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

To fulfill the requirements of Section 2.8.3 (Equation 2-3), design provisions for initial stiffness were calibrated based on the observed stiffness values of the proposed and reference components. In this example, this was done by adjusting the e_n value for staple slip in the design displacement equation, as discussed in Section D.5.1. By completing the calibration of the displacement prediction equation, the ratio was made equal unity. A value of 1.0 easily falls within the range of 0.75 to 1.33 determined by Equation 2-3, so the design stiffness provisions fulfill this acceptance criterion.

$$0.75 \le \frac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \le 1.33$$

$$0.75 \le \frac{1.04}{1.04} \le 1.33 \qquad \underline{\underline{OK}}$$

D.8.4 Requirements Based on Cyclic-Load Test Data: Effective Ductility Capacity

In order to fulfill the requirements of Section 2.8.4 (Equation 2-4), and to ensure at least rough parity between the post-yield deformation capacity (and energy dissipation capacity) of the proposed and reference components, the median ductility capacity of the proposed components, $\tilde{\mu}_{eff,PC}$, must be at least half as large as the median ductility capacity of the reference components, $\tilde{\mu}_{eff,RC}$. In this example, $\tilde{\mu}_{eff,PC} = 2.3$ and $\tilde{\mu}_{eff,RC} = 4.8$, so this requirement is not met.

Because Equation 2-4 was not fulfilled, the exception in Section 2.8.4 would be invoked in this example, and the design requirements would need to limit the lateral force in proposed component to less than 25% of the design seismic force along any framing line in the building.

D.8.5 Requirements Based on Monotonic-Load Test Data: Ultimate Deformation

As discussed previously, monotonic-load test data for stapled wood shear wall components were not available, so monotonic criteria were not evaluated.

D.8.6 Summary of Component Equivalency Evaluation

In summary, the stapled wood shear wall components passed the acceptance criteria related to initial stiffness but failed the criteria related to ultimate deformation capacity. Stapled wood shear wall components also failed the ductility capacity criterion, but this can be addressed though modification of design requirements.

D.9 Iteration: Evaluate Component Equivalency with Modifications

In order to demonstrate equivalency stapled wood shear wall design requirements would need to be modified, or stapled wood shear wall test data would need to be expanded. Possible alternative approaches include the following:

- Changes to reduce the strength penalty factor, P_Q, to 1.0. One approach would be to refine the stapled wood shear wall design strength requirements (Sections 2.5.1 and D.5.1). Since the proposed component is stronger than the reference component, another approach would be to modify the stapled wood shear wall design provisions to prohibit mixing of the reference and proposed components in a building (Sections 2.8.2 and D.5.4).

- Generation of more stapled wood shear wall data to achieve a data quality rating of Superior and a resulting value of $P_U = 0.95$.
- Identification of the behavioral problem with the 8'x8' panel configuration, fix the problem, and perform more 8'x8' panel testing to prove that the problem has been resolved.
- Improvement in stapled wood shear wall detailing requirements to achieve larger ultimate deformations in tested wall components.

Data in this chapter suggest that stapled wood shear wall components could pass the component equivalency criteria if the above items were implemented.

D.10 Summary of Component Equivalency Evaluation of Stapled Wood Shear Walls

Possible equivalency between stapled wood shear walls and nailed wood shear walls was evaluated. Due to limitation in available data this application focused on a narrow range of possible stapled wood shear wall configurations. Additionally, monotonic-load test data were not available. Results are summarized in Table D-10. Overall, stapled wood shear wall components were not found to be equivalent to nailed wood shear wall components based on the criteria of the Component Methodology.

Table D-10 Summary of Acceptance Criteria Evaluation for Proposed Stapled Wood Shear Component

Acceptance Criteria		Equation Reference	Pass/Fail
Requirement Based on Cyclic-Load Test Data			
Ultimate Deformation Capacity (performance group)	$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q$	2-1	Fail
Ultimate Deformation Capacity (individual configurations)	$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \; P_Q$	2-2	Pass 3, Fail 1
Initial Stiffness Ratio	$0.75 \leq \dfrac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33$	2-3	Pass
Effective Ductility Capacity	$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC}$	2-4	Fail
Requirements Based on Monotonic-Load Test Data			
Ultimate Deformation Capacity (Option 1)	$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC} \; P_U \; P_Q$	2-5	n/a
Ultimate Deformation Capacity (Option 2)	$\tilde{\Delta}_{UM,PC} \geq 1.2\,\tilde{D}_C \; \tilde{\Delta}_{U,RC} \; P_U \; P_Q$	2-6	n/a

Appendix E
Test Application: Buckling-Restrained Brace Components

E.1 Introduction

This appendix illustrates the application of the Component Methodology to the possible substitution of buckling-restrained braces for conventional braces in a special steel concentrically-braced seismic-force-resisting system. The objective is to determine whether or not equivalency based on comparison of test data for the proposed component (buckling-restrained braces) and the reference component (conventional braces) can be achieved. This test application assumes that buckling-restrained braces (BRBs) are not currently approved, and that they are proposed as an alternative to conventional braces in the code-approved special steel concentrically braced frame system listed in Table 12.2-1 of ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). Seismic design coefficients for the reference system are $R = 6$, $C_d = 5$, and $\Omega_o = 2$.

The findings of this test application are presented with two caveats. First, differences between buckling-restrained braces and conventional braces may warrant a system-level evaluation approach, as outlined in the FEMA P-695 Methodology, rather than the component-level approach of the Component Methodology. A detailed evaluation of system behavior may be appropriate because of: (1) differences in the transfer of forces from the component to the system for BRBs as compared with conventional braces; and (2) the potentially important role of beams, columns and connections in the overall behavior of braced frame systems. Second, available data are limited, and conventional brace (reference component) data do not reflect the full range of the possible brace designs and properties and may represent a lower bound on brace ductility and performance. More data are needed for a complete evaluation.

The steps in this test application follow the requirements of the Component Methodology defined in Section 2.2 through Section 2.8.

E.2 Description of Buckling-Restrained Braces

In steel braced frame seismic-force-resisting systems lateral forces create axial forces in brace components, and energy is dissipated through the hysteretic response of the braces. Conventional braces typically have significant strength and deformation capacity under tensile loading, but can buckle in compression, leading to a reduction in compressive load-carrying capacity and energy dissipation in the member. Under repeated cycles of loading, braces can fracture, leading to a redistribution of loads to the beams and columns in the framing system. On the other hand, buckling-restrained braces (BRBs), in use in Japan since the mid-1980s (Black et al., 2002), have exhibited nearly symmetric behavior under tension and compression loads. Failure can occur if deformations are large enough to fracture the core plate or exceed compression deformation limits. Figure E-1 shows typical force deformation response from cyclic loading on conventional and buckling-restrained braces.

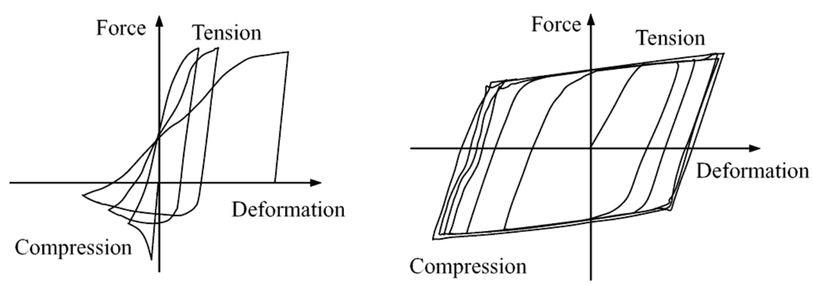


Figure E-1 Conventional (left) and buckling-restrained (right) braces under cyclic loading (from Kumar et al., 2007).

Steel braces can be made of wide-flange or HSS sections with bolted or welded gusset plate connections to beams and columns. Buckling-restrained braces consist of a ductile steel core that carries the entire axial load in the member with a mortar- or concrete-filled steel tube confining the core. The concrete in the tube restrains the steel core from buckling, but is prevented from bonding with the core so that axial loads cannot be transferred to outer steel tube. Isolation of the steel core is accomplished with a layer of unbonding material. Figure E-2 illustrates the different parts of a BRB.

This test application considers buckling-restrained braces designed according to AISC 341-10 *Seismic Provisions for Structural Steel Buildings* (AISC, 2011). These design provisions ensure that the braces are able to permit axial elongation and shortening in the steel core up to deformations corresponding to twice the design story drift. The design strength is based on the size and strength of the steel core material, and must be sufficient to resist relevant load combinations. In order to show that a buckling-restrained brace meets the requirements for strength and inelastic deformation, individual braces and brace assemblages are required to undergo cyclic qualification testing according to Section K3 of AISC 341-10. Design of brace connections and adjoining members of the braced frame is based on the maximum strength of the buckling-restrained brace. Buckling-restrained braces are not designed to be part of the gravity framing system.

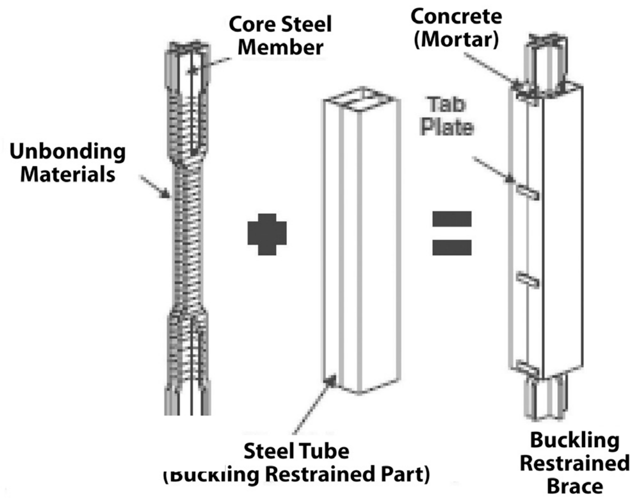


Figure E-2 Features of a typical buckling-restrained brace (from Tsai and Hsiao, 2008).

E.3 Evaluation of Applicability Criteria

The Component Methodology applicability criteria address the suitability of the reference SFRS, the adequacy of the reference and proposed component design criteria and test data, and the characteristics of the proposed component. The adequacy of the design criteria and test data for the reference and proposed component data are assumed to be sufficient for the purpose of establishing equivalency in the Component Methodology.

An important part of evaluating the applicability of the Component Methodology is the definition of the component boundary, which separates the component of interest from the seismic-force-resisting system. As illustrated in Figure E-3, the reference component is defined as a pair of braces. Pairs of braces (rather than individual braces) were selected because conventional braces have very different behavior in the tensile and compressive directions, and a single brace (acting in either tension or compression) will not have the same seismic response as a pair of braces. Furthermore, AISC 341-10 requires that in special steel concentrically braced frames "along any line of bracing braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30%, but no more than 70% of the total horizontal force is resisted by tension braces." Although there are various design alternatives that meet this requirement, a pair of braces with one brace acting in compression and one acting in tension is a commonly used solution. The compressive strength of braces with slenderness ratios used in special concentrically braced frames have approximately 30% to 40% of the strength of a pair of tension and compression braces.

Defining the reference component as a pair of braces ensures that the component boundary and transfer of forces across the component boundary are as similar as possible for reference and proposed components. Based on this definition, it is assumed that the reference component can be tested either in pairs of braces (e.g., X-brace configurations) or as single braces assumed to resist lateral forces in parallel with another identical single brace.

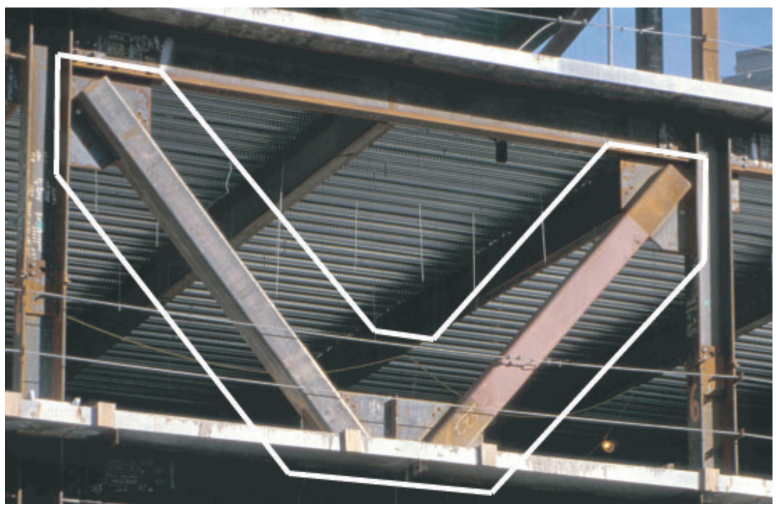


Figure E-3 Illustration of reference component and definition of component boundary (Engelhardt, 2007).

In this test application, the proposed component is defined as a single buckling-restrained brace and its end connections, as shown in Figure E-4. Provided that gusset plates and connections are designed for the maximum brace force, inelastic action is expected to occur in the BRB.

With the component boundaries of proposed and reference components defined, the balance of seismic-force-resisting system is essentially unchanged by the replacement of reference components with proposed components. In both cases, the inelastic deformation occurs primarily in the braces through compression buckling or tension and compression yielding.

In actuality, buckling-restrained and conventional braces may induce significantly different axial forces in columns because of differences in configuration and angle between the proposed and reference component. In addition, beams, columns, and beam-column connections may influence structural response. These differences are set aside in order continue with the test application. Nevertheless, there are significant questions as to whether the applicability criteria are fully satisfied in this example.

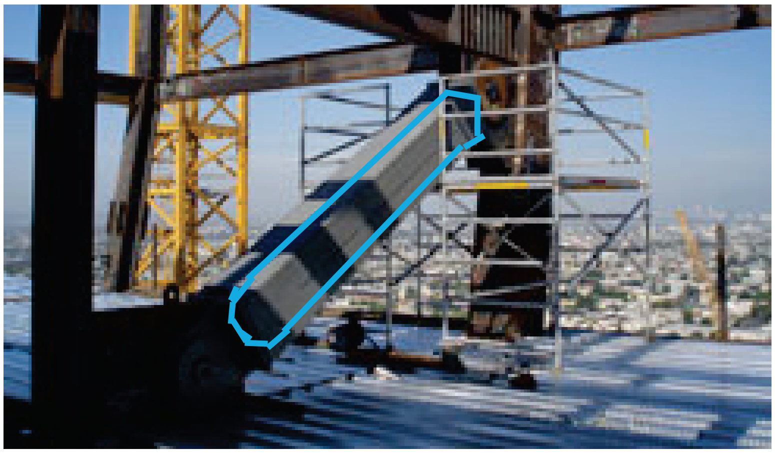


Figure E-4 Illustration of proposed component and definition of component boundary (Photo from Star Seismic).

E.4 Reference Component Test Data

This section describes the conventional brace reference component test data set compiled specifically to include braces that may be employed in the special steel concentrically braced frame system.

E.4.1 Define Reference Component Design Space

In this test application, the reference component design space is intended to represent the range of possible braces in the special steel concentrically braced frame system. The design space includes braces of varying size and strength, and braces with different sections, including wide-flange, angle, tube, and others. While brace detailing, brace slenderness, or width-to-thickness ratios may also have a critical influence on the response, and should be considered, special steel concentrically braced frames must satisfy limitations on width-to-thickness ratios for highly ductile members such that the range of sections that can be used is significantly restricted (AISC, 2011). The component design space is intended to represent conventional brace systems with any of the approved connection types, such as welded or bolted gusset plate connections.

The reference component design space includes a variety of system configurations, including diagonal, chevron, V-brace, and X-brace configurations. However, since it is assumed that the primary configuration issue is the number of braces acting in tension and compression, the single-brace and X-brace data collected are taken to be representative of the commonly used brace configurations. K-brace configurations are prohibited in special steel concentrically braced frames, and have been excluded from the reference component data set.

The reference component data set also has limited variation in section properties, size, and configuration such that it does not fully represent the full reference component design space. These limitations affect the test data quality ratings that are assigned.

E.4.2 Define Reference Component Performance Groups

Within the reference component design space, differences in brace slenderness, brace configuration, or section properties may lead to different failure modes and load-deformation responses. Such fundamental differences in behavior could lead to the assigment of different performance groups. However, performance groups defined for proposed and reference components should be compatible. Since conventional and buckling-restrained braces have different failure modes and component strengths (e.g., buckling-restrained braces are often strong enough to replace several conventional braces), it is difficult to divide reference and proposed components into an analogous set of performance groups that can be systematically compared. Therefore, all buckling-restrained braces are seen as a potential substitute for conventional braces in the special steel concentrically braced frame system, allowing all conventional braces to be placed in a single performance group. In addition, all buckling-restrained braces are also placed in a single proposed component performance group.

E.4.3 Compile or Generate Reference Component Test Data

In this test application, data on "special" or similar conventional braces are obtained from Johnson (2005), Kotulka (2007), Clark, (2009) and Powell (2010). Much of the available test data, however, including Astaneh-Asl et al. (1982), El-Tayem and Goel (1985), Aslani and Goel (1989), and Wakabayashi et al. (1977 and 1980) were not included in the reference component data set because the characteristics of the tested braces or connections might not satisfy AISC 341-10 provisions for special braces.

As a result, a limited set of brace tests that includes special or similar braces for the Component Methodology equivalency comparison were assembled. Even so, in some cases, specific specimens may have characteristics that differ from those that would be permitted in AISC 341-10. These limitations should be considered in interpreting the reference component data.

The data gathered include 19 configurations, summarized in Tables E-1 and E-2. Most configurations include just one specimen, although two configurations have two or more identical specimens. Among the X-brace tests listed in Table E-1, both tube and wide flange sections are used for test specimens, all with welded gusset plate connections. The single brace tests listed in Table E-2 include tubular and wide flange sections, with either welded or bolted gusset plate connections with varying properties.

Table E-1 Summary of Conventional Brace Configurations in the Reference Component Data Set: X-Brace Tests

Test Index	Test Specimen ID	Area (in^2)	Length (in)	Brace Section*	Connection Type	Failure Mode
1	TCBF1-HSS-R	6.15	275.7	HSS 125x125x9	Gusset Plate(Weld)	Brace Buckling
2	TCBF1-HSS-T	6.15	275.7	HSS 125x125x9	Gusset Plate(Weld)	Brace Buckling
3	TCBF1-WF-R	7.94	275.7	H 175x175x7.5x11	Gusset Plate(Weld)	Brace Buckling

* Section properties defined in millimeters.

Table E-2 Summary of Conventional Brace Configurations in the Reference Component Data Set: Single Brace Tests

Test Index	Test Specimen ID	Area (in^2)	Length (in)	Brace Section*	Connection Type	Failure Mode
4	HSS-01	6.94	134.6	HSS-5x5x3/8	Gusset Plate(Weld)	Weld Fracture
5	HSS-02	6.94	157.8	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
6	HSS-03	6.94	157.8	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
7	HSS-05	6.94	157.8	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
8	HSS-04	6.94	153.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
9	HSS-12	6.94	134.6	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
10	HSS-13	6.94	157.3	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
11	HSS-17	6.94	157.3	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
12	HSS-14	6.94	157.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
13	HSS-15	6.94	162.1	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
14	HSS-16	6.94	131.3	HSS-5x5x3/8	Bolted Splice	Plate Fracture
15	HSS-18	6.94	157.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
16	HSS-19	6.94	103.6	HSS-5x5x3/8	Gusset Plate(Bolt)	Plate Fracture
17	HSS-20	6.94	157.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
18	HSS-21	6.94	157.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
19	HSS-22	6.94	157.3	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
20	WF-23	7.34	158.7	W 6x25	Gusset Plate(Weld)	Weld Fracture
21	HSS-24	6.94	157.7	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture
22	HSS-25	6.94	148.8	HSS-5x5x3/8	Gusset Plate(Weld)	Brace Fracture

* Section properties defined in inches.

All specimens have slenderness ratios, *Kl/r*, between 49 and 100. For braces in V or inverted-V configurations in special concentrically braced frames, the ratios are limited to $Kl / r \leq 4\sqrt{E / F_y}$. For 50 ksi steel, this limit corresponds to a maximum slenderness ratio of 96. Width-to-thickness ratios, *b/t*, of the specimens are all approximately 13 to 14. Since the prescribed maximum *b/t* for highly ductile compression elements is $0.55\sqrt{E / F_y} = 13.2$ in AISC 341-10, these specimens are near the maximum allowable slenderness and width-to-thickness ratios for special steel concentrically braced frames.

This data set satisfies the requirement that the reference component include at least four tested configurations per performance group. The Component Methodology also requires a minimum of two identical test specimens for each component configuration. The data compiled do not meet this requirement, as few of the configurations have been tested repeatedly. Even so, the lack of repeated configurations may be acceptable according to the provisions in Chapter 2, provided that there is no significant variation in the response among the different test specimens. Otherwise, more data may be needed.

There are no monotonic-load data included in the reference component test data set but the Component Methodology permits the use of reference component data with cyclic-load data only. Examples of reference component test data are shown in Figures E-5 and E-6. The data were available graphically (rather than digitally) so key force and deformation quantities were digitized from plots of cyclic-load test data.

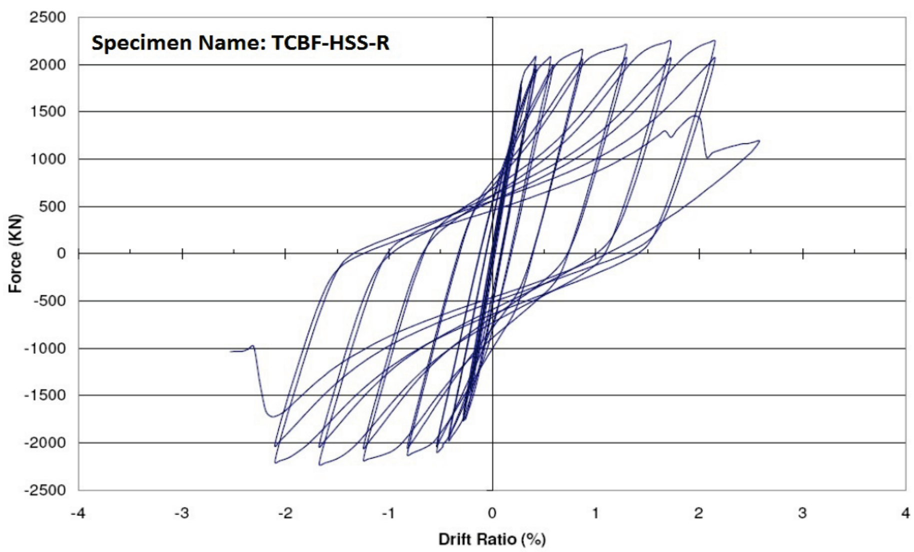


Figure E-5 Illustration of cyclic response of X-brace configuration tested by Clark (2009) for a two-story frame configuration.

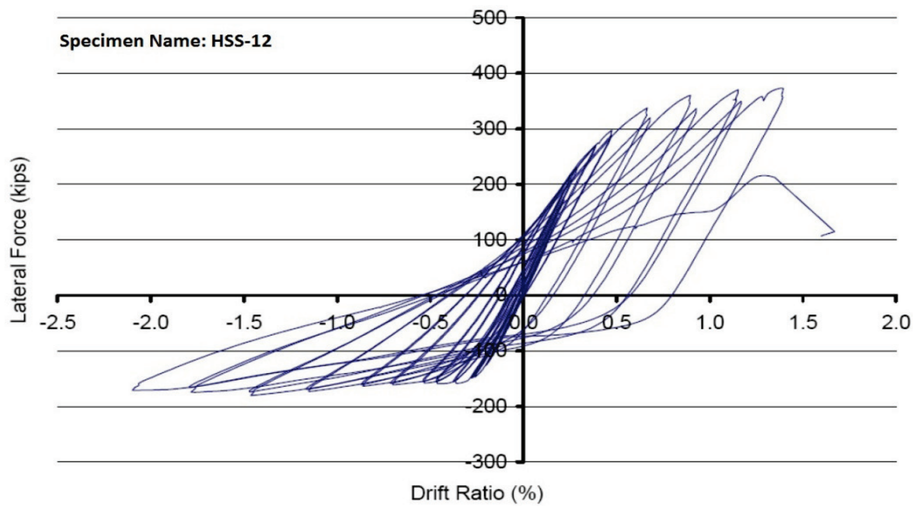


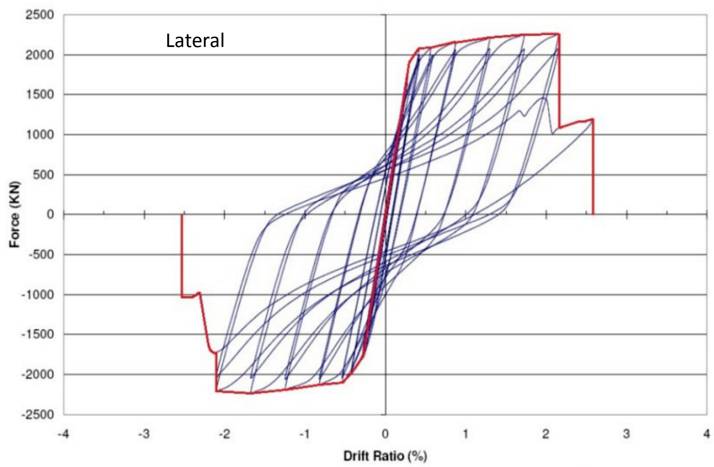
Figure E-6 Illustration of cyclic response of single HSS-section brace from Kotulka (2007) for a 1-story, 1-bay frame.

E.4.4 Interpret Reference Component Test Results

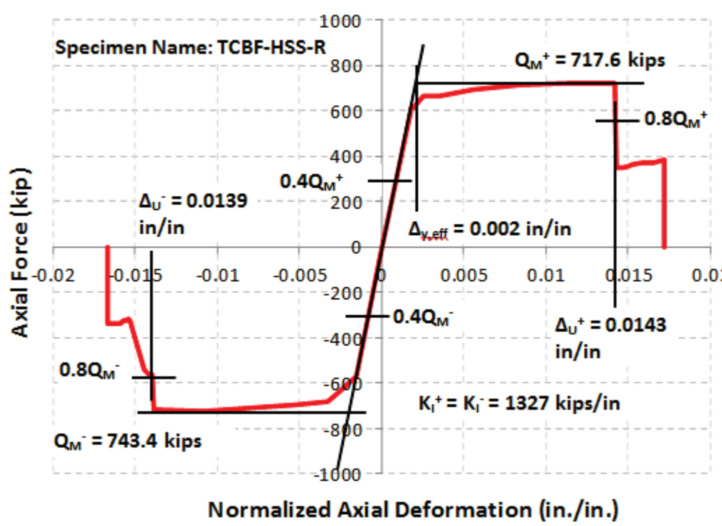
Figure E-7a shows the cyclic response of one X-brace test, with the cyclic envelope curve drawn according to the requirements of Section 2.2.2 superimposed. The envelope curve is used to determine the parameters that are utilized in the acceptance criteria of the Component Methodology.

The force quantity used to compare response between the reference and proposed component is the axial force in the brace or the total axial force in the X-braces (kips). The displacement quantity is the axial elongation in the brace, *l,* normalized by the length of the brace, i.e., in/in. In Figure E-7b, the cyclic envelope curve has been converted from horizontal force and deformation quantities obtained from frame tests to axial force and normalized axial deformation quantities. This estimation assumes that the lateral force is taken entirely by the brace and that all lateral deformation in the frame comes from brace axial elongation and shortening. The axial force, F_A, is calculated as $F_A = F_H / \cos(\theta)$ where F_H is the horizontal force on the frame and θ is the brace orientation angle measured counterclockwise from horizontal to brace axis. F_A is approximately $1.41F_H$ for $\theta = 45°$. The axial deformation, D_A, can be computed from the horizontal deformation, D_H, as $D_A = D_H \cos(\theta)$. If the horizontal deformation is reported as lateral drift (such that $D_H = \Delta_H h$) and the axial deformation is normalized by brace length l (such that $\Delta_A = D_A / l$), $\Delta_A = \Delta_H \cos(\theta) h/l \approx 0.707 \Delta_H h/l$.

For tests conducted in a frame, these assumptions overly simplify the actual behavior as the frame members will carry some of the lateral force and connections and other members contribute to the total frame deformation. For example, the test data in Figure E-5 show very little strength loss after brace buckling, likely because the moment resistance of frame members is significant. As a result, when force and deformation quantities of the frame are converted to force and deformation quantities of a brace under the assumptions described above, the values obtained overestimate both the force and the deformation in the brace. For the reference component, overestimating the ultimate brace deformation is conservative, since it requires that the proposed component data satisfy a higher ultimate deformation threshold.



(a)



(b)

Figure E-7 Cyclic test data for X-brace specimen TCBF-HSS-R (Test Index 1) tested by Clark (2009) showing: (a) cyclic response and cyclic envelope curve; and (b) cyclic envelope curve and component response quantities, in terms of brace axial force and elongation. The frame height and width are 6660 mm.

The specimen in Figure E-7 has maximum axial strength in positive and negative directions of 717.6 kips and 743.4 kips, respectively, for an average Q_M = 730.5 kips. For consistency, units were converted from kN to kips.

The design strength, Q_D, of the braces is computed as the sum of the brace design strength in tension and compression, because the component is defined as a pair of braces, one in compression and one in tension. The design tensile strength is given by AISC 341-10 as follows:

$$\varphi_t P_n = \varphi_t F_y A_g$$

where $_t$ = 0.90, F_y is the nominal yield stress of the type of steel being used, and A_g is the gross area of the member. The design compressive strength is

$$\varphi_c P_n = \varphi_c F_{cr} A_g$$

where $_c$ = 0.90. F_{cr} is the flexural buckling stress and depends on the slenderness ratio of the brace, Kl/r. For braces, the effective length factor, K, is equal to 1.0. For specimens with $Kl/r \leq 4.71\sqrt{E/F_y}$, F_{cr} is determined from:

$$F_{cr} = [0.658^{\frac{F_y}{F_e}}]F_y$$

The elastic critical buckling stress, F_e, is:

$$F_e = \frac{\pi^2 E}{(Kl/r)^2}$$

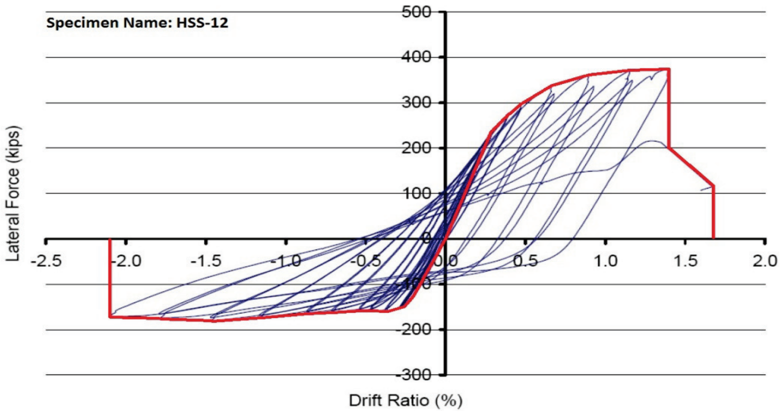
For the example in Figure E-7, the yield stress is 46 ksi, the area is 6.15 in^2, l/r is 78, and Q_D = 424 kips. The ratio of maximum strength to design strength, $R_Q = Q_M/Q_{D,}$ is 1.72.

Initial stiffness is computed as the secant stiffness at 40% of the peak strength, and the average of the positive and negative loading directions is K_I = 1327 kips/in. The design stiffness, K_D, is computed simply as the stiffness of brace itself, i.e., $K = EA/L$. For the specimen of interest and the simple assumption, K is calculated as 646.5 kips/in. Since there are two braces acting in parallel to resist the lateral force, K_D is 1293 kips/in and $R_{K,}$ the ratio of measured to design stiffness, is 1.03. (Note that these computations ignore the influence of other frame elements on lateral stiffness. The calculation of design stiffness could easily be improved by estimating the stiffness of the entire frame/brace/connection specimen that was tested).

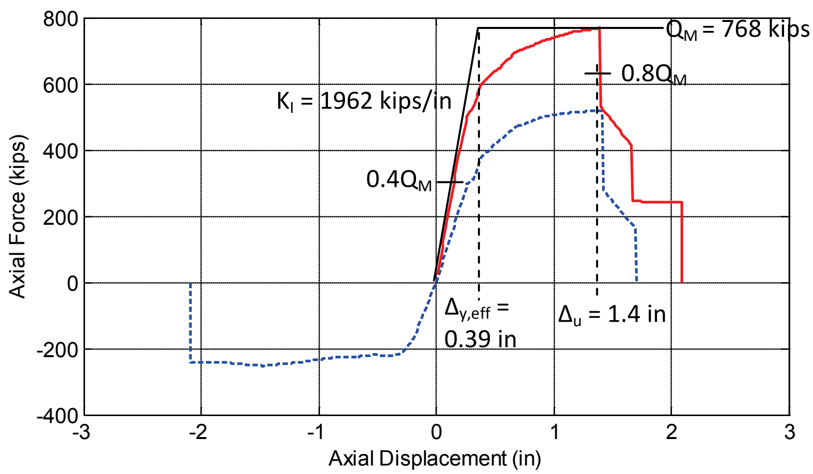
The ultimate deformation, Δ_U, is 3.84 inches in the negative loading direction and 3.94 inches in the positive loading direction. The final average ultimate deformation is 3.89 inches, which corresponds to $\Delta_U = 0.014$ in/in for this specimen, when normalized. Per Section 2.2.2, the value of Δ_U would be affected if the vertical load carrying ability of the specimen was compromised. Since braces are assumed to carry no gravity load, consideration of vertical load carrying capacity is not relevant.

The Component Methodology also requires determination of the effective yield displacement, $\Delta_{Y,eff}$. For this specimen, $\Delta_{Y,eff}$ is computed separately in the positive and negative directions and the average value is $\Delta_{Y,eff} = 0.55$ in. When normalized by brace length, $\Delta_{Y,eff} = 0.0020$ in/in. Chapter 2 defines effective ductility capacity as the ratio of the ultimate deformation, Δ_U, to the effective yield displacement, $\Delta_{Y,eff}$, in each direction. The specimen shown has an average effective ductility capacity, $\mu_{eff} = 7.1$.

Figure E-8a shows the cycle response of one of the single-brace tests with the cyclic envelope curve drawn according to the requirements of Chapter 2. Because seismic-force-resisting system requirements in AISC 341-10 do not allow single braces to be used, the single-brace test data are modified to represent a pair of braces. Assuming that there is no significant interaction between the two braces, we assume that, for any given level of deformation, the response in the compressive and tensile directions can be added to represent the behavior of a pair of braces. This process is illustrated in Figure E-8b, where the total backbone curve represents the sum of the cyclic envelope curves in the two loading directions. The design values of strength and stiffness are determined as before. (Note: In some tests, the brace buckles before any excursions in the tensile direction occur. In such a case, estimates of initial stiffness are based only on the first loading cycle in the compressive direction).



(a)



(b)

Figure E-8 Plot of test data for specimen HSS-12 (Test Index 9) from Kotulka (2007) showing: (a) cyclic response and cyclic envelope curve; and (b) combination of response in the positive and negative direction for calculation of component parameters for single brace specimen, tested in a 1-bay, 1-story frame with height and width of 12 ft.

For each of the tests included in the reference component data set, a cyclic envelope curve was drawn similar to those in Figures E-7 and E-8 in order to obtain component parameters for establishing equivalency. Table E-3 provides values of important response quantities, such as strength, stiffness, ductility, and deformation capacity, for each specimen of the reference component data set.

E.4.5 Compute Summary Statistics

The summary statistics for each component parameter are computed according to the requirements of Chapter 2 and presented in Table E-4. Statistical values of component parameters combine data from X-braces and single braces (converted to pairs).

Table E-3 Summary of Important Component Parameters for the Reference Component Data Set

Test Index	Strength			Stiffness			Ductility		Deformation Capacity
	Q_M (kips)	Q_D (kips)	R_Q	K_I (kip/in)	K_D (kip/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)
1	730	424	1.7	1327	1293	1.0	0.0020	7.1	0.014
2	642	429	1.5	1398	1293	1.1	0.0017	11.3	0.019
3	668	573	1.2	1653	1670	1.0	0.0015	14.5	0.021
4	747	459	1.6	2204	2802	0.79	0.0025	4.9	0.012
5	709	465	1.5	2025	2551	0.79	0.0022	4.5	0.010
6	742	465	1.6	1841	2551	0.72	0.0026	3.9	0.010
7	708	465	1.5	1643	2551	0.64	0.0027	4.3	0.012
8	695	470	1.5	1803	2618	0.69	0.0025	4.0	0.010
9	768	490	1.6	1962	2990	0.66	0.0029	3.6	0.010
10	715	466	1.5	2106	2559	0.82	0.0022	6.1	0.013
11	719	466	1.5	1617	2559	0.63	0.0028	4.9	0.014
12	716	465	1.5	1704	2552	0.67	0.0027	5.0	0.013
13	696	461	1.5	1562	2483	0.63	0.0028	4.1	0.011
14	628	493	1.3	1447	3066	0.47	0.0033	6.3	0.021
15	665	471	1.4	2650	2552	1.04	0.0016	6.6	0.010
16	439	424	1.0	2317	3885	0.60	0.0018	2.0	0.004
17	708	489	1.4	2804	2552	1.10	0.0016	6.7	0.011
18	717	472	1.5	2391	2552	0.94	0.0019	5.4	0.010
19	604	465	1.3	2330	2559	0.91	0.0016	5.8	0.010
20	683	481	1.4	2681	2683	1.00	0.0016	9.4	0.015
21	698	496	1.4	2412	2552	0.95	0.0018	6.7	0.012
22	814	590	1.4	3824	2705	1.41	0.0014	4.0	0.006

Table E-4 Summary Statistics for the Reference Component

Summary Statistic	Perf. Group	RQ (=Q_M / Q_D)	RK (=K_I / K_D)	μ_{eff} (= $\Delta_U / \Delta_{Y,eff}$)	Δ_U (in/in)
Median	ALL	$\tilde{R}_Q = 1.45$	$\tilde{R}_K = 0.79$	$\tilde{\mu}_{eff} = 5.5$	$\tilde{\Delta}_U = 0.011$
Variability	ALL	$\sigma_{RQ} = 0.11$	$\sigma_{RK} = 0.26$	$\sigma_{\mu,eff} = 0.42$	$\sigma_{\Delta U} = 0.38$

E.5 Proposed Component Design Requirements

The buckling-restrained brace design requirements are based on AISC 341-10. In an actual application of the Component Methodology, code requirements may not exist for the proposed component, and such requirements would need to be created. Necessary design requirements include design strength and stiffness requirements, as well provisions to ensure that the component has adequate deformation capacity and detailing.

E.5.1 Component Design Strength and Stiffness

According to AISC 341-10 the steel core is designed to resist the total axial force in the brace. The brace design axial strength is

$$\varphi P_{ysc} = \varphi F_{ysc} A_{sc}$$

where is F_{ysc} is the specified minimum yield stress of the steel core or actual yield stress of the steel core as determined from a coupon test, A_{sc} is the net area of steel core, and $\varphi = 0.90$. In the evaluation that follows, the design strength is taken as $Q_D = P_{ysc}$, using the specified minimum yield stress.

Design requirements are also needed to predict the initial stiffness. The total elastic stiffness of the BRB is the in-series sum of the individual stiffnesses of the different brace segments as proposed by Black et al. (2002):

$$K_{\text{total}} = \frac{1}{\left(\frac{1}{K_y} + 2\frac{1}{K_{con}} + 2\frac{1}{K_{tr}}\right)}$$

where $K_y = EA_y/L_y$ is the elastic stiffness of the yielding portion, $K_{con} = EA_{con}/L_{con}$ is the stiffness of the connection portion and $K_{tr} = EA_{tr}/L_{tr}$ is the stiffness of the transition portion. The “yielding,” “connection,” and “transition” portions of a BRB are illustrated in Figure E-9.

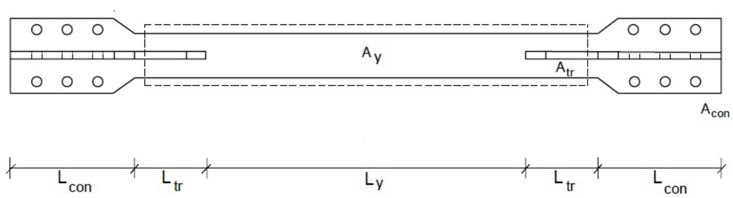


Figure E-9 Dimensions needed for computing the design stiffness for BRBs (Figure from Black et al., 2002).

For the 28 specimens studied, there was insufficient information about the dimensions needed to compute K_{total}. Instead, an approximate relationship proposed for typical buckling-restrained brace dimensions, story heights, and bay widths (Nippon Steel Engineering) was used:

$$K_{total} \sim 0.83 \frac{A_y E}{L_y}$$

This approximation has been shown to provide good agreement with the original equation for BRB specimens of typical sizes. The design stiffness value K_D is taken equal to K_{total}.

E.5.2 Component Detailing Requirements

Detailing provisions follow AISC 341-10. The primary requirement is that buckling-restrained braces are able to permit axial elongation and shortening in the steel core without failure up to deformations that are twice the design story drift. Additional requirements specify the minimum stiffness and notch toughness requirements for the steel core plates.

E.5.3 Component Connection Requirements

According to AISC 341-10, the required strength of columns, beams and connections must be designed to include the amplified seismic load, which accounts for the adjusted strength of BRBs in tension and compression. The adjusted brace strength is based on qualification testing and accounts for differences in strength in compression and tension. These connection and load transfer design requirements ensure that the inelastic behavior occurs in the BRB and fulfill the connection capacity design requirements of Section 2.5.3.

In using the AISC 341-10 requirements for buckling-restrained braces as the basis for the proposed component design requirements, it is assumed that if buckling-restrained braces were substituted for conventional braces in the special concentrically braced frame system, the design of the rest of the system would also comply with the AISC 341-10 specifications for buckling-restrained braces. Specifically, connection design in the balance of the system would need to account for the large overstrength that is associated with the use of buckling-restrained braces.

E.5.4 Limitations on Component Applicability and Use

The Component Methodology requires that the design provisions restrict the design space to the range of configurations that are considered in the testing program. Since the test data described in Section E.6 cover the range of size and other properties of BRBs that may be reasonably expected in practice, there are no specific additional limitations on component applicability and use.

E.5.5 Component Construction, Inspection, and Maintenance Requirements

Construction, inspection, and maintenance requirements for BRBs follow AISC 341-10 provisions for quality assurance which include requirements for written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications, and contract documents.

E.6 Proposed Component Test Data

Experimental work conducted by a variety of researchers provided test data for the proposed component. These data include 33 geometric configurations and a total of 46 component tests. Table E-5 summarizes the components tested, providing general information about core plates, dimensions, and failure modes. Configurations of identical specimens are grouped by the color of rows.

The following references were used to collect the BRB data in this section: Black et al. (2002), Merritt et al. (2003a), Merritt et al. (2003b), Reaveley et al. (2004), Newell et al. (2005), Christopulos (2005), Newell et al. (2006), Benzoni and Innamorato (2007), Kim et al. (2010), and Sim et al. (2010). Data were provided by a number of researchers and includes experimental tests of products from three major BRB manufacturers: CoreBrace, Star Seismic and Nippon Steel.

Although a number of tests do not show any failure, commonly occurring failure modes included core plate fracture or local buckling at end plates. Figure E-10 illustrates the fracture that can occur in buckling-restrained braces.

Table E-5 Summary of Buckling-Restrained Brace Configurations in the Proposed Component Data Set

Test Index	Test Specimen ID	Number of Core Plates/Shape	Area of Core Plate (in^2)	Length, L_y (in)	Connection Type	Tube Section	Failure Mode
1	PC1100A-1	3/Flat	24.1	185.9	Pin	Rectangular	Not failed
2	PC1100B-1	3/Flat	24.1	185.9	Pin	Rectangular	Not failed
3	PC1000A-1	2/Flat	23.6	185.9	Pin	Rectangular	Core plate rupture
4	1D	1/Flat	10.0	133.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
5	2D	1/Flat	10.0	133.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
6	3D	1/Cruciform	16.0	131.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
7	4D	1/Cruciform	16.0	131.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
8	5D	1/Cruciform	23.1	130.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
9	6D	1/Cruciform	23.1	130.0	Pin (with cruciform gusset bracket)	Rectangular	Core plate fracture
10	1E	1/Flat	4.0	100.2	Gusset Plate (with bolt)	Rectangular	Core plate fracture
11	2E	1/Flat	4.0	73.8	Gusset Plate (with bolt)	Rectangular	Core plate fracture
12	3E	1/Flat	9.0	92.7	Gusset Plate (with bolt)	Rectangular	Gusset plate bending
13	4E	1/Flat	9.0	65.8	Gusset Plate (with bolt)	Rectangular	Core plate fracture

Table E-5 Summary of Buckling-Restrained Braces in the Proposed Component Data Set (continued)

Test Index	Test Specimen ID	Number of Core Plates/Shape	Area of Core Plate (in^2)	Length, L_y (in)	Connection Type	Tube Section	Failure Mode
14	5E	1/Cruciform	20.0	54.5	Gusset Plate (with bolt)	Rectangular	Gusset plate bending
15	6E	1/Cruciform	20.0	54.5	Gusset Plate (with bolt)	Rectangular	Gusset plate bending
16	1F	1/Cruciform	27.0	144.5	Gusset Plate (with bolt)	Rectangular	Core plate fracture
17	2F	1/Cruciform	27.0	144.5	Gusset Plate (with bolt)	Rectangular	Core plate fracture
18	1G	1/Flat	12.0	132.5	Gusset Plate (with bolt)	Rectangular	Not failed
19	2G	1/Flat	12.0	132.5	Gusset Plate (with bolt)	Rectangular	Not failed
20	3G	1/Cruciform	27.0	144.4	Gusset Plate (with bolt)	Rectangular	Core plate rupture
21	4G	1/Cruciform	27.0	144.4	Gusset Plate (with bolt)	Rectangular	Not failed
22	H1	1/Flat	4.0	179.9	Pin	Circular	Buckling at end of plates
23	H2	1/Flat	11.5	125.8	Pin	Rectangular	HSS Casing damaged
24	H3	1/Flat	18.0	113.2	Pin	Rectangular	Test stopped
25	H4	1/Flat	27.0	178.8	Pin	Rectangular	Core plate rupture
26	J2	1/Flat	2.0	217.0	Gusset Plate (with weld)	Circular	Core plate rupture
27	J3	1/Flat	6.0	203.0	Gusset Plate (with weld)	Rectangular	Core plate fracture
28	J4	1/Flat	18.0	176.0	Gusset Plate (with weld)	Circular	Core plate rupture
29	Star - 1	2/Flat	3.8	176.0	Pin	1-Rectangular	Core plate fracture
30	Star - 2	2/Flat	6.0	179.4	Pin	1-Rectangular	Core plate fracture
31	Star - 3	2/Flat	8.3	183.3	Pin	1-Rectangular	Not failed
32	Star - 4	2/Flat	12.7	185.1	Pin	1-Rectangular	Not failed
33	Star - 5	4/Flat	17.9	184.2	Pin	2-Rectangular	Not failed
34	Star - 6	6/Flat	17.9	179.4	Pin	2-Rectangular	Not failed
35	Star - 7	6/Flat	28.5	185.2	Pin	2-Rectangular	Not failed
36	Star - 8	8/Flat	28.6	181.3	Pin	4-Rectangular	Not failed
37	99-1	1/Flat	4.5	121.7	Gusset Plate (with bolt)	Rectangular	Not failed
38	99-2	1/Flat	6.0	117.7	Gusset Plate (with bolt)	Rectangular	Not failed
39	99-3	1/Cruciform	8.0	135.8	Gusset Plate (with bolt)	Rectangular	Not failed
40	00-11	1/Cruciform	11.0	134.3	Gusset Plate (with bolt)	Rectangular	Not failed
41	00-12	1/Cruciform	11.0	134.3	Gusset Plate (with bolt)	Rectangular	Not failed
42	REF-BRB	1/Flat	4.8	93.2	Gusset Plate (with bolt)	Rectangular	Core plate yielding
43	BRB02	1/Flat	4.8	93.2	Gusset Plate (with bolt)	Rectangular	Core plate yielding
44	BRB03	1/Flat	4.8	93.2	Gusset Plate (with bolt)	Rectangular	Core plate yielding
45	BRB04	1/Flat	4.8	93.2	Gusset Plate (with bolt)	Rectangular	Core plate yielding
46	BRB01	1/Flat	4.8	93.2	Gusset Plate (with bolt)	Rectangular	Core plate yielding

Figure E-10 Photo of steel core after fracture in buckling-restrained brace specimen (Merritt et al., 2003a).

E.6.1 Define Proposed Component Design Space

The proposed component design space includes buckling-restrained braces that comply with the design and detailing requirements of Section E.5. Buckling-restrained braces used in practice vary in terms of:

- Steel core length,
- Cross-sectional area and geometry of the steel core plates,
- Type of connection,
- Cross-section of the steel brace section ,
- The presence of mortar fill as the buckling-restraining mechanism, and
- Failure mode.

E.6.2 Select Component Configurations for Testing

Component configurations selected for testing should span the component design space in accordance with Section 2.6.2. In this test application, the configurations listed in Table E-5 are utilized. The specimens cover the reasonable range of configurations for key variables, including steel core plate length, which varies from 54.5 to 217 inches, the type of the connection, which includes pins, gusset plates with different cross-sectional shape and bolts or welds, different numbers and geometries of core plates, including both flat and cruciform plates, core cross-section areas ranging from 4 in^2 to 28.62 in^2, and shape.

All buckling-restrained brace specimens considered are mortar-filled. Therefore, the findings of the Component Methodology should be applicable only to these mortar-filled tubes, unless it can be shown that specimens without concrete or mortar filling have comparable performance.

E.6.3 Perform Cyclic-Load and Monotonic-Load Tests

In this test application, only cyclic-load test data are utilized because monotonic-load test data were not found in the available references. Although monotonic-load test data for the proposed component are required in most cases, under certain conditions, the cyclic-load data can be used in lieu of monotonic-load test data.

Figure E-11 shows an example of the cyclic response for Specimen 3, which has 2 flat plates in the steel core. This specimen failed because of core plate rupture during the test.

Most of the tests were conducted according to the loading protocol specified in AISC 341-10 for cyclic qualification of buckling-restrained braces (Section K3). This loading history is defined by the axial deformation in the brace, Δ_b, and depends on the value of axial deformation corresponding to the design story drift, Δ_{bm}, and the value of axial deformation at first significant yield of the test specimen, Δ_{by}. The loading history imposed can be shown to be equivalently damaging to the histories used in the reference component tests, as described in Section E.9.

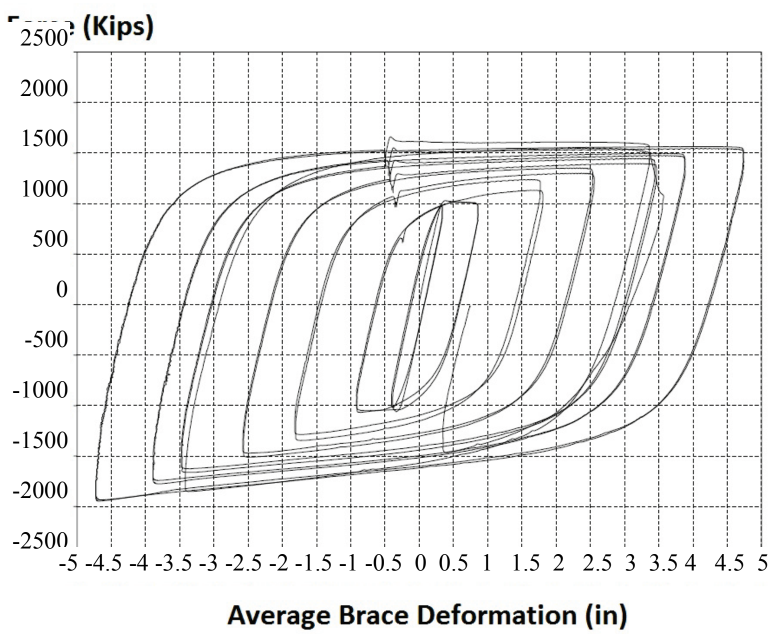


Figure E-11 Illustration of cyclic response of buckling-restrained brace Specimen 3 (Benzoni and Innamorato, 2007).

E.6.4 Interpret Proposed Component Test Results

An example cyclic envelope curve is shown in Figure E-12 illustrating the calculation of component response quantities. The component parameters are obtained from experimental data following the same steps described previously for the reference component data.

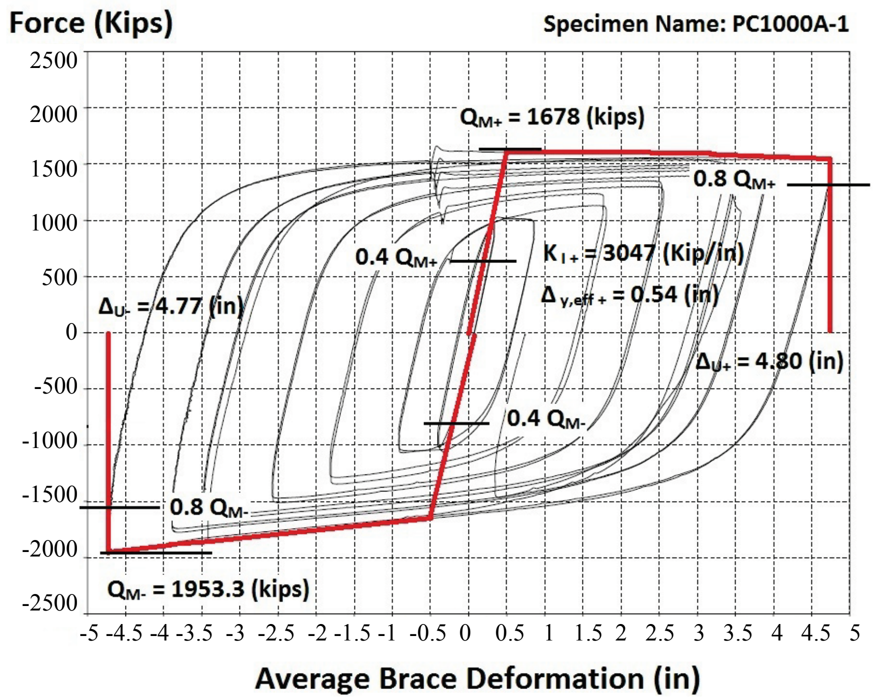


Figure E-12 Illustration of cyclic response, cyclic envelope curve and calculation of component response quantities of BRB Specimen 1 (Benzoni and Innamorato, 2007).

Important component parameters were compiled for each of the 46 buckling-restrained brace tests, and summarized in Table E-6. As was done with the reference component data, proposed component data was placed in one performance group, implying that any of the buckling-restrained braces are conceived as a replacement for any of the conventional brace components. Note that some configurations have only one test specimen. According to the Component Methodology, each proposed component configuration should have at least two identical test specimens. Repeated specimens would therefore need to be added to the data set to complete this example.

Most of the braces were tested axially and test data were provided in terms of axial force and deformation. However, specimens 42 through 46 were tested in a frame including beam, columns, and connection. These test results have been converted from lateral force-story drift to axial force and deformation in accordance with the formulas provided in Section E.4.4 for comparison with component parameters from other tests.

Table E-6 Summary of Important Component Parameters for the Proposed Component Data Set

Test Index	Strength			Stiffness			Ductility		Deformation Capacity	
	Q_M (kips)	Q_D (kips)	R_Q	K_I (kips/in)	K_D (kips/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)	$\tilde{\Delta}_{U,I}$
1	1850	780	2.4	3107	3118	1.0	0.00300	7.0	0.021	0.021
2	1780	780	2.3	3270	3118	1.0	0.00279	7.5	0.021	
3	1816	763	2.4	2988	3051	1.0	0.00302	8.5	0.026	0.026
4	700	324	2.2	1751	1846	0.9	0.00276	10.7	0.029	0.029
5	706	324	2.2	1843	1846	1.0	0.00270	10.8	0.029	
6	1139	518	2.2	2844	2610	1.1	0.00291	10.3	0.029	0.029
7	1140	518	2.2	2839	2610	1.1	0.00287	10.1	0.029	
8	1495	749	2.0	4474	3664	1.2	0.00248	10.0	0.024	0.026
9	1630	749	2.2	3930	3664	1.1	0.00309	9.6	0.029	
10	294	130	2.3	931	961	1.0	0.00317	10.7	0.034	0.034
11	286	130	2.2	1040	1304	0.8	0.00366	7.1	0.025	0.025
12	602	292	2.1	1836	2336	0.8	0.00356	8.6	0.029	0.029
13	655	292	2.2	2140	3292	0.6	0.00457	8.9	0.040	0.040
14	1291	648	2.0	5860	8833	0.7	0.00413	7.2	0.030	0.030
15	1313	648	2.0	5613	8833	0.6	0.00433	7.0	0.029	
16	1985	875	2.3	4723	3869	1.2	0.00299	12.5	0.037	0.037
17	1897	875	2.2	4329	3869	1.1	0.00253	14.0	0.036	
18	894	389	2.3	1974	1522	1.3	0.00344	10.2	0.035	0.034
19	940	389	2.4	1533	1522	1.0	0.00423	8.0	0.034	
20	1826	875	2.1	4015	3721	1.1	0.00306	10.4	0.031	0.034
21	1934	875	2.2	4228	3721	1.1	0.00300	12.2	0.036	
22	345	130	2.7	501	535	0.9	0.00364	9.5	0.033	0.033
23	813	373	2.2	2166	2201	1.0	0.00273	12.0	0.033	0.033
24	1205	583	2.1	3905	3828	1.0	0.00265	11.1	0.029	0.029
25	1622	875	1.9	3292	3636	0.9	0.00234	9.2	0.020	0.020
26	200	65	3.1	216	232	0.9	0.00307	9.0	0.027	0.027
27	475	194	2.4	645	726	0.9	0.00319	9.2	0.029	0.029
28	1406	583	2.4	2467	2173	1.1	0.00274	12.1	0.033	0.033
29	306	123	2.5	428	571	0.7	0.00404	6.1	0.024	0.024
30	447	193	2.3	636	845	0.8	0.00357	7.1	0.024	0.024
31	525	270	1.9	1000	1118	0.9	0.00319	5.9	0.019	0.019
32	915	410	2.2	1430	1625	0.9	0.00374	6.7	0.025	0.025

Table E-6 Summary of Important Component Parameters for the Proposed Component Data Set (continued)

Test Index	Strength			Stiffness			Ductility		Deformation Capacity	
	Q_M (kips)	Q_D (kips)	R_Q	K_I (kips/in)	K_D (kips/in)	R_K	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)	$\tilde{\Delta} U_I$
33	1359	578	2.3	2142	2291	0.9	0.00409	6.5	0.027	0.027
34	1378	579	2.4	1875	2438	0.8	0.00418	6.2	0.026	0.026
35	1787	924	1.9	2000	3529	0.6	0.00371	5.2	0.019	0.019
36	1764	927	1.9	2500	3660	0.7	0.00359	5.3	0.019	0.019
37	325	146	2.2	949	976	1.0	0.00282	7.1	0.020	0.020
38	328	194	1.7	994	1265	0.8	0.00281	7.4	0.021	0.021
39	599	259	2.3	1681	1528	1.1	0.00262	7.6	0.020	0.020
40	725	358	2.0	1997	2083	1.0	0.00253	8.6	0.022	0.021
41	719	358	2.0	2531	2083	1.2	0.00203	10.1	0.020	
42	475	155	3.1	1043	1244	0.8	0.00384	6.0	0.023	0.022
43	445	155	2.9	1043	1244	0.8	0.00437	5.9	0.026	
44	459	155	3.0	1043	1244	0.8	0.00490	4.6	0.022	
45	464	155	3.0	1043	1244	0.8	0.00490	5.0	0.025	
46	409	155	2.6	1043	1244	0.8	0.00490	3.2	0.016	

E.6.5 Compute Summary Statistics

The summary statistics for buckling-restrained brace data are computed in accordance with the requirements of Section 2.6.5, and reported in Table E-7.

Table E-7 Summary Statistics for the Proposed Component

Summary Statistic	Perf. Group	RQ (=Q_M/ Q_D)	RK (=K_I/ K_D)	μ_{eff} (=$\Delta_U/\Delta_{y,eff}$)	ΔU (in/in)
Median	ALL	$\tilde{R}_Q = 2.27$	$\tilde{R}_K = 0.92$	$\tilde{\mu}_{eff} = 8.1$	$\tilde{\Delta}_U = 0.026$
Variability	ALL	$\sigma_{RQ} = 0.13$	$\sigma_{RK} = 0.19$	$\sigma_{\mu,eff} = 0.31$	$\sigma_{\Delta U} = 0.22$

E.7 Evaluate Quality Ratings

E.7.1 Quality Rating of Test Data

Judged according to the criteria described in Section 2.7, the quality of reference component test data was rated as Fair. Although there is good agreement between test results and numerical and analytical predictions of test data, the test specimens do not cover the range of possible component configurations that would be used in practice, such that the data set was assigned a medium confidence rating. In addition, because frame data were used to approximate axial force and elongation parameters, the dataset was also rated with medium completeness and robustness. According to the Component Methodology, reference component test data with a Fair quality rating are not acceptable. As a result, more testing of reference component specimens would be required. However, for the purpose of this test application, the reference data quality rating has been artificially inflated to a rating of Good.

The quality of proposed component test data was rated as Superior. This rating is based on high confidence in test results due to the large number of tests completed by independent investigators. Tests represent a reasonable range of design parameters to be representative of design practice. There is substantially smaller uncertainty in the buckling-restrained brace data than the conventional brace data. The completeness and robustness of the test data were also judged to be high. Material component and connection behavior is well understood, and there is good agreement between experimental testing and analytical modeling. The primary limitation in the data set is the lack of configurations with two or more test specimens.

E.7.2 Quality Rating of Design Requirements

The quality rating for proposed and reference component design requirements depends on the completeness and robustness of requirements and the confidence in the underlying methods and design data.

For conventional braces there is high confidence in design requirements because there are extensive experimental data and analytical studies, and good agreement between the two. Completeness and robustness of design requirements is also high since the usage of capacity design requirements ensures that inelastic action concentrates primarily in braces. Seismic compactness requirements delay brace buckling in compression. On the basis of these evaluations, the reference component design requirement quality rating was judged to be Superior.

The proposed component design requirements were also assigned a quality rating of Superior. Extensive experimental data and analytical studies provide high confidence that the material properties, strength criteria and other parameters are representative of actual behavior. In addition, BRB design is based on prototype testing that demonstrates that the component has good performance (no failure) up to deformations that are twice the design displacement level of the structure. Fabrication, erection, and other construction issues are carefully controlled by BRB manufacturers.

E.8 Evaluate Component Equivalency

E.8.1 Overview

This section discusses the acceptance criteria and evaluates whether the conventional and buckling-restrained braces can be judged to be equivalent according to the Component Methodology. If equivalent, the proposed components could hypothetically be substituted for brace components of the reference seismic-force-resisting system, subject to design requirements and seismic design category restrictions on the use of the reference seismic-force-resisting system.

E.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation

The median ultimate deformation of each proposed component performance group is required to fulfill Equation 2-1, as follows:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q$$

In this example, the reference and proposed components are each treated as a single performance group.

As reported in Table E-4, the median ultimate deformation, $\tilde{\Delta}_{U,RC}$, of the reference component data set is 0.011 in/in. The uncertainty penalty factor, P_u, is based on the quality ratings discussed in Section E.7.1. Incorporating design requirement ratings of Superior and test data ratings of Good and Superior, the uncertainty penalty factor $P_u = 1.00$. The penalty factor for strength is based on the ratio of the median proposed and reference component strength ratios, as follows:

$$\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} = \frac{2.27}{1.45} = 1.57$$

Based on linear interpolation in Table 2-4 with a strength ratio of 1.57, the penalty factor to account for differences in strength, P_Q, is 1.21. The ratio of $\tilde{R}_{Q,PC} \big/ \tilde{R}_{Q,RC}$ exceeds the limit of 1.2, but is allowed because buckling-restrained brace design provisions ensure that force-controlled and capacity-designed components are designed for the expected strength of the component.

Incorporating these values leads to the following requirement:

$$\tilde{\Delta}_{U<PC} = 0.026 \geq 0.011 \times 1.00 \times 1.21 = 0.013$$

This requirement is easily satisfied since the median ultimate deformation obtained from buckling-restrained brace data is 0.026 (from Table E-7).

The ultimate deformation for each geometric configuration, $\tilde{\Delta}_{Uj,PC}$, must also meet the requirement of Equation 2-2, repeated here:

$$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \; P_Q$$

According to Table E-4, the variability in ultimate deformation for the reference component, $\sigma_{\Delta U,RC}$, is 0.38. Since $\sigma_{\Delta U,RC}$ exceeds 0.3, 0.3 is used. Therefore, each proposed component configuration must exceed the lower bound:

$$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\times 0.30\right)\left(0.011\right) 1.00\times 1.21 = 0.0074$$

Referring back to Table E-6, all proposed component configurations have median ultimate deformation capacities of 0.0074 or larger, and therefore satisfy the requirement.

E.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

The median value of the proposed component initial stiffness ratio, $\tilde{R}_{K,PC}$, should be within the range specified in Equation 2-3, and repeated here:

$$0.75 \leq \frac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} = \frac{0.92}{0.81} = 1.14 \leq 1.33$$

The stiffness ratio computed is 1.14, which falls within the allowable range.

E.8.4 Requirements Based on Cyclic-Load Test Data: Effective Ductility Capacity

The median value of the effective ductility capacity of the proposed component should not be less than 50% of the median value of the effective ductility capacity of the reference component, such that

$$\tilde{\mu}_{eff,PC} = 8.1 \geq 0.5\,\tilde{\mu}_{eff,RC} = 0.5(5.5) = 2.75$$

The median effective ductility for the reference brace component is 5.5 from Table E-4. The median effective ductility for the proposed component is 8.1 from Table E-7, satisfying the effective ductility capacity requirement.

E.8.5 Requirements Based on Monotonic Load Test Data: Ultimate Deformation

As discussed previously, the proposed and reference component data compiled in this test application do not include monotonic data. When monotonic data for the reference component are not available, the monotonic load test data requirements in Equation 2-6 specify that:

$$\tilde{\Delta}_{UM,PC} \geq 1.2\, \tilde{D}_C\, \tilde{\Delta}_{U,RC}\, P_U\, P_Q$$

However, since monotonic test data are not available, it is allowable to take $\tilde{\Delta}_{UM,PC}$ as equal to $\tilde{\Delta}_{U,PC}$, which is 0.026. As before $\tilde{\Delta}_{U,RC}$ = 0.011, P_U = 1.00, and P_Q = 1.21. The median value of the cumulative damage factor, $\tilde{D}_C$, depends on the number of cycles in the reference component testing protocols. In this data set, the reference components are tested with a number of different loading histories but an average of 35 cycles before failure. Therefore, according to Section 2.8.5, $\tilde{D}_C$ = 1.5. Substituting these values into Equation 2-6 results in

$$\tilde{\Delta}_{UM,PC} = 0.026 \geq 1.2\,(1.5)(0.011)(1.00)(1.21) = 0.024$$

Since the median cyclic deformation capacity for the proposed component exceeds this limit with 0.026, the monotonic load test data requirements are implicitly met.

If Equation 2-6 were not satisfied with the cyclic data, at a minimum, monotonic data would be required for selected BRB configurations. Ideally, experimental monotonic load testing would be conducted on selected reference component configurations as well, though this is not required. Monotonic load test data provide important insights into the seismic behavior of new components.

E.8.6 Summary of Component Equivalency Evaluation

In summary, buckling-restrained brace components passed all acceptance criteria, including those related to ultimate deformation capacity, initial stiffness, and effective ductility. Results are summarized in Table E-8.

Based on the criteria of the Component Methodology, buckling-restrained braces are equivalent to conventional braces, and could be substituted into the special steel concentrically braced frame seismic-force-resisting system. This substitution is contingent on incorporation of the design rules proposed for buckling-restrained braces in Section E.5.

Table E-8 Summary of Acceptance Criteria and Equivalency Evaluation

Acceptance Criteria		Equation Reference	Pass/Fail
Requirement Based on Cyclic-Load Test Data			
Ultimate Deformation Capacity (performance group)	$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} P_U P_Q$	2-1	Pass
Ultimate Deformation Capacity (individual configurations)	$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U\ P_Q$	2-2	Pass
Initial Stiffness Ratio	$0.75 \leq \dfrac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33$	2-3	Pass
Effective Ductility Capacity	$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC}$	2-4	Pass
Requirements Based on Monotonic-Load Test Data			
Ultimate Deformation Capacity (Option 1)	$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC}\ P_U\ P_Q$	2-5	N/A
Ultimate Deformation Capacity (Option 2)	$\tilde{\Delta}_{UM,PC} \geq 1.2\,\tilde{D}_C\,\tilde{\Delta}_{U,RC}\ P_U\ P_Q$	2-6	Pass

E.9 Loading Protocol Suitability

In order for the comparisons described above to be appropriate, the loading protocols used to test reference and proposed component test specimens must be equivalently damaging. The method described in Appendix B is used to compare the loading histories used in cyclic load testing for the buckling-restrained and conventional brace specimens. According to the Component Methodology, other quantitative methods may also be used to compare cyclic loading protocols.

In all cases, the reference components were tested with a cyclic loading protocol based on a combination of ATC-24 (ATC, 1992) and SAC Steel protocols (Clark et al., 1997). The control variable was based on the interstory drift angle of the frame at the first onset of yielding or buckling. Based on these loading histories, the normalized cumulative deformation (*NCD*) imposed before reaching the ultimate deformation was computed for each specimen. The median normalized cumulative deformation imposed in the reference component tests is 16.7, with a maximum computed value of 24.5 (Specimen 19) and a minimum value of 12.2 (Specimen 15).

Most of the proposed component tests were conducted according to the loading protocol specified in AISC 341-10 Section K3, as described in Section E.6.3. The median normalized cumulative deformation imposed in the proposed component tests is 15.4; individual specimen normalized cumulative deformations vary from 9.2 to 23.5.

The first criterion in Appendix B is that the normalized cumulative deformation demand from each test, NCD_i, should fall within the range defined by either the equations below, such that

$$0.5N\tilde{C}D_{RC} \leq NCD_{RC,i} \leq 2N\tilde{C}D_{RC}$$

and

$$0.5N\tilde{C}D_{PC} \leq NCD_{PC,i} \leq 2N\tilde{C}D_{PC}$$

For the median cumulative deformations calculated for the reference and proposed components, respectively, these limits become:

$$8.4 \leq NCD_{RC,i} \leq 33.4$$

and

$$7.7 \leq NCD_{PC,i} \leq 30.8$$

Although not reported here, all specimens tested have a normalized cumulative deformation value, NCD_i, that fall within these ranges.

The second criterion states that no tests with normalized cumulative deformation smaller than 6.0 are permitted, ensuring that all loading histories are sufficiently damaging. The normalized cumulative deformation imposed on every test specimen exceeds this lower bound.

Finally, the proposed component shall be tested under a loading protocol(s) that imposes approximately the same, or more, normalized cumulative deformation on the component, compared to the loading histories used to establish the reference component data set. Quantitatively, this limit is provided in Equation B-3 and repeated here:

$$N\tilde{C}D_{PC} = 15.4 \geq 0.75N\tilde{C}D_{RC} = (0.75)(16.7) = 12.5$$

This implies that proposed component specimens should be tested with a protocol that imposes (on average) at least a normalized cumulative deformation of 12.5. The median proposed component normalized cumulative deformation is 15.4, exceeding this lower bound. As a result, the cyclic loading histories imposed on the reference and proposed components are deemed equivalently damaging for use in the Component Methodology.

E.10 Summary of Component Equivalency Evaluation of Buckling-Restrained Braces

Overall, the comparison of buckling-restrained braces to special steel braces found that the buckling-restrained braces have a larger median deformation capacity and satisfy Component Methodology equivalency criteria based on deformation capacity. Although buckling-restrained braces have higher overstrength than conventional braces, connections are designed using a capacity design philosophy that satisfies Component Methodology requirements for strength. Requirements based on stiffness and effective ductility capacity are also met. As a result, the evaluation concludes that buckling-restrained braces are equivalent to conventional braces in special steel concentrically braced frame systems.

E.11 Limitations of Test Application

This section summarizes limitations in the test application comparison between buckling-restrained braces and steel concentric braces, which limit the general applicability of the conclusions and the usability of the reference and proposed component data sets developed.

E.11.1 Reference Component Test Data Do Not Fully Represent the Design Space

The reference component data gathered in this example are based primarily on HSS sections of the same length and area, with different connection types. There are also a few wide flange tests. Most of the test specimens have section properties that are close to the limit in terms of allowable slenderness and width-to-thickness ratios. As a result, the reference component data set may present a pessimistic view of brace behavior that does not represent the typical behavior of braces in real buildings. In addition, some of these tests examine changes in connection design that may not be typical special concentrically brace frames as currently constructed in practice.

E.11.2 The Equivalency Evaluation May Not Adequately Account for System Differences

While most of the buckling-restrained braces were tested in an axial test setup (with the exception of specimens 42-46), the reference component braces were tested in frames and the data recorded were in terms in lateral force and drift ratio. This difference in test setup poses problems in obtaining a consistent metric of force and deformation for comparison of proposed and reference components. In this evaluation, data obtained from frame tests were used to estimate the axial force and axial elongation in the brace assuming that the entire lateral force is taken by the brace. In fact, depending on the properties of the frame and connection, some portion of the load is taken by the frame itself. For one specimen (Specimen 6), Johnson (2005) reported that about 85% of the horizontal load is in the brace. However, this information is not available for other specimens and the actual amount transferred depends on the frame and connection properties. In addition, the calculation of the ultimate deformation in the brace assumes that there is no significant deformation in the connections or other framing elements. When there is significant deformation in the connections, the ultimate deformation of the brace itself is overestimated. In this example, the overestimation occurs primarily with the reference component tests, introducing additional conservatism to the comparison and evaluation of the proposed components. The overstrength in the reference components is also overestimated.

The differences in frame versus brace behavior also illustrate the important influence that frame and connection elements in special concentrically braced frame have on overall system behavior. For example, Figure E-6 shows that the frame tested shows very little strength loss after buckling, which occurs in the range of 0.2% to 0.5% drift. This ductility is likely provided by the moment resistance of frame members, and it is not clear whether the frame ductility evident in these tests is generally representative of buildings designed according to AISC 341-10. In addition, since a single buckling-restrained brace may be used to replace more than one conventional brace, the transfer of forces across the component boundary and engagement of the frame in the system behavior may be different for the two different components. These observations indicate that a system analysis of buckling-restrained brace frames as described in FEMA P-695, and illustrated in NIST GCR 10-917-8 (NIST, 2010) may be more appropriate.

E.11.3 Component Parameters are Approximate

Because component parameters were obtained from graphical interpretation of load-deformation plots, rather than digitized data. This approach may lead to errors in estimation of parameters, especially those related to initial stiffness and effective yield displacement.

Appendix F
Test Application: Pre-Fabricated Wall Components

F.1 Introduction

This appendix illustrates the application of the Component Methodology to the possible substitution of pre-fabricated wall components for nailed wood shear wall components in wood light-frame construction. In accordance with the request of the manufacturer, the specific product is not identified, but is representative of a number of proprietary wall products from different manufacturers, including Weyerhaeuser, Hardy Frames, Simpson Strong-Tie, and others. These products are typically slender elements with very high-aspect ratios that are used at the boundaries of large openings where available wall length is limited (e.g., garage wall lines in residential construction), and are particularly common in regions of high seismicity or strong winds.

Available design information and test data were limited for the pre-fabricated wall component, consisting of cyclic-load testing on three specimens of one configuration. As such, these data are not sufficient to permit a complete application of the Component Methodology. Nonetheless, these data are useful in terms of testing the methods for construction of cyclic envelope curves, extraction of key parameters from these curves, and the evaluation of component parameters using the acceptance criteria of Section 2.8.

Although incomplete as a full application of the Component Methodology, this test application illustrates a special case that might be consistent with a preliminary check of equivalency during the development of a new product. In this case, prototypical specimens of a representative (or critical) configuration are constructed and cyclic-load tested, and key performance parameters evaluated, to determine if the product will have reasonably adequate deformation capacity and ductility. In such an application, at least two specimens should be tested to assess the repeatability of key parameters.

F.2 Description of Pre-Fabricated Wall Component

The pre-fabricated wall component is intended for use in wood light-frame construction. The component is 9 feet (108 inches) tall and is very short in length. The details of the materials and construction were not available.

Figures F-1 and F-2 show typical applications of two similar high-aspect ratio, pre-fabricated wall products.

Figure F-1 Example residential building application of high-aspect ratio pre-fabricated wall components (www.sbebuilders.com/images/hardy-frame-2.jpg).

Figure F-2 Example commercial building application of high-aspect ratio, pre-fabricated wall components (www.sbebuilders.com/images/strong-wall.jpg)

F.3 Evaluation of Applicability Criteria

The applicability of the Component Methodology is based largely on the comparison of characteristics of components of the reference system, in this case, the light-frame (wood) wall system of line A.15 in Table 12.2-1 of ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures*

(ASCE, 2010) and those of the proposed component. While the applicability of the Component Methodology could not be rigorously evaluated, due to lack of specific design and configuration data, the Component Methodology is expected to generally apply to these types of pre-fabricated wall components.

Of particular importance to the applicability of high-aspect wall components is the definition of the component boundary, and related boundary conditions, which must be realistically represented by the test setup. Possible differences to consider would include additional overturning demands due to differences in aspect ratio between pre-fabricated wall components and typical configurations of nailed wood shear wall reference components.

The boundary of pre-fabricated wall components is typically well defined by the manufacturer's drawings of typical details for anchoring and connecting these products to the balance of the seismic-force-resisting system. This test application presumes that these types of drawings would be available for evaluating force transfer at the component boundary and for establishing boundary conditions of tests.

F.4 Reference Component Test Data

The reference component test data in this example is taken as the same nailed wood shear wall data set developed for the example application in Chapter 4. Table F-1 shows the summary statistics for the reference component data set.

Table F-1 Summary Statistics for Reference Component Parameters (from Table 4-3)

Summary Statistic	R_Q $(=V_M / V_D)$	R_K $(=K_I / K_D)$	μ_{eff}	Δ_U (in/in)
Median	$\tilde{R}_Q = 2.7$	$\tilde{R}_K = 1.00$	$\tilde{\mu}_{eff} = 6.3$	$\tilde{\Delta}_U = 0.035$
Variability	$\sigma_{RQ} = 0.11$	$\sigma_{RK} = 0.42$	$\sigma_{\mu,eff} = 0.38$	$\sigma_{\Delta U} = 0.16$

F.5 Proposed Component Design Requirements

Design data and requirements were not available for the pre-fabricated component, other than an ASD design load of Q_D = 580 lbs, and a drift displacement of 0.38 inches (i.e., drift ratio of 0.0035 in./in.) at the ASD design load. This corresponds to a design stiffness of K_D = 1,514 lbs/in.

In general, manufacturers of pre-fabricated wall components provide detailed product guides that describe installation requirements, including typical anchorage and connection details and construction requirements, and tables of design loads and other design criteria. This test application presumes that these types of design and construction requirements would be available for establishing design loads and evaluating the quality of design requirements.

F.6 Proposed Component Test Data

Cyclic-load test data were provided by the manufacturer for three specimens of one proposed component configuration. While this configuration is somewhat different from the configurations contained in the reference component data set, this is acceptable and does not violate any of the applicability criteria of Section 2.3. No monotonic-load test data were available.

Cyclic-load test data were plotted, envelope curves fitted, and values of various load, deformation and related parameters determined from the envelop curves, as shown in Figure F-3.

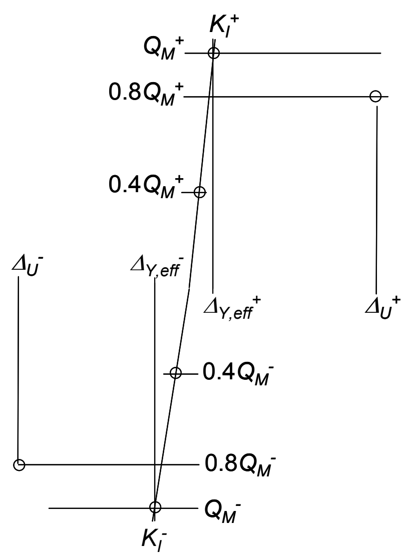


Figure F-3 Illustration of cyclic-load testing data and cyclic envelope curve (from Figure 2-2).

Plots of the cyclic-load test data and envelopes are shown in Figures F-4, F-5 and F-6 for the three test specimens. The test loops are very stable and similar for the three test specimens. Table F-4 summarizes values of strength, stiffness, ductility and deformation capacity parameters obtained for these three test specimens, and shows the summary statistics (median and variability) for these four parameters.

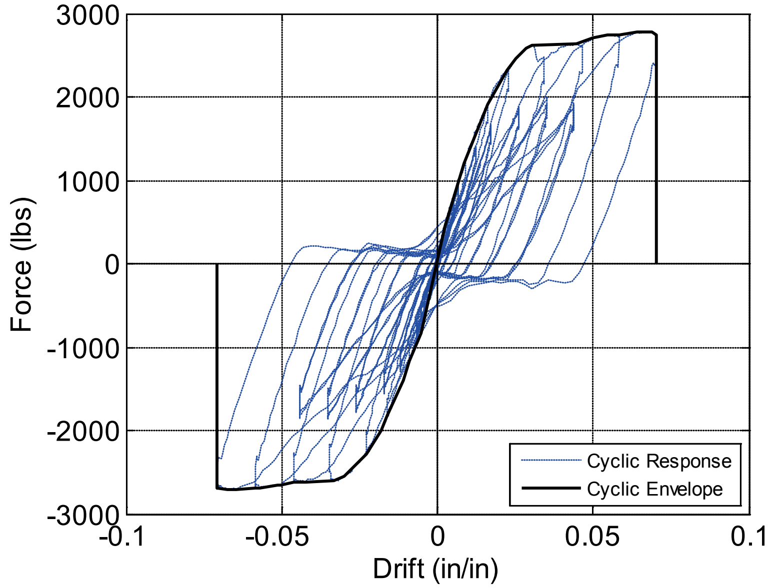


Figure F-4 Cyclic response and envelope curve for pre-fabricated component test specimen No.1.

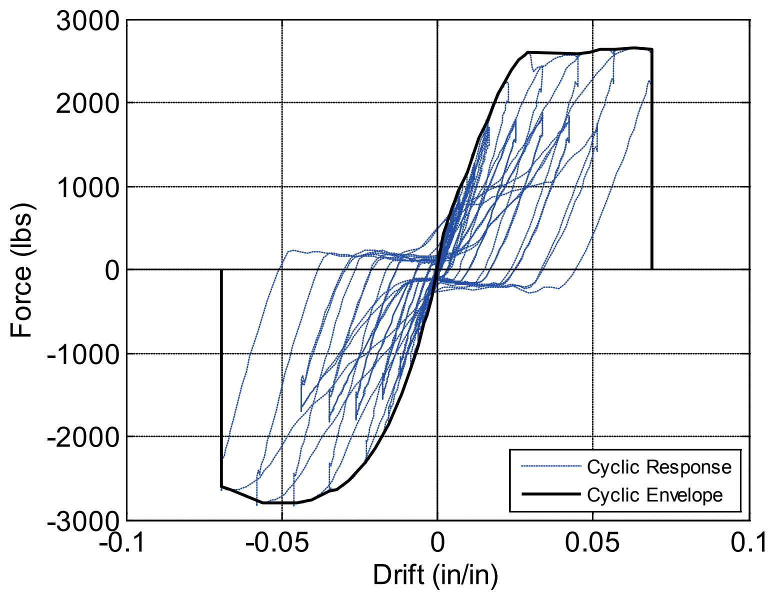


Figure F-5 Cyclic response and envelope curve for pre-fabricated component test specimen No.2.

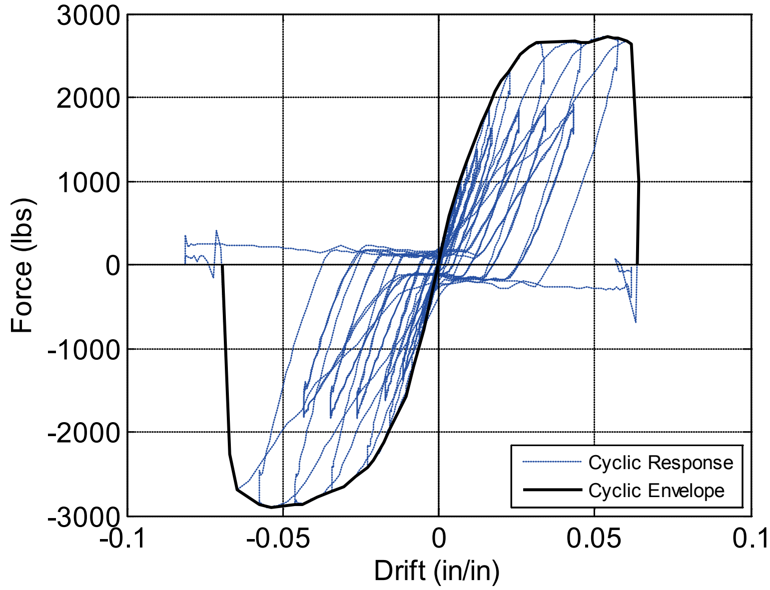


Figure F-6 Cyclic response and envelope curve for pre-fabricated component test specimen No.3.

Table F-2 Values of Strength, Stiffness, Ductility and Deformation Capacity Parameters and Summary Statistics for the Proposed Component

Test Index	Strength			Stiffness			Ductility		Deformation Capacity	
	V_M (lb)	V_D (lb)	$R_Q = V_M / V_D$	K_I (lb/in)	K_D (lb/in)	$R_K = K_I / K_D$	$\Delta_{Y,eff}$ (in/in)	μ_{eff}	Δ_U (in/in)	Δ_{Uj}
1	2,745	580	4.7	1,248	1,514	0.8	0.020	3.5	0.071	
2	2,730	580	4.7	1,211	1,514	0.8	0.021	3.3	0.070	0.068
3	2,810	580	4.8	1,317	1,514	0.9	0.020	3.3	0.065	
Median:	$\tilde{R}_Q = 4.8$			$\tilde{R}_K = 0.83$			$\tilde{\mu}_{eff} = 3.4$		$\tilde{\Delta}_U = 0.068$	
Variability:	$\sigma_{RQ} = 0.02$			$\sigma_{RK} = 0.04$			$\sigma_{\mu,eff} = 0.03$		$\sigma_{\Delta U} = 0.05$	

F.7 Evaluate Quality Ratings

F.7.1 Quality Rating of Test Data

Consistent with the example of Chapter 4, reference component test data is rated as Superior.

There was not sufficient information to rate the quality of proposed component test data due to limited number of available test data. For the purpose of this test application, it was assumed that proposed component test data would be rated as Good, if all required cyclic-load and monotonic-load tests were performed.

F.7.2 Quality Rating of Design Requirements

Consistent with the example of Chapter 4, the quality rating for the reference component design requirements is Good.

There was not sufficient information to rate the quality of proposed component design requirements. For the purpose of this test application, it was assumed that proposed component design requirements would be rated as Superior, if product information, including design and construction requirements, were comparable to that typically provided by manufacturers of pre-fabricated wall product. The higher quality rating of the proposed component design requirements recognizes the greater control that can be achieved in pre-fabricated products that are manufactured in a controlled environment (rather that site-built), provided they are properly installed in the field.

F.8 Evaluate Component Equivalency

F.8.1 Overview

This section discusses the acceptance criteria of the Component Methodology and evaluates whether the proposed pre-fabricated component can be judged to be equivalent for use in light-frame wood construction. The evaluation of equivalency is necessarily incomplete due to the limited amount of available test data. Nonetheless, results provide a useful evaluation of equivalency to the degree that a single configuration can represent the balance of all possible configurations of the pre-fabricated wall component of interest.

The Component Methodology evaluates equivalency of proposed and reference components by performance group (components grouped in terms of comparable characteristics). In this test application, there is only one performance group, and the proposed component is represented by a single configuration with three test specimens. The summary statistics of the proposed component performance group are assumed to be the same as those of the three specimens of the single configuration.

If test data were available for all configurations of the proposed component, then the summary statistics of the proposed component performance group would likely be different from those based on the single configuration of this test application. In general, median values of parameters would be expected to vary modestly, while values of log standard deviation would be expected to increase significantly.

F.8.2 Requirements Based on Cyclic-Load Test Data: Strength and Ultimate Deformation

Section 2.8.2 requires that the median ultimate deformation of each proposed component performance group satisfy Equation (2.1), as follows:

$$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} \; P_U \; P_Q$$

The median ultimate deformation, $\tilde{\Delta}_{U,RC}$, of the reference component, expressed in terms of drift ratio, is 0.035 in/in. (from Table F-1), and the median ultimate deformation, $\tilde{\Delta}_{U,PC}$, of the proposed component, expressed in terms of drift ratio, is 0.068 in/in. (from Table F-2). The penalty factor for strength is based on the ratio of median strength ratios of proposed and reference components. The ratio of proposed component strength (i.e., overstrength) is 4.8 (from Table F-2) and the ratio reference component strength is 2.7 (from Table F-1) and the ratio of these values is:

$$\frac{\tilde{R}_{Q,PC}}{\tilde{R}_{Q,RC}} = \frac{4.8}{2.7} = 1.78$$

Based on this ratio and from Table 2-4, the value of the penalty factor is P_Q = 1.24.

Since the ratio exceeds 1.2, Section 2.8.2 requires force-controlled and capacity designed elements of the reference SFRS to be designed to develop the expected strength of the proposed component (i.e., 4.8 x 580 lbs = 2,784 lbs). Regardless of the value of this ratio, Section 2.5.2 requires the attachment of the proposed component to the balance of the reference SFRS to be strong enough to develop the full ultimate strength of the proposed component, such that inelastic behavior occurs in the proposed component, and not at the boundary between the proposed component and the balance of the reference SFRS.

From Table 2-3, based on the quality ratings described in Section F.7, the value of uncertainty factor is P_U= 1.0. In this case, differences in the quality ratings of test data and design requirements of the proposed and the reference components essentially cancel out.

Incorporating these values into the first equation leads to the following check of ultimate deformation equivalency:

$$0.068 \geq 0.035\,(1.0)\,(1.24) = 0.043 \qquad \underline{\underline{OK}}$$

This requirement is easily satisfied since the median ultimate deformation capacity of the pre-fabricated wall is quite large.

In addition to the equivalency requirement for the median of each component performance group, $\tilde{\Delta}_{U,PC}$, the median ultimate deformation of each configuration, $\tilde{\Delta}_{Uj,PC}$, must meet the requirement of Equation (2-2), as follows:

$$\tilde{\Delta}_{Uj,PC} \geq \left(1 - 1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \; P_Q$$

This equation checks for "outlier" configurations, which cannot be meaningfully evaluated without having summary statistics for all configurations of the proposed component.

F.8.3 Requirements Based on Cyclic-Load Test Data: Effective Initial Stiffness

Section 2.8.3 requires that the median initial stiffness ratio of the proposed component be reasonably similar to median initial stiffness ratio of the reference component, in accordance with Equation (2-3), as follows:

$$0.75 \leq \frac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33$$

As reported earlier, the median initial stiffness ratio, $\tilde{R}_{K,RC}$, of the reference component is 1.00 (from Table F-1) and the median initial stiffness ratio, $\tilde{R}_{K,PC}$, of the proposed component is 0.83 (from Table F-2).

Incorporating these values leads to the following check of initial stiffness equivalency:

$$0.75 \leq \frac{0.83}{1.00} \leq 1.33 \qquad \underline{\underline{OK}}$$

As shown, the initial stiffness requirement is satisfied. While the initial stiffness of the proposed component is much less than that of the reference component, the design stiffness of the proposed component is also much less than that of the reference component.

F.8.4 Requirements Based on Cyclic Test Data: Effective Ductility Capacity

Section 2.8.4 requires that the median effective ductility of the proposed component be not less than one-half of the median effective ductility of the reference component, in accordance with Equation (2-4), as follows:

$$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC}$$

The median effective ductility, $\tilde{\mu}_{eff,RC}$, of the reference component is 6.3 (from Table F-1) and the median initial stiffness ratio, $\tilde{\mu}_{eff,PC}$, of the proposed component is 3.4 (from Table F-2).

Incorporating these values into Equation (F-7) leads to the following check of effective ductility:

$$3.4 \geq (0.5)6.3 = 3.2 \qquad \underline{\underline{OK}}$$

As shown, the effective ductility requirement is satisfied, although with a modest margin, essentially using all of the proposed component's ultimate displacement capacity to meet the effective ductility criterion.

F.8.5 Requirements Based on Monotonic Load Test Data: Ultimate Deformation

Section 2.8.5 requires that monotonic-load data be used to show that the proposed component has some additional ultimate displacement capacity with respect to the cyclic-load ultimate displacement capacity of the reference component, in accordance with either Equation (2-5) or (2-6), as follows:

$$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC}\ P_U\ P_Q$$

$$\tilde{\Delta}_{UM,PC} \geq 1.2\ D_C\ \tilde{\Delta}_{U,RC}\ P_U\ P_Q$$

Since monotonic data were not available, the latter equation was used to evaluate this requirement, where monotonic-based median ultimate displacement, $\tilde{\Delta}_{UM,PC}$, is taken as equal to the cyclic-based median ultimate displacement, $\tilde{\Delta}_{U,PC}$, of 0.068. The value of $\tilde{D}_C$ is 1.0, based on the use of CUREE cyclic-load test protocol to perform the cyclic-load tests of the reference component. Values of penalty factors, P_U and P_Q, are as previously defined.

Incorporating these values leads to the following check of ultimate displacement capacity for monotonic-loading:

$$0.068 \geq 1.2(1.0)0.035(1.0)(1.24) = 0.052 \quad \underline{\underline{OK}}$$

As shown, the check of monotonic ultimate displacement capacity is satisfied, using the cyclic-load test ultimate deformation of the proposed component as a surrogate for the monotonic-load test ultimate deformation of the proposed component. This result shows that monotonic-load tests would not be required for the pre-fabricated wall component.

Table F-3 provides a summary of the acceptance criteria and the results of the evaluation of the equivalency of the pre-fabricated wall components and nailed wood shear walls components.

Table F-3 Summary of Acceptance Criteria Evaluation for Pre-Fabricated Wall Components

Acceptance Criteria		Equation Reference	Pass/Fail
Requirement Based on Cyclic-Load Test Data			
Ultimate Deformation Capacity (performance group)	$\tilde{\Delta}_{U,PC} \geq \tilde{\Delta}_{U,RC} \cdot P_U \cdot P_Q$	2-1	Pass
Ultimate Deformation Capacity (individual configurations)	$\tilde{\Delta}_{Uj,PC} \geq \left(1-1.5\sigma_{\Delta_U,RC}\right)\left(\tilde{\Delta}_{U,RC}\right) P_U \; P_Q$	2-2	N/A
Initial Stiffness Ratio	$0.75 \leq \dfrac{\tilde{R}_{K,PC}}{\tilde{R}_{K,RC}} \leq 1.33$	2-3	Pass
Effective Ductility Capacity	$\tilde{\mu}_{eff,PC} \geq 0.5\,\tilde{\mu}_{eff,RC}$	2-4	Pass
Requirements Based on Monotonic-Load Test Data			
Ultimate Deformation Capacity (Option 1)	$\tilde{\Delta}_{UM,PC} \geq \tilde{\Delta}_{UM,RC} \; P_U \; P_Q$	2-5	N/A
Ultimate Deformation Capacity (Option 2)	$\tilde{\Delta}_{UM,PC} \geq 1.2\,\tilde{D}_C \; \tilde{\Delta}_{U,RC} \; P_U \; P_Q$	2-6	Pass

F.9 Summary of Component Equivalency Evaluation of Pre-Fabricated Wall Components

This test application of the Component Methodology for pre-fabricated wall components, although limited to the evaluation of just three cyclic-load test specimens on a single configuration, provided useful information on the Component Methodology when applied to the proposed substitution of a product that is inherently more flexible than that of the reference system. In this case, the high-aspect ratio pre-fabricated wall component is a relatively strong, short (slender) proprietary product intended for substitution in light-frame wood wall construction, and is representative of similar proprietary products from a number of manufacturers.

Overall, the test application found that the pre-fabricated wall component was equivalent. This finding is consistent with findings of the International Code Council Evaluation Services (ICC ES) for similar products using the acceptance criteria of AC322.

References

ACI, 2001, *Acceptance Criteria for Moment Frames Based on Structural Testing* (ACI T1.1-01) and *Commentary* (ACI T1.1R-01), Innovation Task Group 1 and collaborators, American Concrete Institute, Farmington Hills, Michigan.

AF&PA, 2005, *Special Design Provisions for Wind and Seismic Standard with Commentary,* ANSI/AF&PA SDPWS-2005, American Forest & Paper Association, Washington, D.C.

AF&PA, 2009, *Special Design Provisions for Wind and Seismic Standard with Commentary*, ANSI/AF&PA SDPWS-2008, American Forest & Paper Association, Washington, D.C.

AISC, 2011, *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-10, American Institute for Steel Construction, Chicago, Illinois.

ASCE, 2005, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, Virginia.

ASCE, 2010, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, Virginia.

Aslani, F. and Goel, S.C., 1989, *Experimental and Analytical Study of the Inelastic Behavior of Double Angle Bracing Members under Severe Cyclic Loading*, Research Report UMEE 89-5, University of Michigan.

Astaneh-Asl, A., Goel, S.C., and Hanson, R.D., 1982, *Cyclic Behavior of Double Angle Bracing Members with End Gusset Plates*, Research Report UMEE 82R7, University of Michigan.

ASTM, 2009, *Standard Test Method for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Load Force Resisting System for Buildings*, ASTM E2126-09, American Society for Testing and Materials International, West Conshohocken, Pennsylvania.

ATC, 1992, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*, Report No. ATC-24, Applied Technology Council, Redwood City, California.

Benzoni, G. and Innamorato, D, 2007, *StarSeismic Brace Tests. Mercy San Juan Hospital Project*, Report No. SRMD-2007-05-rev2, University of California, San Diego.

Bernal, D., 1992, "Instability of buildings subjected to earthquakes," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 118, No. 8, pp. 2239-2260.

Bernal, D., 1998, "Instability of buildings during seismic response," *Engineering Structures*, Vol. 20, No. 4-6, pp. 496-502.

Berry, M., Parrish, M., and Eberhard, M., 2004, *PEER Structural Performance Database User's Manual*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, 38 pp. Available at http://nisee.berkeley.edu/spd/ and http://maximus.ce.washington.edu/~peera1/.

Black, C., Makris, N., and Aiken, I., 2002, *Component Testing, Stability Analysis and Characterization of Buckling-Restrained Unbonded Braces*, PEER Report 2002/08, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

CEN, 1995, *Timber Structures–Test Methods–Cyclic Testing of Joints Made with Mechanical Fasteners*, IN TC 124.117, European Committee for Standardization, Brussels, Belgium.

Cheung, P.C., Pauley, T., Park, R., 1991, "New Zealand tests on full-scale reinforced concrete beam-column-slab sub-assemblages designed for earthquake resistance," ACI Special Publication SP-123, *Design of Beam-Column Joints for Seismic Resistance*, American Concrete Institute, Detroit, Michigan, pp. 1-37.

Christopulos, A.S., 2005, *Improved Seismic Performance of Buckling Restrained Braced Frames*, M.Sc. Thesis, University of Washington.

Clark, K.A., 2009, *Experimental Performance of Multi-Story X-Braced SCBF Systems*, M.Sc. Thesis, University of Washington.

Clark, P., Frank, K., Krawinkler, H., and Shaw, R., 1997, *Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens*, SAC Steel Project Background Document, Report No. SAC/BD-97/02.

Clough, R.W. and Johnston, S.B., 1966, “Effects of stiffness degradation on earthquake ductility requirements,” *Proc. 2nd Japan Earthquake Engrg. Symp.*, pp. 227–232.

CoLA, 2001, *Report of a Testing Program of Light-Framed Walls with Wood-Sheathed Panels*, final report to the City of Los Angeles Department of Building and Safety, by Structural Engineers Association of Southern California and COLA-UCI Light Frame Test Committee, Department of Civil Engineering, University of California, Irvine, California, 93 pp.

Dolsek, M. and Fajfar, P., 2004, “Inelastic spectra for infilled reinforced concrete frames,” *Earthquake Engineering and Structural Dynamics*, Vol. 33, pp. 1395–1416.

El Tayem, A. and Goel, S.C., 1985, *Cyclic Behavior of Angle X-Bracing with Welded Connections*, Research Report UMEE 85-4, University of Michigan.

Engelhardt, M., 2007, *Teaching Principles of Seismic-Resistant Design of Steel Building Structures,* prepared for the American Institute of Steel Construction. Available at http://www.aisc.org/content.aspx?id=21490. Last accessed February 25, 2010.

Fardis, M.N. and Biskini, D.E., 2003, “Deformation capacity of RC members, as controlled by flexure or shear,” *Proceedings, Otani Symposium*, University of Tokyo, Japan, pp. 511- 530.

FEMA, 2007, *Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components,* FEMA 461, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2009a, *Effects of Strength and Stiffness Degradation on Seismic Response,* FEMA P-440A, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA, 2009b, *Quantification of Building Seismic Performance Factors*, FEMA P-695, prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

Filiatrault, A., Wanitkorkul, A., and Constantinou, M.C., 2008, *Development and Appraisal of a Numerical Cyclic Loading Protocol for Quantifying Building System Performance*, Technical Report

MCEER-08-0013, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, New York.

Folz, B., and Filiatrault, A., 2004a, "Seismic analysis of woodframe structures I: model formulation," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 130, No. 8, pp. 1353-1360.

Folz, B. and Filiatrault, A., 2004b, "Seismic analysis of woodframe structures II: model implementation and verification," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 130, No. 8, pp. 1361-1370.

Foutch, D.A. and Shi, S., 1998, "Effects of hysteresis type on the seismic response of buildings," *Proceedings of the Sixth U.S. National Conference on Earthquake Engineering*, Seattle, Washington.

Gatto, K. and Uang, C.M., 2002, *Cyclic Response of Woodframe Shearwalls: Loading Protocol and Rate of Loading Effects*, CUREE Report No. W-16, Consortium of Universities for Earthquake Engineering Research, Richmond, California.

Gupta, A. and Krawinkler, H., 1998, "Effect of stiffness degradation on deformation demands for SDOF and MDOF structures," *Proceedings of the Sixth U.S. National Conference on Earthquake Engineering*, Seattle, Washington.

Gupta, A. and Krawinkler, H., 1999, *Seismic Demands for Performance Evaluation of Steel Moment-Resisting Frame Structures*, John A. Blume Earthquake Engineering Center Technical Report No. 132, Stanford University, Stanford, California.

Haselton, C.B. and Deierlein, G.G., 2007, *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings*, John A. Blume Earthquake Engineering Center Technical Report No. 156, Stanford University, Stanford, California.

Haselton, C.B., Mitrani-Reiser, J., Goulet, C., Deierlein, G.G., Beck, J., Porter, K.A., Stewart, J., and Taciroglu, E., 2008, *An Assessment to Benchmark the Seismic Performance of a Code-Conforming Reinforced-Concrete Moment-Frame Building*, PEER Report 2007/12, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California.

Ibarra, L., 2003, *Global Collapse of Frame Structures Under Seismic Excitations*, Ph.D. Dissertation, Department of Civil and

Environmental Engineering, Stanford University, Stanford, California.

Ibarra, L.F. and Krawinkler, H., 2005, *Global Collapse of Frame Structures Under Seismic Excitations*, PEER Report 2005/06, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, and John A. Blume Earthquake Engineering Center Technical Report No. 152, Department of Civil Engineering, Stanford University, Stanford, California.

Ibarra, L.F., Medina, R.A., and Krawinkler, H., 2005, "Hysteretic models that incorporate strength and stiffness deterioration," *International Journal for Earthquake Engineering and Structural Dynamics,* Vol. 34, No.12, pp. 1489-1511.

ICC, 2006, *International Building Code,* International Code Council, Washington, D.C.

ICC, 2009, *International Building Code,* International Code Council, Washington, D.C.

ICC-ES, 2007, *Acceptance Criteria for Prefabricated, Cold-Formed, Steel Lateral-Force-Resisting Vertical Assemblies*, AC322, International Code Council Evaluation Services, Whittier, California.

ISO, 2003, *Timber Structures-Joints Made with Mechanical Fasteners – Quasi-Static Reversed-Cyclic Test Method*, ISO 16670:2003, International Organization for Standardization, Geneva, Switzerland.

ISO, 2005, *General Requirements for the Competence of Testing and Calibration Laboratories*, ISO/IEC 17025:2005, International Organization for Standardization, Geneva, Switzerland.

Iwan, W.D., 1973, "A model for the dynamic analysis of deteriorating structures," *Proc. Fifth World Conference on Earthquake Engineering,* Rome, Italy, pp. 1782-1791.

Iwan, W.D., 1977, "The response of simple stiffness degrading structures," *Proc. Sixth World Conference on Earthquake Engineering*, pp. 1094-1099.

Iwan, W.D., 1978, "The earthquake response of strongly deteriorating systems including gravity effects," *Proc. Sixth European Conference on Earthquake Engineering*; pp. 23-30.

Johnson, S., 2005, *Improved Seismic Performance of Special Concentrically Braced Frames*, M.Sc. Thesis, University of Washington.

Karacabeyli, E., 1998, "Lateral resistance of nailed shear walls subjected to static and cyclic displacements," *Forest Products Society 49th Annual Meeting*, Portland, Oregon.

Kim, D.W., Sim, H.B., Uang, C.M., and Benzoni, G., 2010, *Subassemblage Testing of CoreBrace Buckling-Restrained Braces (H series)*, Report No. TR 09-01, University of California, San Diego.

Kotulka, B.A., 2007, *Analysis for a Design Guide on Gusset Plates used in Special Concentrically Braced Frames*, M. Sc. Thesis, University of Washington.

Krawinkler, H., 2009, "Loading histories for cyclic tests in support of performance assessment of structural components," *Proceedings 3AESE Conference*, San Francisco, California.

Krawinkler, H. and Lignos, D, 2009, "How to predict the probability of collapse of non-ductile building structures," book chapter in Seismic Assessment and Retrofitting, Geotechnical, Geological, and Earthquake Engineering, Vol. 10, published by Springer, NY.

Krawinkler, H., Gupta, A., Medina, R., and Luco, N., 2000, *Development of Loading Histories for Testing of Steel Beam-to-Column Assemblies*, SAC Background Report SAC/BD-00/10.

Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., and Medina, R., 2000, *Development of a Testing Protocol for Woodframe Structures*, CUREE Report No. W-02, Consortium of Universities for Earthquake Engineering Research, Richmond, California.

Kumar, G.R., Kumar, S.R., and Kalyanaraman, V., 2007, "Behaviour of frames with non-buckling bracings under earthquake loading," *Journal of Constructional Steel Research*, Vol. 63, No. 2.

Lee, L.H., Han, S.W., and Oh, Y.H., 1999, "Determination of ductility factor considering different hysteretic models," *Earthquake Engineering and Structural Dynamics*, Vol. 28, 957–977.

Line, P., Waltz, N., and Skaggs, T., 2008, "Seismic equivalence parameters for engineered wood frame wood structural panel shear walls," *Wood Design Focus*, Vol. 18 No. 2.

Mahin, S.A. and Bertero, V.V., 1981, "An evaluation of inelastic seismic design spectra," *Journal of the Structures Division*, American Society of Civil Engineers, Vol. 107, No. 9, pp. 1777-1795.

Medina, R. and Krawinkler, H., 2004, "Influence of hysteretic behavior on the nonlinear response of frame structures," *Proc. 13th World* Conference on Earthquake Engineering, Vancouver, Canada.

Merritt, S., Uang, C.M., and Benzoni, G., 2003a, *Subassemblage Testing of CoreBrace Buckling-Restrained Braces*, Report No. TR-03/01, University of California, San Diego.

Merritt, S., Uang, C.M., and Benzoni, G., 2003b, *Subassemblage Testing of StarSeismic Buckling-Restrained Braces*, Report No. TR-03/04, University of California, San Diego.

Miranda, E. and Akkar, S.D., 2003, "Dynamic instability of simple structural systems," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 129, No. 12, pp 1722-1727.

Newell, J., Uang, C.M., and Benzoni, G., 2005, *Subassemblage Testing of CoreBrace Buckling-Restrained Braces (F series)*, Report No. TR-05/01, University of California, San Diego.

Newell, J., Uang, C.M., and Benzoni, G., 2006, *Subassemblage Testing of CoreBrace Buckling-Restrained Braces (G series)*, Report No. TR-06/01, University of California, San Diego.

Nippon Steel Engineering, "Unbonded Brace: Basic Design Information," *Nippon Steel Engineering Co. Ltd.,* Building Construction and Steel Structures Div., Undated.

NIST, 2010, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors,* NIST GCR 10-917-8, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Researh in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, Maryland.

OpenSees, 2009, *Open System for Earthquake Engineering Simulation*, Pacific Earthquake Engineering Research Center, University of California, Berkeley. Available at http://opensees.berkeley.edu/.

Otani, S., 1981, "Hysteresis model of reinforced concrete for earthquake response analysis," *Journal, Fac. of Eng.,* University of Tokyo, Series B, Vol. XXXVI-11, 2, 407-441.

Panagiotakos, T.B., and Fardis, M.N., 2001, "Deformations of reinforced concrete members at yielding and ultimate," *ACI Structural Journal*, Vol. 98, No.2.

PEER/ATC, 2010, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings*, PEER/ATC-72-1 Report, prepared by the Applied Technology Council in cooperation with the Pacific Earthquake Engineering Research Center, Redwood City, California.

Pincheira, J.A, Dotiwala, F.S., and D'Souza, J.T., 1999, "Spectral displacement demands of stiffness- and strength-degrading systems," *Earthquake Spectra*, Vol. 15, No. 2, pp. 245–272.

Porter, M.L., 1987, "Sequential phased displacement (SPD) procedure for TCCMAR testing," *Proceedings, 3rd Meeting of the Joint Technical Coordinating Committee on Masonry Research*, US-Japan Coordinated Research Program.

Powell, J.A., 2010, *Evaluation of Special Concentrically Braced Frames for Improved Seismic Performance and Constructability*, M.S.E. Thesis, University of Washington.

Rahnama, M. and Krawinkler, H., 1993, *Effects of Soft Soil and Hysteretic Models on Seismic Demands*, John A. Blume Earthquake Engineering Center Technical Report No. 108, Stanford University, Stanford, California.

Riddell, R. and Newmark, N.M., 1979a, *Statistical Analysis of the Response of Nonlinear Systems Subjected to Earthquakes*, Rep. No. UILI-ENG79-2016, Structural Research Series, Dept. of Civil Engineering, University of Illinois, Urbana.

Riddell, R. and Newmark, N.M., 1979b, "Force-deformation models for nonlinear analyses," *J. Struct. Div.*, American Society of Civil Engineers, 105, pp. 2773–2778.

Reaveley, L.D., Okahashi, T., and Farr, C.K., 2004, Corebrace Series E Buckling-Restrained Brace Test Results, University of Utah Report.

Rosowsky, D., Elkins, L., and Carrol, C., 2004, *Cyclic Tests of Engineered Shear Walls Considering Different Plate Washers*, prepared by Oregon State University for the American Forest and Paper Association.

Ruiz-Garcia, J. and Miranda, E., 2005, *Performance-Based Assessment of Existing Structures Accounting for Residual Displacements*, John A. Blume Earthquake Engineering Center Technical Report No. 153, Stanford University, Stanford, California.

Sabelli, R., 2007, *Teaching Principles of Seismic-Resistant Design of Steel Building Structures: Special Plate Shear Walls,* prepared for the

American Institute of Steel Construction. Available at http://www.aisc.org/content.aspx?id=21490. Last accessed February 25, 2010.

Sim, H.B., Kim, D.W., Uang, C.M., and G. Benzoni, G., 2010, *Subassemblage Testing of CoreBrace Buckling-Restrained Braces (J series),* Report No. TR-09/02, 2010, University of California, San Diego.

Song J.K. and Pincheira, J.A., 2000, "Seismic analysis of older reinforced concrete columns," *Earthquake Spectra*, Vol. 16, No. 4, pp. 817–851.

SST, 2009, *Wood Construction Connectors Catalogue*, Simpson Strong-Tie Company, Inc. Available at http://www.strongtie.com/literature. Last accessed November 20, 2009.

Talbot, K.C., Reaveley, L.D., and Pantelides, C.P., 2009, "Structural performance of stapled wood shear walls under dynamic cyclic loads," *Earthquake Spectra*, Vol. 25, No. 1, pp. 161-183.

Tsai, K.C. and Hsiao, P.C., 2008, "Pseudo-dynamic test of a full-scale CFT/BRB frame—Part II: Seismic performance of buckling-restrained braces and connections," *Earthquake Engineering and Structural Dynamics,* Vol. 37, No. 7.

Uang, C.M., Yu, K., and Gilton, C., 2000, *Cyclic Response of RBS Moment Connections: Loading Sequence and Lateral Bracing Effects*, Structural Steel Research Project Report No. SSRP-99/13.

Vamvatsikos, D. and Cornell, C.A., 2002, "Incremental Dynamic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 31, Issue 3, pp. 491-514.

Vian, D. and Bruneau, M., 2003, "Tests to structural collapse of single degree of freedom frames subjected to earthquake excitations," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 129, No. 112, pp. 1676-1685.

Wakabayashi, M., Nakamura, T., and Yoshida, N., 1977, "Experimental studies on the elastic-plastic behavior of braced frames under repeated horizontal loading: Part 1 experiments of braces with an H-shaped cross section in frame," *Bulletin of the Disaster Prevention Research Institute,* Vol. 27, Part 3, No. 251.

Wakabayashi, M., Nakamura, T., and Yoshida, N., 1980, "Experimental studies on the elastic-plastic behavior of braced frames under repeated horizontal loading: Part 1 experiments of braces composed of steel circular tubes, angle-shapes, flat bars and round bars," *Bulletin of the Disaster Prevention Research Institute,* Vol. 29, Part 3, No. 264.

Zareian, F., 2006, *Simplified Performance-Based Earthquake Engineering*, Dissertation, Department of Civil and Environmental Engineering, Stanford University.

Zareian, F. and H. Krawinkler, 2010, "Structural system parameter selection based on collapse potential of buildings in earthquakes," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 136, No. 8, pp. 933-943.

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