

GLOBAL TOPICS REPORT ON THE PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS



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PRESTANDARD AND COMMENTARY FOR THE
SEISMIC REHABILITATION OF BUILDINGS**

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Reston, Virginia**

**Prepared for
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ASCE Standards Program and the Structural Engineering Institute

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Foreword

Among the FEMA documents covering the topic of making existing buildings more resistant to the effects of earthquakes, this volume occupies a unique position: it is the only one that fulfills a historical need. When the decision was made to convert the performance-based *Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, into a prestandard containing mandatory language (FEMA 356), there was considerable concern among design professionals that some of the major characteristics and salient features of the original document (or indeed its very fabric) would be adversely affected in the conversion process. This volume was purposely conceived to allay such concerns by providing a transparent and permanent record of the changes that were made and the reasons for such changes, as well as the major challenges encountered in the conversion process and how they were resolved. It is hoped that this volume will also serve as a useful tool in facilitating the further conversion of the prestandard into an ANSI-approved standard by the American Society of Civil Engineers.

FEMA and the FEMA Project Officer are warmly thankful to the Project Team and consultants, the Project Advisory Committee, and the staff of the American Society of Civil Engineers for their dedicated efforts in completing this unique volume.

The Federal Emergency Management Agency

Preface

This Global Topics Report is the third in a series of reports chronicling the development of the FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* into the FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. The purpose of this report is to provide a narrative discussion and permanent record of the technical changes made to *Guidelines* as the document evolved into the *Prestandard*. It is the vehicle by which new technical information was introduced into the *Prestandard*, as issues were identified and, when possible, resolved by the Prestandard Project Team. For completeness, this report also includes a brief discussion of new concepts introduced to the engineering profession in the publication of the original FEMA 273 *Guidelines* and FEMA 274 *Commentary* documents.

As the *Guidelines* were used by the industry, questions arose regarding application of certain procedures, interpretation of some provisions, and results stemming from portions of the methodology. These questions have been formulated into statements, termed global issues, and recorded in this report for reference during the prestandard project and future revisions of the document.

At the time the *Guidelines* were published, it was known that additional research was needed to refine the accuracy and applicability of certain procedures, and analytical studies were required to test and substantiate certain new concepts and philosophical themes. Unresolved issues, reported by BSSC to be present at the time of publication, are incorporated into this report and identified with the designation 'previously unresolved' in the classification of the issue.

The purpose of Global Topics Report 1, Identification of Global Issues, dated April 12, 1999, was to formulate a statement and classify global issues that had been identified as of the date of the report. The issues identified in that report were presented and discussed at the ASCE Standards Committee Meeting on March 3, 1999, in San Francisco. The discussions resulted in clarifications to some of the issues, as well as a consensus on the recommended classification of each issue. Comments from Standards Committee members were incorporated into the report, and were used by the Project Team in moving issues toward resolution.

Global Topics Report 2 was published on March 22, 2000. The purpose of the second report was to formulate statements for new global issues identified since Global Topics Report 1, and to document resolution of issues that were incorporated into the Second Draft of the Prestandard.

This third and final Global Topics Report contains new global issues identified since the publication of the previous two reports, and final resolutions of previously identified issues. The appendices to this report contain the results of special focused studies, which serve as back-up data to the resolution of selected issues. These studies are referenced in the body of this report, where applicable, and included in the appendices for future reference.

Upon completion of the Case Studies Project, the final report FEMA 343 *Case Studies: An Assessment of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings* was made available to the Prestandard Project Team. Issues identified in FEMA 343 have been incorporated as global issues in this report, and a cross-reference to these issues is contained in Appendix C.

In April, 2000, a *Prestandard* draft document was distributed to the ASCE Standards Committee on the Seismic Rehabilitation of Buildings in an unofficial letter ballot. Ballot comments were reviewed and considered by the Project Team, and changes, where applicable, were incorporated into the Prestandard. The results of that balloting are documented in the *Ballot Comment Resolution Report on the Unofficial Letter Ballot on the Second Draft of FEMA 356 Prestandard for the Seismic Rehabilitation of Existing Buildings*, included in Appendix L of this report.

This report is organized based on the chapter numbering and sequence of information contained in the original *Guidelines*. New section numbers are referenced for information that was relocated during the development of the *Prestandard*. Included in the body of this report are global technical or editorial issues that merited expanded discussion. Each issue was classified as one or more of the following:

Technical Revision — Issue requiring a revision or clarification of the technical content of the *Prestandard*

Editorial Revision — Issue requiring a revision or clarification of the technical verbiage of the *Prestandard* that does not substantially change the technical content.

Commentary Revision — Issue requiring a revision, clarification or expanded discussion in the *Commentary*

FEMA 343 Case study Consensus Revision — Issue resolved with the help of information gained from the FEMA 343 Case Study Project

Application of Published Research — Issue for which additional research has been published and can be used to supplement the *Prestandard*

Recommended for Basic Research — Issue that requires more information and further detailed study before a resolution can be reached.

Non-persuasive — Issue that was reviewed by the Project Team and the resolution resulted in no change to the *Prestandard*.

Once classified, issues were presented to the Project Team for resolution. Issues that were successfully resolved with the consensus of the Project Team were then incorporated into the *Prestandard* document. Resolved or not, the history of each issue that was identified over the course of the prestandard project is recorded in this report for future reference. Appendix B contains a summary of unresolved issues recommended for future research. It is the hope of the Prestandard Project Team that this Global Topics Report will serve as a resource and a reference for improvements to the FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* as the document is developed into a standard and incorporated into the practice of seismic rehabilitation.

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1. Introduction

Chapter 1 provides an introduction to the *Guidelines*. It describes how the document relates to other documents and explains how it is to be used in a seismic rehabilitation program. It also provides an overview of significant new features (concepts) that are introduced in the following chapters.

1.1 New Concepts

Chapter 1 provides a brief discussion of major new concepts introduced in the *Guidelines*. These concepts are listed below for information only, and discussed in greater detail in the following chapters.

- Seismic performance levels and rehabilitation objectives.
- Simplified and systematic rehabilitation methods.
- Varying methods of analysis.
- Quantitative specifications of component behavior.
- Procedures for incorporating new information and technologies into rehabilitation.

1.2 Global Issues

1-1 Reorganization of Chapters 1 and 2

Overlap and redundancy between Chapters 1 and 2 of the *Guidelines* makes it difficult to find and apply all provisions applicable to a given rehabilitation project.

Section: Chapter 1, all; Chapter 2, all.

Classification: *Editorial Revision.*

Discussion: None.

Resolution: Information contained in these chapters has been combined and reorganized so that Prestandard Chapter 1 now contains all information related to an overview of the rehabilitation process including the definition and selection of rehabilitation objectives, performance levels, and seismic hazard. Prestandard Chapter 2 now contains all general information related to applying the rehabilitation methodology. All non-mandatory information related to use of the standard for local or directed risk mitigation programs has been split out into Prestandard Appendix A.

2. General Requirements

(Simplified and Systematic Rehabilitation)

Chapter 2 describes the overall framework of the methodology. It describes performance levels rehabilitation options and how rehabilitation objectives are set. It discusses the basis of the seismic hazard determination and the component acceptance criteria. It sets general limitations on the application of the various analysis procedures and describes general analysis requirements.

2.1 New Concepts

- Rehabilitation using new and existing components: The procedures for simplified and systematic rehabilitation utilize existing elements to their fullest capacity. Basic, enhanced, partial and reduced rehabilitation objectives are defined that allow for the selection of a range of rehabilitation strategies using existing components to varying degrees.
- Displacement-based design: The analysis methodology uses a displacement-based philosophy that evaluates the behavior of individual components of the building at the maximum expected displacements of the structure. This philosophy was adopted as being more indicative of actual member performance than traditional force-based analysis procedures. In the linear procedures of the methodology, displacement-based concepts are translated back to force-based calculations to facilitate application by using procedures that are more familiar to engineers.
- Performance levels and rehabilitation objectives: Building performance is characterized by the performance of structural and nonstructural elements. Performance levels are related to certain limiting damage states of structural and nonstructural elements. A rehabilitation objective is a statement of the desired building performance level when subjected to the selected earthquake hazard level, and must be selected in order to use the methodology.
- Primary and secondary elements: Primary elements provide the overall resistance of the structure against collapse, and must not be damaged beyond usable limits. Secondary elements are those elements for which damage does not compromise the integrity of the structure, and higher levels of damage can be permitted. The concept of primary and secondary elements was introduced to take advantage of the inherent redundancy in some structures by allowing a few selected elements to experience excessive damage, and prevent less important elements from controlling the rehabilitation objective.
- Design parameters from physical tests: Destructive and nondestructive testing is required by the methodology in order to determine physical parameters in sufficient detail to reliably evaluate component strengths. A reliability coefficient, κ , was introduced to reduce calculated strengths considering the quality and uncertainty of information about the existing structure.
- Determination of regular and irregular structures: The regularity or irregularity of a structure affects the applicability of the analysis procedures. If a regular building has relatively limited inelastic demands, linear procedures are sufficiently accurate for evaluation. Regularity is determined by calculation of element Demand to Capacity Ratios (DCRs). Low DCRs are an indication of low inelastic demands. However, if calculated DCRs are high, there is a high potential for a concentration of inelastic activity at an irregularity that may not be accurately reflected in an elastic analysis.

- Hazard parameter determination: The seismic hazard in conjunction with building performance is used to define the rehabilitation objective. The *Guidelines* consider two hazard levels, Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). These correspond to a 10%/50 year earthquake and 2%/50 year earthquake respectively. In addition to BSE-1 and BSE-2, rehabilitation objectives may be formed using seismic hazards from earthquakes with any defined probability of exceedance. Procedures are included for determining hazard parameters for these other earthquakes, which can then be used for enhanced or reduced rehabilitation objectives.
- Simplified and Systematic Rehabilitation: Simplified rehabilitation allows for the design of building rehabilitation measures without requiring full building analysis or strengthening. Simplified rehabilitation can only be used in applications of limited rehabilitation. Systematic rehabilitation consisting of a comprehensive evaluation of the entire structural system is required to achieve the Basic Safety Objective of the *Guidelines*.
- The absence of drift control checks or limits: The analysis methodology evaluates the acceptability of elements in their displaced state at maximum expected displacements. Since displacements and their effects are explicitly calculated, drift limits are implicitly evaluated and not included.

2.2 Global Issues

2-1 Overtuning Appears Overly Conservative

Overtuning calculations at pseudo lateral force levels appear to be overly conservative and can predict overturning stability problems that are not well correlated with observed behavior.

Section: 2.11.4 (new sections 2.6.4 and 3.2.10).

Classification: *Technical and Basic Research (previously unresolved).*

Discussion: Related to issue 2-23 regarding R_{OT} for IO performance. Upon completion of the *Guidelines*, BSSC identified the need to develop improved procedures for evaluating overturning. The *Guidelines* evaluate overturning stability at seismic force levels representing expected building displacements. Thus overturning effects are larger than typically calculated for new buildings using current code-based analytical procedures that reduce earthquake forces by an R -factor. In spite of this force reduction, however, code-based design procedures have yielded satisfactory performance with regard to overturning. It, therefore, seems unnecessary to require buildings to remain stable at full pseudo lateral force levels. While the LSP will permit incorporation of foundation flexibility in the analysis, this does not fully resolve the problem. Simplified rocking calculation procedures are available in the literature, but have not yet been incorporated into the prestandard. Nonlinear analytical techniques are currently the best methods available to reconcile the difference between calculated and observed results.

Resolution: Prestandard Sections 2.6.4 and 3.2.10 have been revised to incorporate the overturning sidebar from the *Guidelines* into the Prestandard. The intent of the sidebar was to provide alternative overturning criteria that would be consistent with NEHRP provisions for new buildings. The sidebar overturning equation has been revised to reduce the earthquake force demand, Q_E , by C_1 , C_2 , and C_3 , which are displacement amplifiers. Due to the 0.75 factor on demands present in NEHRP, R_{OT} has been revised to 10 and 8 for collapse prevention and life safety respectively to calibrate overturning criteria for consistency with UBC $K=1.0$ force levels.

2-2**Ground Motion Pulses Not Covered**

Ground motion duration and pulses are not explicitly considered in the analysis procedures except for the use of higher acceleration values specified in regions near active faults.

Section: 2.6 (new section 1.6)

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to develop procedures for evaluating near field ground motion effects. The results of the NSP, in particular, may be very sensitive to earthquake pulses. Proper consideration of duration and pulses may require a time-history analysis, and records may or may not be available. No guidance on appropriate consideration of these effects is provided.

Resolution: Unresolved pending future research.

2-3

MCE Exceeds Probabilistic Values

In some areas (primarily areas of moderate to high seismicity), there are locations that have mapped acceleration response parameters on MCE maps that exceed the probabilistic response acceleration parameters for the 2%/50 years earthquake hazard.

Section: 2.6, 2.6.1, 2.6.1.1, 2.6.1.2, 2.6.2 (new sections 1.6.1.1, 1.6.1.2, 1.6.2).

Classification: *Commentary Revision.*

Discussion: Related to issue 2-16 regarding the definition of design earthquake. The latest seismic design maps, the Maximum Considered Earthquake (MCE) ground motion maps, were developed by the USGS in conjunction with the Seismic Design Procedure Group appointed by the BSSC. The effort utilized the latest seismological information to develop design response acceleration parameters with the intent of providing a uniform margin against collapse in all areas of the United States. The MCE ground motion maps are based on seismic hazard maps which are (1) 2%/50 years earthquake ground motion hazard maps for regions of the United States which have different ground motion attenuation relationships and (2) deterministic ground motion maps in regions of high seismicity with the appropriate ground motion attenuation relationships for each region. The deterministic maps are used in regions of high seismicity where frequent large earthquakes are known to occur, and the rare earthquake ground motions corresponding to the 2%/50 years hazard are controlled by the large uncertainties in the hazard studies which results in unusually high ground motions. These high ground motions were judged by the Seismic Design Procedures Group to be inappropriate for use in design. The use of these different maps to develop the MCE maps required the Seismic Design Procedure Group to define guidelines for integrating the maps into the design ground motion maps.

The most rigorous guideline developed was for integrating the probabilistic and the deterministic maps. To integrate the probabilistic maps and the deterministic map, a transition zone set at 150% of the level of the 1994 NEHRP Provisions was used and is extensively discussed in the 1997 NEHRP Provisions Commentary. The goal of this guideline was to not exceed the deterministic ground motion in these areas of high seismicity where the earthquake faults and maximum magnitudes are relatively well defined. The remaining guidelines were more subjective, and were related to smoothing irregular contours, joining contours in areas where closely spaced contours of equal values occurred (particularly in areas where faults are known to exist, but the hazard parameters are not well defined), increasing the response acceleration parameters in small areas surrounded by higher parameters, etc.

2-3 (continued)

Based on the process used to develop the MCE maps, there are some locations where the mapped acceleration response parameters on the MCE maps exceed the probabilistic 2%/50 years seismic hazard maps. These locations primarily occur in the New Madrid, Missouri area, the Salt Lake City area, coastal California, and in the Seattle, Washington area. The areas where this exceedance occurs are relatively small and the exceedance in general is less than about 10 to 15 percent. The maximum exceedance in very small areas varies from about 30 to 50 percent. The areas where these larger exceedances occur are in areas where there is a large uncertainty in the seismic hazard, and as more information is obtained the likelihood that the 2%/50 years maps increasing is relatively high. In addition, where these larger exceedances occur, the acceleration response parameters are high (short period values vary from about 1.25g to 1.8g and long period values range from 0.5g to 0.8g for B soil conditions). In these locations, the rehabilitation costs will be high, which makes these locations good candidates for site specific seismic hazard studies and non-linear analyses of the structures. Consideration of the site-specific studies and non-linear analyses should reduce the cost impact of the higher values.

Change in the definition of BSE-2 to consider probabilistic maps in conjunction with the MCE maps is not recommended for the following reasons:

1. The areas where the differences between the MCE maps and the 2%/50 years maps occur are considered to be small.
2. The differences in these areas are generally small and even the larger differences are considered to be well within the uncertainty associated with the maps in these areas.
3. The acceleration response parameters in these areas are generally high values and will result in high rehabilitation cost which should lead to consideration of site specific seismic hazard studies and non-linear analyses in order to minimize the cost.
4. The use of maps other than the MCE maps will result in differences with other codes and standards which will result in confusion and present an unneeded complexity in the design process.
5. A standing subcommittee was formed by BSSC in 1997 to address seismic hazard mapping issues and the subcommittee will continue to evaluate new data and information to ensure the MCE maps reflect the best scientific and engineering knowledge available.

In summary, the MCE maps were developed using a careful process of integrating probabilistic and determinist maps considering uncertainties in available knowledge. The resulting mapped values are an intentional result of this process so the BSE-2 hazard level will continue to be defined from the MCE maps.

Resolution:

The commentary of Section 1.6 has been revised to reflect the above discussion.

2-4 **Minimum Safety Level Not Specified**

The *Guidelines* should specify a minimum safety level, and that level should be set at the Basic Safety Objective (BSO).

Section: 2.4.1 (new section 1.4).

Classification: *Commentary Revision.*

Discussion: The *Guidelines* are intended to permit the selection of the rehabilitation objective that is most appropriate for a given situation. This is a policy issue that should be decided by the local authority having jurisdiction. However, the document must provide sufficient information so that informed decisions can be made.

Resolution: The commentary of Prestandard Section 1.4 has been expanded with additional text from FEMA 274 to provide additional information on selection of rehabilitation objectives.

2-5 **BSO Should Use Collapse Prevention**

The BSO should be based on the Collapse Prevention Performance Level instead of the Life Safety Performance Level. Consider a single level evaluation approach using BSE-2 at the collapse prevention performance level.

Section: 2.5.1.

Classification: *Non-persuasive.*

Discussion: Collapse prevention implies that the building is on the verge of collapse, but has not yet collapsed. If the building does not collapse, in part or in total, some may consider that the life safety objective has been met. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive. The Life Safety Performance Level, as defined in the *Guidelines*, includes an intentional margin of safety against collapse for the lower level earthquake. The collapse prevention check at the higher level was intended to safeguard the building against collapse due to a rare earthquake. Neither case governs in all situations. The definition of BSO as a two-level approach was set with this in mind, and use of a single level evaluation at the collapse prevention performance level would substantially change the intent.

Resolution: No change proposed.

- 2-6** **Baseline Adjustments to Acceptance Criteria Needed**
Use of experimental data to set acceptance criteria has led to some inconsistency in calculated versus expected results. It may be appropriate to consider some baseline adjustments to acceptance parameters.
- Section:** 2.9.4 (new section 2.4.4), Chapters 5 through 8.
- Classification:** *Technical Revision and Basic Research.*
- Discussion:** Baselining adjusts values to make sense. However, just because experimental results are contrary to historically used R-values does not mean the experiments are wrong. Special Study 6 – Acceptability Criteria (Anomalous *m*-values) was funded to research this issue. The study concluded that even non-ductile components have some limited level of inelastic deformation capacity, and that *m*-factors for deformation-controlled actions could be conservatively adjusted to minimum values of 1.25, 1.50 and 1.75 for IO, LS and CP performance levels respectively. This conclusion did not impact *m*-factor tables in Chapters 7 and 8. The results of this study are still under consideration by the Project Team. Changes to *m*-factor tables in Chapters 5 and 6 are on hold pending further discussion.
- Resolution:** Unresolved pending future research.
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- 2-7** **Software Not Commercially Available**
Nonlinear software capable of performing 3-D nonlinear analyses is not commercially available to the building engineering community. Any building that requires this analysis based on *Guidelines* provisions cannot be rehabilitated to meet the provisions.
- Section:** 2.9 (new section 2.4).
- Classification:** *Recommended for Basic Research.*
- Discussion:** None.
- Resolution:** Unresolved pending future research.
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- 2-8** **Force-Based Anchorage Criteria Not Consistent**
Wall anchorage and non-structural force-based evaluation criteria are inconsistent with the overall displacement-based methodology.
- Section:** 2.11.7, 2.11.8 (new sections 2.6.2, 2.6.8), Chapter 11.
- Classification:** *Non-persuasive.*
- Discussion:** Force-based evaluation criteria use force amplification factors to increase reliability. This procedure is not based on an evaluation of displacements or deformations. Similarly, this issue would apply to any force-based evaluation procedure in the *Guidelines*. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive. Force-based procedures are not inconsistent with the methodology. Wall anchors are treated as force-controlled elements with a defined force level.
- Resolution:** No change proposed.

2-9**Application Based on Rehabilitated Condition**

It is not clear that the limitations in the application of linear versus nonlinear procedures or static versus dynamic procedures apply to the condition of the rehabilitated building.

Section: 2.9 (new section 2.4).

Classification: *Technical Revision.*

Discussion: The applicability of analysis procedures depends on the condition of the structure that is being analyzed. If the structure is being rehabilitated, the configuration of the rehabilitated structure is important. If the analysis is intended to justify that no rehabilitation is required, then the configuration of the existing structure is important.

Resolution: Prestandard Section 2.4 has been revised to clearly state that the configuration of the rehabilitated structure determines whether the structure is classified as irregular or not.

2-10**No Public Input or Consensus on Acceptable Risk**

The present definitions of performance levels and acceptable risk have been developed by engineers with little input from the public, and may not be consistent with popular notions.

Section: 2.5 (new section 1.5).

Classification: *Commentary Revision and Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to develop a popular consensus on performance levels and acceptable risk.

Resolution: The commentary of Prestandard Section 1.5 has been expanded to provide additional clarification on the definition of performance levels. Prestandard commentary tables C1-3 through C1-7 provide detailed descriptors of damage. Further resolution of this issue is recommended for future research.

- 2-11** **Statistical Basis of Ground Motion Not Stated**
The statistical basis of ground motion hazards is not explicitly stated in the *Guidelines*. This information is needed to properly develop site specific hazard information.
- Section:** 2.6.2.1 (new section 1.6.2.1).
- Classification:** *Technical Revision.*
- Discussion:** It is unclear if ground motion hazards are to be expressed using mean spectra, median spectra, mean plus one standard deviation or some other statistical basis. The *Guidelines* are silent on how to develop BSE-1 and BSE-2 parameters when using site-specific hazard information.
- Resolution:** New prestandard Sections 1.6.2.1.3, 1.6.2.1.5, and 1.6.2.1.6 were developed to specify the statistical basis of site-specific hazard information. The BSE-1 hazard corresponds to mean spectra at the 10%/50 year probability of exceedance. Probabilistic BSE-2 hazard corresponds to mean spectra at the 2%/50 year probability of exceedance. Deterministic BSE-2 hazard corresponds to 150% of the median spectra for the characteristic event.
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- 2-12** **Vertical Drop in Component Curve**
The vertical drop in the idealized component load versus deformation curve is computationally difficult and leads to computer convergence problems.
- Section:** 2.9.4, 5.4.2.2.B, 6.4.1.2.B, 7.4.2.3.B, 8.4.4.3, (new sections 2.4.4, 5.5.2.2.2, 6.4.1.2.2, 7.4.2.3.2).
- Classification:** *Technical Revision.*
- Discussion:** The idealized force versus deformation backbone curves show a vertical drop when components reach their deformation capacity limits at collapse prevention (point C to point D). Point D is not related to any particular level of deformation and is not keyed to any acceptance criteria. This vertical drop is an unnecessary simplification that leads to computational difficulties.
- Resolution:** Prestandard figures C2-1, 5-1, 6-1, 7-1 and 8-1 have been revised to show a slight slope from point C to Point D. The commentary in Section 2.4.4 has been expanded to discuss the reason for the slope.
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- 2-13** **Equation for Mean Return Period Specific to 50 Years**
Equation 2-2, calculating the mean return period at the desired probability of exceedance, is more complex than necessary and is only specific to recurrence intervals of 50 years.
- Section:** 2.6.1.3 (new section 1.6.1.3, Eq 1-2).
- Classification:** *Technical Revision.*
- Discussion:** A more general equation can be used that is simpler, technically correct and can be used for recurrence intervals other than 50 years.
- Resolution:** Prestandard Equation 1-2 has been revised to the more general form $P_R = -T/\ln(1-P_E)$, where P_R is the mean return period and P_E is the probability of exceedance in time T.

2-14**Performance Levels Imply a Guarantee**

The detailed specification of performance levels may imply a “guarantee” of building performance in an earthquake, and increase liability of engineers.

Section: 2.5 (new sections 1.2.2 and 1.5).

Classification: *Editorial and Commentary Revision.*

Discussion: Building owners, and the public, may interpret designing to specific performance levels as implying a guarantee that selected performance will be achieved. Some have expressed concern over this notion while others feel it is no different than the current situation in which designing to current code is expected to provide life safe performance. It does not result in any more liability than is already implicit in the practice of design professionals.

Resolution: The commentary of Prestandard Sections 1.2.2 and 1.5 have been expanded to clarify that an uncertainty exists in predicting damage states and emphasize that there is still a possibility for damage in excess of the predicted damage state to occur in some cases. The word “Target” has been added to the designation of Building Performance Levels in the prestandard to imply the notion that the selected performance level is a goal and not a certainty.

2-15**Inconsistency in Response Spectrum Nomenclature**

The response spectrum nomenclature used in the *Guidelines* is not consistent with the nomenclature used in the 1997 NEHRP Provisions.

Section: 2.6 (new section 1.6), Figure 2-1 (new Figure 1-1).

Classification: *Technical Revision.*

Discussion: Differences in nomenclature for the response acceleration parameters S_{XS} and S_{X1} were intentional on the part of the FEMA 273 project team to distinguish parameters that can be related to any selected damping level from those in NEHRP that are related to 5% damping. Differences in nomenclature for period, T_0 and T_S , are not intentional (they were changed in NEHRP after FEMA 273 was published) and should be revised for consistency. In 1997 NEHRP, T_S designates the period at which the constant velocity and constant acceleration portions of the spectrum intersect. T_0 designates the beginning of the region of constant acceleration, taken as $0.2T_S$.

Resolution: The period nomenclature, T_0 and T_S , in Prestandard Section 1.6 has been revised for consistency with the 1997 NEHRP Provisions.

2-16**Inconsistency in Definition of Design Earthquake**

The definition of the design earthquake in FEMA 273 is not consistent the design earthquake in the 1997 NEHRP Provisions.

Section: 2.6, 2.6.1.2 (new sections 1.6, 1.6.1.2).

Classification: *Commentary Revision.*

Discussion: The latest MCE hazard maps were developed based on a 2%/50 earthquake hazard level. Because of conservatism present in the actual design of structures there is a margin (seismic margin) against collapse in the event the design level earthquake is exceeded. Popular consensus is that the minimum seismic margin for all buildings is on the order of 150%. This margin is used to set the design values at a level less than if taken directly from the actual hazard. The NEHRP design value is $1/1.5 = 2/3 * MCE$. Because of differences in seismicity throughout the country, the variation in probability is not directly proportional to the variation in the response acceleration parameters. This means that applying a $2/3$ factor on the MCE results in a design earthquake with a different probability of exceedance at each location, but gives a uniform margin against collapse. However, this is inconsistent with the intent of the *Guidelines*, which is to permit design for specific levels of performance in earthquakes with specific probabilities of exceedance. For this reason the *Guidelines* intentionally adopted a slightly different definition for the design earthquake. BSE-1 was taken as the ground motion with a 10%/50 year probability of exceedance, but not exceeding $2/3 * MCE$. The 10%/50 hazard level is consistent with what has traditionally been accepted as the basis for new construction. The $2/3 * MCE$ limit is included so that the design requirements for the BSO do not exceed the requirements for new construction under the 1997 NEHRP Provisions.

Resolution: The commentary of Prestandard Sections 1.6 and 1.6.1 have been expanded to explain the difference in design earthquakes.

2-17**Incorrect Adjustment for Damping at T=0**

Damping adjustments to response spectrum values have been incorrectly applied at T=0.

Section: 2.6.1.5, Eq 2-8, Figure 2-1 (new section 1.6.1.5, Eq 1-8, Figure 1-1).

Classification: *Technical Revision.*

Discussion: Adjustments of response spectrum values for damping should not occur at T=0.

Resolution: Prestandard Equation 1-8 and Figure 1-1 have been revised to correct this.

2-18**Knowledge Factor Requirements Unclear**

The requirements for the knowledge factor κ , specified in multiple sections, are unclear.

Section: 2.7.2 (new section 2.2.6.4), 5.3.4, 6.3.4, 7.3.4, 8.3.4.

Classification: *Technical Revision.*

Discussion: This issue is related to issues 5-4 and 6-3 regarding too much required testing. The selection of a knowledge factor depends on the selected analysis procedure, the level of information available on the building, and the amount of testing and condition assessment performed to confirm unknown information. These requirements are distributed throughout multiple sections across different chapters.

Resolution: Prestandard Section 2.2.6 was created to clearly outline data collection requirements. New Table 2-1 was created to provide a matrix of information used for selection of a knowledge factor. New Section 2.2.6.4 was created to centralize requirements for the knowledge factor. Prestandard Sections 5.3.4, 6.3.4, 7.3.4 and 8.3.4 now refer back to Section 2.2.6.4, and contain only knowledge factor information specific to the material in question.

2-19**Upper Limit on DCRs for LSP Needed**

There should be an upper limit on DCR values that should not be exceeded if linear procedures are to be applicable, regardless of the presence or absence of structural irregularities.

Section: 2.9.1 (new section 2.4.1).

Classification: *Recommended for Basic Research.*

Discussion: None.

Resolution: Unresolved pending future research.

2-20**General Design Requirements Keyed to BSO**

The general analysis and design requirements in Section 2.11 apply to the BSO or Enhanced Rehabilitation Objectives. References to this section in Chapter 3 apply to all rehabilitation objectives. Should application of these requirements be based on performance levels instead?

Section: 2.11 (new section 2.6)

Classification: *Technical Revision.*

Discussion: Related to issue 3-24 regarding redundancy between Sections 2.11 and 3.2. With few exceptions, application of the general design requirements applies to all rehabilitation objectives and would be necessary to achieve Life Safety at any seismic hazard. Therefore, keying application of these requirements to the BSO would be unconservative for a limited objective involving only life-safety performance.

Resolution: Prestandard Section 2.6 has been revised to require application of the general design requirements for systematic rehabilitation to any performance level or seismic hazard, unless otherwise noted. Section 2.11.9 (new Section 2.6.9) regarding common building elements has been revised to apply to all objectives. Application of Section 2.11.10 (new Section 2.6.10) regarding building separation is now keyed to the Life Safety Performance Level.

2-21**Building Separation Requirements Too Severe**

The requirements for building separation are too severe, and the analysis required by the *Guidelines* to achieve the BSO is beyond the current state of the practice.

Section: 2.11.10 (new section 2.6.10).

Classification: *Technical Revision.*

Discussion: Related to issue 2-20 regarding general design requirements. Building separation requirements are better keyed to the Life Safety Performance Level. Buildings that are approximately the same height with floor levels that align have demonstrated life safety performance in past earthquakes. The concern for catastrophic damage is really only related to gravity elements, such as columns, that are damaged by impact from misaligned floors, or buildings of substantially different height that impact and alter the distribution of seismic forces in each building.

Resolution: Prestandard Section 2.6.10 has been revised to soften the application of building separation requirements for life safety and lower performance levels when the buildings are substantially the same height and the floor levels align. Prestandard Equation 2-8 has been revised to permit an alternative conservative assumption for adjacent building deflection to simplify calculation.

- 2-22** **Revise Default Site Class from E to D**
The default site class should be revised from Class E to Class D.
- Section:** 2.6.1.4 (new section 1.6.1.4).
- Classification:** *Technical Revision.*
- Discussion:** The original intent was for the *Guidelines* and the 1997 NEHRP Provisions to be consistent. The *Guidelines* went to print before the Provisions, and a change in default site class was made from Class E to Class D in the Provisions.
- Resolution:** The default site class specified in Prestandard Section 1.6.1.4 has been revised from Class E to Class D. A new subsection within 1.6.1.4 has been created to clarify the selection of default site class.
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- 2-23** **R_{OT} Needed for IO Performance**
An overturning force reduction factor, R_{OT}, for IO performance is needed to complete the alternative procedure for evaluating overturning stability.
- Section:** 2.11.4 (new Section 3.2.10.1).
- Classification:** *Technical Revision and Basic Research.*
- Discussion:** Related to issue 2-1 regarding conservatism in overturning criteria. The overturning sidebar from the *Guidelines* was incorporated into the Prestandard to provide an analytical method of evaluating overturning that would achieve a level of overturning stability that was consistent with current code procedures for new buildings. The sidebar required the use of full LSP forces for the IO Performance Level. This criteria appears overly conservative in comparison to current code procedures for new hospital construction, which only requires an importance factor of 1.5 on design forces to raise performance to the Immediate Occupancy Level. Using this criteria as a model, R_{OT} has been developed for IO performance as:
- $$R_{OT(L.S.)/1.5} = 8/1.5 = 5.3, \text{ and then conservatively reduced to } 4.0.$$
- Resolution:** Prestandard Section 3.2.10.1, which includes the overturning sidebar discussion from the *Guidelines*, has been revised to include an R_{OT} factor equal to 4.0 for IO performance. Further study is recommended to determine if a value larger than 4.0 may be appropriate.
-
- 2-24** **LS Performance Level Should be Clarified or Eliminated**
The Life Safety Performance Level should be more clearly defined in terms of structural performance, or it should be eliminated as a performance goal.
- Section:** 2.5.1.2 (new Section 1.5.1.2).
- Classification:** *Recommended for Basic Research.*
- Discussion:** Defined as retaining a margin against the onset of collapse, the Life Safety Performance Level corresponds to a structural damage state that is not related to a clearly definable post earthquake condition of the building.
- Resolution:** Unresolved pending future research.

- 2-25** **The 2/3 Factor Estimating Vertical Seismic Forces is Not Accurate**
The 2/3 factor used to estimate the relationship between vertical response spectra and horizontal response spectra is not accurate.
- Section:** 2.6.1.5 (new section 1.6.1.5.2)
- Classification:** *Application of Published Research and Basic Research.*
- Discussion:** Research presented in a paper by Bozorgnia, et al, “Relationship Between Vertical and Horizontal Response Spectra for the Northridge Earthquake,” Eleventh WCEE, 1996, suggests that the 2/3 factor underestimates the ratio between vertical and horizontal spectra for short periods, especially in the near-field region. At longer periods, the 2/3 factor appears to overestimate the ratio.
- Resolution:** Unresolved pending further study of available information and future research.
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- 2-26** **Additional Guidance on Damping Needed**
There is more variation in damping of actual buildings than addressed in the document. Additional guidance on damping values is needed.
- Section:** 2.6.1.5 (new section 1.6.1.5.3)
- Classification:** *Application of Published Research.*
- Discussion:** Additional guidance on damping for various systems can be found in the Tri-Services Manual. This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Unresolved pending further study of available information.
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- 2-27** **Application of Site Coefficients Not Consistent with the IBC**
The application of site coefficients F_a and F_v occurs before application of the 2/3 reduction factor on MCE spectral response acceleration parameters for the BSE-1 earthquake hazard level. This is not consistent with the procedure in the IBC, which applies the coefficients first, and then applies the 2/3 reduction factor.
- Section:** 2.6.1.1, 2.6.1.2 (new Sections 1.6.1.1, 1.6.1.2)
- Classification:** *Technical Revision*
- Discussion:** The selection of site factors F_a and F_v depends on the magnitude of the spectral response acceleration parameters S_s and S_1 . As spectral acceleration increases, site factors decrease. Application of the 2/3 reduction factor before selecting the site coefficient in Tables 1-4 and 1-5 will result in the use of more conservative site factors than would be selected in conjunction with the IBC.
- Resolution:** Prestandard Sections 1.6.1.1 and 1.6.1.2 discussing BSE-1 and BSE-2 parameters S_s and S_1 have been revised to refer to the design spectral response acceleration parameters S_{xs} and S_{x1} , which have been adjusted for site class in accordance with Section 1.6.1.4. The BSE-1 hazard level design parameters will therefore be taken as the minimum of the values calculated using the 10%/50 mapped parameters, or 2/3 of the values calculated using the MCE mapped parameters.

2-28

Equation for Building Separation is Overconservative

Equation (2-16) for required building separation based on SRSS combination of building displacements is overconservative.

Section: 2.11.10.1 (new Section 2.6.10.1, Equation 2-8)

Classification: *Application of Published Research*

Discussion: This issue was raised at the 8/23/00 Standards Committee meeting. SRSS combination of maximum estimated building displacements assumes the buildings are moving out-of-phase, with some consideration that the maximum response in each building might occur at different times. While this is less conservative than a direct sum of building displacements, it may overconservative if the buildings are moving under forced oscillations from the same ground motion. It was the opinion of those in attendance that recent published research was available that might justify reduced separation requirements in consideration of potential in-phase response of buildings moving under the same forced input.

Resolution: Unresolved pending further study.

3. Modeling and Analysis

(Systematic Rehabilitation)

Chapter 3 describes modeling and analysis procedures for the systematic evaluation and rehabilitation of buildings. It describes, in detail, four new analysis procedures including the Linear Static Procedure, Linear Dynamic Procedure, Nonlinear Static Procedure and Nonlinear Dynamic Procedure. It addresses loading and mathematical modeling requirements and the basic acceptance criteria.

3.1 New Concepts

- Analysis procedures: The Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic procedures are new concepts because they use a displacement-based philosophy addressing the behavior of individual components of the building at the maximum expected displacements of the structure. This philosophy was adopted as being more indicative of actual member performance than traditional force-based analysis procedures. In the linear procedures of the methodology, displacement-based concepts are translated back to force-based calculations to facilitate application by using more familiar procedures.
- Deformation- and force-controlled actions: These concepts were introduced to better define when excess strength can substitute for a lack of ductility. Deformation-controlled actions occur in elements that can undergo inelastic deformation without failure. Force-controlled actions occur in brittle elements or elements that would experience failure when subjected to inelastic deformation. Demands on force-controlled actions are limited by the maximum force that can be delivered to the element due to inelastic activity in the surrounding structure.
- Load combinations: The specified gravity load combinations are intended for seismic evaluation only, and are intentionally smaller than total loads that would be calculated for new buildings. They include the use of 25% of the live load. The resulting total loads are modified because the *Guidelines* require on-site verification of loads so uncertainties are smaller, the building is known to have existed under the loads present, and the performance levels for rehabilitation are not necessarily the same as intended for new construction.
- Mathematical Modeling: Modeling procedures are new concepts because they have never before been prescribed to the level of detail contained in FEMA 273.
- Acceptance criteria: New component-based acceptance criteria have been developed to evaluate components of the lateral force resisting system on an individual basis for deformation- or force-controlled actions considering individual element ductility. Common code-based procedures use a single value for all elements in a building.
- Expected strength: The concept of expected strength was introduced to take full advantage of element capacities at maximum deformation considering overstrength, actual material properties, strain hardening, and composite action. Capacity reduction factors, ϕ , are taken equal to 1.0.
- Lower bound strength: The concept of lower bound strength was developed for force-controlled actions and is the minimum capacity of a force controlled element.
- C factors: The factors C0, C1, C2, and C3, have been introduced to assist in estimating the likely building roof displacement in the design earthquake. The factors make adjustments for higher mode effects, inelastic displacements, shape of the hysteretic behavior of the structure, and P-delta effects.

3.2 Global Issues

3-1 **C_T=0.06 for Wood Buildings Not Documented**

The accuracy of $C_T = 0.06$ for use in the period calculation for small wood buildings is not documented.

Section: 3.3.1.2, Method 2.

Classification: *Recommended for Basic Research.*

Discussion: The number was selected qualitatively based on some limited case study information and was calibrated to expected results for flexible structures.

Resolution: Unresolved pending future research.

3-2 **Application of Method 3 Period Calculation Not Clear**

It is not clear that the period calculation for one-story buildings with flexible diaphragms applies to all rigid element flexible diaphragm systems. Calculation of wood diaphragm deflection at 1.0g force level does not appear reasonable.

Section: 3.3.1.2 (new Section 3.3.1.2.3).

Classification: *Technical Revision.*

Discussion: Method 3 applies to all systems in which the response amplification of the ground motion occurs primarily in the flexible diaphragms elements and not in the rigid vertical elements. Use on Method 2 in this situation will significantly underestimate the period of the system and may result in erroneously high pseudo lateral forces. The calculation of period using the diaphragm deflection under a 1.0g force level is a fictitious calculation used for estimating period only. It does not represent actual diaphragm demands or expected displacements. For this calculation the diaphragm is considered to remain elastic.

Resolution: The commentary to Prestandard Section 3.3.1.2 has been expanded to provide additional direction on the use of Method 3. A new Section 3.3.1.2.4 was created to specify a new empirical equation for use specifically with URM buildings.

3-3 **Empirical Formulas Underestimate Period**

Empirical formulas for period intentionally underestimate building periods and add an unnecessary layer of conservatism to the LSP.

Section: 3.3.1.2.

Classification: *Application of Published Research.*

Discussion: Special Study 3 – Improvements to the FEMA 273 Linear Static Procedure was funded to research this issue. The main conclusion was that using empirical equations yielded conservative results when compared eigenvalue analyses or to measured actual response of buildings. Proposed refinements to empirical equations for period are available in the literature.

Resolution: Method 2 empirical calculation of period in Prestandard Section 3.3.1.2 has been refined to reduce conservatism. The coefficients have been refined to better match measured building performance as recommended in the literature.

3-4 **Multidirectional Effects Need Clarification**

Further direction on consideration of multidirectional effects, including vertical seismic forces, is required.

Section: 3.2.7.

Classification: *Technical Revision and Basic Research.*

Discussion: When a structure is displaced to its limit state in one direction, there is no reserve capacity to resist additional demands caused by displacements in the perpendicular direction. Also the addition of displacements in perpendicular directions is not intuitive and requires further explanation. It is unclear how to combine the acceptance criteria to elements receiving demands from multiple directions, particularly in the case of non-linear push-over analyses. Special Study 5 – Report on Multidirectional Effects and P-M Interaction on Columns was funded to research this issue. The major conclusions of this study were that information is available in the literature supporting the use of simplified 100% + 30% combinations, but that further research should be conducted in this area.

Resolution: Prestandard Section 3.2.7 was revised to specify code-based 100%+30% combinations for linear procedures. For nonlinear procedures the section was refined to check 100% of the deformations associated with the target displacement in the primary direction plus the forces (not deformations) associated with 30% of the target displacement in the other direction. Prestandard Section 3.2.7.2 was created to state that vertical seismic effects need not be combined with horizontal effects.

3-5 **Mass Participation Effects Not Considered**

The static analysis procedures do not consider mass participation factors and higher mode effects.

Section: 3.3.1.

Classification: *Application of Published Research.*

Discussion: Static analysis procedures which do not consider mass participation factors overstate the first mode contributions and underestimate the effects of higher modes which are likely out of phase with the primary mode of vibration. Consideration of higher mode effects can reduce the total demand on a structure. Special Study 3 – Improvements to the FEMA 273 Linear Static Procedure was funded to research this issue. The study concluded that the benefits of higher mode mass participation effects are documented in the literature, and were specifically, and conservatively, ignored in the development of the LSP. The effects of higher mode mass participation on building response is dependent on the mass and stiffness characteristics of the structure, so resolution has been keyed to structure type and number of stories.

Resolution: The equation for Pseudo Lateral Load in Prestandard Section 3.3.1.3.1 has been revised to include an new C_m factor to account for higher mode mass participation effects that reduce overall building response. New Table 3-1 was created, which specifies the factor based on structure type and number of stories.

3-6 **NSP Uniform Load Pattern Overly Conservative**

The shape of the loading pattern used in NSP significantly affects the results. Specifying a uniform load pattern appears to be overly conservative and can dominate the resulting behavior.

Section: 3.3.3.2.

Classification: *Technical Revision and Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to perform additional research on nonlinear procedures to consider strength and stiffness irregularities in the structure and improve reliability and accuracy as compared benchmark results. As a structure yields during actual nonlinear response, forces and deformations can redistribute due to changes in stiffness. This effect is not captured by the NSP. Consideration of multiple load patterns is intended to envelope the range possible response. The uniform load pattern is intentionally conservative, and unrelated to what may be actually happening in the yielded structure. Procedures that adapt the load pattern to the yielded structure are available, but currently require more computational effort to apply.

Resolution: Prestandard Section 3.3.3.2.3 has been revised to clarify the application of multiple load patterns and permit the use of an approved adaptive load pattern. Development of simplified adaptive load procedures is recommended for future research.

3-7 **Reconcile FEMA 273 and 310**

The potential difference in evaluation results between FEMA 273 and FEMA 310 should be reconciled.

Section: 3.3.

Classification: *Non-persuasive.*

Discussion: This issue is related to Issue 10-4 regarding differences between FEMA 310 and FEMA 356. Special Study 12 – FEMA 310 and FEMA 356 Differences was funded to research this issue further. FEMA 310 is an evaluation document, while FEMA 273 is a rehabilitation design document. The FEMA 310 Tier 3 detailed evaluation procedure uses 0.75 times the force levels used in FEMA 273. The Tier 2 evaluation procedure uses different *m*-factors. Building components that are compliant at FEMA 310 force levels may not be compliant at full FEMA 273 force levels. This issue stems from the controversial concept that force levels for evaluation should be different (lower) than force levels for design. Because the documents are for different purposes, the differences in the two procedures are intentional. See the discussion on Issue 10-4 for further information.

Resolution: No change proposed.

3-8 URM Special Procedure Not Included

The URM Special Procedure is not included in the *Guidelines*. Some building types, such as URM or tilt-up structures, may be more appropriately evaluated as systems rather than components. Flexible wood diaphragms in rigid wall buildings may need special treatment.

Section: 3.3 (new section 3.3.1.3.5).

Classification: *Technical Revision.*

Discussion: The response amplification of ground motion occurs in the diaphragm of rigid wall flexible diaphragm systems. As such, the behavior of individual components such as wall anchors depends overall system behavior. The Special Procedure was considered and specifically excluded from the *Guidelines*, and Special Study 2 – Analysis of Special Procedure Issues was funded to research this issue. The major conclusions of this study were that the Special Procedure should not be added to the Prestandard, specific portions of the procedure necessary to recognize the unique behavior of URM building should be added, and a revised method to empirically calculate the period of URM buildings is needed.

Resolution: Prestandard Section 3.3.1.3.5 was created to specify a lateral force distribution procedure that considers the unique behavior of URM buildings. A new method for calculating the period of URM buildings was added in Prestandard Section 3.3.1.2.4.

3-9 Reconcile FEMA 273 and Other Procedures

The potential difference in evaluation results between FEMA 273 and other evaluation procedures (other than FEMA 310) should be reconciled.

Section: 3.3.

Classification: *Non-persuasive.*

Discussion: The detailed evaluation procedures described in FEMA 273 may not agree with other procedures that are based more on qualitative information such as engineering judgment or past experience. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive. A potential resolution would be to assign other procedures to an appropriate FEMA 273 performance level. This idea met with considerable disagreement. It would require bringing all other procedures into the document in some way, directly or by reference, and imply alternative methods for obtaining the same performance.

Resolution: No change proposed.

- 3-10** **Upper Limit on Pseudo Lateral Force**
The LSP forces appear to be too high. FEMA 273 does not contain an upper bound limit on maximum base shear similar to the 0.75W limit in FEMA 310.
- Section:** 3.3.1.3.
- Classification:** *Technical Revision and Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to conduct soil-structure interaction research to study limiting ground motion input to buildings in cases where the ground may not be able to transmit motion through the foundation to the structure. For short and stiff buildings the pseudo lateral force may exceed the force required to cause sliding at the foundation, and the strength of the structure should not need to exceed the capacity of the soil-structure interface. Prestandard Section 3.2.6 provides methods for considering soil-structure-interaction effects.
- Resolution:** Unresolved pending future research.
-
- 3-11** **Clarify Primary, Secondary, Force-, and Deformation-Controlled**
Further explanation and clarification of primary and secondary components and deformation- and force-controlled actions is required.
- Section:** 2.9.4 (new section 2.4.4), 3.2.2.4, Chapters 5, 6, 7 and 8.
- Classification:** *Technical and Commentary Revision.*
- Discussion:** The concepts are partially explained in multiple sections, and the references between sections are circular. Materials chapters are not complete or consistent about specifying the force- or deformation-controlled nature of component actions.
- Resolution:** The definitions of primary and secondary components and deformation- and force-controlled actions have been centralized in Prestandard Section 2.4.4. The commentary has been expanded to further clarify the distinction. Materials Chapters 5 through 8 have been editorially clarified to specify force- or deformation-controlled actions for components.
-
- 3-12** **Reference to Alternative NSP Procedures Needed**
The *Guidelines* utilize the target displacement, or coefficient, method of evaluating nonlinear response, and do not include other alternative methods for performing nonlinear analyses.
- Section:** 3.3.3.3.
- Classification:** *Commentary Revision.*
- Discussion:** The Commentary in FEMA 274 describes the Capacity Spectrum Method as an acceptable alternative, but this procedure has not been directly incorporated into the analysis methodology of the *Guidelines*.
- Resolution:** Commentary has been added to Prestandard Section 3.3.3.3.2 to reference the Capacity Spectrum Method as an acceptable alternative method for nonlinear analysis.

- 3-13** **LSP and NSP Results Need Calibration**
The Linear Static Procedure is not always more conservative than Nonlinear Static Procedure.
- Section:** 3.3.1.
- Classification:** *Recommended for Basic Research.*
- Discussion:** The concern is that a building passing the LSP may fail the NSP. It is generally expected that simplified methods yield more conservative results so that a reduction in conservatism can then be achieved with additional computational effort.
- Resolution:** Unresolved pending future research.
-
- 3-14** **Reliability Information Not Provided**
No specific information on reliability is provided in the *Guidelines*.
- Section:** 3.3.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** No procedures exist for taking reliability into account in setting parameters or performing evaluations. Upon completion of the *Guidelines*, BSSC identified the need to perform reliability studies using statistical techniques to develop the degree to which rehabilitation objectives could be met.
- Resolution:** Unresolved pending future research.
-
- 3-15** **LSP Should be a Displacement Calculation**
The Linear Static Procedure should be changed to a displacement-based calculation procedure.
- Section:** 3.3.1.
- Classification:** *Non-persuasive.*
- Discussion:** The LSP is a displacement-based procedure that has been translated back to force-based calculations for simplicity. The concern is that the use of force-based calculations hides the real intent of the displacement-based philosophy and is confusing to engineers who are used to dealing with lower magnitude forces. Special Study 3 – Improvements to the FEMA 273 Linear Static Procedure was funded to research this issue, but was unsuccessful in developing a simplified displacement-based calculation procedure for incorporation into the Prestandard. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive.
- Resolution:** No change proposed.

- 3-16** **Combined with 2-2, 3-5, 3-6**
Combined with Global Issues 2-2, 3-5, 3-6 and omitted.
- Section:** None.
- Classification:** *None.*
- Discussion:** None.
- Resolution:** None.
-
- 3-17** **C1 Factor Overly Conservative**
Introduction of the C_1 factor overly penalizes buildings with short calculated fundamental periods.
- Section:** 3.3.3.3.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to research the effects of foundation flexibility on increasing the period of short and stiff structures and the associated impact on the C_1 factor.
- Resolution:** Unresolved pending future research.
-
- 3-18** **Duration Effects Not Considered**
The analytical procedures of the *Guidelines* do not consider duration effects to take into account cyclic degradation.
- Section:** 3.3.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to develop simplified methods for establishing degraded pushover properties and approximating complex duration effects.
- Resolution:** Unresolved pending future research.
-
- 3-19** **Marginal Gravity Load Capacity Not Considered**
Further study of LSP acceptance criteria is required for building components with marginal gravity load capacity.
- Section:** 3.4.2.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to further research this issue.
- Resolution:** Unresolved pending future research.

- 3-20** **Inelastic Cyclic Properties Needed**
More information is needed to develop inelastic cyclic component properties for use in complex nonlinear dynamic analyses.
- Section:** 3.3.4.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to develop consensus models for inelastic cyclic behavior of components.
- Resolution:** Unresolved pending future research.
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- 3-21** **Combined with 3-10**
Combined with Global Issue 3-10 and omitted.
- Section:** None.
- Classification:** *None.*
- Discussion:** None.
- Resolution:** None.
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- 3-22** **Amplification of Torsion Needs Clarification**
The definition of torsion and the procedure for amplification of torsion need further clarification.
- Section:** 3.2.2.2.
- Classification:** *Technical Revision.*
- Discussion:** The current definition does not discuss dynamic torsion, or torsion due to rotational modes of building response. This is a dynamic characteristic of the system that may produce torsion in excess of that due to eccentricity between the center of mass and center of rigidity. Currently the *Guidelines* only require accidental torsion to be amplified.
- Resolution:** Resolution expected, but not yet developed.
-
- 3-23** **Substantiation of C1, C2, C3 Needed**
Further research is needed to substantiate the coefficients C_1 , C_2 , and C_3 .
- Section:** 3.3.1, 3.3.3.
- Classification:** *Commentary Revision and Basic Research.*
- Discussion:** Special study 7 – Report on Study of C-Coefficients was funded to research this issue, resulting in minor clarifications to C coefficient definitions and additional commentary.
- Resolution:** Commentary from FEMA 274 has been added to Prestandard Section 3.3.1.3.1, and definitions in Section 3.3.3.3.2 have been clarified for consistency. Further resolution of this issue is recommended for future research.

3-24**Reorganization of Sections 3.2 and 2.11**

Overlap and redundancy between Sections 3.2 and 2.11 (new section 2.6) makes it difficult to find and apply general analysis and design provisions applicable to a given rehabilitation project.

Section: 3.2, 2.11 (new section 2.6).

Classification: *Editorial Revision.*

Discussion: None.

Resolution: Information contained in these sections has been combined and reorganized in the Prestandard so that Section 2.6 contains general design provisions applicable to any rehabilitation project, and Section 3.2 now contains general analysis provisions needed to properly apply the analysis procedures.

3-25**Definition of Pushover Curve Not Complete**

The idealized force-displacement curve shown in Figure 3-1 is not well defined. Further guidance is needed to properly, and consistently, define the pushover curve.

Section: 3.3.3.2 (new section 3.3.3.2.4).

Classification: *Technical Revision.*

Discussion: The idealized force-displacement curve is used to set the effective stiffness and, in turn, calculate the target displacement. Consistent definition of this curve is necessary for proper application of the NSP.

Resolution: Prestandard Section 3.3.3.2.4 has been revised to better define the construction of the idealized curve. Revisions include balancing the area above and below the actual curve, and requiring the idealized curve to pass through the actual curve at the calculated target displacement.

3-26**Application of the J -factor Not Clear**

The technical justification and proper application of the J -factor is not clear. It is also not clear why the J -factor should be related to the spectral response coefficient S_{XS} , in Equation 3-17.

Section: 3.4.2.1, Equation 3-17 (new Equation 3-21).

Classification: *Commentary Revision and Basic Research.*

Discussion: The technical justification of the J -factor is not described in the FEMA 274 *Commentary*. Consequently the factor is not widely understood. For force-controlled actions, the preferred method to calculate demands is a limit state analysis to determine the maximum force that can be delivered to a component. The intent of the J -factor is to provide an alternative method of calculating the maximum demand based on the pseudo lateral force. The J -factor is a force reduction factor that limits forces on components due to nonlinear actions on other ductile components in the system. It is intended to account for ductility inherent in systems that have elements that are behaving inelastically, even if the component under consideration is nonductile. The concept of a limit state analysis means that the maximum force delivered to a component is not governed by the severity of the ground motion. In the original *Guidelines*, J was related to S_{XS} , so that when it was used in Equation 3-15 (new Equation 3-19) the resulting force was also not dependent on the severity of the ground motion. At the 2/15/00 Standards Committee meeting, the committee voted to delete Equation 3-17 (new Equation 3-21) relating J to S_{XS} . The PT concurs that relating J to S_{XS} is questionable. It does, however, feel that the concept of a force-reduction factor is appropriate, and that some more appropriate formulation of it should remain in the Prestandard.

Resolution: The commentary to prestandard Section 3.4.2.1 has been expanded to reflect the above discussion. Prestandard Equation 3-21 relating J to S_{XS} has been deleted and replaced with a revised Section 3.4.2.1 that provides values of J judged to be conservative, and emphasizes the use of DCR values in the load path which is more rational. Further study on this issue is recommended.

3-27**Degradation Effects Double Counted in LSP**

Calculation of demands in the *Guidelines* analysis procedures include coefficients that account for degradation, but acceptance criteria do not permit components to respond beyond the elastic or plastic limits of response.

Section: 2.9.4 (new section 2.4.4), 3.3.1.

Classification: *Technical Revision.*

Discussion: Coefficients C_2 and C_3 are intended to account, in part, for increased displacements caused by degradation of components or the structural system. Component load-deformation curves in Figure 2-5, and acceptance criteria specified in 2.9.4, state that acceptance for primary elements is within the elastic or plastic portions of response, so components meeting the acceptance criteria will not experience degradation that would lead to increased displacements. Special Study 3 – Improvements to the FEMA 273 Linear Static Procedure was funded to research this issue. The main conclusion was that the effects of component degradation are counted on both the demand side as well as the capacity side of the equation for acceptance, and that this conservatism should be eliminated.

Resolution: The definition of C_2 in Prestandard Section 3.3.1.3.1 has been revised so that the coefficient is taken as 1.0 for linear procedures.

3-28**Global Acceptance Criteria Needed**

Tracking acceptance on a component basis is conservative with respect to overall building behavior. Global nonlinear acceptance criteria are needed to better calibrate observed performance with performance predicted by the procedures in the *Guidelines*.

Section: 3.3.3.2, 3.4.3.2.

Classification: *Technical Revision.*

Discussion: This issue is related to 3-27, and was studied as part of Special Study 3 – Improvements to the FEMA 273 Linear Static Procedure. The main conclusion was that a global nonlinear analysis criterion was needed. Further study concluded that a global criteria was implicit in the current NSP procedure, but not explicitly defined or well understood. If all components are modeled with full degrading backbone curves, the effects of component degradation can be evaluated in the analysis, and acceptance can be permitted out to secondary component limits of response.

Resolution: Prestandard Section 3.3.3.2 was expanded to clarify modeling requirements, including the use of full component backbone curves. The concept of a simplified NSP analysis was introduced for situations where degradation cannot be modeled. The acceptance criteria of Section 3.4.3.2 was revised to permit acceptance out to secondary component limits of response when degradation is explicitly modeled. A new Section 3.4.3.2.2 was created to define acceptance criteria for the simplified NSP analysis.

3-29**Snow Load Should be Specified**

The *Guidelines* are not specific regarding the magnitude of snow load to be considered in combination with seismic forces.

Section: 3.3.1.3 (new Section 3.3.1.3.1).

Classification: *Technical Revision.*

Discussion: This issue was raised at the 2/15/00 Standards Committee meeting. It is considered critical in regions with large snowpack. The verbiage incorporated in the Prestandard was based on the 1997 NEHRP Provisions, with permissive language allowing the reduction of snow loads with the approval of the local jurisdiction. The issue is that a more definitive statement on the amount of snow load to be considered in the calculation of seismic weight is needed in the Prestandard. The IBC, which specifies 20% of snow loads exceeding 30 psf, was recommended as a source for information on an appropriate snow load.

Resolution: The definition of snow load to be considered in the calculation of seismic weight has been revised to match the IBC. The permissive language regarding reduction of the snow load has been replaced with the specification of 20% of snow loads exceeding 30psf.

3-30**Application of η -factor is Overconservative**

Amplifying forces and displacements by the η -factor to account for torsion is overconservative for lateral force resisting elements located near the center of rigidity.

Section: 3.2.2.2 (new Section 3.2.2.2.2).

Classification: *Recommended for Basic Research.*

Discussion: Lateral force resisting elements located near the center of rigidity will not experience the same increase in forces and displacements as elements located farther away. It is suggested that η should vary with distance between the element and the center of rigidity.

Resolution: Unresolved pending further study.

3-31**Consider Reduced Demands Due to Actual Torsion**

Actual torsion will reduce the demands on some elements. It is overconservative and analytically difficult when using finite element programs to require that torsion never reduce the total demand on an element.

Section: 3.2.2.2 (new Section 3.2.2.2.2).

Classification: *Technical Revision.*

Discussion: Actual torsion is due to the actual eccentricity between the centers of mass and rigidity in the structure. This eccentricity is a source of real torsion that always adds to the critical elements and subtracts from the non-critical ones. When modeling in 3-D, it is analytically difficult to make sure the actual torsion does not reduce the demand on some elements. Uncertainty in torsion is addressed by accidental torsion. Since this torsion is uncertain in nature, it makes sense that accidental torsion effects should never reduce the demands on a component. It is recommended that only accidental torsion fall under this requirement.

Resolution: Prestandard Section 3.2.2.2.2 has been revised to specify that only accidental torsion shall not be used to reduce force and deformation demands on components.

3-32**No Maximum Limit on Method 1 Period**

Method 1 for analytical calculation of period has no maximum limit.

Section: 3.3.1.2.

Classification: *Commentary Revision.*

Discussion: Codes for new buildings include an upper limit on periods determined using analytical methods in order to maintain a minimum design base shear. Prestandard Method 1 calculation of period using eigenvalue analysis has no upper bound limit. Use of analytically calculated period to determine design actions without limit was intentionally permitted in the *Guidelines* to encourage more advanced analyses and reward additional computational effort. It was thought that sufficient controls are present in analysis procedures and acceptance criteria to yield appropriate results.

Resolution: Commentary to Prestandard Section 3.3.1.2 regarding Method 1 has been expanded to explain this departure from current code procedures.

3-33**Omit C2 Factor For Nonlinear Procedures**

The C2 factor should be omitted for nonlinear procedures because recent research has shown that inelastic displacements are not significantly affected by the pinched hysteretic behavior of components.

Section:

3.3.3.3.2.

Classification:

Technical Revision and Basic Research.

Discussion:

Related to issue 3-27 regarding degradation effects in the LSP. The C2 factor is intended to account for increased inelastic displacements due to pinched hysteretic behavior, stiffness deterioration and strength degradation of components. Recent research in SAC state of the art reports indicates that hysteretic behavior does not significantly affect inelastic displacements. Since the C3 factor already amplifies displacements for global strength and stiffness deterioration of the system, a direct result of component deterioration, current consensus is that the C2 factor can be eliminated. At the 2/15/00 Standards Committee meeting the committee voted to omit the C2 factor. The Prestandard has been revised to permit the use of C2=1.0 for nonlinear procedures, however, the original formulation of the factor has been preserved in the document because the information is new and evolving. Further research is recommended to confirm the relationship between inelastic displacements and component hysteretic behavior.

Resolution:

The definition of C2 for nonlinear procedures has been revised to permit the use of C2=1.0. The commentary to Prestandard Section 3.3.3.3.2 has been expanded to reflect the above discussion.

3-34**Alternate Empirical Period Calculation for Flexible Diaphragms**

An alternate empirical equation can be developed for single span flexible diaphragms consisting of $T=C_{td} (L)^{1/2}$, where L is the span length and C_{td} is a materials based coefficient.

Section:

3.3.1.2.3

Classification:

Application of Published Research.

Discussion:

This formulation was proposed as an alternate to the current Method 3 period calculation in response to the unofficial letter ballot of the Prestandard distributed to the SC. The proposed equation is based on preliminary studies made by Freeman, et al.

Resolution:

Unresolved pending further study of available information and future research.

- 3-35** **Omit C_1 C_2 C_3 Factors from the Denominator of Diaphragm F_p**
The presence of C_1 C_2 and C_3 in the denominator of the equation for diaphragm F_p forces is not consistent with the calculation of force- or deformation-controlled demands with the acceptance criteria of Section 3.4.
- Section:** 3.3.1.3 (new section 3.3.1.3.4, Equation 3-13)
- Classification:** *Technical Revision.*
- Discussion:** Chapters 5 through 8 provide specific direction regarding consideration of force- or deformation-controlled actions on diaphragm components. Calculation of forces using Equation 3-13 is not consistent with force- or deformation-controlled acceptance criteria in Section 3.4. Equation 3-22 would permit the use of m-factors with F_p forces reduced by C_1 C_2 C_3 for deformation –controlled actions, and Equation 3-19 would permit the further reduction of F_p forces by C_1 C_2 and C_3 a second time for force-controlled actions. This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Prestandard Equation 3-13 has been revised to omit the factors from the denominator. Section 3.3.1.3.4 has been expanded to reference Chapters 5 through 8 for direction on force- or deformation-controlled actions.
-
- 3-36** **Application of the NSP With Non-Rigid Diaphragms Needs Revision**
Further guidance is required on the proper application of the NSP in buildings with non-rigid diaphragms.
- Section:** 3.3.3.3 (new section 3.3.3.3.1)
- Classification:** *Recommended for Basic Research.*
- Discussion:** In buildings with non-rigid diaphragms, some of the deformation demand can be taken up in diaphragm deflection. This could be unconservative in estimating deformation demands on vertical seismic framing elements. To approximately account for this, original FEMA 273 included provisions for amplifying the calculated target displacement by the ratio of the maximum diaphragm displacement to the displacement at the center of mass. However, pushing the vertical elements to the full target without consideration of diaphragm deflections is overconservative. Development of methods to explicitly apply the NSP to non-rigid diaphragms is recommended. The solution may center around the development of C_0 factors relating horizontal displacements along the length of the diaphragm or revising the control node location to push the third points of the diaphragm to the target.
- Resolution:** Unresolved pending future research.

3-37**C₀ Factors Overconservative for Uniform Load Pattern**

Pushing buildings with the uniform load pattern to target displacements calculated using C₀ factors based on an inverted triangular load pattern is overconservative.

Section: 3.3.3.3

Classification: *Technical Revision.*

Discussion: The current C₀ factors were developed for an inverted triangular distribution of loading, which is essentially the first mode response with all floors moving in phase. The uniform load pattern is intended to capture higher mode effects, which occur when floors are moving out of phase. In buildings responding dynamically in a manner consistent with the uniform load pattern, the relationship between the spectral displacement of the equivalent SDOF system and the roof displacement of the actual MDOF system will be different (lower) than the case of a triangular distribution. Additional C₀ factors specific to the uniform load pattern should be developed.

Resolution: Prestandard Table 3-2 has been revised and expanded to consider buildings dominated by shear or cantilever behavior, and to include reduced values for the uniform load pattern in the case of shear buildings. The commentary has been expanded to explain that explicit calculation of C₀ is preferred and could be beneficial.

3-38**Procedures for Torsional Amplification are Unconservative**

Procedures for torsional amplification do not account for torsional degradation and are unconservative in determining increased forces and displacements for this effect.

Section: 3.2.2.2

Classification: *Recommended for Basic Research*

Discussion: This issue was raised at the 8/23/00 Standards Committee meeting. Traditional practice has permitted the analysis of buildings along each principle axis independently. Reportedly there have been recent studies in Japan indicating that further amplification of forces and displacements is required to properly account for torsion as the stiffness of the structure degrades in the direction perpendicular to the direction under consideration. This issue is related to issue 3-30 which suggests that current procedures are overconservative.

Resolution: Unresolved pending future research.

4. Foundation and Geotechnical Hazards (Systematic Rehabilitation)

Chapter 4 provides guidance on geotechnical aspects of foundations and site hazards. It describes acceptability criteria for foundation systems and foundation soils. It includes procedures for developing soil design and analysis parameters.

4.1 New Concepts

- Soil cannot fail: The procedures contained in the *Guidelines* presume that the soil will not be susceptible to a significant loss in strength due to earthquake loading. Soils such as this will continue to mobilize load with increasing deformations after reaching ultimate soil capacity. The amount of acceptable soil deformation depends primarily on the effect of the deformation on the structure, and the two cannot be evaluated independently. If the soil underlying the building in question is subject to strength loss, the resulting structural deformations must be explicitly considered in the evaluation.
- Mitigation of site hazards: Site hazard mitigation is considered in the context of overall building performance. If the consequences of fault rupture, liquefaction, differential settlement, landslide or flood result in excessive structural deformations that do not meet the performance level, mitigation is recommended. Methods of site hazard mitigation are listed.
- Consideration of seismic forces on retaining walls: In general, past earthquakes have not caused damage to building walls below grade. The *Guidelines*, however, include guidance on conditions for which it may be advisable to check walls for seismic demands such as poor construction, light reinforcement, use of archaic materials, or the presence of damage.

4.2 Global Issues

4-1 Spring Limitations Required in NSP

Some of the problems identified in a NSP analysis can be fixed by the addition of foundation springs in the analysis. There is insufficient guidance on the limitations in the application of foundation springs to increase building flexibility.

Section: 4.4, 3.2.6

Classification: *Technical Revision*

Discussion: The addition of foundation springs, if sufficiently flexible, can provide additional displacement capacity to reach the target displacement without exceeding structural deformation limits. Special Study 4 – Foundation Issues was funded to research this issue further. The main conclusion of this study was that additional limitations on the use of soil-structure interaction (SSI) with the NSP are not required. Additional flexibility in the system will increase the target displacement, which can make it more difficult to achieve the desired performance, even when that flexibility is coming from the foundation level. The study also concluded that the intent of the original 25% limitation on maximum reduction due to SSI effects in Section 3.2.6 applies to linear procedure only. If the results of an NSP analysis are bounded by parametric studies of soil parameters, this limitation is not needed.

Resolution: Prestandard Section 3.2.6 has been revised to limit the 25% maximum reduction due to SSI effects to linear procedures only. No other changes proposed.

4-2 **Spring Procedure Not Applicable to Strip Footings**

The procedure for developing foundation spring constants using an equivalent circular footing is not directly applicable to strip footings below shear walls.

Section: 4.4.2.1, Figures 4-2, 4-3 (new Figures 4-4 and C4-1).

Classification: *Application of Published Research.*

Discussion: At the 3/3/99 Standards Committee meeting this issue was reclassified as recommended for basic research. Special Study 4 – Foundation Issues was funded to research this issue further. The study concluded that new spring stiffness solutions directly applicable to a general rectangular footing of any size are available in the literature, and can be incorporated into the Prestandard.

Resolution: Prestandard Figure 4-4 has been revised to include new equations for spring constants that are directly applicable to rectangular footings. Figure C4-1 is a graphical representation of information in the equations that has been added to the commentary for information only.

4-3 **Lateral Soil Spring Procedure Needs Refinement**

The procedure for developing lateral soil spring stiffness based on displacement results in unrealistically high calculated lateral soil pressures. More information is needed on the force-displacement behavior of geotechnical materials and foundations under short term loading.

Section: 4.4.2.1.

Classification: *Application of Published Research and Basic Research (previously unresolved).*

Discussion: Geotechnical engineering has traditionally focused on long-term force-displacement behavior of soils. Upon completion of the *Guidelines*, BSSC identified the need to conduct additional research on characteristics of soils under short term loading. Special Study 4 – Foundation Issues was funded to research this issue further. The study concluded that the *Guidelines* procedure for developing lateral soil springs at a certain displacement implies that unrealistically high passive pressures are developed in the soil. A revised formulation for lateral strength due to passive pressure and base traction is included.

Resolution: Prestandard Section 4.4.2.1.5 has been revised to specify the use of principles of soil mechanics to determine the lateral capacity of shallow foundations. The commentary has been expanded to provide guidance on this.

4-4 **Nonlinear Soil Spring Information Needed**
More information is needed on nonlinear force-displacement behavior of foundation systems for inclusion in nonlinear analyses.

Section: 4.4.2.1, Figure 4-4 (new Figure 4-6).

Classification: *Application of Published Research and Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to conduct additional research on this issue. Special Study 4 – Foundation Issues was funded to research this issue further. The study concluded that the present linear relationship for passive pressure mobilization shown in *Guidelines* Figure 4-4 is unrealistic. The actual relationship is highly nonlinear.

Resolution: Prestandard Figure 4-6 has been revised to reflect the actual nonlinear relationship for mobilization of passive pressure.

4-5 **Shear Modulus Factors Inconsistent with NEHRP**
Shear modulus reduction factors presented in Table 4-3 are significantly different from those presented in Table 5.5.2.1.1 of the 1997 NEHRP Provisions.

Section: 4.4.2.1, Table 4-3 (new Table 4-7).

Classification: *Technical Revision.*

Discussion: Special Study 4 – Foundation Issues was funded to research this issue further. The study concluded that the values in Table 4-3 should be revised to reflect recent research on the subject, consider sensitivity to realistic variation in key parameters, and reflect softening of soils due to free-field response and inertial interaction.

Resolution: Values of effective shear modulus in Prestandard Table 4-7 have been revised in accordance with this research.

4-6 **Soil Parametric Range Appears Extreme**
Variation in soil parameters by factors of ½ and 2 appears to be extreme. A more appropriate range between upper and lower bound should be specified.

Section: 4.4.2.

Classification: *Non-persuasive.*

Discussion: Special Study 4 – Foundation Issues was funded to research this issue further. Variation in soil parameters is intended to account for many factors including rate of loading, assumed elasto-plastic soil behavior, cyclic loading, and variability of soil properties. The study concluded that variation in parameters of ½ and 2 is consistent with other standards, and is appropriate. With additional soil investigation, this factor could be reduced to 1.5.

Resolution: No change proposed.

- 4-7** **Classification of Foundation Rigidity**
Quantitative guidance on the classification of foundations as rigid or flexible with respect to the underlying soil is required.
- Section:** 4.4.2.1.
- Classification:** *Application of Published Research.*
- Discussion:** Special Study 4 – Foundation Issues was funded to research this issue further. The commentary of Prestandard Section 4.4.2.1.1 has been expanded to provide guidance on the classification of foundations as rigid or flexible with respect to the underlying soil.
- Resolution:** The commentary of Prestandard Section 4.4.2.1.1 has been expanded to provide guidance on the classification of foundations as rigid or flexible with respect to the underlying soil.
-
- 4-8** **Guidance for Rocking Needed**
Although rocking behavior is discussed in Section C4.4.2.1 of FEMA 274, no guidance is provided on the inclusion of such behavior in the analysis procedures of the *Guidelines*.
- Section:** 4.4.
- Classification:** *Application of Published Research.*
- Discussion:** Special Study 4 – Foundation Issues was funded to research this issue further. The study presented an outline of a response spectrum design approach for considering rocking, based research published in the literature. This information has not yet been incorporated into the Prestandard.
- Resolution:** Commentary has been added to Prestandard Section 4.4.2 to provide guidance on how to consider rocking when using the LSP. References to published literature on rocking have been added to Section C4.9.
-
- 4-9** **Presumptive Values for Piles Missing**
Information on presumptive capacities for pile foundations is not included in the *Guidelines*.
- Section:** 4.4.1.
- Classification:** *Application of Published Research.*
- Discussion:** Special Study 8 – Incorporation of Selected Portions of Recent Related Documents was funded to research this issue further. Information on presumptive capacities of pile foundations is available in ATC-43.
- Resolution:** Information on presumptive capacities for pile foundations has been added to Prestandard Section 4.4.1.1.

5. Steel and Cast Iron

(Systematic Rehabilitation)

Chapter 5 provides guidance on systematic rehabilitation of steel structural systems including moment frames, braced frames, plate shear walls and steel frames with infill. It includes procedures for obtaining material properties and the condition assessment of steel structures, and describes the acceptance criteria for steel components.

5.1 New Concepts

- Cast iron values: The *Guidelines* include design values for evaluating the capacity of cast iron elements
- Brittle connections: *m*-values have been specified for fully restrained welded moment connections, permitting limited inelastic activity on potentially brittle elements.
- Testing requirements: The *Guidelines* include new requirements on testing and condition assessment for determination of design and analysis parameters for steel structures.
- Rehabilitation measures: The procedure includes a discussion of possible rehabilitation strategies to address deficiencies identified in various steel structural systems.

5.2 Global Issues

5-1 *m*-factors Appear Overly Conservative

Certain values of acceptance criteria (*m*-factors) and deformation limits for steel components appear to be too conservative.

Section: Tables 5-3, 5-4, 5-5, 5-6, 5-7, 5-8; Sections 5.8.x.3.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to augment data used to develop acceptance criteria. Existing values were determined on a rational basis using available experimental results. This issue is related to issue 2-6 regarding baselining of acceptance criteria. Special Study 6 – Acceptability Criteria (Anomalous *m*-values) was funded to research this issue. The results of this study are still under consideration by the Project Team. Changes to *m*-factor tables in Chapter 5 are on hold pending further discussion.

Resolution: Unresolved pending future research.

5-2 **Steel Default Values Too Low**

Default expected material strength values for steel are too low.

Section: 5.3.2.5.

Classification: *Technical Revision.*

Discussion: This issue is related to issue A-7 regarding expected and lower bound strengths. Default expected values for steel in the *Guidelines* have been conservatively set at mean less two standard deviations. In general, however, default values in the *Guidelines* are intended to be lower bound, not expected material properties. Use of default values as expected strengths in Chapter 5 is not consistent with section 2.9.4 or other material chapters.

Resolution: Tables of default values in Prestandard Chapter 5 have been revised to reflect lower bound material strengths. Values were conservatively based on historic data using mean less two standard deviations. Values remain unchanged, but have been assigned to lower bound properties.

5-3 **Insufficient Limits for Cast Iron**

There are not enough limitations on using cast iron to resist seismic forces, particularly in bending.

Section: 5.4.2.3, 5.4.3.3, 5.5.2.3, 5.5.3.3.

Classification: *Technical Revision.*

Discussion: Except for a few locations, cast iron is not explicitly discussed. Tables of acceptance criteria do not clearly distinguish between steel and cast iron, which have very different responses to inelastic deformations.

Resolution: Cast iron requirements were centralized in Prestandard Section 5.11. This section clearly prohibits the use of cast iron components as primary elements of the lateral force resisting system.

5-4 Too Much Testing is Required

The *Guidelines* require too much testing of in-place materials for the determination of design and analysis parameters.

Section: 5.3.2, 5.3.3.

Classification: *Technical Revision (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to develop non-destructive test and inspection procedures for in-situ evaluation of materials. This issue is related to issues 2-18 and 6-3 regarding knowledge factor and too much required testing of concrete. Acceptance criteria depend on reliable knowledge of the material properties and condition of the components. Nonlinear procedures in particular require an in-depth understanding of the condition and material properties of components. Testing and condition assessment decreases the potential uncertainty and increases the reliability of results. However, the level of testing and destructive condition assessment specified in the *Guidelines* is extreme, and far in excess of standard practice. The amount of required testing is related to the selected analysis procedure, the level of information available on the building and the knowledge factor used in the analysis.

Resolution: Prestandard Section 2.2.6 was created to clearly outline data collection requirements. Minimum, comprehensive, and a new classification called usual data collection have been clearly defined. New provisions for usual data collection in Prestandard Sections 5.3.2 and 5.3.3 are intended to match current standard practice with regard to testing and condition assessment. Original FEMA 273 materials testing and destructive condition assessment provisions have been assigned to comprehensive data collection. New Table 2-1 was created to provide a matrix of information used for determination of testing requirements as related to rehabilitation objective, analysis procedure and knowledge factor

5-5 Presentation by System Type is Redundant

The presentation of material evaluation and acceptance criteria by system type, such as moment frame, braced frame, etc. is redundant, difficult to follow, and makes it difficult to compare the criteria for each system

Section: 5.4, 5.5, 5.6, 5.7, 5.8, 5.

Classification: *Non-persuasive.*

Discussion: This change would require editorial reorganization of information in all materials chapters. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive.

Resolution: No change proposed.

- 5-6** **Aluminum is Not Included**
Parameters for design, analysis and acceptance of aluminum structural systems are not included in the document.
- Section:** 5.4, 5.5, 5.6, 5.7, 5.8, 5.9.
- Classification:** *Non-persuasive.*
- Discussion:** At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive. The infrequent occurrence of aluminum in lateral force resisting systems does not warrant further consideration of this issue.
- Resolution:** No change proposed.
-
- 5-7** **Infill Evaluation Criteria Not Complete**
The *Guidelines* reference Chapters 6 and 7 for acceptance criteria when addressing steel frame structures with infills. The procedures in other materials chapters are not fully developed and not directly applicable for evaluating steel frame elements in infill systems.
- Section:** 5.7 (new section 5.8).
- Classification:** *Commentary Revision.*
- Discussion:** At the 3/3/99 Standards Committee meeting this issue was reclassified as commentary revision. It was the consensus opinion that the necessary information is already contained within the *Guidelines*, but that additional commentary could be added to further clarify the procedures.
- Resolution:** Commentary to Prestandard Section 5.8 has been expanded to provide additional direction regarding steel frame with infills.
-
- 5-8** **Inconsistent Specification of Acceptance Criteria**
The specification of acceptance criteria in Chapter 5 is inconsistent with the criteria specified in Chapter 6.
- Section:** 5.4, 5.5, 5.6, 5.7, 5.8, 5.9.
- Classification:** *Technical Revision.*
- Discussion:** Chapter 5 specifies deformation ratios (Δ/Δ_y), whereas Chapter 6 specifies deformation limits (maximum plastic hinge rotations). Ideally the acceptance criteria should be specified in the same way for similar actions in all materials. Special Study 9, Incorporating the Results of the SAC Joint Venture Steel Moment Frame Project was funded to research this issue further. Related to issue 5-14 regarding the relationship between Chapter 5 acceptance criteria and component length.
- Resolution:** Prestandard Table 5-6 containing nonlinear acceptance criteria for steel components has been revised to provide plastic hinge rotations or plastic deformation limits in a format that is more consistent with other chapters.

5-9***m*-factors Less Than 1.0 Too Low**

Component modification factors (*m*-factors) less than 1.0 are specified for some brittle components of significant concern. Values less than 1.0 imply these components require strengths in excess of pseudo lateral force elastic demands, which does not make sense.

Section: 5.4.2.3, 5.4.3.3, 5.5.2.3, 5.5.3.3, 5.6.3, 5.9.3

Classification: *Technical Revision.*

Discussion: None.

Resolution: Prestandard Tables in Chapter 5 have been revised so that all *m*-factors less than 1.0 are set equal to 1.0. Notes requiring the use of tabulated values divided by 2.0 have been revised to specify *m*=1.0 as a minimum value. Similarly, deformation ductility ratios for nonlinear acceptance criteria that were less than 1.0 have been revised to a minimum of 1.0.

5-10**Chapter 5 Acceptance Criteria Inconsistent and Unclear**

The acceptance criteria in Chapter 5 tables of *m*-factors and deformation limits is internally inconsistent and appears to contain errors. The treatment of P-M interaction needs clarification.

Section: 5.4, 5.5, 5.6, 5.7, 5.8, 5.9, Tables — all.

Classification: *Technical Revision.*

Discussion: The treatment of axial loads on beam-columns needs clarification. IO requirements for braces are more stringent than columns. Table headings are inconsistent with tabular values and it is unclear what the entries are intended to be.

Resolution: Prestandard Chapter 5 has been revised to correct these issues. Table headings and entries have been clarified and corrected based on errata published by ATC on November 2, 1999. Prestandard Section 5.5.2.4 has been revised to clarify beam-column acceptability requirements.

5-11 **Guidance on calculation of strength of anchor bolts needed**

Guidance on calculating the strength of anchor bolts is needed.

Section: 5.4, 5.5, 5.6

Classification: *Technical Revision*

Discussion: Prestandard Section 5.5 on FR frames references the limit states to be considered at the interface between steel columns and concrete foundations. (Sections for other systems reference FR frames as the basis for strength and acceptability calculations.) These limit states include consideration of anchor bolt bond to concrete, and failure of concrete. A new procedure for calculation of anchor bolt strength called the Concrete Capacity Design (CCD) Method has been developed and incorporated in Section 1916 of the IBC. The procedure explicitly evaluates the various failure states of the steel anchor or the concrete. Anchor bolt failure modes related to concrete failures should be treated as force controlled actions. Related to issue 5-16 regarding permissible nonlinearity in column base plates.

Resolution: Prestandard Section 5.5.2.3.2, Item 5 has been revised to reference Section 1913 of the IBC for calculation of anchor bolt strength, using ϕ equal to 1.0. Anchor bolt failure modes governed by concrete are designated as force-controlled actions.

5-12 **Braced Frame Connection Requirements Need Clarification**

Braced frame connection provisions appear too restrictive for applications where braces are lightly loaded and the connections are required to develop brace capacities that will not be utilized. Provisions are difficult to understand and should be clarified.

Section: 5.5 (new Section 5.6).

Classification: *Technical Revision*.

Discussion: The original *Guidelines* required that connections develop 1.25 times the compression capacity of the brace, or the brace m -factors were to be reduced by one half. This requirement is inconsistent with the overall methodology of force- and deformation-controlled actions. Brace connections should be treated as force-controlled and brace m -factors should not be related to connection capacity.

Resolution: Prestandard Section 5.6.2.4 has been revised to delete this requirement on brace connection capacity and associated adjustment in brace m -factors. Additionally, brace connection demands have been clearly defined as force-controlled actions.

- 5-13** **Incorporate SAC Research Into Chapter 5**
The acceptance criteria for steel moment resisting frame components in Chapter 5 should be updated to reflect the results of SAC research.
- Section:** 5.4 (new Section 5.5)
- Classification:** *Application of Published Research.*
- Discussion:** Special Study 9 – Incorporating Results of the SAC Joint Venture Steel Moment Frame Project was funded to research this issue. This study reviewed results of SAC research, and translated test results and reliability studies into plastic hinge rotation limits for FR and PR moment frame connections that are consistent with the format of acceptance criteria in other chapters.
- Resolution:** Section 5.5, Table 5-4, and Table 5-5 in the Prestandard have been revised to incorporate SAC research results.
-
- 5-14** **Steel Acceptance Criteria is Based on Component Length**
Nonlinear acceptance criteria for certain steel components are expressed as a multiple of yield rotation, which is based on the length of the component.
- Section:** 5.4, 5.5
- Classification:** *Recommended for Basic Research*
- Discussion:** Related to issue 5-8 regarding inconsistent specification of acceptance criteria. Values in Table 5-6 have been revised to express acceptance criteria in terms of plastic rotations as a multiple of yield rotation to be more consistent with other chapters. This however, has not changed the fundamental basis of the acceptance criteria for steel components. Calculation of yield rotation is based on chord rotation, and is proportional to the length of the component. This means that as the length of the component increases, the permissible plastic deformation increases. This is inconsistent with plastic rotation limits for concrete moment frames specified in Chapter 6, that are independent of component length. It is not immediately obvious why a given steel section would have a different plastic rotation limit when used in a component of a different length. In addition, as the length of the member decreases, the permissible plastic rotation tends toward zero.
- Resolution:** Unresolved pending future research.
-
- 5-15** **The Ratio Between IO and LS Acceptance Criteria Appears Too Large**
The ratio between IO and LS acceptance criteria for certain steel components appears to be too large. IO values for these components appear to be too low.
- Section:** 5.4, 5.5, 5.6 (new Tables 5-5 and 5-6)
- Classification:** *Recommended for Basic Research*
- Discussion:** Special Study 6, Acceptability Criteria (Anomalous m-values), identified this issue. One conclusion of this study was that based on Section 2.13 (Prestandard Section 2.8) Immediate Occupancy acceptance criteria should be on the order of 25% to 50% of the values for Life Safety. Values for diagonal brace, steel plate shear wall, and diaphragm components exceed these ratios.
- Resolution:** Unresolved pending future research.

5-16**Nonlinearity is Permitted in Column Base Plates**

For certain controlling actions, nonlinearity is permitted in column base plates. Column bases should be treated as force-controlled.

Section: 5.4.2.3, 5.4.3.3 (new Section 5.5.2.3.2, Item 5)

Classification: *Recommended for Basic Research*

Discussion: This issue was raised at the 8/23/00 Standards Committee meeting. Exception was taken to the use of m -factors on column base connections. It was stated that nonlinearity should be forced to occur in the structure above the base connection. This is contrary to the original intent of the *Guidelines*, which permitted nonlinear activity on ductile behavior such as the base plate yielding.

Resolution: Unresolved pending future research.

5-17**Tension-only Braces Have Full Nonlinear Deformation Limits**

Tension-only braces have the same nonlinear deformation limits as tension/compression braces.

Section: 5.5 (new Section 5.6)

Classification: *Technical Revision*

Discussion: The behavior of tension-only bracing systems is very different than systems in which the braces act in both tension and compression. Tension-only systems have extremely pinched hysteretic behavior and are subject to impact loading as the braces alternately stretch, buckle and then re-tension. Linear acceptance criteria (m -factors) for these systems are adjusted to half the values for tension/compression braces, but no such adjustment is provided for nonlinear acceptance criteria.

Resolution: A footnote has been added to Prestandard Table 5-6 to reduce nonlinear deformation limits by one-half for tension-only brace components, similar to the original note applying to m -factors.

6. Concrete

(Systematic Rehabilitation)

Chapter 6 provides guidance on systematic rehabilitation of concrete structural systems including moment frames, braced frames, shear walls, diaphragms and foundations. It includes procedures for obtaining material properties and the condition assessment of concrete structures, and describes the acceptance criteria for concrete components.

6.1 New Concepts

- Testing requirements: The *Guidelines* include new requirements on testing and condition assessment for determination of design and analysis parameters for the concrete structure.
- Non-conforming components and elements: Procedures are included for quantitatively evaluating the capacity of elements and components that may have limited ductility because they do not conform to the reinforcing requirements of modern day codes, standards or construction.
- Modeling parameters: Specific guidance is provided on modeling parameters for concrete elements including effective stiffness, and material properties.
- Flanged construction: Intersecting components will act compositely, and the response will differ substantially from that of isolated components. Specific guidance is provided for assigning a portion of perpendicular intersecting components as effective flanges for the component under consideration.
- Rehabilitation techniques: Specific guidance is provided on selecting appropriate rehabilitation techniques for concrete systems. Among traditional measures including addition of shear walls or shotcrete elements to the structural system, rehabilitation techniques include jacketing non-conforming elements to improve confinement.
- Infill frames: The *Guidelines* include enhanced discussion of the interaction between infill walls and frame elements, and new evaluation techniques for rehabilitation of infill frame systems.

6.2 Global Issues

6-1 *m*-factors Appear Overly Conservative

Certain values of acceptance criteria (*m*-factors) and deformation limits for concrete components appear to be too conservative and are not consistent with other chapters. Of particular concern is an inconsistency with Chapter 7, Masonry.

Section: Tables 6-6, 6-7, 6-8, 6-10, 6-11, 6-12, 6-13, 6-14, 6-15, 6-16, 6-17, 6-18, 6-19, 6-20; Sections 6.5.x.4, 6.6.x.4, 6.7.x.4, 6.8.x.4, 6.9.2.4, 6.10.5, 6.11.2, 6.12.2, 6.13.3.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to augment data used to develop acceptance criteria. Existing values were determined on a rational basis using available experimental results. This issue is related to issue 2-6 regarding baselining of acceptance criteria. Special Study 6 – Acceptability Criteria (Anomalous *m*-values) was funded to research this issue. The results of this study are still under consideration by the Project Team. Changes to *m*-factor tables in Chapter 6 are on hold pending further discussion.

Resolution: Unresolved pending future research.

6-2 Presentation by System Type is Redundant

The presentation of material evaluation and acceptance criteria by system type, such as moment frame, shear wall, etc. is redundant, difficult to follow, and makes it difficult to compare the criteria for each system.

Section: 6.5, 6.6, 6.7, 6.8, 6.9, 6.10, 6.11, 6.12, 6.13.

Classification: *Non-persuasive.*

Discussion: This change would require editorial reorganization of information in all materials chapters. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive.

Resolution: No change proposed.

6-3 Too Much Testing is Required

The *Guidelines* require too much testing of in-place materials for the determination of design and analysis parameters.

Section: 6.3.2, 6.3.3.

Classification: *Technical and Commentary Revision (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to develop non-destructive test and inspection procedures for in-situ evaluation of materials. This issue is related to issues 2-18 and 5-4 regarding knowledge factor and too much required testing of steel. Acceptance criteria depend on reliable knowledge of the material properties and condition of the components. Nonlinear procedures in particular require an in-depth understanding of the condition and material properties of components. Testing and condition assessment decreases the potential uncertainty and increases the reliability of results. However, the level of testing and destructive condition assessment specified in the *Guidelines* is extreme, and far in excess of standard practice. The amount of required testing is related to the selected analysis procedure, the level of information available on the building and the knowledge factor used in the analysis.

Resolution: Prestandard Section 2.2.6 was created to clearly outline data collection requirements. Minimum, comprehensive, and a new classification called usual data collection have been clearly defined. New provisions for usual data collection in Prestandard Sections 5.3.2 and 5.3.3 are intended to match current standard practice with regard to testing and condition assessment. Original FEMA 273 materials testing and destructive condition assessment provisions have been assigned to comprehensive data collection. New Table 2-1 was created to provide a matrix of information used for determination of testing requirements as related to rehabilitation objective, analysis procedure and knowledge factor.

6-4 Guidance for Concrete Infill Panels Needed

The section on infill frames does not provide guidance on evaluation of concrete infill panels.

Section: 6.7.

Classification: *Commentary Revision.*

Discussion: At the 3/3/99 Standards Committee meeting this issue was reclassified as a commentary revision.

Resolution: Commentary to Prestandard Section 6.7.1.3 has been added to provide additional guidance on concrete infill.

- 6-5** **Inconsistent Definition of Weak Story**
Definition of weak story in Section 6.5.2.4 is not consistent with the definition in Section 2.9.1.1. DCR requirements should be centralized in one location with additional explanation regarding their use.
- Section:** 6.5.2.4, 2.9.1.1 (new section 2.4.1.1).
- Classification:** *Technical Revision.*
- Discussion:** Section 2.9.1.1 is a trigger measuring relative story strengths. Section 6.5.2.4 is a trigger measuring relative strengths of beams and columns. Section 6.5.2.4 should refer to weak column elements, so there is no conflict in definitions. Material specific DCR requirements are best located in the appropriate materials chapter. Proposed changes regarding DCRs were found non-persuasive by the Prestandard Project Team.
- Resolution:** Prestandard Section 6.5.2.4.1 has been revised to refer to weak column elements.
-
- 6-6** **Clarify Shear Wall Component Definitions**
Clarification is required regarding evaluation of pierced shear walls. Classification of components as wall segments, beams or coupling beams needs further guidance. The acceptance criteria are not consistent between classifications.
- Section:** 6.8.2.
- Classification:** *Application of Published Research.*
- Discussion:** It is not clear how to select the most appropriate classification for components of pierced shear walls. Acceptance criteria in terms of plastic hinge rotation are more stringent for wall segments than they are for non-ductile concrete frame elements, which seems inconsistent with expected performance of the two systems. Special Study 8 – Incorporation of Selected Portions of Recent Related Documents was funded to research this issue. The main conclusion of this study was that useful information is available in FEMA 306, 307 and 308, to assist in classifying and evaluating the concrete components, but since these documents are not standards themselves, they could not be referenced directly by the Prestandard.
- Resolution:** Information consisting of a table of component types and figure showing various wall component configurations has been extracted from FEMA 306 and added as new commentary to Prestandard Section 6.8.1 to assist in the identification of wall component classifications.
-
- 6-7** ***m*-factors Less Than 1.0 Too Low**
Component modification factors (*m*-factors) less than 1.0 imply certain concrete components require strengths in excess of pseudo lateral force elastic demands, which does not make sense.
- Section:** All.
- Classification:** *Technical Revision.*
- Discussion:** No *m*-values less than 1.0 appear in Chapter 6.
- Resolution:** No change proposed.

- 6-8** **Tables 6-13 and 6-14 Reversed**
Tables 6-13 and 6-14 regarding m -values and deformation acceptance criteria for flat plate moment frames are interchanged and incorrectly referenced within the text.
- Section:** Tables 6-13 and 6-14; Section 6.5.4.4.
- Classification:** *Editorial Revision.*
- Discussion:** None.
- Resolution:** The Prestandard has been corrected to properly reference the tables.
-
- 6-9** **m -factors Less Than 2.0 Worse Than Force-Controlled**
Considering actions associated with m -factors less than 2.0 as deformation-controlled may be more restrictive than considering the same action as force-controlled and using the J factor.
- Section:** 3.4.2.
- Classification:** *Commentary Revision.*
- Discussion:** J can be between 1.0 and 2.0. Force-controlled actions are less desirable than deformation-controlled actions, and the criteria should be more restrictive. When m is less than about 1.5 it may appear to be more favorable to treat elements as forced-controlled. However, calculation of demand on force-controlled actions requires a limit state analysis, and capacity is calculated using lower bound strengths. If these concepts are properly applied, the method will yield a safe result whether the action is considered force- or deformation-controlled.
- Resolution:** Commentary from FEMA 274, Section 3.4.2.1 has been added to Prestandard Section 3.4.2.1.2 to clarify the application of force-controlled acceptance criteria.
-
- 6-10** **Column Acceptance Criteria Overly Conservative**
The acceptance criteria for concrete columns appear to be overly conservative, even for secondary elements. Concrete shear strength goes to zero at high ductility demands, which may be too stringent.
- Section:** Table 6-7, 6-11 (new Tables 6-8, 6-12); Sections 6.4.4, 6.5.
- Classification:** *Technical Revision.*
- Discussion:** Special Study 5 – Report on Multidirectional Effects and P-M Interaction on Columns was funded to research this issue. The major conclusion of this study was that more data on concrete column failures in the range of interest is available, and revisions of the acceptance criteria can be made.
- Resolution:** Column acceptance criteria in Prestandard Section 6.5.2.3.1 have been revised in accordance with this study. Prestandard equation 6-4 for concrete contribution to shear capacity has been revised to better match results from tests. Prestandard Tables 6-8 and 6-12 have been revised to increase acceptance criteria for concrete columns based on data from recent tests.

- 6-11** **Footnote 1, Table 6-20 Incorrect**
Footnote 1 in Table 6-20 incorrectly reads ‘stress’ when it should read ‘capacity’.
- Section:** Table 6-20 (new Table 6-21).
- Classification:** *Non-persuasive.*
- Discussion:** Footnote 1 sets limits on application of deformation acceptance criteria based on axial load and shear demands on the element. The term ‘capacity’ is not appropriate.
- Resolution:** Prestandard Table 6-21 has been revised to read ‘demand’ in Footnote 1.
-
- 6-12** **Table 6-17 Missing Headings**
Table 6-17 regarding numerical acceptance criteria for nonlinear procedures is missing column headings. Rotation limits for coupling beams should be entitled chord rotations.
- Section:** Table 6-17.
- Classification:** *Editorial Revision.*
- Discussion:** The missing headings imply the acceptance criteria listed for coupling beams are plastic hinge rotation limits. This is incorrect and significantly different from the correct limits which are actually chord rotation limits.
- Resolution:** Column headings in Prestandard Tables have been corrected.
-
- 6-13** **Column P-M Interaction Unclear**
Acceptance criteria for P-M interaction in concrete columns is unclear.
- Section:** 6.4.3.
- Classification:** *Technical Revision.*
- Discussion:** This issue was raised at the 3/3/99 Standards Committee meeting. Flexure in concrete columns is treated as deformation-controlled, while axial loads are force-controlled. For concrete braced frames in Section 6.10.5, axial actions in braces are considered deformation controlled. It is unclear how to check the interaction between force-controlled and deformation-controlled actions when they occur simultaneously on one component. Special Study 5 – Report on Multidirectional Effects and P-M Interaction on Columns was funded to research this issue.
- Resolution:** Prestandard Section 6.4.3 has been expanded to provide direction on how to address P-M interaction and biaxial bending of concrete columns. Axial force actions are considered force-controlled and a squared interaction relationship for biaxial bending has been introduced.

6-14 **Guidance for Lightweight Concrete Needed**
Guidance is required on how to address lightweight concrete in capacity calculations.

Section: Chapter 6, all.

Classification: *Technical Revision.*

Discussion: The current document refers to ACI 318 for calculation of component strengths. Since ACI 318 addresses lightweight concrete, it can be interpreted that consideration of lightweight concrete has already been included. However, this consideration could be made more explicit.

Resolution: Prestandard Sections 6.4.2.2 and 6.4.2.3 have been revised to explicitly reference ACI 318 adjustments for lightweight concrete in the calculation of component strengths.

6-15 **Guidance for Square Rebar Needed**
Guidance is required on how to address square reinforcing steel in capacity calculations.

Section: Chapter 6, all.

Classification: *Technical Revision.*

Discussion: None.

Resolution: Prestandard Section 6.4.5.1, Square Reinforcing Steel, has been created to provide direction on square bars. Twisted square bars are to be treated as deformed bars and straight square bars are to be treated as plain bars. For calculation of required development length or maximum developed stress in square reinforcing bars (Prestandard Section 6.4.5), the area of the square bars, or an effective bar diameter, d_b , calculated based on the area of the square bars, will be used as appropriate.

6-16 ***m*-factors for Concrete Diaphragms Needed**
Acceptance criteria for concrete diaphragms are based on DCR values. Diaphragm criteria should be base on *m*-factors.

Section: 6.11, 6.11.2.4.

Classification: *Technical Revision.*

Discussion: Cast-in-place concrete diaphragm components can be considered to behave like shear wall components. The current criteria using DCR values is overconservative.

Resolution: Prestandard Section 6.11.2.4 on concrete diaphragms has been revised to reference acceptance criteria for shear walls. Section 6.12.2 has been revised to incorporate conservative *m*-factors, based on judgement, for topping slabs on precast concrete diaphragms

- 6-17** **Acceptability for Columns in Tension Missing**
Acceptability requirements for concrete columns in tension are not provided.
- Section:** 6.4.
- Classification:** *Recommended for Basic Research.*
- Discussion:** None.
- Resolution:** Unresolved pending future research.
-
- 6-18** **Calculation of M_y for Shearwalls Unconservative**
The procedure in Section 6.8.2.3 for calculating the yield moment of reinforced concrete wall sections may underestimate the actual flexural capacity. This result would be unconservative for use in a limit state analysis.
- Section:** 6.8.2.3.
- Classification:** *Recommended for Basic Research*
- Discussion:** None.
- Resolution:** Unresolved pending future research.
-
- 6-19** **Omit Sampling of Prestressing Steel**
Sampling of prestressing steel is unnecessary and dangerous. Requirements for testing of prestressing steel should be deleted.
- Section:** 6.3.2.4 (new Section 6.3.2.4.4).
- Classification:** *Non-persuasive*
- Discussion:** Prestandard Section 6.3.2.4.4 currently only calls for sampling of prestressing steel for lateral force resisting elements, and suggests that sampling should occur beyond the anchorage to avoid loss of prestress. If a prestressed component is going to be used for lateral force resistance in the rehabilitated structure, the material properties of the prestressing steel must be subject to the same data collection requirements of other materials. For linear procedures, BSO performance, and minimum or usual data collection with information from drawings, testing would not be required. However, for enhanced objectives, or in the absence of drawings, testing would be necessary.
- Resolution:** No change made.
-
- 6-20** **Concrete Flange Provisions Unconservative**
Provisions for flanged sections in Section 6.4.1.3 may underestimate the frame action of the system when applied to joist construction.
- Section:** 6.4.1.3.
- Classification:** *Recommended for Basic Research.*
- Discussion:** None.
- Resolution:** Unresolved pending future research.

6-21**Clarify Definition of Closed Stirrups, Ties and Hoops**

The terms closed stirrups, ties and hoops are not used consistently in tables of concrete acceptance criteria.

Section: Tables 6-7, 6-8, 6-9, 6-18, Section 6.14

Classification: *Technical Revision.*

Discussion: Table 6-7 for beams reads closed stirrups at hinge locations. Table 6-8 for columns reads closed hoops at hinge locations. Table 6-9 for joints reads closed hoops with 135 degree hooks and no lap splices within the joint. Table 6-18 for wall segments reads closed stirrups along entire length. Since these terms are important for selection of appropriate acceptance criteria, clarification is needed regarding the necessity for 135 degree hooks and absence of lap splices.. The intent of the original FEMA 273 *Guidelines* was that, in the case of beam, column and joint components of concrete moment frames, conforming transverse reinforcement meant ACI hoops with no lap splices and 135 degree hooks on the ends (with 90 degree hooks permitted on cross-ties). This requirement was not intended to apply to concrete wall segments.

Resolution: The terms “hoops” and “closed ties or stirrups” have been added to the list of definitions in the Prestandard. “Hoops” refers to ACI 318 hoops, with seismic hooks and no lap splices. “Closed ties or stirrups” refers to ACI 318, Section 7.11 for lateral reinforcement of flexural members, which permits 90 degree hooks and lap splices. The footnotes of tables 6-7, 6-8 and 6-9 for concrete frame components have been revised to refer to hoops as defined above. The footnotes of Table 6-18 for shear wall components have been revised to refer to closed ties or stirrups.

7. Masonry

(Systematic Rehabilitation)

Chapter 7 provides guidance on systematic rehabilitation of masonry structural systems including shear walls, infill walls, wall anchorage and foundations. Types of masonry covered by this chapter include solid or hollow clay-unit masonry, solid or hollow concrete-unit masonry and hollow clay tile, but excludes glass block and stone masonry. It includes procedures for obtaining material properties and the condition assessment of masonry elements, and describes the acceptance criteria for masonry components.

7.1 New Concepts

- Testing requirements: The *Guidelines* include new requirements on testing and condition assessment for determination of design and analysis parameters for masonry components.
- Rehabilitation techniques: Specific guidance is provided on selecting appropriate rehabilitation techniques for masonry elements. Techniques include infilling openings, enlarging openings, applying shotcrete or other exterior structural bracing.
- Infill walls: The *Guidelines* include enhanced discussion of the interaction between infill walls and frame elements, and new evaluation techniques for rehabilitation of masonry infill wall components.
- Ductility in URM walls: The evaluation of unreinforced masonry walls now considers two new failure modes consisting of bed-joint sliding shear and toe crushing that are defined and quantified. Depending on which failure mode governs the behavior, the walls can be considered deformation-controlled, and *m*-values are provided.

7.2 Global Issues

7-1 *m*-factors Appear Overly Conservative

Certain values of acceptance criteria (*m*-factors) and deformation limits for masonry components appear to be too conservative and are not consistent with other chapters. Of particular concern is an inconsistency with Chapter 6, Concrete.

Section: Tables 7-1, 7-4; Sections 7.4.2.3, 7.4.4.3, 7.5.2.3, 7.7.2.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to augment data used to develop acceptance criteria. Additional studies of inelastic behavior of elements are recommended to refine acceptance criteria. Acceptance criteria for masonry elements appear to result in higher capacities than similar elements in concrete, which is counter-intuitive. This issue is related to issue 2-6 regarding baselining of acceptance criteria. Special Study 6 – Acceptability Criteria (Anomalous *m*-values) and Special Study 10 – Issues related to Chapter 7 were funded to research this issue further. The conclusions of Special Study 6 did not impact *m*-factor tables in Chapter 7. Special Study 10 concluded that *m*-factors for shear controlled reinforced masonry walls were necessary to make Chapter 7 more consistent with Chapter 6. These factors were subsequently incorporated into Prestandard Tables 7-6 and 7-7, but neither study concluded that significant changes to the remaining *m*-factors were required.

Resolution: Unresolved pending future research.

7-2 URM h/t Limits Independent of Performance Level

Height to thickness ratio acceptance criteria for URM walls out-of-plane does not change for CP, LS, and IO performance levels.

Section: Tables 7-3; Section 7.4.3.3.

Classification: *Non-persuasive.*

Discussion: Height to thickness ratios are not applicable to the IO performance level. Meeting the ratios satisfies the LS performance level, but there is no technical basis for relaxing the criteria for the CP performance level.

Resolution: No change proposed.

7-3 Interpolation Not Specified

Not all acceptance values are defined as a “sliding scale” between limits.

Section: All Tables, 7.4.4.2.

Classification: *Editorial Revision.*

Discussion: All tables note that interpolation between values is permitted. In Section 7.4.4.2, it is not clear that for values of M/Vd between limits for equations 7-9 and 7-10, interpolation is intended.

Resolution: Prestandard Section 7.4.4.2.2 has been revised to specify interpolation between limits.

7-4

Guidance for Infill Panels with Openings Needed

Evaluation of masonry infills does not provide adequate guidance for addressing masonry infill panels with openings.

Section: 7.5.2.

Classification: *Commentary Revision and Basic Research.*

Discussion: At the 3/3/99 Standards Committee meeting this issue was reclassified as a commentary revision. While the equivalent diagonal compression strut analogy may not be directly applicable when openings are present in the infill panel, some guidance is provided on how to modify the procedure when openings are present. Further research is necessary to develop simplified methods for considering openings in infill panels.

Resolution: Additional information from FEMA 274 was added to the commentary for Prestandard Section 7.5.2. Further resolution of this issue is recommended for basic research.

7-5

Quantitative Definition of Masonry Terms Needed

The acceptance criteria for masonry components in Chapter 7 depend on the condition of the masonry. Qualitative terms such as good, fair, poor, significant cracking, etc. are used throughout. A quantitative measure or definition of these terms is required to properly apply the provisions of the standard.

Section: 7.3.2.1, 7.8.

Classification: *Application of Published Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to establish an improved relationship between crack widths and performance of damaged masonry components. For the standard to be enforceable, qualitative terms must be defined with some quantitative measurement. The ATC-43 project (FEMA 306, 307 and 308) is a potential source for information on crack widths. Special Study 8 - Incorporation of Selected Portions of Recent Related Documents was funded to research this issue. The main conclusion of this study was that useful information is available in FEMA 306, 307 and 308, to assist in evaluating the condition of masonry, but since these documents are not standards themselves, they could not be referenced directly by the Prestandard.

Resolution: Commentary was added in Prestandard Section 7.3.2.1, and in the definitions of Section 7.8, to reference more detailed information on the condition of masonry contained in FEMA 306, 307 and 308.

- 7-6** **1.25 fy Not Specified for Masonry**
Expected strength calculations for reinforced masonry components do not utilize $1.25 \cdot f_y$ for strength of reinforcement, similar to concrete components.
- Section:** 7.3.2.10, 7.4, 7.4.4.2.1
- Classification:** *Commentary Revision.*
- Discussion:** Calculation of expected strength of masonry components calls for the use of expected material properties. The expected strength of reinforcing steel is intended to include consideration of material overstrength and strain hardening expected in yielding components. Section 7.3.2.10 on default properties references Chapter 6 for reinforcing steel, which includes a 1.25 factor used to convert lower bound yield stress to expected strength. Section 7.4 was previously revised to include reference to using $1.25 \cdot \text{nominal yield stress}$, but this is redundant with the use of expected strength.
- Resolution:** Commentary has been added to Prestandard Sections 6.4.2.2, and Section 7.4 to clarify that the use of expected strength material properties for reinforcing steel includes a 1.25 factor to account for material overstrength and strain hardening that is expected in yielding components.
-
- 7-7** **h/t Ratios for S_{X1} Exceeding 0.5g Needed**
The spectral response acceleration values in the headings of Table 7-3 for URM h/t ratios are limited to 0.50g. There is no guidance for sites with S_{X1} values exceeding 0.50g.
- Section:** Section 7.4.3.3, Table 7-3 (new Table 7-5).
- Classification:** *Technical Revision.*
- Discussion:** The h/t ratios in Table 7-3 were developed with a different definition of seismic hazard in mind. Values for S_{X1} between 0.37g and 0.50g are applicable above 0.50g.
- Resolution:** Table 7-5 in the Prestandard has been revised so that the column of h/t ratios for the highest seismic hazard is not limited to 0.50g.
-
- 7-8** **Clarify Application of Equations 7-5 and 7-6**
The application of Equations 7-5 and 7-6, particularly outside of specified L/h_{eff} limits, is unclear.
- Section:** Section 7.4.2.2.
- Classification:** *Editorial Revision.*
- Discussion:** None.
- Resolution:** Prestandard Sections 7.4.2.2 and 7.4.2.2.2 have been expanded to clarify the proper application of Equations 7-5 and 7-6.

- 7-9** **Clarify Definition of Effective Height**
The definitions of parameters Δ_{eff} and h_{eff} require additional clarification.
- Section:** Section 7.9, 7.4.2.3.2 (related to Figure 7-1)
- Classification:** *Commentary Revision.*
- Discussion:** This issue was raised in the BSSC Case Studies Report and Special Study 1 - Early Input from the BSSC Case Studies Report was funded to research this issue further.
- Resolution:** Prestandard definitions of parameters Δ_{eff} and h_{eff} have been clarified. Commentary to Prestandard Section 7.4.2.3.2 has been added with a figure to clarify what is meant by these terms.
-
- 7-10** **Masonry Shear Strength Based on Average Test Values is Unconservative**
The calculation of expected masonry shear strength using average values of brick shear tests overestimates the actual shear strength.
- Section:** 7.3.2.4
- Classification:** *Application of Published Research*
- Discussion:** This issue was raised at the 8/23/00 Standards Committee meeting. Use of average shear test values to estimate shear strength by calculation reportedly does not correlate well with results of full-scale wall tests. Special Study 10 – Issues related to Chapter 7 was funded to research this issue further. This study concluded that average brick shear test values was the intended value, although this resolution has not found consensus with all members of the standards committee.
- Resolution:** Unresolved pending further study.
-
- 7-11** **URM Shear Strength Should be Force-Controlled**
Shear strength of URM walls is brittle and unreliable and should be treated as a force-controlled action.
- Section:** 7.4.2.2
- Classification:** *Recommended for Basic Research*
- Discussion:** This issue was raised at the 8/23/00 Standards Committee meeting. The shear strength of URM walls is limited by diagonal tension failure that originates at the weakest point in the brick and mortar matrix. Shear failure is brittle and the ultimate values are unreliable. This type of action should not have m -factors that permit significant inelastic activity. This is contrary to the concept introduced in the original *Guidelines* that URM walls governed by bed-joint sliding or rocking have some level of ductility. Special Study 10 – Issues related to Chapter 7 was funded to research this issue further. This study concluded that certain shear failures in URM walls could be considered deformation-controlled, although this resolution has not found consensus with all members of the standards committee.
- Resolution:** Unresolved pending future research.

8. Wood and Light Metal Framing (Systematic Rehabilitation)

Chapter 8 provides guidance on systematic rehabilitation of wood and light metal framing systems including shear walls, diaphragms and foundations. It includes procedures for obtaining material properties and performing the condition assessment, and describes the acceptance criteria for wood and light metal framing components.

8.1 New Concepts

- Testing requirements: The *Guidelines* include new requirements on testing and condition assessment for determination of design and analysis parameters for wood and light metal framing components.
- Rehabilitation techniques: Specific guidance on selecting appropriate rehabilitation techniques for wood and light metal framing elements is provided. Techniques include the addition of wood structural panel overlays on existing assemblies, and increased attachment between sheathing and framing.
- Strength varies with aspect ratio: Because excessive deflection can result in major damage to the structure and its contents, acceptance criteria for wood components is based on the height/length or length/width ratios.
- Non-conforming components and elements: Procedures are included for quantitatively evaluating the capacity of elements and components that do not conform to construction based on modern day codes and standards.

8.2 Global Issues

8-1 *m*-factors Appear Overly Conservative

Certain values of acceptance criteria (*m*-factors) and deformation limits for wood components appear to be too conservative.

Section: Table 8-1.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to augment data used to develop acceptance criteria. Additional studies of inelastic behavior of elements are recommended to refine acceptance criteria. This issue is related to issue 2-6 regarding baselining of acceptance criteria. Special Study 6 – Acceptability Criteria (Anomalous *m*-values) and Special Study 11 – Wood Issues were funded to research this issue further. The conclusions of Special Study 6 did not impact *m*-factor tables in Chapters 8, however, Special Study 11 concluded that, based on current available research, tabulated *m*-factors appear to be appropriate given the expected strengths provided.

Resolution: Unresolved pending future research.

- 8-2** **Guidance for Diaphragm Chord Area Needed**
More guidance on how to determine the area of the chord for use in a diaphragm deflection calculation is required.
- Section:** 8.5.7.1.
- Classification:** *Commentary Revision.*
- Discussion:** Chapter 8 covers acceptance criteria for wood diaphragms that is applicable to all building types with wood diaphragms. The area of the chord can be different on each side particularly when concrete walls are present and only the reinforcing steel can be considered effective in tension. Further clarification is required on what to consider as diaphragm chords.
- Resolution:** Commentary to Section 8.5.7.1 has been added in the Prestandard to provide additional guidance.
-
- 8-3** **Wood Values Based on Judgment**
Values for wood components are based on engineering judgment rather than tests.
- Section:** All.
- Classification:** *Recommended for Basic Research.*
- Discussion:** Special Study 11 – Wood Issues was funded to research this issue further. This study reviewed historic research as well as preliminary results from current research underway at UCI, and proposed revisions to tabulated strength and stiffness values for wood shear wall and diaphragm assemblies.
- Resolution:** Revised tabulated strength and stiffness values for wood shear wall and diaphragm assemblies, and revised equations for calculation of shear wall and diaphragm deflections have been incorporated into Prestandard Chapter 8.
-
- 8-4** **Anomalous *m*-factors for Different Assemblies**
There are apparent anomalies when *m*-values for different assemblies are compared.
- Section:** Table 8-1.
- Classification:** *Commentary Revision.*
- Discussion:** As an example, *m*-values for gypsum plaster are higher than values for structural panels, implying better performance. However, since expected strengths for gypsum plaster are much lower than structural panels, the combination of $m \cdot Q_{ce}$ is higher for structural panels, as expected. There is no real anomaly.
- Resolution:** Commentary has been added to the Prestandard to explain this apparent anomaly.
-
- 8-5** **Combined with 3-8**
Combined with Global Issue 3-8 and omitted.
- Section:** None.
- Classification:** *None.*
- Discussion:** None.
- Resolution:** None.

8-6 Use of Default Values Needs Clarification

The shear wall and diaphragm sections list capacities and non-linear parameters for various assemblies. It is not clear whether these values are directly applicable to the NSP, or if verification testing is required before the specified nonlinear parameters can be used.

Section: 8.3.2.5.

Classification: *Editorial Revision.*

Discussion: Capacity values and nonlinear acceptance criteria in Chapter 8 are similar in concept to acceptance criteria specified for other materials. These values are intended to be used directly, without verification testing of mock-up assemblies.

Resolution: Prestandard Section 8.3.2.5 has been revised to clarify the use of default capacities for assemblies. Section 8.3.4 has been revised to make knowledge factor, κ , requirements consistent with this intent.

8-7 Inconsistent Requirements for Connections

The sections on various types of shear wall assemblies require connections to be checked or not checked depending on the perceived strength of the assembly. The sections are not consistent. In some cases weaker assemblies require verification of connections, and stronger assemblies do not.

Section: 8.4.x.4.

Classification: *Technical Revision.*

Discussion: For example, Section 8.4.11 for plaster on wood lath lists a capacity of 400 lbs/ft and does not require the connections to be checked, while Section 8.4.4 for horizontal siding lists a capacity of 80 lbs/ft and requires connections to be checked. The original distinction between assemblies requiring verification of connections and those that did not was related to ease of inspection and ability to verify connections without destroying the assembly.

Resolution: Prestandard Sections 8.4.x.4 have been revised for consistency with regard to verification of connections.

8-8 Guidance on Wood Components in Compression Needed

Guidance on the evaluation of wood posts below discontinuous shear walls, components of knee-braced frames, and braced horizontal diaphragms is needed.

Section: 8.4.

Classification: *Technical Revision.*

Discussion: Wood components are generally considered deformation-controlled. Provisions on how to address wood components in compression are necessary because this situation requires a force-controlled application of the criteria.

Resolution: Prestandard Section 8.4 has been revised to provide direction on consideration of posts below discontinuous shear walls. Prestandard Section 8.8 was created to provide direction on strength and acceptance criteria for knee-braced frames and other miscellaneous wood components.

8-9**Lower-Bound Capacities for Wood Components Needed**

Direction on calculation of lower-bound capacities for wood components is needed for evaluation of force-controlled actions.

Section: 8.3.2.5

Classification: *Technical Revision.*

Discussion: Wood components and connections are generally considered deformation-controlled. Because of this, Chapter 8 lacks defined criteria for calculation of lower-bound capacities. These capacities are needed for evaluation of force-controlled actions on wall anchorage components, bodies of connections, posts below shear walls. Special Study 11 – Wood Issues was funded to research this issue further. The factor proposed in this study (0.85) is based on mean minus one standard deviation values for the recently completed CoLA/UCI testing of shear walls.

Resolution: Prestandard Section 8.3.2.5 has been revised to include a 0.85 factor for conversion from expected strength to lower bound for use when needed.

8-10**Stiffness Values for Wood Assemblies are Not Supported by Tests**

Stiffness values that are provided for wood shear wall and diaphragm assemblies are inconsistent and not supported by tests.

Section: 8.3.2.5, 8.4, 8.5 (new Tables 8-1 and 8-2)

Classification: *Application of Published Research*

Discussion: This issue was raised at the 8/23/00 Standards Committee meeting. Values for assemblies when used as shear walls are different for the same assemblies when used as diaphragms. Special Study 11 – Wood Issues was funded to research this issue further. This study reviewed preliminary results from the recently completed CoLA/UCI testing of shear walls to develop proposed revisions to tabulated shear wall and diaphragm assembly stiffness.

Resolution: Revised tabulated stiffness values for wood shear wall and diaphragm assemblies, and revised equations for calculation of shear wall and diaphragm deflections have been incorporated into Prestandard Chapter 8.

8-11**Wood Conversion Factors are not Supported by Tests**

Factors used to convert allowable values to expected strength are not supported by tests.

Section: 8.3.2.5

Classification: *Application of Published Research*

Discussion: This issue was raised at the 8/23/00 Standards Committee meeting. Factors consisting of $2.16 \times 0.8 \times 1.6 = 2.8$ are not representative of the actual factors of safety present between allowable values of wood components and tested ultimate strengths. Special Study 11 – Wood Issues was funded to research this issue further. This study reviewed preliminary results from the recently completed CoLA/UCI testing of shear walls to develop revised conversion factors based on the test results

Resolution: The methodology for calculating component capacities has been revised to a strength-based procedure using wood LRFD provisions. Revised conversion factors from allowable to expected strength have been provided in the commentary to retain this method as an alternative.

9. Seismic Isolation and Energy Dissipation (Systematic Rehabilitation)

Chapter 9 provides guidance on systematic rehabilitation of buildings using base isolation or passive energy dissipation systems. It includes specific direction on both linear and nonlinear modeling and analysis procedures for structures with isolators or energy dissipation devices. It also includes requirements for verification and testing of the design properties of isolators and energy dissipation devices.

9.1 New Concepts

Passive energy dissipation systems: The *Guidelines* provide direction on the implementation of energy dissipation devices in the systematic rehabilitation of structures. While design provisions for seismic isolation have been in place for some time, comprehensive provisions for energy dissipation have not been published before the *Guidelines*.

9.2 Global Issues

9-1 Procedures Require Validation

Analytical procedures for energy dissipation systems require validation.

Section: 9.3.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to validate energy dissipation procedures through analytical studies comparing results of linear static and nonlinear static analyses with results of nonlinear time-history analyses.

Resolution: Unresolved pending future research.

9-2 Inconsistent Nomenclature

Response acceleration parameter nomenclature in Chapter 9 is not consistent with the nomenclature in the rest of the document.

Section: 9.2, 9.3, 2.6.1.5.

Classification: *Editorial Revision.*

Discussion: The names of the spectral response acceleration parameter variables in Chapter 9 are different from those elsewhere in the document. Section 2.6.1.5 includes a cross-reference between the variables.

Resolution: The nomenclature in Chapter 9 of the Prestandard has been revised to be consistent with the rest of the document. Section 2.6.1.5, which previously provided cross-reference information for the nomenclature has been deleted.

9-3**Clarify Use of C1, C2, C3 with Isolation**

Clarification regarding the use of coefficients C_1 , C_2 , C_3 , and J for seismically isolated structures is required in Chapter 9.

Section: 9.2.1.

Classification: *Editorial Revision.*

Discussion: Procedures for seismic isolation calculate design displacements directly. Additional modification of response using these coefficients is incorrect.

Resolution: A sentence was added in Prestandard Section 9.2.1 clarifying that coefficients C_1 , C_2 , C_3 , and J shall be taken as 1.0 for seismically isolated structures.

9-4**Chapter 9 Needs Controls for Proper Application**

Chapter 9 needs sufficient controls to ensure proper application of provisions.

Section: Chapter 9 – all.

Classification: *Recommended for Basic Research.*

Discussion: This issue was raised by the Project Advisory Committee who felt that the chapter was too complex and contains too much information to be properly applied by practicing engineers with limited experience.

Resolution: Unresolved pending future research.

10. Simplified Rehabilitation

Chapter 10 outlines the Simplified Rehabilitation Method. Simplified Rehabilitation is an alternative to Systematic Rehabilitation that can be used to achieve the Life Safety Performance Level in buildings that conform to certain type, size and regularity requirements. It is based on the provisions of FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, and includes a cross-reference between the *Guidelines* and FEMA 178. It contains a section on amendments to FEMA 178, listing new potential deficiencies in building systems identified in earthquakes subsequent to the publication of FEMA 178. Chapter 10 also suggests specific corrective measures for the rehabilitation of certain deficiencies.

10.1 New Concepts

- Amendments to FEMA 178: Since the development and publication of FEMA 178, several damaging earthquakes have occurred. These earthquakes have exposed new potential deficiencies in building systems that were not addressed by the FEMA 178 methodology. The *Guidelines* contain amendments to FEMA 178 that incorporate lessons learned from these earthquakes.
- Simplified Rehabilitation: The localized correction of deficiencies is sufficient to rehabilitate simple buildings to the Life Safety Performance Level without the need for a full-scale global analysis.

10.2 Global Issues

10-1 FEMA 310 as Basis for Chapter 10

Chapter 10 is based on FEMA 178. FEMA 178 has since been fully updated with the publication of FEMA 310, *Handbook for the Seismic Evaluation of Buildings – A Prestandard*. FEMA 310 should be used as the basis for Chapter 10.

Section: All.

Classification: *Technical Revision.*

Discussion: FEMA 178, based on early 80's technology, is a force-based methodology that uses traditional building code force level analysis techniques. FEMA 310 includes issues identified in recent earthquakes, and utilizes a displacement-based analysis approach that is consistent with the methodology of the *Guidelines*.

Resolution: Chapter 10 of the Prestandard has been revised for consistency with FEMA 310.

10-2**Simplified Rehabilitation Equivalent to BSO**

If Chapter 10 is revised to reference FEMA 310, can the Simplified Rehabilitation Method be judged to satisfy the Basic Safety Objective (BSO) for buildings eligible for simplified rehabilitation?

Section: 10.1.

Classification: *Non-persuasive.*

Discussion: This issue is related to issue 3-7. Limited performance expectations for buildings passing the Chapter 10 provisions were due in part to the lateral force level used in FEMA 178. FEMA 310 utilizes a displacement-based methodology consistent with the *Guidelines*, however, there are differences between the two methods. The analysis criterion in FEMA 310 is based on a single level of earthquake shaking hazard and the BSO requires a two-level approach consisting of life safety performance for the BSE-1 earthquake hazard level, and collapse prevention performance for the BSE-2 earthquake hazard level. It may not be reasonable to assume that the BSE-1 level evaluation will always govern. There are different *m*-values in the two documents, and FEMA 310 uses a 0.75 factor for a Tier 3 detailed evaluation using the procedures in the *Guidelines*.

Resolution: No change proposed.

10-3**Chapter 10 Too Complex to be Simplified Rehabilitation**

The procedures of Chapter 10 are too complex to be considered Simplified Rehabilitation.

Section: Chapter 10 – all.

Classification: *Non-persuasive.*

Discussion: This issue was raised by the Project Advisory Committee who felt that the Chapter was too complex, particularly for buildings in regions of low seismicity. The PT considered this comment non-persuasive with the opinion that the checklist methodology and deficiency-only analysis and rehabilitation were not too complex, but only required more familiarity on the part of practicing engineers.

Resolution: No change proposed.

10-4

Reconcile Differences Between FEMA 310 and FEMA 356

Since the ASCE Standards Committee is producing both the evaluation standard and rehabilitation standard, the two documents should be consistent. In addition, FEMA 310 has been revised through the committee ballot process. Therefore, FEMA 356 should be checked and updated to reflect these changes.

Section: Chapter 10

Classification: Technical Revision

Discussion: The ASCE Standards Committee on Seismic Rehabilitation of Buildings is now responsible for producing both of the standards for seismic evaluation (FEMA 310) and seismic rehabilitation (FEMA 356). These two documents, while similar, were produced at different times in separate forums. FEMA 310 has already gone through standards committee ballot and has had numerous revisions. FEMA 356 has had many global topic studies performed, resulting in significant changes. The goal of these two documents is that they be used together. FEMA 310 would be used for the initial evaluation of buildings and FEMA 356 would be used either for advanced analysis or rehabilitation. Therefore, the two documents need to be checked for consistency against one another. Special Study 12 – FEMA 310 and FEMA 356 Differences was funded to research this issue further.

In examination of both documents, two major differences are apparent:

1. There is a difference in the seismic demands in evaluation versus design. The difference is philosophical and extends back to FEMA 178 when a 0.85 and 0.67 were applied to the static base shear. FEMA 310 was developed to maintain this consistency with FEMA 178. FEMA 356 is a rehabilitation document, so the forces remain at design level. After much discussion, it was decided that the difference would remain between the two documents since the documents are used for different purposes. However, FEMA 310 commentary would be revised to indicate that evaluation level demands would have a lower probability of achieving the desired performance level.
2. The FEMA 310 analysis methodology is less complex than FEMA 356. When FEMA 310 was developed, it was recognized that the requirements for evaluation should be less strenuous than for rehabilitation. Therefore, only the LSP was used and the terms and analysis requirements were simplified. Other requirements, such as material properties and materials testing were also relaxed. Since the FEMA 310 methodology is really a simplified subset of FEMA 356, it was decided that the difference would remain, once again acknowledging the difference between evaluation and design.

Once these two differences were recognized, the two documents were very consistent. Changes to the methodology due to FEMA 356 global topic studies, such as foundations and period formulation, would be made to FEMA 310 during public ballot. Changes to definitions and cross-references due to the FEMA 310 ballot process would be made to FEMA 356 prior to standards committee ballot.

Resolution: Modify definitions in Chapter 10 of FEMA 356 to match FEMA 310. Update cross-references in Chapter 10 of FEMA 356 to reflect changes to FEMA 310.

11. Architectural, Mechanical, and Electrical Components (Simplified and Systematic Rehabilitation)

Chapter 11 outlines the rehabilitation criteria for architectural, mechanical and electrical components, collectively referred to as nonstructural components. It defines nonstructural components and systems, describes the expected behavior, and outlines the acceptance criteria for various architectural, mechanical and electrical systems.

11.1 New Concepts

- Deformation-sensitive Components: Nonstructural components are classified as acceleration-sensitive, deformation-sensitive, or both. The *Guidelines* include specific acceptance criteria for evaluating drifts of deformation-sensitive nonstructural components.
- Designation of life safety considerations: The *Guidelines* specifically identify which nonstructural components and systems represent potential life safety concerns based on level of seismicity.
- Rehabilitation requirements for IO: The acceptance criteria include specific requirements for meeting the Immediate Occupancy Performance Level.
- Discussion of the Operational Performance Level: Prescriptive requirements for the Operational Performance Level are beyond the scope of the *Guidelines*, however, the *Guidelines* include a definition of it, and describe a procedure for developing Operational Performance criteria.

11.2 Global Issues

11-1 Preservation of Egress Not Required

Statements about preserving egress for the life safety performance level may not be necessary.

Section: 11.4.4.

Classification: *Non-persuasive.*

Discussion: Issues related to egress were specifically separated from requirements for the Life Safety Performance Level to avoid triggering unintended upgrades of emergency lighting, emergency power, disabled access, and security and fire alarm systems that are related to egress, but not directly related to seismic concerns. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive.

Resolution: No change proposed.

11-2 **Extent of Nonstructural Investigation Unclear**
The *Guidelines* are not specific as to how many occurrences of typical conditions must be checked for each different nonstructural component.

Section: 11.2.

Classification: *Technical Revision.*

Discussion: In large buildings nonstructural components, such as light fixtures, can occur hundreds of times throughout the structure. There is no discussion regarding an appropriate level of investigation for nonstructural components (i.e.: does every fixture need to be inspected?).

Resolution: Prestandard Section 11.2.2 was created to specify nonstructural sample size. The new nonstructural sampling provisions are modeled after the comprehensive condition assessment provisions for structural components.

11-3 **Vertical Acceleration Criteria Missing**
Vertical accelerations as well as horizontal accelerations are required to be considered in the rehabilitation of canopies and marquees. Sections 11.7.3 and 11.7.4 do not specify vertical acceleration criteria.

Section: 11.7.3, 11.7.4.

Classification: *Technical Revision.*

Discussion: Related to issue 2-5 regarding inaccuracies in estimating vertical accelerations using the 2/3 factor.

Resolution: Prestandard Sections 11.7.3 and 11.7.4 have been revised to include equations for vertical acceleration based on 2/3 of horizontal acceleration. In 11.7.4, vertical acceleration has been separated from the requirements for variation over the height of the building.

11-4 **Effects of Nonstructural on Structural Response**
There is insufficient guidance on how to consider the effects of nonstructural components in the structural analysis of the building.

Section: 3.2.2.3, 11.5.1.

Classification: *Recommended for Basic Research (previously unresolved).*

Discussion: Upon completion of the *Guidelines*, BSSC identified the need to further study the effects of nonstructural components on the behavior of the structure. Partial resolution should focus on providing additional commentary to highlight what guidance is provided.

Resolution: Unresolved pending future research.

- 11-5** **Sensitivity of Nonstructural to Deformation**
More information is needed regarding the sensitivity of nonstructural components to building deformations and drift.
- Section:** 11.6.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to further research the interaction between structural movements and nonstructural components, particularly glass, heavy cladding, and components and re-entrant corners.
- Resolution:** Unresolved pending future research.
-
- 11-6** **Glazing Acceptance Criteria Outdated**
The analysis and acceptance criteria for glazed exterior wall systems is not consistent with the latest research.
- Section:** 11.9.1.5.
- Classification:** *Application of Published Research.*
- Discussion:** Recent published research on this topic include the following: Behr, R.A., et al, “Seismic Performance of Architectural Glass in a Storefront Wall System”, EERI Spectra, vol. 11, no. 3, 8/95; Pantelides, C.P., et al, “Dynamic In-plane Racking Tests of Curtain Wall Glass Elements”, Earthquake Engineering and Structural Dynamics, vol. 23, 1994, among others. Changes to these provisions would be consistent with proposed changes to other documents governing glazed exterior wall systems.
- Resolution:** Prestandard Section 11.9.1.5 has been revised to incorporate new definitions of glazed exterior wall systems, and new analysis and acceptance criteria based on the referenced research.
-
- 11-7** **Acceptance Criteria Needed for Other Performance Levels**
Acceptance criteria for nonstructural components specified in Chapter 11 refer only to the Life Safety Performance Level and the Immediate Occupancy Performance Level. Other levels are not covered.
- Section:** Chapter 11, all, Table 11-1, Section 1.5.2.4.
- Classification:** *Technical Revision.*
- Discussion:** The Operational Performance Level is outside the current scope of the Prestandard. The nonstructural performance criteria for the Life Safety Performance Level was intended to be the basis for the Hazards Reduced criteria. Special Study 13 – Study of Nonstructural Provisions was funded to research this issue further.
- Resolution:** Prestandard Section 11.3.2 has been revised to state that analysis and rehabilitation requirements for the Hazards Reduced Performance Level shall follow the requirements for the Life Safety Performance Level. The definition of Hazards Reduced Nonstructural Performance has been clarified in Prestandard Section 1.5.2.4. Prestandard Table 11-1 has been revised to explicitly define the subset of nonstructural components addressed by the Hazards Reduced Performance Level.

- 11-8** **Equation 11-2 (11-3) Variation with Height**
Equation 11-2 used to calculate the seismic force on nonstructural components varies in an inverted triangular distribution over the height of the building. This distribution is not justified by recorded data or dynamic analysis results.
- Section:** 11.7.4, Equation 11-2 (new equation 11-3).
- Classification:** *Application of Published Research and Basic Research.*
- Discussion:** The equation in the *Guidelines* is consistent with the 1997 NEHRP Provisions and the 1997 UBC. This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Unresolved pending further study of available information and future research.
-
- 11-9** **Heavy Partitions—Scope and Definition**
In zones of low seismicity, the *Guidelines* should require heavy partitions to be reviewed for adequacy. In Section 11.9.2.1 heavy is defined as greater than 5 psf, which means metal stud and gypsum board partitions would fall under this classification.
- Section:** 11.9.2.1, Table 11-1.
- Classification:** *Technical Revision.*
- Discussion:** Review of heavy partitions in regions of low seismicity was considered by the Prestandard PT and found non-persuasive. The evaluation procedure in the *Guidelines* was judged appropriate, although the 5 psf limitation is not consistent with what was intended to be heavy (masonry partitions).
- Resolution:** Prestandard Section 11.9.2.1 was revised to omit the 5 psf criteria for heavy partitions. Table 11-1 remains unchanged with regard to evaluation of heavy partitions.
-
- 11-10** **Guidance on Nonstructural Operational Performance Needed**
Guidance is needed on establishing nonstructural Operational Performance acceptance criteria.
- Section:** 11.3.2
- Classification:** *Application of Published Research.*
- Discussion:** Related to issue 11-7 regarding acceptance criteria for other performance levels. Nonstructural Operational Performance is outside the current scope of the Prestandard. This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Unresolved pending further study of available information.

- 11-11** **Nonstructural IO and LS Criteria need calibration**
The distinction between nonstructural IO and LS performance criteria needs investigation. Design forces for each performance level need to be calibrated between the two methods.
- Section:** 11.7.3, 11.7.4, 11.9
- Classification:** *Recommended for Basic Research.*
- Discussion:** Throughout Section 11.9, references to Sections 11.7.3 and 11.7.4 are made for seismic design force criteria. For LS, either section is permissible, but for IO only 11.7.4 is used. The equations in 11.7.3 are conservative empirical equations that are always greater than those in 11.7.4. This results in LS force levels that can be more stringent than IO force levels, depending on the method chosen. This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Unresolved pending future research.
-
- 11-12** **Storage Racks as Non-Building Structures**
Storage racks should be treated differently than other nonstructural components because they behave more like a multi-story building than a rigid block. Provisions should be developed to address non-building type structures.
- Section:** 11.7.3, 11.7.4, 11.11.1.3
- Classification:** *Application of Published Research.*
- Discussion:** This issue was raised by the SC in response to the unofficial letter ballot of the Prestandard.
- Resolution:** Unresolved pending further review of available information.
-
- 11-13** **Floating Concrete Isolation Floors are not Addressed**
Isolation floors consisting of concrete slabs “floating” above the structural slab on a layer of isolation material are not addressed by the *Guidelines*.
- Section:** 11.9
- Classification:** *Recommended for Basic Research*
- Discussion:** This type of isolation floor system has been used on occasion in the past and is gaining popularity. To maintain the integrity of the noise or vibration barrier, the concrete slab is not anchored to the structural system, but should be restrained by a system of curbs or keys. Direction on how to address these systems is needed in the Prestandard.
- Resolution:** Unresolved pending future research.

Appendices

A. Miscellaneous Issues

This section addresses miscellaneous issues that are not directly related to any one chapter of the FEMA 273 *Guidelines*.

A.1 Global Issues

A-1 Reference to Other Standards Incomplete

References to other standards (e.g. ACI 318) throughout the *Guidelines* are not sufficient to determine how to apply them properly.

Section: All.

Classification: *Technical Revision.*

Discussion: None.

Resolution: Specific occurrences have been identified in the development of the Prestandard and additional direction has been provided on a case-by-case basis.

A-2 Quality Assurance Not Specified

The *Guidelines* are generally silent on design quality assurance provisions related to computer codes, engineer qualifications, peer reviews, and plan checking.

Section: All.

Classification: *Non-persuasive.*

Discussion: The omission of specific guidance on design quality assurance is inconsistent with the requirements for materials testing and construction inspection. At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive.

Resolution: No changes proposed.

A-3 Permissive Language Not Standard Compatible

Permissive language present in the *Guidelines* is not compatible with the provisions of a standard. Consider the use of the term “authority having jurisdiction” (AHJ) in the document to allow permissive requirements to be tightened as decided by local jurisdictions.

Section: All.

Classification: *Editorial Revision.*

Discussion: The purpose of the prestandard effort is to convert the verbiage of the *Guidelines* to standards language. Permissive requirements have been tightened where possible and where appropriate. It is implied in every code or standard that the authority having jurisdiction has the authority to specify criteria or approve alternative rational analysis procedures. It is not necessary to add this phrase throughout the standard.

Resolution: In the Prestandard permissive requirements have been converted to standards language. Where it is appropriate for leeway to remain in the provisions, the term “or approved” has been used. In Chapter 1, implications that the building owner has the authority to enforce the provisions of this standard have been removed.

- A-4** **Triggers for Seismic Rehabilitation Missing**
Should enabling statements and triggers for seismic rehabilitation be added?
- Section:** All.
- Classification:** *Non-persuasive.*
- Discussion:** At the 3/3/99 Standards Committee meeting this issue was reclassified as non-persuasive. The decision regarding triggers for mandatory rehabilitation is a policy decision intentionally left to the local authority having jurisdiction.
- Resolution:** No changes proposed.
-
- A-5** **Drift Limits Omitted**
Drift limits and acceptance criteria based on calculation of interstory drift are not included in the document.
- Section:** All.
- Classification:** *Non-persuasive.*
- Discussion:** A displacement base analysis procedure eliminates the need for drift limits. The analysis methodology evaluates the acceptability of elements in their displaced state at maximum expected displacements. Since displacements and their effects are explicitly calculated, drift limits are not relevant.
- Resolution:** No change proposed.
-
- A-6** **Behavior of Rehabilitated Elements**
More information is needed regarding the behavior of rehabilitated elements and components.
- Section:** Chapters 5, 6, 7 and 8.
- Classification:** *Recommended for Basic Research (previously unresolved).*
- Discussion:** Upon completion of the *Guidelines*, BSSC identified the need to conduct additional research on the behavior of rehabilitated elements.
- Resolution:** Unresolved pending future research.

A-7**Expected and Lower Bound Strengths Unclear**

The concepts of expected strength and lower bound strength are not clearly defined or used consistently throughout the document.

Section: Section 2.9.4 (new section 2.4.4), Chapters 5, 6, 7, and 8.

Classification: *Technical Revision.*

Discussion: This issue is related to issues 5-2 and 8-6. It is not clear what material properties should be used in the calculation of expected strength and lower bound strength. It is also not clear if default properties provided in the document are expected or lower bound properties, or if specified material properties are considered expected or lower bound. The correct use of strength reduction (ϕ) factors is not clearly stated.

Resolution: Prestandard Section 2.4.4 has been revised to clearly introduce the concept of expected and lower bound strengths and material properties. Expected material properties have been defined as mean values of tested properties. Lower bound material properties have been defined as mean minus one standard deviation of tested material properties. All relevant sections have been revised to state that $\phi = 1.0$ in all cases when strength reduction factors are used in the calculation of expected or lower bound strengths. All references to default values have been made consistent with lower bound material properties, with the exception of Chapter 8. Default wood material properties are considered expected material properties. All references to expected and lower bound strengths in Chapters 5, 6, 7 and 8 have been revised to be consistent with this revision.

A-8**Paragraphs Contain Multiple Provisions**

Many paragraphs throughout the *Guidelines* contain multiple provisions and several important concepts lumped together. Lists throughout the *Guidelines* have bullet points that are not numbered. In codes and standards, major concepts and mandatory provisions are usually separated and numbered individually.

Section: All.

Classification: *Editorial Revision.*

Discussion: This issue was raised at the 3/3/99 Standards Committee meeting. Separation and numbering of major concepts and mandatory provisions will make it easier to locate or cross-reference between requirements.

Resolution: Long paragraphs with multiple provisions in the Prestandard have been split and numbered individually to the extent possible. Sections with letter designations have been revised to numeric designations only. Bulleted lists in the Prestandard have been numbered sequentially.

A-9**Rehabilitation Measures as Commentary**

Sections describing specific rehabilitation measures for various structural systems should not be mandatory. Engineers should be free to determine an appropriate rehabilitation measure that meets the acceptance criteria.

Section: All.

Classification: *Editorial Revision.*

Discussion: This issue was raised at the 3/3/99 Standards Committee meeting. Inclusion of rehabilitation measures in the standard implies they are mandatory and limits options for rehabilitating buildings.

Resolution: Prestandard Section 2.5, Rehabilitation Strategies, has been left in the standard. This section describes the overall general approach to rehabilitation. All other sections that describe specific rehabilitation measures in Chapters 5 through 8 of the Prestandard have been shifted to commentary.

A-10**Standard/Commentary Split**

The First SC Draft of the Prestandard contains text that is not mandatory itself, or necessary to the mandatory requirements of the document. The split between standard and commentary needs to be improved to reduce the text of the standard to the mandatory requirements alone.

Section: All.

Classification: *Editorial Revision.*

Discussion: This issue was raised at the 3/3/99 Standards Committee meeting.

Resolution: The split between standard and commentary in the Prestandard has been reviewed in each subsequent draft since the First SC Draft. Non-mandatory verbiage has been removed from the Prestandard to the extent possible.

A-11**No Acceptance Criteria for Secondary IO**

The *Guidelines* have no acceptance criteria for secondary components at the IO performance level.

Section: All.

Classification: *Editorial Revision.*

Discussion: Because the Immediate Occupancy Performance Level is related to damage control, the intent of the *Guidelines* is that acceptability for IO performance is not related to primary or secondary element classifications. Components damaged to the extent they are performing at the secondary limits of response do not meet the intent of IO performance. This means that components which might otherwise be classified as secondary for other performance levels, may end up controlling a design for the IO performance level.

Resolution: Tables of acceptance criteria in the Prestandard have been revised to remove IO from under the heading of “Component Type” to clarify that IO criteria is independent of primary or secondary classifications.

A-12**Acceptance Criteria for Archaic Materials Needed**

Some archaic materials such as hollow clay tile and plain concrete do not have explicit acceptance criteria or modeling information in the *Guidelines*. A procedure should be developed, other than testing, to estimate this information when engineering data is available.

- Section:** All, 2.13 (new section 2.8).
- Classification:** *Recommended for Basic Research.*
- Discussion:** None.
- Resolution:** Unresolved pending future research.

B. Research and Study Needs

To facilitate future improvements to the *Prestandard*, this section summarizes issues that are currently unresolved and recommended for basic research. Issues are listed in numerical order.

- 2-1 Overturning Appears Overly Conservative**
Overturning calculations at pseudo lateral force levels appear to be overly conservative and can predict overturning stability problems that are not well correlated with observed behavior.
- 2-2 Ground Motion Pulses Not Covered**
Ground motion duration and pulses are not explicitly considered in the analysis procedures except for the use of higher acceleration values specified in regions near active faults.
- 2-6 Baseline Adjustments to Acceptance Criteria Needed**
Use of experimental data to set acceptance criteria has led to some inconsistency in calculated versus expected results. It may be appropriate to consider some baseline adjustments to acceptance parameters.
- 2-7 Software Not Commercially Available**
Nonlinear software capable of performing 3-D nonlinear analyses is not commercially available to the building engineering community. Any building that requires this analysis based on *Guidelines* provisions cannot be rehabilitated to meet the provisions.
- 2-10 No Public Input or Consensus on Acceptable Risk**
The present definitions of performance levels and acceptable risk have been developed by engineers with little input from the public, and may not be consistent with popular notions.
- 2-19 Upper Limit on DCRs for LSP Needed**
There should be an upper limit on DCR values that should not be exceeded if linear procedures are to be applicable, regardless of the presence or absence of structural irregularities.
- 2-23 R_{OT} Needed for IO Performance**
An overturning force reduction factor, R_{OT} , for IO performance is needed to complete the alternative procedure for evaluating overturning stability.
- 2-24 LS Performance Level Should be Clarified or Eliminated**
The Life Safety Performance Level should be more clearly defined in terms of structural performance, or it should be eliminated as a performance goal.

- 2-25 The 2/3 Factor Estimating Vertical Seismic Forces is Not Accurate**
The 2/3 factor used to estimate the relationship between vertical response spectra and horizontal response spectra is not accurate.
- 2-26 Additional Guidance on Damping Needed**
There is more variation in damping of actual buildings than addressed in the document. Additional guidance on damping values is needed.
- 2-28 Equation for Building Separation is Overconservative**
Equation (2-16) for required building separation based on SRSS combination of building displacements is overconservative.
- 3-1 $C_T=0.06$ for Wood Buildings Not Documented**
The accuracy of $C_T=0.06$ for use in the period calculation for small wood buildings is not documented.
- 3-4 Multidirectional Effects Need Clarification**
Further direction on consideration of multidirectional effects, including vertical seismic forces, is required.
- 3-6 NSP Uniform Load Pattern Overly Conservative**
The shape of the loading pattern used in NSP significantly affects the results. Specifying a uniform load pattern appears to be overly conservative and can dominate the resulting behavior.
- 3-10 Upper Limit on Pseudo Lateral Force**
The LSP forces appear to be too high. FEMA 273 does not contain an upper bound limit on maximum base shear similar to the 0.75W limit in FEMA 310.
- 3-13 LSP and NSP Results Need Calibration**
The Linear Static Procedure is not always more conservative than Nonlinear Static Procedure.
- 3-14 Reliability Information Not Provided**
No specific information on reliability is provided in the *Guidelines*.
- 3-15 LSP Should be a Displacement Calculation**
The Linear Static Procedure should be changed to a displacement-based calculation procedure.
- 3-17 C_1 Factor Overly Conservative**
Introduction of the C_1 factor overly penalizes buildings with short calculated fundamental periods.

- 3-18** **Duration Effects Not Considered**
The analytical procedures of the *Guidelines* do not consider duration effects to take into account cyclic degradation.
- 3-19** **Marginal Gravity Load Capacity Not Considered**
Further study of LSP acceptance criteria is required for building components with marginal gravity load capacity.
- 3-20** **Inelastic Cyclic Properties Needed**
More information is needed to develop inelastic cyclic component properties for use in complex nonlinear dynamic analyses.
- 3-23** **Substantiation of C1, C2, C3 Needed**
Further research is needed to substantiate the coefficients C_1 , C_2 , and C_3 .
- 3-30** **Application of η -factor is Overconservative**
Amplifying forces and displacements by the η -factor to account for torsion is overconservative for lateral force resisting elements located near the center of rigidity.
- 3-34** **Alternate Empirical Period Calculation for Flexible Diaphragms**
An alternate empirical equation can be developed for single span flexible diaphragms consisting of $T=C_{td} (L)^{1/2}$, where L is the span length and C_{td} is a materials based coefficient.
- 3-36** **Application of the NSP With Non-Rigid Diaphragms Needs Revision**
Further guidance is required on the proper application of the NSP in buildings with non-rigid diaphragms.
- 3-38** **Procedures for Torsional Amplification are Unconservative**
Procedures for torsional amplification do not account for torsional degradation and are unconservative in determining increased forces and displacements for this effect.
- 4-3** **Lateral Soil Spring Procedure Needs Refinement**
The procedure for developing lateral soil spring stiffness based on displacement results in unrealistically high calculated lateral soil pressures. More information is needed on the force-displacement behavior of geotechnical materials and foundations under short term loading.
- 4-4** **Nonlinear Soil Spring Information Needed**
More information is needed on nonlinear force-displacement behavior of foundation systems for inclusion in nonlinear analyses.

- 5-1** ***m*-factors Appear Overly Conservative**
Certain values of acceptance criteria (*m*-factors) and deformation limits for steel components appear to be too conservative.
- 5-14** **Steel Acceptance Criteria is Based on Component Length**
Nonlinear acceptance criteria for certain steel components are expressed as a multiple of yield rotation, which is based on the length of the component.
- 5-15** **The Ratio Between IO and LS Acceptance Criteria Appears Too Large**
The ratio between IO and LS acceptance criteria for certain steel components appears to be too large. IO values for these components appear to be too low.
- 5-16** **Nonlinearity is Permitted in Column Base Plates**
For certain controlling actions, nonlinearity is permitted in column base plates. Column bases should be treated as force-controlled.
- 6-1** ***m*-factors Appear Overly Conservative**
Certain values of acceptance criteria (*m*-factors) and deformation limits for concrete components appear to be too conservative and are not consistent with other chapters. Of particular concern is an inconsistency with Chapter 7, Masonry.
- 6-17** **Acceptability for Columns in Tension Missing**
Acceptability requirements for concrete columns in tension are not provided.
- 6-18** **Calculation of M_y for Shearwalls Unconservative**
The procedure in Section 6.8.2.3 for calculating the yield moment of reinforced concrete wall sections may underestimate the actual flexural capacity. This result would be unconservative for use in a limit state analysis.
- 6-20** **Concrete Flange Provisions Unconservative**
Provisions for flanged sections in Section 6.4.1.3 may underestimate the frame action of the system when applied to joist construction.
- 7-1** ***m*-factors Appear Overly Conservative**
Certain values of acceptance criteria (*m*-factors) and deformation limits for masonry components appear to be too conservative and are not consistent with other chapters. Of particular concern is an inconsistency with Chapter 6, Concrete.
- 7-4** **Guidance for Infill Panels with Openings Needed**
Evaluation of masonry infills does not provide adequate guidance for addressing masonry infill panels with openings.

- 7-10 Masonry Shear Strength Based on Average Test Values is Unconservative**
The calculation of expected masonry shear strength using average values of brick shear tests overestimates the actual shear strength.
- 7-11 URM Shear Strength Should be Force-Controlled**
Shear strength of URM walls is brittle and unreliable and should be treated as a force-controlled action.
- 8-1 *m*-factors Appear Overly Conservative**
Certain values of acceptance criteria (*m*-factors) and deformation limits for wood components appear to be too conservative.
- 9-1 Procedures Require Validation**
Analytical procedures for energy dissipation systems require validation.
- 9-4 Chapter 9 Needs Controls for Proper Application**
Chapter 9 needs sufficient controls to ensure proper application of provisions.
- 11-4 Effects of Nonstructural on Structural Response**
There is insufficient guidance on how to consider the effects of nonstructural components in the structural analysis of the building.
- 11-5 Sensitivity of Nonstructural to Deformation**
More information is needed regarding the sensitivity of nonstructural components to building deformations and drift.
- 11-8 Equation 11-2 (11-3) Variation with Height**
Equation 11-2 used to calculate the seismic force on nonstructural components varies in an inverted triangular distribution over the height of the building. This distribution is not justified by recorded data or dynamic analysis results.
- 11-10 Guidance on Nonstructural Operational Performance Needed**
Guidance is needed on establishing nonstructural Operational Performance acceptance criteria.
- 11-11 Nonstructural IO and LS Criteria need calibration**
The distinction between nonstructural IO and LS performance criteria needs investigation. Design forces for each performance level need to be calibrated between the two methods.
- 11-12 Storage Racks as Non-Building Structures**
Storage racks should be treated differently than other nonstructural components because they behave more like a multi-story building than a rigid block. Provisions should be developed to address non-building type structures.

11-13

Floating Concrete Isolation Floors are not Addressed

Isolation floors consisting of concrete slabs “floating” above the structural slab on a layer of isolation material are not addressed by the *Guidelines*.

A-6

Behavior of Rehabilitated Elements

More information is needed regarding the behavior of rehabilitated elements and components.

A-12

Acceptance Criteria for Archaic Materials Needed

Some archaic materials such as hollow clay tile and plain concrete do not have explicit acceptance criteria or modeling information in the *Guidelines*. A procedure should be developed, other than testing, to estimate this information when engineering data is available.

C. Special Study 1— Early Input from the BSSC Case Studies Report

ASCE/FEMA 273 Prestandard Project Early Input from the BSSC Case Studies Project

William T. Holmes

October 12, 1999

Purpose

The purpose of this study was to monitor progress of the BSSC Case Studies Project and review early drafts of the Case Studies Project Report to enable inclusion of significant findings into the ASCE/FEMA 273 Prestandard.

Summary of Findings

Five existing Global Topics were classified as Case Study Consensus Revision—that is, they possibly could be resolved by the Case Study Project. We found that none of these were resolved by the case studies.

Twenty-six of the major issues documented in the Case Studies Report were already contained in the Global Topics Report.

Twenty-seven new Global Topics were raised by the report. Of these, it is judged herein that sixteen should be classified as *Recommended for Future Research*, or will require further study and analysis for resolution.

Eleven new Global Topics resulted in development of proposed changes in the Prestandard. These are listed in Attachment 2.

Procedure

The Case Studies Project Report (Final Draft-6/30/99) was reviewed. The lists of recommendations contained in tables for Usability Comments (“U” items) and Technical Issues (“T” items) were cross-checked with the Global Topics Report (April 12, 1999). A.T. Merovich assisted in interpreting the Case Studies Report and in recommending changes to the Prestandard.

The U and T-items were categorized as 1) Non-persuasive, 2) already contained in the Global Topics Report, 3) New Global Topic that needs further study or research for resolution, or 4) New Global Topic for which a clarification or change can be recommended. The cross references between the U and T items and the Global Topics, as well as the categorizations are contained in tables in Attachment 1.

The new Global Topics for which changes can be formulated, as well as action that the Project Team has taken on them (when applicable) are listed in Attachment 2.

A summary of the results of this review is given below:

- 5 Global Topics classified *Case Study Consensus Revision*
 - none resolved
- 42 Usability Issues, 25 Technical Issues
 - number of issues studied 67
- Number found non-persuasive: 14
- Number already covered by Global Topics 26
- Number of new Global Topics: 27
 - Future study or research 13
 - Might be resolved or clarified with focused study 3
 - T12 (C2 counterintuitive)
 - T18 (multiple comments on chapter 6)
 - T23 (multiple comments on chapter 11)
 - Clarifications proposed by this study 5
 - U3 (default site class E to D)
 - U9 (clarification of roof loads)
 - U15 (new concrete elements)
 - U18 (L/h_{eff} limits in certain circumstances)
 - U36 (reference to regularity re Table 10-1)
 - Technical Revisions identified by this study 6
 - U7, U37 (Definition and use of DCRs)
 - U17 (definition of h_{eff})
 - U22 (use of Cs and J in Chapter 9)
 - U28 (heavy partitions in low seismic zones)
 - U34 (Change BSO to single level—CP @MCE)
 - Ground motion (BSE use of 2 maps; MCE use of 2 maps; conflict with FEMA 310)

ATTACHMENT 1

**Recommendations for Change or Clarification to FEMA 273
from the Case Studies Report (6/30/99 Draft)
and
Cross Reference to Global Topics Report (April 12, 1999)
with
Classifications for Action for the ASCE Prestandard**

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U1.All formulae in the <i>Commentary</i> that are required to be used for meeting a provision in the <i>Guidelines</i> should be relocated into the <i>Guidelines</i> . All associated parameters should be defined.	A-10	In Global Topics Report
U2.A more precise procedure for relating site location to mapped hazard parameters must be developed and integrated into the <i>Guidelines</i> .	N/A	Not in scope of ASCE/FEMA 273 Prestandard project
U3.The default site class should be revised from Class E to Class D.	2-22	New GT (Technical Revision)
U4.Section 2.6 and 1.3.3 should be rewritten to unambiguously define BSE-1, BSE-2, 10%/50 year, 2%/50 year hazards and their relationships for use in the <i>Guidelines</i> and to the map set. There appears to be no practical value for separate MCE and 2%/50 maps. They should be combined to prevent misapplication. Note also that 10%/50 maps are not available for Alaska. This should be addressed	2-3	In Global Topics Report
U5.The definitions of seismicity and the site class coefficients must be the same in FEMA 310 and FEMA 273. The term "seismicity" should be replaced with the word "shaking" when site effects have been included in the characterization. Seismic zones are now shaking zones.	3-7	The two are different by the site factor F. In Global Topics Report
U6.The current requirements to achieve a kappa of 1.0 require more expense than the Case Study engineering firms believe is necessary given the inherent uncertainty in the calculation procedures. Alternative variations should be evaluated that include finer gradations between the values of 0.75 and 1.0. Additionally, it is recommended that a study be undertaken to establish the appropriateness of expanding the range of values permissible for this coefficient and to provide a rationally derived basis that reflects performance reliability.	5-4 6-3	In Global Topics Report
U7.All provisions relating to the use of DCRs should be located in one section. The definition of DCRs should be revised to be consistent with the parameters used for checking component acceptability (force-controlled) to eliminate an additional round of calculations.	6-5 (related to T10, U37)	New GT (Technical Revision)

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U8. The definitions of force-controlled and deformation-controlled component actions require more robust development for unambiguous application. The <i>Guidelines</i> concept of defining actions in this manner is a significant technical advancement for which application must be made clear.	3-11	In Global Topics Report
U9. Clarification regarding the inclusion of roof loads and the definition of measured loads is necessary.	N/A	Editorial clarification part of Prestandard process
U10. The procedures that are used to define K_e (section 3.3.3.2D) require a determination of V_y . For many real structures, a clearly defined yield plateau does not exist. Engineers have requested more guidance and rules for establishing V_y so as to more uniformly establish the K_e parameter. Expanded discussion on this subject with representative examples would greatly enhance usability.	3-25	New GT (Technical Revision)
U11. Nonlinear software capable of performing 3-D <i>Guidelines</i> conforming analysis is not commercially available to the building engineering community. Any building that requires this analysis according to the <i>Guidelines</i> cannot be rehabilitated to meet the provisions. An alternative strategy for these buildings must be developed.	2-7 (related to U42 and T9)	New GT (future study or research)
U12. The J factor is used to reduce the demand for reviewing the sufficiency of force-controlled component actions. It is intended to reflect the force limitations imposed by the yielding of deformation-controlled components along the load path. Case Study firms expressed concern that use of an equation which included ground acceleration does not seem rational. It is recommended that an alternative equation be developed that more rationally reflects the basis for this parameter and that further guidance is provided explaining how to calculate this parameter.	N/A	Non-persuasive

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U13. The procedure for evaluating components such as columns for multiple actions (such as axial and flexural) to determine force or deformation controlled behavior and acceptability criteria needs elaboration and clarification. When numerous actions are potentially the controlling actions, engineers need more detailed guidance in establishing how to classify a component to establish its acceptability.	6-13	In Global Topics Report
U14. Chapter 5 is difficult to use because it does not include a broad enough range of component/element types, section shapes, steels and irons. The interrelationship with AISC is not developed in sufficient detail to prevent confusion. "m" values of Section 5.8 should be consolidated and presented in tabular form. It is recommended that this chapter be rewritten with the above improvements.	5-5, 5-10 (related to T6)	In Global Topics Report
U15. When replacement of a concrete element is required (Section 6.3.5), the <i>Guidelines</i> generally require the element be designed to meet the requirements for new buildings. This is problematic in that design for new buildings will require a complete re-analysis of the building to establish demand. The <i>Guidelines</i> should require that the design of new elements is deemed sufficient if these components are shown to meet the requirements of the <i>Guidelines</i> .	N/A	Make it clear that new code requirements are detailing. Editorial clarification part of Prestandard process)
U16. Inconsistencies to the reference standards for design and expected strength in the masonry chapter should be eliminated.	A-7	In Global Topics Report
U17. The Chapter 7 definitions for the parameters h_{eff} and Δ_{eff} require clarification. A graphical depiction of these parameters would be helpful but further explanation is necessary.	7-9	New GT (editorial revision)
U18. Equations 7-5 and 7-6 do not provide guidance to users on L/h_{eff} limits outside the applicable bounds noted for these equations. Guidance on this subject is necessary.	7-8	New GT (editorial)
U19. Equations 7-9 and 7-10 must be clarified to indicate how users are to determine strength if M/Vdv is greater than 0.25 and less than one. Is the correct parameter in these equations f_m or f_{me} ?	7-3	In Global Topics Report. Interpolate values between limits

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U20. Clarification is necessary regarding the procedure used to determine if a masonry wall is controlled by shear (force) or flexure (deformation). Should a demand/capacity comparison be made or just a capacity check?	3-11	In Global Topics Report
U21. Guidance needs to be provided to users as to how to treat discontinuous posts and beams under wood shear walls. The wood section does not define a procedure for determining lower bound strengths to be used in determining requirements for force-controlled components. Guidance on this subject is necessary.	8-8	New GT (Technical Revision)
U22. Chapter 9 should address use of the C_1 , C_2 and C_3 coefficients.	9-3 (related to T22)	New GT (Technical Revision)
U23. FEMA 310 and 273 do not provide adequate guidance on correcting out-of-plane wall deficiencies using strongbacks. Chapter 10 defines system performance criteria but does not reference equations to determine demand. Section 10.3.3.3E should be amended to include this information.	N/A	Non-persuasive. In Guidelines 2.11.7
U24. Structural irregularity as defined by FEMA 302 should be consistent with the <i>Guidelines</i> if they are to be cross-referenced as standards. At present, FEMA 310 is less severe than FEMA 302 regarding the definition of structural irregularities. If this is intentional, reference to FEMA 302 should be deleted and supportive discussion provided in the <i>Commentary</i> .	3-9	In Global Topics Report

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
<p>U25. Confusion exists in the application of tier one, FEMA 310 checklists. Questions are asked that require tier two numerical calculations to be performed. FEMA 310 requires clarification on this subject and a fundamental statement that tier one evaluations may require a significant level of tier two calculation for various items. Engineers are being misled into expecting that a tier one analysis is a rapid series of yes/no questions to be answered and are frustrated to find that they must calculate the lateral force capacity of every vertical component on every floor to determine if a weak story exists. Engineers should be advised that a tier one evaluation may require substantial engineering effort for some building types. Such a statement would significantly improve usability by alerting engineers to the potential level of effort to complete a tier one scope of evaluation.</p>	N/A	FEMA 310 not in scope of ASCE/FEMA 273 Prestandard project
<p>U26. FEMA 310, tier one does not require a minimum strength for diaphragm to wall connections or lath and plaster attachments. The acceptance requirements for these items is ambiguous and needs to be clarified.</p>	N/A	FEMA 310 not in scope of ASCE/FEMA 273 Prestandard project
<p>U27. FEMA 310 does not address hollow clay tile or ungrouted/partially grouted block walls as written. This should be corrected. These are very common building materials.</p>	N/A	FEMA 310 not in scope of ASCE/FEMA 273 Prestandard project
<p>U28. In zones of low seismicity the <i>Guidelines</i> do not require heavy partitions to be reviewed for adequacy. Section 11.4.4 describes items of concern for maintaining building egress to meet a Life Safety performance level. This discussion includes heavy partitions. Further discussion should be added to this section noting that in zones of low seismicity the risk of heavy partitions blocking egress is sufficiently low to be ignored.</p>	11-9 (related to T23a)	New GT (Technical Revision)
<p>U29. Remove explanatory text from the <i>Guidelines</i> and provide equations, definitions and provisions without a discussion of intent. Transfer necessary explanatory material to the <i>Commentary</i>.</p>	A-10	In Global Topics Report

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U30. Reorganize, consolidate and cross reference design requirements to eliminate "loose end" provisions that are isolated from similar requirements. This is a common problem among codes that familiarity improves over time, however the users have indicated that an improvement would significantly improve usability.	1-1, 3-24	In Global Topics Report
U31. Renumber figures, formula and tables to correspond to the related section number where the provision requiring application is located. This will make it easier to keep linkages among requirements. Locate figures, tables and definitions at the end of the chapter to make them easier to find.	N/A	Non-persuasive
U32. Alternative methods to that illustrated in Figure C7-3 for modeling perforated infills should be developed to simplify application. Consideration should be given to use of a single strut with reduced properties.	7-4	Combine with 7-4 (future study or research)
U33. The concept of primary and secondary components requires further clarification.	Related to 3-11	Combine with 3-11
U34. The BSO requires analytical reviews for both Life Safety at BSE-1 and Collapse Prevention at BSE-2. The Case Studies indicate that the BSE-2 and Collapse Prevention generally govern design requirements. Eliminate the Life Safety review for BSE-1 to reduce the computational burden and improve usability. This will also eliminate the possibility of requiring engineers to use nonlinear procedures for BSE-2 while having used linear procedures for BSE-1.	2-5 (related to T3)	Combine with T3 for incorporating CP @ MCE and single level. In Global Topics Report
U35. Review and incorporate the various minor editorial corrections in Appendix 10.2.2 labeled [2] and [3].	N/A	Editorial clarification. Part of Prestandard process
U36. Section 2.8.1 should delete the reference to Table 10-1 that suggests regularity is a feature of the table.	N/A	Editorial clarification. Part of Prestandard process
U37. Clarify inconsistent definitions of weak story given in Sections 2.9.1.1 and 6.5.2.4A.	6-5 (related to U7)	New GT. Conflict exists

Case Study Issue— Usability Comment	Corresponding Global Topic	Action
U38. Reference to the requirement to increase all numerical values by 1.25 for Immediate Occupancy in Section 2.11 should be removed and a pair of values provided at all affected locations to prevent omissions.	N/A	Clarify use of 1.25 factor. Editorial clarification part of Prestandard process
U39. Insufficient guidance provided in Chapter 7 for use of the cracked and uncracked stiffness and force-deformation characteristics of reinforced masonry wall systems. Guidance for establishing fraction of gross section stiffness (shear and flexure) not provided in <i>Guidelines</i> (see <i>Commentary</i>). It is recommended that the <i>Guidelines</i> be expanded to include this information.	N/A	Editorial clarification part of Prestandard process
U40. The <i>Guidelines</i> ' requirements for nonlinear analysis using both uniform and triangular load patterns should be relaxed to reduce the computational burden of the NSP. Procedures should be specified that identify which patterns are most appropriate for analysis on certain building configurations.	3-6	In Global Topics Report
U41. Tilt-up buildings are very common and force-controlled requirements should be footnoted in Table 6-20. (See C6.9.1.3)	N/A	Editorial clarification part of Prestandard process (see C6.9.1.3)
U42. The generalized shape of the component force-deformation behavior is a simplification that does not seem computationally practical. The instantaneous drop in strength from point C and D and from point E to the abscissa have presented difficulties in nonlinear software application. Given the failure of currently available software to incorporate this characterization of nonlinear behavior, it is recommended that a study be undertaken to investigate alternative formulations and programming limitations so that production software can be expediently developed.	2-12	In Global Topics Report

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<p>T1. The treatment of overturning in the Linear Procedures produces results that are much more severe than observations of past building performance imply are necessary. The <i>Guidelines</i> provide a sidebar that can be used to adjust overturning demands to levels consistent with that of new construction designed by current code procedures. At a minimum, the sidebar should be modified to include a reduction in earthquake demand consistent with the removal of coefficients C_1, C_2 and C_3. This modification should generally produce overturning demands consistent with current codes for new construction. This modification, however, does not address the resulting inconsistency in demand forces above the foundation interface and those reduced forces below it. It is therefore recommended that the sidebar be further clarified to require that all components of the superstructure have adequate capacity to mobilize the dead loads assumed effective in the overturning calculation. These modifications will improve application of the Linear Procedure for overturning effects, however, for many buildings (braced frame, shear wall) these improvements may not be sufficient to reduce the requirements for overturning to levels consistent with past observations of building performance and engineering judgment. It is therefore recommended that further study to develop a more comprehensive solution to this dilemma be undertaken and <i>Guidelines</i> users be advised that for certain building types use of the nonlinear procedures could significantly reduce the scope of foundation rehabilitation work predicted by the Linear Procedures.</p>	2-1	In Global Topics Report

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<p>T2. The <i>Guidelines</i> presently do not permit any component to exceed its acceptance criteria under any circumstance. Case Study engineering firms and the DAP have expressed the concern that for some buildings this may be too extreme a requirement. Comparative studies of internal consistency have shown that some buildings cannot achieve the drift limits descriptive of the target damage state (performance level) without component actions exceeding their <i>Guidelines</i> limits. Rather than generally increasing component acceptance limits (which does not appear justified on the basis of Case Study findings alone), it is recommended that procedures be developed that permit a relaxation of component acceptance criteria when the global performance of the structure can be shown to be capable of accommodating this more severe component damage state. For the nonlinear procedures, this might be done by assessing story strength degradation. For the Linear Procedures, it might be done by relaxing or eliminating acceptance criteria for non-load bearing components, horizontal components or displacement-controlled vertical load bearing components. A comprehensive study of this issue is strongly urged as it can have significant cost implications and serve to tie a much tighter bond between global and component performance than presently exists in the <i>Guidelines</i>.</p>	<p>3-27, 3-28</p>	<p>In Global Topics Report</p>

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<p>T3. The <i>Guidelines</i> put forth the BSO as the suggested rehabilitation goal. The BSO requires a demonstration of sufficiency for Collapse Prevention performance under the action of BSE-2. For many parts of central and eastern United States, this requirement will necessitate costly rehabilitations. Consideration should be given to the economic consequences of meeting this requirement in areas of the country where rehabilitation is rare at present. Study of this issue and the importance of selecting performance objectives to reflect local economic risk/reward considerations should be undertaken as part of the development of the <i>Guidelines</i> into a national building code. Consideration should also be given to a potential recalibration of lower bound component capacities to acknowledge the probability of occurrence of a very rare event.</p>	<p>2-5 (related to U34)</p>	<p>Combine with U34 for incorporating CP @ MCE and single level. In Global Topics Report</p>
<p>T4. The acceptability criteria for secondary components that consist of non-vertical load bearing elements and flexurally-controlled columns could be relaxed. Additional research and study should be done to focus on the level of damage and deformation components can sustain when they lose their ability to support gravity loads. This research is necessary to permit the <i>Guidelines</i> procedures to be used to the fullest measure of their technical development and to boost their cost effectiveness.</p>	<p>6-10 (related to T5)</p>	<p>In Global Topics Report</p>
<p>T5. All <i>m</i> values should be revised so they are not less than the product of $C_1C_2C_3J$ to eliminate the possibility of creating non-ductile structural mechanisms instead of ductile or semi-ductile ones.</p>	<p>5-1, 5-9, 6-1, 6-7, 6-9, 7-1, 8-1</p>	<p>In Global Topics Report</p>
<p>T6. Chapter 5 was found to contain several items that require modification to improve technical adequacy. It is recommended that this chapter be redrafted with the following modifications: Revise Table 5-2 to reflect default material strengths that are mean values and are consistent with the other chapters.</p>	<p>5-10 A-7</p>	<p>In Global Topics Report In Global Topics Report</p>

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
Revise Table 5-4 to express parameters as plastic rotations and not multiples of yield rotation	5-8	In Global Topics Report
Revise Tables and text so m is never less than one	5-9	In Global Topics Report
Revise treatment of columns as force or deformation-controlled and modify equations to improve usability	5-10	In Global Topics Report
Revise definition of permissible plastic rotation to be consistent with SAC and other chapters	5-8	In Global Topics Report
Correct the references cited in Section 5.5.2.3 to more current standards	N/A	Part of Prestandard process
Braced frame connection provisions appear too restrictive for applications where braces are lightly loaded and the connections are required to develop a brace capacity that will not be utilized.	5-12	
Application of braced frame connection provisions were found to be difficult to understand and apply and could be rewritten to clarify	5-12	
The <i>Guidelines</i> ' treatment of braces and columns as force and deformation-controlled components led to user confusion. For IO performance, deformation-controlled braces have more stringent requirements than force-controlled columns. This should be corrected and the treatment of braces and columns clarified	5-12	In Global Topics Report
Expected strengths for foundation anchor bolts is not provided.	5-11	New GT
Diaphragm capacities appear to be too restrictive and inconsistent with past building performance.	6-16	New GT
The <i>Guidelines</i> should provide consistent guidance for diaphragms of the same materials. Metal deck with concrete fill has a series of m values for IO, LS, CP while concrete diaphragms have a single DCR value. In general, the correctness of these values and the procedures for establishing capacity should be reviewed. Diaphragms were found to be a significant factor in higher construction costs for <i>Guidelines</i> design solutions	6-16	New GT
Improve the explanations for which reference standards are applicable to capacity calculations	A-1	In Global Topics Report

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
T7. Procedures for estimating the sliding capacity of foundations produce answers inconsistent with observed performance and engineering judgment. Information has not been provided in Chapter 4 for friction piles (subject to uplift and overturning) and procedures for determining lateral soil springs require clarification. It is recommended that these concerns be studied and appropriate modifications to Chapter 4 be developed.	4-3, 4-4, 4-9	In Global Topics Report
T8. All chapters should be revised to consistently reflect mean values for expected strengths.	A-7	In Global Topics Report
T9. As presently written, Section 3.2.2.2 requires 3-D analyses when the maximum displacement exceeds the average floor displacement by 50%. At present, nonlinear software capable of 3-D analysis is not commercially available. For all buildings that must be analyzed by the nonlinear procedures and must use 3-D analyses, the <i>Guidelines</i> may not be a practical rehabilitation approach. It is recommended that some guidance be developed for use in the <i>Commentary</i> to help users until software is available.	2-7 (related to U11)	New GT (Future Study or Research)
T10. Limitations on the use of the linear procedures require calculation of DCRs. As currently written, the <i>Guidelines</i> require that linear procedures can be used if all DCRs are less than 2.0 or if structural irregularities exist when some DCRs are greater than 2.0. It is recommended that a study be undertaken to determine if there is an upper limit for DCR values that should not be exceeded if linear procedures are to be applicable regardless of the presence or absence of structural irregularities. The study should also determine the need to include consideration of the relative differences among the DCRs and their distribution.	2-19 (related to U17)	New GT (Technical Revision or Editorial)

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<p>T11. Case Study firms expressed concern that some materials such as hollow clay tile and plain concrete do not have explicit acceptance criteria or modeling information in the <i>Guidelines</i>. These firms suggested that a generalized procedure that does not require extensive component testing be developed to permit estimation of acceptance and modeling values for these and other materials. It is recommended that these archaic materials and any others for which engineering data is available be incorporated into the <i>Guidelines</i> and that a generalized procedure with reduced testing requirements be investigated.</p>	A-12	<p>New GT to include these materials or to develop generalized method without testing (Future study or research)</p>
<p>T12. Specification of the C_2 coefficient leads to counter-intuitive demands (higher for Life Safety than Immediate Occupancy) and would be better defined on the basis of the amount of nonlinearity anticipated in the structural response. No numerical procedures are provided for characterizing system strength and stiffness deterioration to permit definitive engineering determinations to be made regarding classification. Further study of alternative formulations for the C_2 coefficient is recommended. The use of DCRs may be an appropriate alternative.</p>	3-23	<p>In Global Topics Report (see Coefficient Study)</p>
<p>T13. Calculation of the C_3 coefficient is very difficult in the nonlinear procedures and probably more difficult than is appropriate with the extent of our existing knowledge. In section 3.3.1, the C_3 coefficient is used to amplify the entire building response but is calculated on the basis of the critical story. This appears unnecessarily restrictive. Further study of alternative formulations for calculation and use of the C_3 coefficient is recommended.</p>	3-23	<p>In Global Topics Report (Future Study or Research)</p>

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
T14. Method 3 period formulation appears unduly conservative for multi-span diaphragm systems when maximum pseudo lateral load is used for entire building. Further guidance on the application of equation 3-5 to various wood and metal deck systems would greatly facilitate correct usage. Further study of the application of this equation is recommended and development of supplemental text describing how it is to be applied is recommended.	3-2, 3-8	In Global Topics Report
T15. Technical concerns have been raised regarding the use of response spectrum analysis techniques with 90% of the effective building mass that are unscaled to a minimum base shear. This approach could be unconservative since ten percent of the effective translational mass is being ignored. Further study of this requirement is recommended.	3-5	In Global Topics Report
T16. The validity of the methods used to determine the target displacement for the NSP have not been satisfactorily demonstrated to the engineering community at large. It is recommended that research and studies be conducted to demonstrate the validity of this approach.	3-23	In Global Topics Report
T17. In Chapter 10 applications of FEMA 310, applying strength and stiffness ratio limitations to floors above (and below) each story to define weak and soft story irregularities seems unnecessarily stringent. By requiring an upper floor to be 80% as strong and 70% as stiff as the floor below, many buildings will be unnecessarily classified as irregular. Study is recommended to determine if this requirement is justified to achieve the Life Safety performance level.	N/A	FEMA 310 not in ASCE/FEMA 273 Prestandard scope
T18. Chapter 6 was found to contain several items for which technical adequacy was questioned or for which information was not provided. These include: T18a For flexure critical walls, the increase in acceptability limits from Life Safety to Collapse Prevention may be too small given the limited number of reported collapses of shear wall buildings.	6-1	In GTR

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
T18b An anchorage to Concrete Walls section similar to that provided in the Masonry section is needed.	N/A	Non-persuasive. Requirements are in Guidelines 2.11.7. However, concrete and masonry are, in fact, treated differently.
T18c Misprints of acceptance criteria values were noted in Tables 6-7 and 6-13	6-1, 6-8	In Global Topics Report
T18d The effects on performance characteristics of lightweight concrete versus normal weight concrete do not appear to be specifically addressed in the acceptance criteria. No information provided on development lengths for square reinforcing bars or welded reinforcing bars	6-14	New GT Missing material (Future study or research)
T18e The <i>Guidelines</i> require 100% of the gross section shear stiffness be used in analysis. For squat walls or other shear dominated elements, this assumption can produce inaccurate results	6-19	New GT (Future study or research)
T18f Inconsistent recommendations for effective flange width of shear walls noted between Sections 6.4.1.3 and 6.8.2.2.A	N/A	Editorial clarification part of Prestandard process
T18g Provisions of Section 6.4.1.3 as applied to joist construction may understate frame action of the system unless specific guidance is provided for these common building systems	6-20	New GT. Future study or research
T18h Section 6.4.2.2 recommends 1.25 times nominal yield stress for tensile strength calculations but Masonry Sections 7.3.2.6 and 7.4.4.2.A do not. Is this inconsistency appropriate? Is a clarification on Section 7 warranted	7-6	Chapter 7 does not exclude use of 1.25. New GT
T18I More discussion of the use of <i>phi</i> factors in conjunction with ACI references for strength determination are necessary	A-7	In Global Topics Report
T18j More guidance is needed to discuss treatment of shear walls with axial loads greater than $0.35P_0$ and with bar spacings greater than 18 inches	N/A	Non-persuasive (too detailed)

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
T18k Concern was expressed that drift ratio limits for walls controlled by shear produce ductility demands of approximately 20, which appears too high	6-1	In Global Topics Report (detailed review of acceptance criteria—future study or research)
T18l Concerns were expressed that Section 6.8.2.3 may predict too low an initial flexural yield moment (point B in Figure 6.1 (a)) particularly for determining shear or flexurally-controlled behavior. Lightly reinforced boundaries may require that point B be defined as a ratio of point C	6-18	New GT
T18m Acceptability limits for columns in tension are not provided	6-17	New GT (Future study or research)
T18n Concrete diaphragms have acceptability defined in terms of DCRs, for consistency this should be changed to an <i>m</i> (see comments on Chapter 5).	6-16	New GT See section 6.11.2.4 (Technical Revision)
T19. Chapter 7 requirements for determining out-of-plane sufficiency when S_{x1} exceeds 0.5g (time history analysis) are not practical. Additional research and study is recommended to develop parameters to extend this table to ranges of acceleration appropriate for MCE demands.	7-2, 7-7	New GT (Technical Revision)
T20. Chapter 7 does not address reinforced masonry infills, and particularly grouted infills. Finite element studies done as part of the Case Studies Project suggest the <i>Guidelines</i> procedures for estimating infill frame capacity underestimate its strength by a significant amount. The <i>Guidelines</i> provisions should be extended to include these common construction materials and further review of infill strength appears justified.	7-4	In Global Topics Report (future study or research)

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<p>T21. The following concerns were expressed regarding Chapter 8. It is recommended that these issues be examined by the <i>Guidelines</i> authors and modifications as deemed appropriate be made:</p> <p>21a Acceptance criteria (<i>m</i> values) for gypsum wall board and plaster are higher than those for structural panels. Engineers expressed concern that this does not seem consistent with historical practices.</p>	8-4	In Global Topics Report
<ul style="list-style-type: none"> 21b Diaphragm deformation acceptance criteria are linked to other <i>Guidelines</i> Sections such as URM, which do not provide the requisite requirements for out of plane deformation limits. Further study is necessary to establish out-of-plane differential floor displacement limits appropriate for the acceptable performance of various wall materials. 	3-8	In Global Topics Report (future study or research)
<ul style="list-style-type: none"> 21c The relative values of strength and stiffness for plywood over diagonal sheathing and the permissible <i>m</i> values for plywood versus diagonal sheathing seem incorrect to engineers. 	8-1, 8-4	In Global Topics Report
<p>T22. Guidance should be provided in Chapter 9 for the use of the <i>C</i> and <i>J</i> coefficients.</p>	9-3 (related to U22)	New GT Add explicit instructions (Technical Revision/editorial)
<p>T23. Technical concerns raised by the Case Studies with regard to Chapter 11 are given below. It is recommended that the authors of this <i>Guideline</i> section review these concerns and develop modifications as may be appropriate.</p> <p>23a Heavy partitions were judged to potentially be a Life Safety threat even in zones of low seismicity and therefore should require some minimum level of resistance to toppling.</p>	11-9 (related to U28)	New GT (Technical Revision)
<ul style="list-style-type: none"> 23b Displacement acceptance criteria for Category C ceilings is not provided. 	N/A	Non-persuasive (Force controlled)

Case Study Issue— Technical Issues	Corresponding Global Topic	Action
<ul style="list-style-type: none"> 23c Inconsistent drift limits provided for similar systems. Some limits appear too large to achieve intended performance. Glass Block and Glazing are limited to .02, while heavy partitions are .01. A 30 foot high window wall could move 7". This does not seem right for life safety. 	11-6	New GT. Reference is to glass. Choice of drift of .02 is unclear. (Technical revision)
<ul style="list-style-type: none"> 23d Mandatory inspection of precast panel connections may not be necessary. 	N/A	Editorial clarification part of Prestandard process
<ul style="list-style-type: none"> 23e Referenced standards in some cases lack the information needed to complete rehabilitation. Category 1 Piping is referenced to SP-58, which has no bracing standards. Electrical distribution to SMACNA, 1980, 1985 which has no bracing standards (reference should be to SMACNA, 1991, Appendix E) 	A-1	Identify and correct references. In Global Topics Report
<p>T24. The Case Studies Project demonstrated a wide range in the performance of engineering firms applying the same set of criteria to the same building. Consistent application of the <i>Guidelines</i> among users will not occur without a program of peer review or design oversight in conjunction with engineer training and the availability of application manuals. Implementation of all these supportive adjuncts to the design process should be included by administrative authorities concerned with a uniform application of the <i>Guidelines</i> as a national building code. Appendices 10.3.3 and 10.3.4 include numerous engineering firm and DAP comments regarding various <i>Guidelines</i> issues. Those comments should be reviewed on a section by section basis for more specific information regarding the above recommendations.</p>	N/A	Review of detailed comments has been performed as part of Prestandard process
<p>T25. Specific requirements for generating <i>Guidelines</i> compatible site specific ground motion characterizations should be developed and added to the <i>Guidelines</i>.</p>	N/A	Non-persuasive. In Guidelines 2.6.2.1

ATTACHMENT 2

**New Global Topics
And Changes to the Prestandard
Developed to Respond to
Case Study Issues**

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U3. The default site class should be revised from Class E to Class D.

Recommended Technical Revision

In section 2.6.1.4 Adjustment for Site Class, under Class F, DELETE, “If insufficient data are available to classify a soil profile as type A through D, a type E profile shall be assumed.

In section 2.6.1.4, under Class D, ADD, “If insufficient data are available to classify a soil profile as type A through C, and there is no evidence in the general area of the site of soft clays characteristic of type E, a type D profile shall be assumed. If there is evidence of the existence of type E soils in the area and no data to classify as type A through D, type E shall be assumed.”

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U7a. All provisions relating to the use of DCRs should be located in one section.

U7b. The definition of DCRs should be revised to be consistent with the parameters used for checking component acceptability (force-controlled) to eliminate an additional round of calculations.

U37. Clarify inconsistent definitions of weak story given in Sections 2.9.1.1 and 6.5.2.4A.

For U7a and U37,

Section 2.9.1.1 is trigger measuring relative story strengths.

Section 6.5.2.4A is a trigger measuring relative strengths of beams and columns.

Therefore incorporated the following:

Recommended Clarifications

Change the term in 6.5.2.4.A from “weak story element” to “weak column element,” eliminating the conflict in definitions.

For U7b,

The capacity must be set at either lower bound or expected strengths. In either case, another calculation would be needed to check the other. Comment is Non-persuasive. T

However, the comment illustrates that the procedures of 2.9.1 are now required. Due to the definition of demand (including C factors) and capacity (expected), a designer may think that a special analysis for this purpose is required. It is suggested that the following wording be added to the commentary.

C2.9.1.1 The magnitude...regularity. ADD “It should also be noted that since these analyses are linear, demand/capacity ratios obtained from previous analyses can be converted to DCRs by developing a multiplier that considers any difference in Sa, the appropriate C factors from Chapter 3, and the change in capacity from nominal to expected.

This clarification found non-persuasive by PT on 9/8/99

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U9. Clarification regarding the inclusion of roof loads and the definition of measured loads is necessary.

U9. Guidelines Section 3.3.1.3 :

The total dead load definition for W does not provide guidance on treatment of non-snow roof loads.

Recommended Clarification

In bulleted items listed under W, add “ Roof live load need not be included except for the applicable snow load....”

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U15 When replacement of a concrete element is required (Section 6.3.5), the *Guidelines* generally require the element be designed to meet the requirements for new buildings. This is problematic in that design for new buildings will require a complete re-analysis of the building to establish demand. The *Guidelines* should require that the design of new elements is deemed sufficient if these components are shown to meet the requirements of the *Guidelines*.

U15. Guidelines Section 6.3.5 :

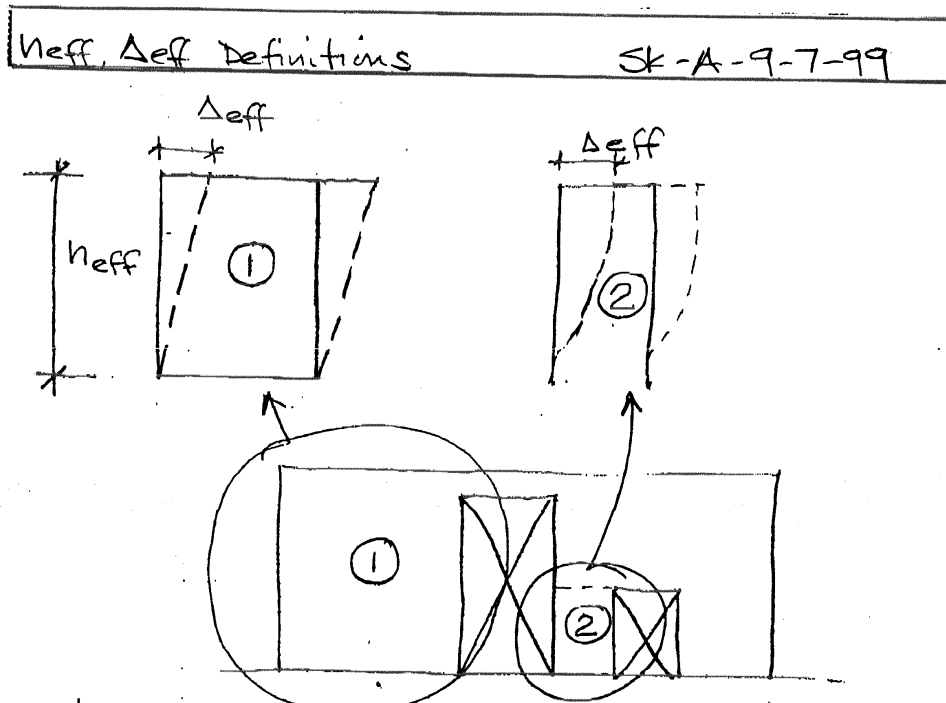
When replacement of a concrete element is required, the Guidelines currently require that the element be designed in accordance with a model code. As written, this would require additional demand and capacity calculations.

Recommended Clarification

Replace the word “design” with the word “detailing”.

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U17. The Chapter 7 definitions for the parameters h_{eff} and Δ_{eff} require clarification. A graphical depiction of these parameters is shown below:



h_{eff} is the effective height of the component under consideration and Δ_{eff} is the differential displacement between the top & bottom of the component. Depending upon wall & Pier geometry, the elevations at which these parameters are defined may vary in the same wall assembly.

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U18. Equations 7-5 and 7-6 do not provide guidance to users on L/h_{eff} limits outside the applicable bounds noted for these equations. Guidance on this subject is necessary.

U18. Guidelines Section 7.4.2.2.B :

Equations 7-5 and 7-6 do not provide guidance to users if L/h_{eff} ratios fall outside the range of 0.67 to 1.00.

Recommended Clarification

Add the following sentence at the end of Section 7.4.2.2.B, before the commentary sentences :

“ For all other L/h_{eff} ratios, Section 7.4.2.2.A is applicable.”

Case Study Issues
New Global Topic
Suggesting Change in FEMA 273 Standard

U22. Chapter 9 should address use of the C_1 , C_2 and C_3 coefficients.

Recommended Clarification

ADD new paragraph in 9.2.1:

For seismically isolated structures, the coefficients C_0 , C_1 , C_2 , C_3 and J shall be taken equal to 1.0.”

Case Study Issues New Global Topic Suggesting Change in FEMA 273 Standard

U 28. In zones of low seismicity the *Guidelines* do not require heavy partitions to be reviewed for adequacy. Section 11.4.4 describes items of concern for maintaining building egress to meet a Life Safety performance level. This discussion includes heavy partitions. Further discussion should be added to this section noting that in zones of low seismicity the risk of heavy partitions blocking egress is sufficiently low to be ignored

T23a Heavy partitions were judged to potentially be a Life Safety threat even in zones of low seismicity and therefore should require some minimum level of resistance to toppling.

Recommended Technical Revision

Change “No” to “Yes in line A2 of Table 11-1.

Found non-persuasive- by PT on 9/8/99

Case Study Issues New Global Topic Suggesting Change in FEMA 273 Standard

U34. The BSO requires analytical reviews for both Life Safety at BSE-1 and Collapse Prevention at BSE-2. The Case Studies indicate that the BSE-2 and Collapse Prevention generally govern design requirements. Eliminate the Life Safety review for BSE-1 to reduce the computational burden and improve usability. This will also eliminate the possibility of requiring engineers to use nonlinear procedures for BSE-2 while having used linear procedures for BSE-1.

Recommended Technical Revision

Revise Section 2.4.1 to define the Basic Safety Objective as rehabilitation to achieve the collapse prevention level of performance for BSE-2. Revise subsequent sections accordingly. Note that non-structural components except parapets and heavy appendages will not require mandatory rehabilitation. Building Performance level 5-E becomes the BSO.

Found non-persuasive by PT on 9/8/99

Related Issues

U5. The definitions of seismicity and the site class coefficients must be the same in FEMA 310 and FEMA 273.

Also other comments about the complexity of using multiple maps:

For **BSE 1** equivalent, FEMA 310 uses $2/3$ MCE.
BSE 1 defined as **lessor** of $10/.50$ **or** $2/3$ MCE (usually $10/50$)

For BSE 2, **lessor** of MCE **or** $2/50$ used.
(PT specifically considered this)

No action recommended by PT on 9/8/99

Case Study Issues New Global Topic Suggesting Change in FEMA 273 Standard

U36. Section 2.8.1 should delete the reference to Table 10-1 that suggests regularity is a feature of the table.

Recommended Clarification

Change the wording of section 2.8.1 in first bullet as follows:

The building conforms to one...limitations indicated in that chapter ~~table~~ with regard...”

D. Special Study 2— Analysis of Special Procedure Issues

ANALYSIS OF SPECIAL PROCEDURE ISSUES FEMA/ASCE FEMA 273 PRESTANDARD PROJECT

Background & Conclusion

In accordance with our proposal to address “Special Procedure Issues” with specific regard to rehabilitation of unreinforced masonry buildings, a team consisting of Daniel Shapiro, Dan Abrams, Mike Mehrain and John Coil has concluded the following:

1. The “Special Procedure” adapted from the UCBC should not be added to the *Guidelines* for the seismic rehabilitation design of unreinforced masonry buildings.
2. The specific portions of the “Special Procedure” deemed necessary to recognize the unique behavior of unreinforced masonry buildings when subjected to earthquake shaking are embedded within the provisions of the *Guidelines* and are adequately identified.
3. Certain revisions to the *Guidelines* may be desirable to clarify the manner in which building periods should be calculated and how lateral forces should be distributed to unreinforced masonry buildings.

Rationale

The following rationale was used to arrive at the conclusions noted above:

The provisions of Appendix Chapter 1 of the 1997 Uniform Code for Building Conservation are intended to meet criteria for life safety for only one particular type of building: i.e. a building with unreinforced masonry walls and timber floors or roofs that are relatively flexible when compared to the walls. Many engineers have expressed concern that the UCBC criterion does not, in fact, meet Life Safety criteria.

Guidelines for seismic rehabilitation given with FEMA 273 are intended to be inclusive of all building types since lateral force resisting elements constructed of concrete, steel, timber or masonry may be combined interchangeably with flexible or stiff floor or roof diaphragms constructed of concrete or timber. The modeling approach inherent with FEMA 273 that will allow engineers to evaluate and rehabilitate a number of different building types is an advancement well beyond the model-building approach of UCBC.

The FEMA 273 *Guidelines* present a more detailed performance-based approach, which is inclusive of not only life safety, but also immediate occupancy and collapse prevention. As a result of this greater versatility, analysis methods given with the *Guidelines* are more diverse than those in UCBC and include linear and nonlinear, static and dynamic methods for estimating peak displacement response. As a result of the displacement-based approach of the *Guidelines*, seismic strength of lateral-force resisting elements are prescribed in terms of expected values rather than the working stress values inherent in the force-based set of requirements of the UCBC. Furthermore, the Guidelines present seismic loads in terms of spectral response curves taken from recent USGS hazard maps that represent the most current expectations of earthquake motions across the country. The seismic demand represented in the UCBC is a much simpler approximation based on one of four seismic zones.

Inasmuch as there would be no easy way to introduce the UCBC Special Procedure into the Guidelines without significant modifications to both the Special Procedure and the *Guidelines* one should instead address the central question of whether the *Guidelines* cover all of the UCBC requirements that are unique to unreinforced masonry buildings, and what, if any, additional guidance is given in FEMA 273 for designing seismic rehabilitation of unreinforced masonry buildings.

A comparison reveals that the Guidelines are not only adequate but advance the state of the art in seismic rehabilitation of unreinforced masonry buildings beyond that provided by the UCBC. The two documents provide similar limitations on masonry piers in a rocking mode and in a shear mode. The *Guidelines* further limits pier lateral strength with equations representing toe compression and diagonal tension. Lateral strengths of piers resisting significant vertical compressive stress, or with relatively strong mortars may be limited by these force-controlled effects, which are not considered by the UCBC.

In the UCBC, lateral forces are distributed to individual piers in proportion to their relative rocking strengths if all piers in a story have a rocking strength less than the allowable shear strength. If one or more piers in a story are governed by shear and not rocking, then the distribution of story shear is in proportion with the D/H ratio of each pier. Any pier that attracts a force greater than its rocking strength is eliminated from the analysis. The distribution of forces to individual piers in accordance with the *Guidelines* simply follows that as calculated with a linear static analysis. For purposes of force distribution, the stiffness of any one pier is estimated with its uncracked stiffness.

h/t limitations in the *Guidelines* for out-of-plane bending of unreinforced masonry walls are adapted directly from the UCBC limitations.

As noted before, the *Guidelines* are intended for use with diaphragms of any stiffness while the UCBC is limited to buildings with flexible diaphragms. In the UCBC a figure is provided for which to determine a basis for establishing h/t values depending on diaphragm configuration and presence of “cross walls.” The Wood Team was unable to verify the values in the figure and determined certain anomalies with its use. They chose not to include it in the *Guidelines*.

In reviewing the period calculations provided in the *Guidelines* it becomes apparent that a method for calculating the period (or periods) of a multi-story unreinforced masonry building is lacking. To rectify this situation it appears that it would be appropriate to modify the period calculations as presented in the *Guidelines* as follows:

A) Modify Section 3.3.1.2 as follows:

Move Method 3 to become a special case of Method 1 and simplify Equation 3-5 to consider the deformation of the diaphragm only as follows:

- Eliminate Method 3
- Add to the end of Method 1 the following:

“It shall be permitted to calculate the fundamental period of a single span flexible diaphragm from Equation 3-5

$$T = (0.078 Dd)^{0.5} \quad (3-5)$$

Where Dd is the maximum in-plane diaphragm displacement in inches, due to a lateral load in the direction under consideration, equal to the weight tributary to the diaphragm. The stiffness of the diaphragm shall be that associated with state of stresses near yield level.”

B) Provide a new section for handling URM building analysis as follows:

For buildings with flexible diaphragms, it shall be permitted to distribute pseudo lateral loads as follows:

- For each span at each level of the building, calculate period from Equation 3-5
- Using Equation 3-6 calculate lateral load for each span
- Apply the lateral loads calculated for all spans and calculate forces in vertical seismic resisting elements, using tributary loads. Equation 3-7 is not applicable in this analysis.
- Diaphragm forces for evaluation of diaphragms are as indicated above (Do not use Equation 3-9)
- Seismic loads shall be distributed along the diaphragm span considering its displaced shape (see existing commentary on this issue).

Finally it should be considered that the just concluded FEMA/BSSC Case Study Project had 5 unreinforced masonry buildings included among the case studies, 3 of which were analyzed by the Linear Static or Linear Dynamic Procedures. None of the Case Study contractors involved suggested that the UCBC Methodology be included in the *Guidelines*.

E. Special Study 3— Improvements to the FEMA 273 Linear Static Procedure

Improvements to the FEMA 273 Linear Static Procedure

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ABSTRACT

The FEMA 273 Linear Static Procedure is appropriate for evaluation of simple, regular structures. Results of case studies, however, have shown that the procedure appears to be overly conservative, and predicts poor performance in buildings that would otherwise be expected to perform satisfactorily. This paper addresses potential sources of conservatism in the LSP including the calculation of building response based on an empirical formula for period, use of 100% of total building weight without regard for higher mode mass participation effects, calculation of pseudo lateral forces based on the initial elastic stiffness of the structure, and acceptance criteria that is inconsistent with assumptions about degradation. Results reported on a database of recent projects show that conservatism in the LSP can be reduced with a few improvements to the procedure.

1. INTRODUCTION

The *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, is a recently published comprehensive reference for performance-based engineering of seismic rehabilitation of buildings. FEMA 273 outlines four analysis tools: the Linear Static Procedure (LSP), Nonlinear Static Procedure (NSP), Linear Dynamic Procedure (LDP), and Nonlinear Dynamic Procedure (NDP), each with different strengths and different limitations in applicability.

The purpose of this paper is to study potential sources of conservatism in the LSP in an effort to improve correlation with expected results based on historic performance of buildings and more advanced analysis techniques. Potential sources of conservatism addressed in this study include the calculation of building response based on an empirical formula for period, use of 100% of total building weight without regard for higher mode mass participation effects, calculation of pseudo lateral forces based on the initial elastic stiffness of the structure, and acceptance criteria that is inconsistent with assumptions about degradation. Data presented in this report is based on results from 25 of the most recent Degenkolb performance-based engineering projects to date, and studies of similar issues published in the literature. The intent of this study is to identify trends observed in data available at this time, and suggest changes that would reduce the conservatism and improve the effectiveness of the LSP for use in situations when linear static procedures are appropriate.

2. FEMA 273 LINEAR STATIC PROCEDURE

Current code procedures rely on elastic analyses for design, with the understanding that in an actual earthquake, structures will be loaded beyond their elastic limits. The difference between actual demands and code design forces is rationalized on the basis of ductility, overstrength and energy dissipation. In FEMA 273, performance-based design is achieved through the explicit evaluation these parameters on a component basis. In the nonlinear range of response, small changes in force demand correspond to large changes in displacement demand and correspondingly large differences in structural damage. For this reason, displacement-based design procedures are considered the best measures of performance, and explicit calculation of displacement demands using nonlinear analysis techniques are considered the best tools for performance-based design of structures.

Nonlinear analyses, however, can be difficult and time consuming to perform. For simple, regular buildings, this level of effort may not be practical, and it can be appropriate to use simplified yet conservative linear procedures to evaluate building performance. The LSP is one such displacement-based approach. Based on the theory of equal displacements, pseudo lateral forces calculated using the LSP are those forces that would push the elastic structure to approximately the same displacements as those expected in the actual inelastic response of the structure subjected to the design earthquake. This relationship is shown graphically in Figure 1. In the LSP, displacement-based concepts have been translated back to force-based calculations for reasons of simplicity and familiarity. This is accomplished with Equation (1), which consists of the building weight (W), the spectral acceleration (S_a), and a series of coefficients (C_1 , C_2 , C_3) that modify calculated displacements to account for inelastic activity, pinched hysteric behavior, and P-delta effects respectively. The coefficients C_1 , C_2 , and C_3 vary with period so the resulting lateral force will vary with period, even if the building response is on the plateau of the spectrum.

$$V = C_1 C_2 C_3 S_a W \quad (1)$$

A logical consequence of simplification is conservatism. In compensation for less precise information, a procedure can be made more conservative. The key to producing reasonable results with a simplified procedure, however, is installing an appropriate level of conservatism. Since the publication of FEMA 273 in 1997, the LSP has been implemented in practice, and has been the subject of verification case studies. In many cases, results using the procedure appear to be overly conservative, and predict poor performance in buildings that would otherwise be expected to perform satisfactorily based on historic earthquake performance.

3. EMPIRICAL FORMULAS FOR PERIOD

FEMA 273 offers three methods for the calculation of building period. Method 1, calculation of period using eigenvalue analysis of the structure, is the most accurate and preferred method. Method 2 uses a formula based on code empirical equations for period. Method 3 is a special case for single story, flexible diaphragm systems.

When using force-based, elastic methods of analysis, a conservative estimate of base shear is obtained by using periods that are shorter than actual periods. Code empirical equations were developed with the intent of underestimating the actual period by 10-20% (Goel and Chopra 1997). Using data recorded from instrumented buildings during the 1989 Loma Prieta and the 1994 Northridge earthquakes, it was shown that empirical equations underestimate measured periods for frame structures on the order of 20-40% (Goel and Chopra 1997), and had very poor correlation with measured periods for shear wall buildings (Goel and Chopra 1998). These results are supported by results on recent Degenkolb projects shown in Table 1. Using data from more recent earthquakes to supplement the data used in the ATC3-06 project, empirical equations can be improved to better correlate with measured building response (Goel and Chopra 1997, 1998). Equations (2), (3), and (4) are best fit equations proposed by Goel and Chopra for steel frame, concrete frame and concrete shear wall buildings respectively, where H is the building height in feet and \overline{A}_e is a ratio based on the shear wall area defined in the paper.

$$T = 0.035 H^{0.80} \quad (2)$$

$$T = 0.018 H^{0.90} \quad (3)$$

$$T = 0.023 H / (\overline{A}_e)^{0.50} \quad (4)$$

Analytically, the best estimate of period comes from an eigenvalue analysis. Empirical equations that more closely approximate eigenvalue periods could help reduce the conservatism in the LSP, even when the building response period is on the plateau of the spectrum. Figure 2 compares empirical equations with eigenvalue periods when the proposed formulas were tested on recent Degenkolb projects. Results were somewhat scattered, showing poor correlation between periods for concrete buildings, and pier spandrel buildings in particular. For steel moment frame buildings, the proposed formulas generally showed improved correlation with eigenvalue periods. Formulas were not available for braced frame systems. Figure 3 compares base shears calculated using different periods, normalized to the base shear resulting from the eigenvalue period. While the results are also scattered, this figure demonstrates that a significant reduction could be achieved if empirical equations could be better correlated with eigenvalue periods. Data suggests that this reduction is on the order of 30% on average across building types, and improved correlation of empirical equations is suggested for future research.

4. HIGHER MODE MASS PARTICIPATION EFFECTS

The LSP, like code-based equivalent lateral force procedures, calculates base shear using 100% of the total building weight. This is contrary to general results of dynamic analyses of MDOF systems in which effective weight can be less than the total weight due to higher mode mass participation effects. In the acceleration-controlled region of the spectrum, base shears determined by response spectrum analyses are less than static base shears based on the total building weight because the effective weight is always less than 100% (Chopra and Cruz 1986). In the velocity- and displacement-controlled regions, higher mode effects can be significant enough that the response may be increased (Chopra and Cruz 1986). These results are dependent upon period as well as the distribution of mass and stiffness within the building, and any potential reductions resulting from these higher mode effects have been explicitly ignored in the development of the LSP (BSSC 1997b).

Dynamic analyses on recent Degenkolb projects shows that response spectrum base shears are always less than static base shears using 100% of total building weight. Data suggests that the effect increases with increasing number of stories, and is closely related to the first mode effective mass. Figure 4 shows the ratio of LDP to LSP base shears as compared to the first mode effective mass. Because the periods for most buildings in this study are on the spectral plateau this result was expected, however, it was also true for taller steel moment frame buildings with periods significantly beyond the plateau.

An adjustment for mass participation effects could be incorporated into the LSP by considering only the effective weight of the building in calculating base shear. This could be done with a matrix of factors, such as that shown in Table 2, developed based on the data in Table 1. The data suggests that mass participation effects could be used to reduce the conservatism in the procedure up to 30%, depending on the building type and number of stories, as indicated in Table 2.

5. INITIAL VERSUS EFFECTIVE STIFFNESS

The pseudo lateral forces of the LSP are those forces that would push the elastic structure to approximately the same displacements as those expected in the actual inelastic response of the structure. The resulting forces are therefore dependent upon an appropriate representation of the elastic stiffness of the structure. One example is the line with slope K_i in Figure 1. In nonlinear analyses, target displacements are calculated using an effective stiffness shown as the line with slope K_e in Figure 1. However, even in elastic analyses, some level of nonlinearity has been traditionally considered in the calculation of the elastic stiffness when the overall response is better characterized by some effective stiffness. In the case of concrete, use of cracked section properties is common practice.

Analogous to using cracked section properties for concrete elements, it was thought that if the effective response of a structure is more appropriately represented by an effective stiffness K_e , then the use of K_i as a basis for pseudo lateral forces may be a source of over conservatism in the LSP. This hypothesis is not supported by data from recent nonlinear analysis projects. The ratio of K_e/K_i is dependent upon the shape of the pushover curve and is shown in Table 1. For most buildings in this study, the ratio of K_e/K_i was nearly equal to 1.0, indicating little or no difference between effective and initial stiffness. Since period, and therefore spectral acceleration, varies with the inverse square root of stiffness, small changes in stiffness would result in even smaller changes in calculated pseudo lateral forces and no significant impact on conservatism in the LSP. As a result, no improvements related to effective stiffness are proposed at this time.

6. ACCEPTANCE CRITERIA AND DEGRADATION

The Collapse Prevention Performance Level is defined as substantial damage, including significant degradation, on the verge of partial or total collapse (BSSC 1997a). The Life Safety Performance Level is defined as significant but repairable damage, with some margin against collapse remaining (BSSC 1997a). In determining demands, the C_2 coefficient is used to account for increased displacements resulting from poor cyclic behavior or pinched hysteresis loops. Pinching of hysteresis loops is a manifestation of structural damage. A smaller degree of nonlinear response results in a smaller degree of pinching (BSSC 1997b). Thus demands multiplied by the C_2 factor are amplified under the presumption that the primary elements of the structure will experience degradation.

FEMA 273 acceptance criteria are set based on generalized component behavior curves corresponding to ductile, limited ductile or nonductile behavior. These curves, reproduced from FEMA 273, are shown in Figure 5. They are characterized by an elastic range, followed by a plastic range (with or without strain hardening), and finally a strength-degraded range. For ductile behavior the strength-degraded range includes significant residual strength. Nonductile behavior has no plastic range and little residual strength.

Using the curves in Figure 5, acceptance criteria for primary elements is set at point 2 for the Collapse Prevention Performance Level, and 75% of point 2 for the Life Safety Performance Level. As defined, the acceptance criteria limit the acceptable response of each component to the elastic or plastic regions of the idealized backbone curves. Primary lateral force resisting elements are not permitted to experience demands in the strength-degraded range. A building will fail the acceptance criteria as soon as the worst case primary element begins to degrade, which means that the overall structure is never permitted to experience degradation. This is not consistent with demands calculated presuming the presence of degradation and not consistent with the descriptions of damage used to distinguish between performance levels.

To establish an appropriate level of conservatism, this “double counting” should be eliminated. If demands are to be calculated presuming the components will degrade, the acceptance criteria should be consistently set permitting some level of degradation. The validity of this approach can be seen when considering the global behavior of a structure. Consider a four-story concrete shear wall structure with the pushover curve depicted in Figure 6. The curve was developed using components modeled with the full degrading backbone curves. Individual components were allowed to exceed collapse prevention acceptance criteria and slip into the degraded range of response. As can be seen by the curve, even as individual elements degrade, the overall structure maintains a stable level of resistance. The performance limit of the building is not reached until a significant number of components have had a chance to degrade.

The acceptance criteria, as currently defined, are not pushing buildings to the limits of performance. Limiting the response of individual components within elastic or plastic behavior results in a much more conservative result when the components are combined in the overall structural system. To reduce the level of conservatism in the LSP, the acceptance criteria shown in Figure 5 could be adjusted so that life safety occurs at the limit of plastic response, point 2, and collapse prevention occurs at the limit of residual strength, point 3 on the behavior curves. This will allow components to respond at extreme limits of performance to better calibrate the resulting global behavior, and will result in potential reductions in conservatism of up to 33%, depending on component m factors.

7. CONCLUSIONS

Results of case studies have shown that the LSP appears to be overly conservative and predicts poor performance in buildings that would otherwise be expected to behave satisfactorily. Potential conservatism in the LSP can be reduced in three ways. Empirical equations for period can be improved to better correlate with actual periods, reducing pseudo lateral forces by an average of 30%, even when the response is on the spectral plateau. A matrix of effective weight factors can be developed to take into account higher mode mass participation effects to reduce pseudo lateral forces up to 30%, depending on building type. Component acceptance criteria can be adjusted to permit degradation of individual components reducing conservatism by up to 33%, depending on component m factors. Results show that the presence of some component degradation can still result in acceptable overall building performance.

8. REFERENCES

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9. KEYWORDS

FEMA 273, LSP, Linear Static Procedure, empirical, period, mass participation, effective stiffness, acceptance criteria, degradation.

10. PROPOSED CHANGES TO FEMA 273

The following changes to the ASCE/FEMA 273 Prestandard Second PT Draft are proposed as a result of this study. Changes are keyed to global issues in the ASCE/FEMA 273 Prestandard Global Topics Report.

GT 2-1: Overturning: Overturning itself was not specifically addressed by this study. The proposed changes, which serve to reduce the overall conservatism in the LSP, will also indirectly affect the overturning problem by reducing pseudo lateral forces and corresponding overturning demands. No changes specifically related to this issue are proposed as part of this study. Further resolution of this issue is recommended for future research.

GT 3-3: Empirical Formulas for Period: One such source of conservatism in the LSP is the current Method 2 empirical formula for period, which yields intentionally conservative estimates of pseudo lateral force. The data in this study, and other recent publications, support the modification of this formula to better correlate the resulting period with measured response in structures. It is proposed that the Goel and Chopra best-fit equations for steel and concrete frame structures be installed in Section 3.3.1.2.2. The proposed change, which serves to reduce the overall conservatism in the LSP, will also indirectly serve to reduce maximum pseudo lateral forces. Further resolution of this issue is recommended for future research.

GT 3-5: Mass participation effects: The data collected in this study supports the consideration of higher mode mass participation effects in the LSP. These effects, within the limitations in application of the LSP, reduce the overall pseudo lateral forces consistent with the first mode effective mass. A table similar to Table 2 of this study is proposed for incorporation into section 3.3.1.3 with the existing limitation on building height of 100 feet and an additional limitation on building period of 1.0 second or less. This limitation is proposed because studies (Chopra and Cruz 1986) have shown that higher mode mass participation effects can increase the effective base shear for longer period structures in the velocity and displacement controlled regions of the spectrum.

GT 3-15: LSP displacement-based calculation: In an effort to make the LSP more transparent, a simplified procedure for estimation of effective stiffness was attempted. The intent was to revise the calculation procedure of the LSP to use an effective stiffness that was more in line with the overall nonlinear response of structures that was observed when using the NSP. The data in this study did not support key assumptions needed in applying the revised procedure, so no changes with respect to this issue are proposed at this time. This issue is recommended for future research.

GT 3-27: Omit degradation in LSP: This study investigated the inconsistency in assumptions about degradation as applied to LSP acceptance criteria and the calculation of demands. Degradation is assumed in the calculation of demands, while acceptance criteria are currently set such that degradation would not be expected to occur. This inconsistency is a source of overconservatism that should be eliminated. Since the adjustment of all acceptance criteria would be a major undertaking, a simpler approach is proposed that would eliminate the amplification of demands based on the assumed presence of degradation. It is proposed that the C_2 factor for pinched hysteretic behavior and the C_3 factor for P-delta effects be eliminated in Section 3.3.1.3, or set equal to 1 for the LSP. These factors relate specifically to amplification of expected displacement demands due to degradation in the system. Removal of these factors will help improve the consistency between calculated demands and acceptance criteria, and reduce the conservatism in buildings where these factors would otherwise be greater than one.

GT 3-28 Global nonlinear acceptance criteria: While this study specifically addressed the LSP, a similar conclusion about degradation can be made for all analysis procedures and acceptance criteria contained in the Guidelines. Nonlinear demands are calculated assuming degradation will occur, while nonlinear acceptance criteria are established such that degradation will not occur. In the NSP, there is an opportunity to model component degradation and explicitly evaluate the overall condition of the structure when degradation occurs. The current procedure, however, does not allow for this. Since the NSP can be used to model degradation, it is appropriate to include the effects of degradation in calculating demands. Since adjustment of all nonlinear acceptance criteria would be a major undertaking, a simpler approach is proposed that would define a global acceptance criteria for the structure when a portion of the individual components have exceeded their acceptance criteria.

The proposed global acceptance criteria involves applying the concept of the idealized component backbone curve to the pushover curve of the entire structure. An example of this is shown in Figure 6. Global acceptance of the structure can be measured by selecting a performance point, such as the point where significant structural degradation occurs in Figure 6. Just as component acceptance criteria are set using the component backbone curve, the global performance levels can be set from the pushover curve as follows: CP – performance point; LS – 75% of the performance point; IO – 50% of the performance point. To draw the pushover curve, components are modeled using full backbone curves, including strength degradation and residual strengths, and are permitted to respond in the strength degraded range up to the current CP limits for secondary elements.

Table 1: Building Data

Building		LSP			LSP			LSP			LDP			NSP	
System	Stories n	Method 1 Period (Eigenvalue)			Method 2 Period (Empirical)			Best Fit Period (Empirical)			Base Shear V _{LDP} (k)	Roof Disp d _{LDP} (in)	Effect Mass 1st Mode	Stiffness ratio Ke/Ki	Target Disp d _T (in)
		Period T ₁ (sec)	Base Shear V ₁ (k)	Roof Disp d ₁ (in)	Period T ₂ (sec)	Base Shear V ₂ (k)	Roof Disp d ₂ (in)	Period T _{BF} (sec)	Base Shear V _{BF} (k)	Roof Disp d _{BF} (in)					
1. CMRF	3	0.78	9325	18.93	0.74	9439	19.16	1.08	8533	17.32	8949	18.17	96%	1.00	24.33
2. CMRF	6	0.97	5652	7.60	0.65	8443	11.35	0.72	7615	10.23	5006	6.73	87%	1.00	6.33
3. CSW	4	0.44	26447	4.56	0.42	26447	4.56	0.68	23040	3.97	19900	3.52	80%	1.00	3.50
4. CSW	4	0.55	21424	6.70	0.38	27260	8.52	0.25	28190	8.81	n/a	5.13	77%	0.86	6.38
5. CSW	6	0.40	20455	3.56	0.50	19214	3.34	0.43	20361	3.54	14578	2.54	68%	0.99	3.85
6. CSW	6	0.45	19900	4.44	0.50	19214	4.29	0.49	19371	4.32	13363	2.98	64%	0.91	3.89
7. Conc P/S	1	0.48	1743	5.06	0.25	1743	5.06	0.37	1845	5.36	1699	4.93	95%	1.00	4.89
8. Conc P/S	3	0.27	3195	1.37	0.38	2413	1.04	0.07	4300	1.84	1983	0.85	67%	1.00	1.31
9. Conc P/S	4	0.32	59652	3.35	0.47	59652	3.35	0.16	59652	3.35	35826	1.94	84%	0.43	6.11
10. Conc P/S	4	0.40	25600	5.44	0.45	24000	5.10	0.13	40900	8.69	19210	4.08	71%	0.78	7.00
11. Conc P/S	5	0.23	86306	2.86	0.39	75659	2.51	0.15	78226	2.60	80630	2.67	66%	0.87	2.67
12. Conc P/S	5	0.39	81559	7.56	0.39	81550	7.56	0.19	93106	8.63	n/a	8.58	83%	0.85	4.99
13. Conc P/S	5	0.37	21700	3.59	0.39	20945	3.46	0.43	20523	3.39	16580	2.74	75%	1.00	2.90
14. Conc P/S	10	0.80	12225	15.85	0.79	12332	15.99	1.47	2750	3.57	8449	10.95	66%	1.00	13.77
15. Conc P/S	10	0.79	12753	15.14	0.79	12547	14.90	1.47	2777	3.30	10613	12.60	73%	1.00	13.19
16. SMRF	2	0.47	692	3.27	0.43	718	3.40	0.50	671	3.17	645	3.05	93%	1.00	2.63
17. SMRF	4	1.82	340	19.17	0.66	936	52.77	0.81	768	43.29	316	17.81	90%	1.00	15.29
18. SMRF	4	1.42	430	15.13	0.64	963	33.88	0.78	789	27.76	392	13.79	87%	1.00	12.30
19. SMRF	4	1.55	400	17.38	0.66	936	40.67	0.81	768	33.37	356	15.47	84%	1.00	13.38
20. SMRF	4	1.24	499	14.22	0.64	963	27.44	0.78	789	22.48	422	12.02	80%	1.00	10.78
21. SMRF	6	2.42	2861	26.07	0.96	7201	65.61	1.19	5810	52.94	2522	22.98	83%	1.00	20.34
22. SMRF	6	1.12	6656	13.05	0.96	7770	15.24	1.19	6311	12.37	5245	10.28	71%	1.00	9.58
23. CBF	2	0.31	5898	1.79	0.28	6379	1.94	n/a	n/a	n/a	4933.00	1.78	84%	1.00	1.90
24. CBF	4	0.84	37290	14.55	0.47	40246	15.70	n/a	n/a	n/a	n/a	n/a	51%	1.00	27.00
25. EBF	8	1.49	2146	11.58	1.07	2993	16.15	n/a	n/a	n/a	1836	9.91	74%	1.00	8.13

Table 2: Proposed Factors for Effective Weight

Stories	CMRF	CSW	Conc P/S	SMRF	CBF	EBF
1-2	1.00	1.00	1.00	1.00	1.00	1.00
3-4	1.00	0.80	0.80	0.90	0.90	n/a
5-7	0.90	0.70	0.80	0.90	n/a	n/a
8-10	n/a	n/a	0.80	n/a	n/a	0.90

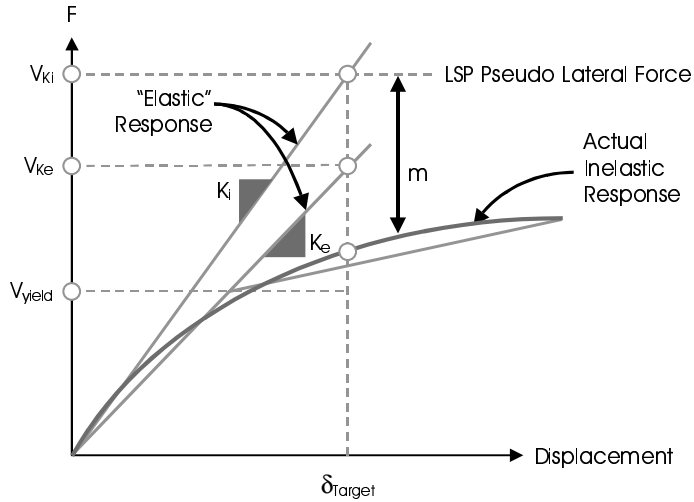


Figure 1:
Graphical Representation of the LSP

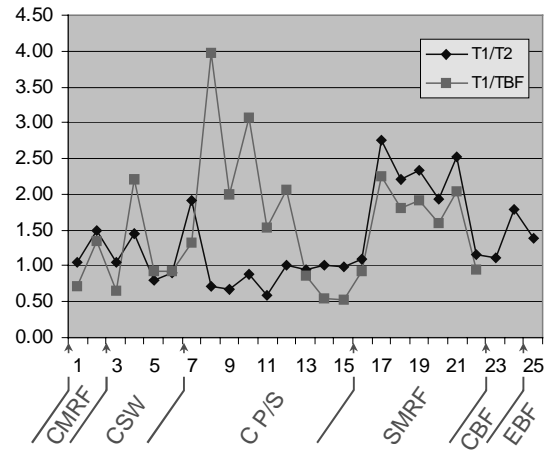


Figure 2: Comparison Between Empirical and Eigenvalue Periods

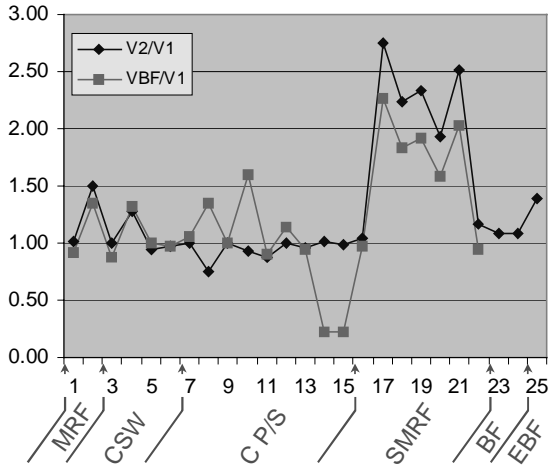


Figure 3: Base Shear Comparison

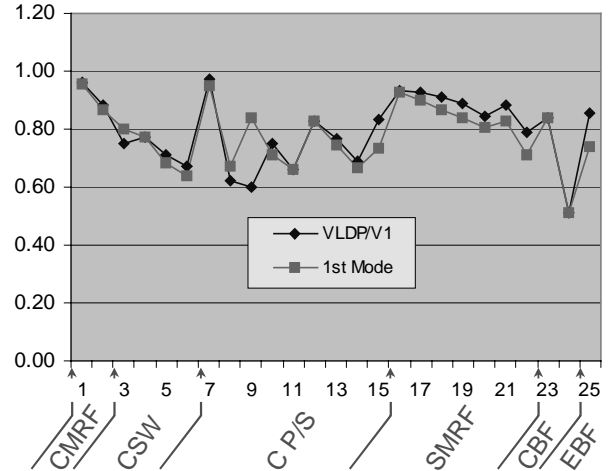


Figure 4: Ratio of LDP to LSP Base Shear and Comparison to First Mode Effective Mass

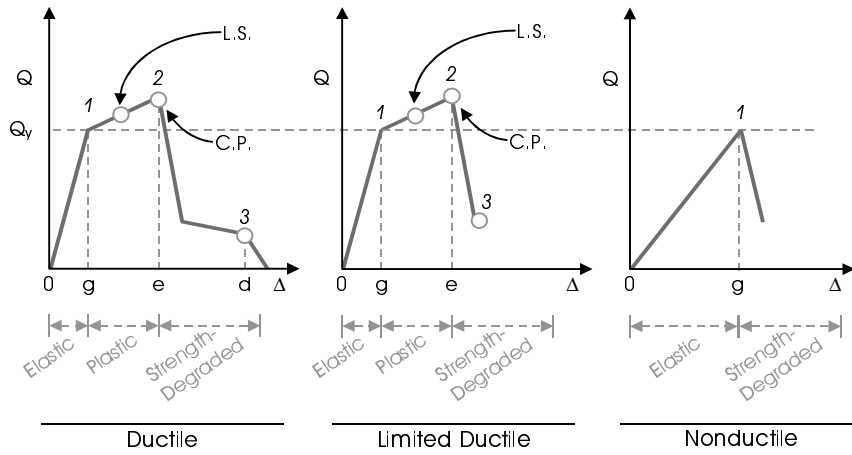


Figure 5:

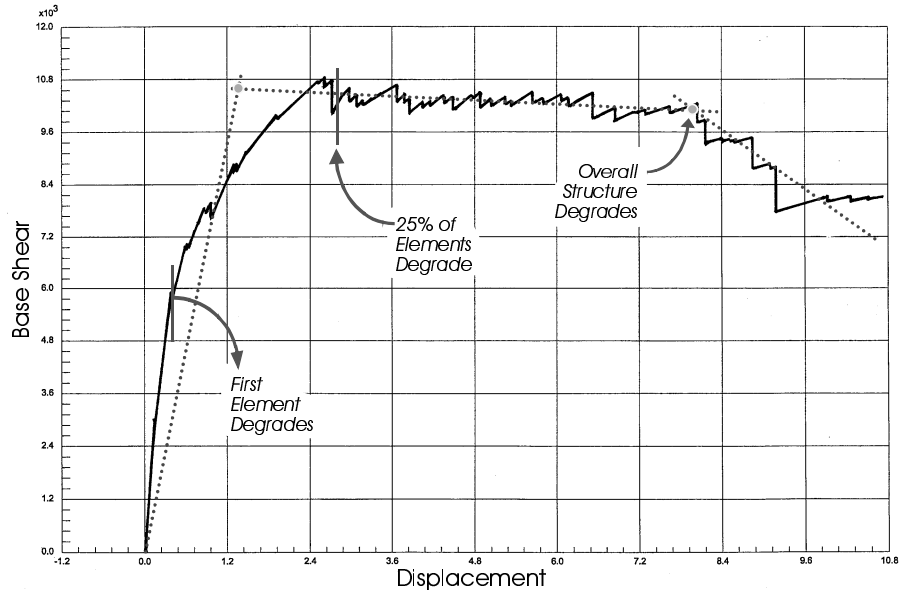


Figure 6: Pushover Curve, Four Story Shear Wall (Building 9)

F. Special Study 4—Foundation Issues

ASCE/FEMA 273 Prestandard Project

Special Study Report
Foundation Issues

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Executive Summary

In this report, four global topics issues are addressed. Four new issues are also identified and addressed.

Two items that require clarification by the Chapter 3 author(s) are identified. These involve clarification of Section 3.2.6 following translation to the prestandard and additional discussion of the limitations placed on the beneficial effects of soil-structure interaction.

Two items are reaffirmed as follows.

- No additional limitations on the beneficial effects of soil-structure interaction are needed for the Nonlinear Static Procedure. We recommend that further clarification be provided to the effect that the 25% limitation in Section 3.2.6 need not be applied to nonlinear analyses.
- The range of variation required in Chapter 4 for strength and stiffness parameters (multiplication and division by a factor of two) is appropriate. A provision for a slightly relaxed range of parameters is recommended when additional testing is provided.

The following revisions are suggested.

- New stiffness solutions for shallow foundations that are applicable to all rectangular foundations are presented. This revision requires the replacement of Figures 4-2 and 4-3.
- Revisions to the calculation procedure for the force-deformation response of lateral soil springs are proposed. These changes produce results for which the stiffness and strength of shallow and deep foundations are consistent with accepted procedures of soil mechanics.
- New effective shear modulus factors are proposed. These new factors are based on research conducted during the course of this project. A revised version of Table 4-3 is presented. The revised table is consistent with recent research and the soil amplification tables found in Chapter 2 of FEMA 273.
- Recommendations are made regarding the classification of the relative stiffnesses of foundations and the supporting soils. Additional guidance (for inclusion in the Commentary) is provided for two-way foundation components.
- Rocking behavior is examined and a rocking design approach suitable for incorporation into commentary is proposed.

Global Topics Report Issues

Four foundation issues are identified in the current Global Topics Report. They are as follows.

- 4-1 **Some of the problems identified in a NSP analysis can be fixed by the addition of foundation springs in the analysis. There is insufficient guidance on the limitations in the application of foundation springs to increase building flexibility.**
Sections: 3.2.6 and 4.4
- 4-2 **The procedure for developing foundation spring constants using an equivalent circular footing is not directly applicable to strip footings below shear walls.**
Section: 4.4.2.1
- 4-3 **The procedure for developing lateral soil spring stiffness using displacement results in unrealistically high soil strengths. More information is needed on the force-displacement behavior of geotechnical materials and foundations under short term loading.**
Section: 4.4
- 4-4 **More information is needed on nonlinear force-displacement behavior of foundation systems for inclusion in nonlinear analyses.**
Section: 4.4

Additional Issues

In the course of this research project, four additional issues were identified.

The shear modulus reduction factors presented in Table 4-3 of FEMA 273 are significantly different from those presented in Table 5.5.2.1.1 of the 1997 Edition (and earlier editions) of the NEHRP *Recommended Provisions*.

Section: 4.4.2.1

What is the appropriate range of parameters for upper and lower bounds of stiffness and capacity?

Section: 4.4.2.1

What quantitative guidance can be given for the classification of foundations as rigid or flexible with respect to the underlying soil?

Section: 4.4.2.1

Although rocking behavior is discussed in Section C4.4.2.1 of FEMA 274, no guidance is given for the inclusion of such behavior in the FEMA 273 procedures.

Section: 4.4

Discussion of Identified Issues

Effect of Soil Flexibility on Displacement Demands (4-1)

Concern has been expressed that the addition of foundation springs, if sufficiently flexible, can provide the necessary displacement capacity to reach the target displacement without exceeding structural deformation limits. Also, the applicability of the 25% limit in Section 3.2.6 to nonlinear procedures is not clear.

It should first be noted that the clarity of the guidelines for selection of appropriate SSI procedures defined in FEMA 273 has been lost in the translation to the prestandard. In particular, the translation lost important damping considerations and fails to clearly identify the SSI approach when the LDP is used. The discussion that follows is based on the original FEMA 273 guidelines.

Linear Procedures

For the linear procedures, only the changes in the elastic, dynamic system (period elongation and increased damping due to soil response) are considered. For the LSP, period elongation is considered using the simplified procedure defined in ASCE 7-98 (which was taken from the NEHRP *Recommended Provisions*). For the LDP, period elongation results from explicit modeling of the stiffness of each foundation element. In both procedures, the effective modal damping is based on the method outlined in the simplified procedure. Because linear analyses (with a rigid base for the LSP) are still prescribed in these cases and the consideration of soil response is approximate, limitations have traditionally been placed on the reduction in base shear (BSSC, 1997c and 1997d). However, the limits placed on the beneficial effects of SSI interaction in the NEHRP *Recommended Provisions* are noticeably less severe than those indicated in FEMA 273. In NEHRP *Recommended Provisions*, the reduction in base shear is limited to 30% of that from the fixed base solution, and no explicit variation of soil parameters is required. Section 3.2.6 of FEMA 273 limits the reduction of component and element actions to 25% of those from the fixed base solution and Section 4.4.2.1 also requires the consideration of upper and lower bound stiffness characteristics. This additional conservatism may be warranted, but some discussion should be provided in the commentary by the chapter authors.

It should also be noted that the limits placed on SSI analysis procedures for nuclear structures are based on parameter variation only; no additional limitation with respect to a fixed base analysis is provided (ASCE, 1986).

Nonlinear Procedures

When properly implemented, the nonlinear procedures include the variation of key parameters including soil strength and stiffness (per Section 4.4.2), gravity load magnitude (per Section 3.2.8), lateral load distribution (for NSP, per Section 3.3.3.1), and lateral load direction (for NSP, per Section 3.3.3.2). In general, increasing the flexibility of the foundation system increases the total displacement demand and decreases the displacement demand on lateral system structural elements. Depending on how the lateral and gravity systems are coupled, the displacement demands on the gravity system (for displacement compatibility) may increase, remain the same, or decrease. To the extent that the prescribed procedures are based on the best estimate of response and the required parameter variations are appropriate, the expected behavior should be bounded.

Concern that incompetent or unscrupulous designers will abuse the SSI provisions is probably unwarranted. Such designers are more likely to ignore requirements already contained in the prestandard. The process, as it is defined, seems to provide an appropriate characterization of the basic behavior and reasonable bounds to capture the expected variations.

Elastic Stiffness of Strip Footings (4-2)

The original issue focuses on the applicability of the circular footing spring stiffness solutions to strip footings; the length to width aspect ratio in the current procedure (Figure 4-3a) is limited to a maximum of four. In the course of this project, it was also noted that the range of embedment reflected in the embedment correction factors (Figure 4-3b) is too small for most practical problems.

Researchers have now developed spring stiffness solutions that are applicable to any solid basemat shape on the surface of, or partially or fully embedded in, a homogeneous halfspace (Gazetas, 1991a). Rectangular foundations are most common in buildings. Therefore, the general spring stiffness solutions were adapted to the general rectangular foundation problem, which includes rectangular strip footings. The results of this adaptation are described in the New Findings section of this report.

Force-Displacement Behavior for Nonlinear Analysis (4-3 and 4-4)

The primary issue raised was that the procedure for lateral soil springs could significantly overestimate the strength of the foundation elements. On a related topic, upon completion of the *Guidelines*, the BSSC identified the need to conduct additional research on characteristics of soils under short term loading. This need was perceived because geotechnical engineering has traditionally focused on the long-term force-displacement behavior of soils. In this report, load rate effects are discussed with other uncertainties in SSI analysis.

Nonlinear analyses that include soil-structure interaction effects must include the force-displacement behavior of foundations subjected to vertical and lateral forces. Therefore, FEMA 273 provides procedures for these calculations. The FEMA 273 approaches to vertical and lateral soil springs differ considerably. These differences are described in detail below.

Vertical Soil Springs

The approach taken by FEMA 273 to define the properties of vertical soil springs for shallow foundations is consistent with the available soil mechanics literature and appears to produce reasonable results. The approach is to define the foundation stiffness and strength. The stiffness is based on a footing embedded in an elastic half-space. The strength is based on the bearing capacity, which may be obtained by classical soil mechanics. The yield displacement is obtained using the calculated stiffness and strength.

Lateral Soil Springs

The basic approach of FEMA 273 for calculation of lateral soil spring properties is different from that taken for vertical bearing springs. Lateral stiffnesses are defined and an assumed yield displacement is stated. (The last paragraph of section 4.4.2.1B states, "The lateral capacity of a footing should [be] assumed to be attained when the displacements, considering both base traction and passive pressure stiffnesses, reaches 2% of the thickness of the footing.") The resulting capacities are not consistent with classical soil mechanics or with measured values. There are two sources of lateral strength and stiffness for shallow foundations: traction and passive pressure. Each will be discussed below.

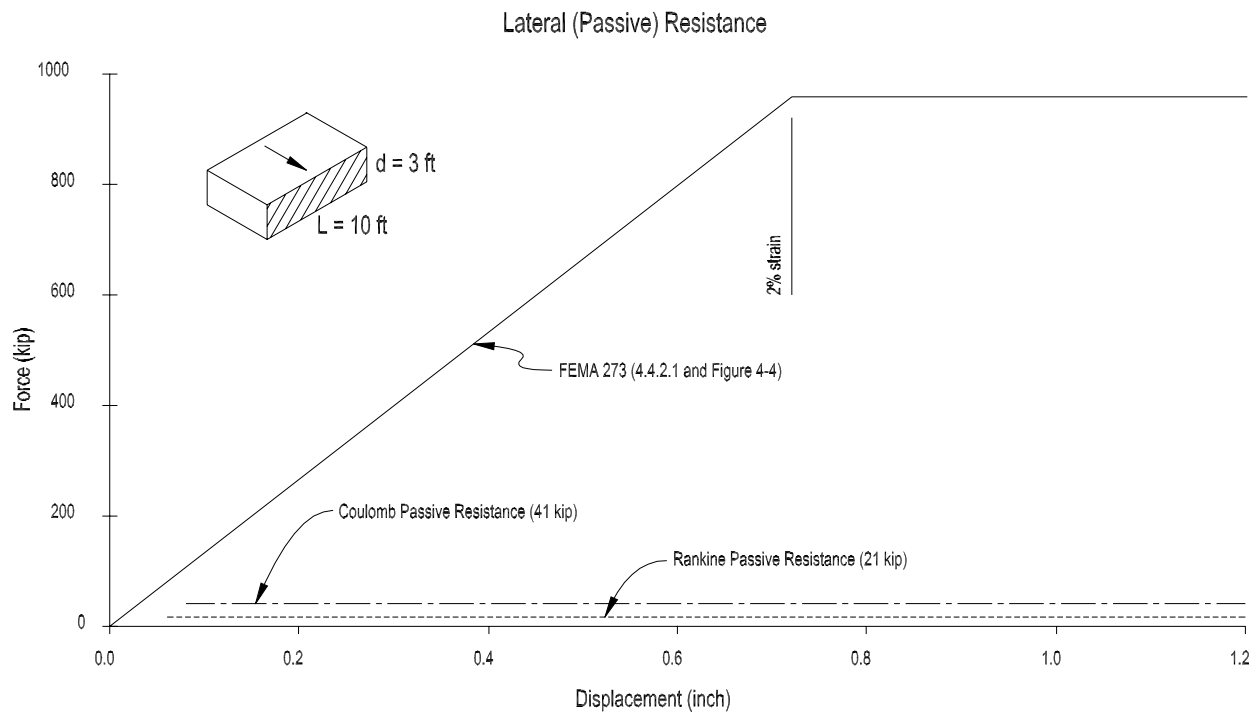
Traction. The stiffness defined for horizontal translation is based on a footing embedded in an elastic half-space. This characterization of the lateral traction stiffness of shallow foundations is consistent with accepted soil mechanics. However, the associated strength should also be based on soil mechanics. The shear strength of soil in force terms is given by $V = C + N \mu$; where C is the effective cohesion force (effective cohesion stress, c' , times footing area), N is the normal (compressive) force and μ is the coefficient of friction. The coefficient of friction is determined by considering the effective internal friction angle of the soil and the friction coefficient between soil and foundation. If the soil is cohesionless and there is no applied compression, the traction strength is exactly zero. In the approach recommended in FEMA 273, the traction "strength" is not a function of the applied compression; this is incorrect. Also, the amount of lateral displacement necessary to mobilize the traction strength of a foundation can be significantly less than or somewhat more than 2% of the thickness of the footing.

Passive Pressure. The lateral stiffness of a typical foundation element was evaluated using the FEMA 273 procedures and the maximum resistive capacity was checked using conventional soil mechanics procedures. The example analyzed was a shallow footing with a depth of 3 feet (d) and a length of 10 feet (L). It was assumed that the footing was located over a Site D soil profile which consisted of a dense sand (N value of 32, friction angle of 36 degrees, and a unit weight of 120 pcf) which had a corresponding average shear wave velocity of 900 fps and a Poisson's ratio of 0.35. Furthermore, it was assumed that the site had an EPA of 0.1 g (thus using the current Table 4-3, $G/Go = 0.5$).

Based on the site conditions described above and Figure 4-4 of FEMA 273, a lateral (passive only) stiffness value of 15,971 kip/ft was computed for the foundation. Then, the procedure in Section 4.4.2.1 was used to compute the ultimate resistance of the foundation (958 kip) corresponding to a foundation displacement of 2% of the footing depth (0.72 in.).

For comparison purposes, the maximum lateral (passive) capacity of the foundation was determined using the Rankine and Coulomb procedures for determining passive resistance. The computed Rankine and Coulomb resistance values were 21 and 41 kips, respectively.

As illustrated above, and also graphically depicted in the figure below, the FEMA 273 procedure for computing lateral resistance is unconservative and may over predict the maximum lateral resistance by a factor of more than 20. To be consistent with the procedures used for calculating the vertical spring constants, FEMA 273 should be modified to base the maximum lateral (passive) capacity of foundations on limiting passive resistance values determined using either the Rankine or Coulomb equations. The recommended procedure is described in more detail in the New Findings section of this report.



Shear Modulus Reduction

The FEMA 273 methodology is based on using the best estimate of expected performance (and considering variations in response as needed). Therefore, estimation of the effective shear modulus of soils in the zone of influence beneath foundations is an important issue. Because the recommended modulus reduction factors found in FEMA 273 differ from those in the NEHRP *Recommended Provisions* (BSSC, 1997c) and ATC 40 (ATC, 1996a), by about a factor of two at both low and high levels of peak ground acceleration, it was unclear whether either of these sets of recommendations was appropriate.

It might be inferred that the values tabulated in ATC 40 and the NEHRP *Recommended Provisions* are more accurate than those in FEMA 273 since more values are reported. However, the Commentary to the NEHRP *Recommended Provisions* says “it should be emphasized that the values in Table 5.5.2.1.1 are first order approximations.” It should also be noted that this table has remained unchanged since it was first published in ATC 3-06 (ATC, 1978); evidently peak ground accelerations in excess of 0.4 g were not considered in its development. It is also unclear whether the “conservative value of v_s/v_{s0} ” used in the development of ATC 3-06 corresponds to a lower bound or upper bound estimate of the soil stiffness. In contrast, Table 4-3 in FEMA 273 reflects a wider range of peak ground accelerations, but contains only two data points. The assumptions made in developing the FEMA 273 table are not documented, and the subsequent ATC 40 project reverted to the values in the NEHRP *Recommended Provisions*.

The recommendations of both documents contain two significant weaknesses. First, as the peak ground acceleration approaches zero, the modulus reduction factor should approach unity. Second, shear modulus reduction is very sensitive to the initial modulus; for a given shear stress, softer soils experience larger strains which, in turn, cause a more pronounced reduction in effective modulus.

Due to the insensitivity of both sets of recommendations to important parameters and the significant differences in the recommendations, this topic was examined in considerably more detail. The results of this work are reported in the New Findings section of this report.

Uncertainties in SSI Analysis

The text of FEMA 273 requires consideration of values ranging from 0.5 to 2 times the expected values for both strength and stiffness. FEMA 274 cites an example in which the range considered varied from 0.67 to 1.5 times the expected values. The concern is two-fold. First, what further guidance can be given for the range; that is, what is appropriate? Second, what considerations are necessary to assure that the starting value is the “average” so that application of the prescribed range produces the desired effect?

The various sources of uncertainty, along with additional recommendations, are discussed in the New Findings section of this report.

Foundation/Soil Stiffness Classification

At the PAC/PT meeting held on June 23, 1999, the project Principal Investigator and the FEMA Consultant requested that the text of the prestandard be revised to provide additional guidance regarding the classification of strip and mat foundations as rigid with respect to the soil.

During translation of FEMA 273 into the First Draft of the prestandard, an error appeared. Recommended corrections and new classification provisions are described in the New Findings section of this report.

Rocking Behavior

There are closed form rocking solutions that have been applied to the seismic problem. However, when using FEMA 273, only the nonlinear procedures can accommodate rocking response. The two questions raised are: 1) Can rocking response be incorporated in the LSP approach?, and 2) Can more guidance be provided for the consideration of rocking in the NSP approach?

An overview of the available research and a recommended approach are provided in the New Findings section of this report.

New Findings

Elastic Stiffness for Rectangular Foundations

Chapter 15 of the *Foundation Engineering Handbook* (Gazetas, 1991) was written by George Gazetas and addresses “Foundation Vibrations.” The chapter contains formulas and graphs for arbitrarily shaped surface and embedded foundations on or in a homogeneous halfspace. This information is based on theoretical and analytical work in the 1970s and 1980s by Gazetas, Dobry, Fotopoulou, Lysmer, Veletsos, Luco, Roesset, Kausel, and others and has been compared with measured values (Gazetas, 1991c).

For use in this prestandard, the solutions for rectangular foundations with dimensions L and B are presented in Appendix B. It is recommended that this figure replace the current Figure 4-2. In general, a two step calculation process is required. First, the stiffness terms are calculated for a foundation at the surface. Then, an embedment correction factor is calculated for each stiffness term. The stiffness of the embedded foundation is the product of these two terms. [Appendix C also contains figures that illustrate the effects of foundation aspect ratio and embedment. Such figures were requested by members of the Project Team at the June 23, 1999 meeting.]

The surface stiffness equations are specific to rectangular foundations (Pais and Kausel, 1988). These equations were chosen over an adaptation of the general solution because they are somewhat simpler. The solutions were modified to apply to rectangular foundations with dimensions L and B , rather than the dimensions $2L$ and $2B$ used by the authors. Pais and Kausel report that the largest error to be expected is “less than a few percent.”

The embedment correction factor equations are based on an adaptation of the general solutions (Gazetas, 1991a and 1991b). The general solutions were modified to apply to rectangular foundations with dimensions L and B , while the original work applied to arbitrarily-shaped foundations circumscribed by a rectangle with dimensions $2L$ and $2B$. Gazetas reports, “Simplicity without any serious compromise in accuracy has been the prime goal when developing these tables. It is believed that, in general, the errors that may result from their use will be well within an acceptable 15 percent.” Gazetas indicates that the height of effective sidewall contact, d , should be taken as “the (average) height of the sidewall that is in ‘good’ contact with the surrounding soil. [It] should, in general, be smaller than the nominal [height] of contact to account for such phenomena as slippage and separation that may occur near the ground surface. Note that ... d will not necessarily attain a single value for all modes of oscillation.” When d is taken larger than zero, the resulting stiffness includes sidewall friction and passive pressure contributions.

Usability Test

The usability of this new approach was tested against the method currently prescribed in FEMA 273. Two engineers who had never performed foundation stiffness calculations and were unfamiliar with FEMA 273 were selected for this test. They were asked to calculate (by hand) foundation stiffnesses for all six degrees of freedom for three typical foundations using both methods. Measures were taken so that they were unaware of the sources of the two methods. They were asked to track the time required to perform each solution for each foundation. The order of solution was varied to avoid penalizing the current FEMA 273 solution method for the time needed to become familiar with each design problem. The users were also asked to comment on issues that arose and to indicate their preferred solution method. The three test problems and their correct solutions (using the recommended equations) are provided in Appendix C.

The results of the usability test are summarized in the table below. It can be seen that the recommended rectangular foundation solution is slightly less time-consuming for hand calculations, lends itself to spreadsheet use, is more generally applicable to the range of foundation conditions that are encountered, and produces more consistent results for different users. On this last point, it should be noted that while all users who correctly apply the rectangular equations would get the same results, the calculations based on the current FEMA 273 approach varied significantly due to table reading and extrapolation.

Solution Times¹ and Results of Usability Study

	User 1		User 2	
	Method A ²	Method B ²	Method A ²	Method B ²
Fdn 1 ³	35 min	25 min	40 min	50 min
Fdn 2 ⁴	25 min	20 min	30 min	25 min
Fdn 3 ⁵	30 min	20 min	30 min	20 min
Preferred method	Method B; would use spreadsheet for multiple conditions		Method B; would use spreadsheet for multiple conditions	
User Comments	Method B is "perfect" for spreadsheet solutions. "Method A is tedious to use when L/B > 4 or D/R > 0.5."		Method B "lends itself to spreadsheet" use. The "circle approximation seems shaky at large aspect ratios, which is a possible reason for the insufficient chart range."	

Notes

- ¹ Solutions with time in bold type were performed first.
- ² Method A is the approach currently prescribed in FEMA 273. Method B is the recommended solution, based on the equations presented by Gazetas.
- ³ Using Method A, minor extrapolation is required for D/R.
- ⁴ Using Method A, significant extrapolation is required for D/R.
- ⁵ Using Method A, moderate extrapolation is required for both L/B and D/R.

Lateral Spring Calculations

We recommend that the force-displacement behavior of lateral soil springs be calculated using the stiffness and strength obtained using established principles of soil mechanics.

Stiffness

For shallow foundations, the stiffness may be calculated using the solutions for footings embedded in an elastic half-space. The shear modulus should be reduced for the effects of large strains. The (shallow) rectangular footing stiffness solutions recommended above include the contributions to stiffness from base traction, sidewall friction, and passive pressure at the leading face. For shallow foundations, passive pressure resistance generally accounts for much less than half of the total strength. Therefore, it is adequate to characterize the nonlinear response as elastic-perfectly plastic using the initial, effective stiffness and the expected strength. Based on a parameter study (details are provided in Appendix C), the actual behavior should fall within the upper and lower bounds prescribed in the prestandard.

The total lateral stiffness of a pile group should include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral stiffness of piles should be based on classical or analytical methods. As the passive pressure resistance may be a significant part of the total strength and deep foundations often require larger lateral displacements than shallow foundations to mobilize the expected strength, it may not be appropriate to base the force-displacement response on the initial, effective stiffness alone. Instead, the contribution of passive pressure should be based on the passive pressure mobilization curve provided in Appendix B. It is recommended that this figure replace the current Figure 4-4.

Strength

For shallow foundations, the calculated strength should include traction at the bottom (and optionally at the sides parallel to motion) and passive pressure resistance on the leading face. The base traction strength is given by $V = C + N \mu$; where C is the effective cohesion force (effective cohesion stress, c' , times footing base area), N is the normal (compressive) force and μ is the coefficient of friction. Side traction is calculated in a similar manner. The coefficient of friction is often specified by the geotechnical consultant. In the absence of such a recommendation, μ may be based on the minimum of the effective internal friction angle of the soil and the friction coefficient between soil and foundation from published foundation references. The ultimate passive pressure strength is often specified by the geotechnical consultant in the form of passive pressure coefficients or equivalent fluid pressures. The passive pressure problem has been extensively investigated for more than two hundred years. As a result, countless solutions and recommendations exist (Terzaghi, Peck, and Mesri, 1996; Bowles, 1988; Martin and Yan, 1995). The method used should, at a minimum, include the contributions of internal friction and cohesion, as appropriate.

The lateral strength of deep foundations includes the contributions of individual piles or piers and the pile cap. The passive strength should be determined as described above for shallow foundations. The lateral strength of piles or piers may be determined by the same methods used to calculate their stiffness, with appropriate modification for yielding if it is anticipated.

Shear Modulus Reduction

The relationship between shear modulus reduction and peak ground acceleration was re-examined. The goals of this effort were

- to reflect recent research on the subject,
- to examine the sensitivity to realistic variations of the key parameters,
- to be consistent with the expected response such that consideration of upper and lower bounds within a factor of two would be appropriate,
- to reflect the softening of soils due to both free-field response and inertial interaction, and
- to produce provisions that are not unduly complex.

Parameters (and Ranges) Considered

Parameter	Range	Typical value ¹	Discussion
PGA	0 to 0.8 g	varies	This is the primary independent variable.
v_s	by Site Class	by Site Class	This is the most significant secondary variable. The range is that used in the Site Class definitions.
Average depth, h	5 to 50 ft	20 ft	This should be representative of the zone of influence (Gazetas, 1991a) which differs with direction. The typical value chosen is consistent with the result of integration of the influence depth for shallow footings with practical dimensions.
Soil weight density, γ	90 to 150 pcf	110 pcf	The effect of weight density variation was found to be negligible.
Surcharge	0 to 3 ksf	0	Surcharge pressures increase the free-field shear stress, but also increase the confining stress. The net effect is slight enough to be ignored.
At-rest pressure coefficient, K_0	0.3 to 1.0	0.5	Values of 0.4 to 0.8 are "usual" (Bowles, 1988). Over-consolidation can produce larger values (Perloff and Baron, 1976).
Inertial effects	$\tau=1$ to 2 times $\tau_{\text{free-field}}$	$\tau=1.5 \tau_{\text{free-field}}$	Approximates phasing of response.

¹ When the results are sensitive to the parameter, the typical values are taken near the middle of the range. When the results are found to be insensitive to the parameter, a convenient value is chosen.

The procedure followed in developing a relationship between shear modulus reduction and peak ground acceleration is as follows. For a given set of parameters (v_s , h , γ , surcharge, K_0 , and τ multiplier) and the full range of accelerations considered,

- Calculate the maximum shear stress at the surface corresponding to rigid body movement of the soil column,
- Modify this value to reflect the average cyclic shear stress at the representative depth (Seed and Idriss, 1982),
- Increase the average cyclic shear stress to approximate inertial effects,
- Solve for average cyclic shear strain and the corresponding modulus reduction factor (by iteration) using Ishibashi's modulus reduction equation (Ishibashi and Zhang, 1993). This modulus reduction equation reflects the expected condition and is consistent with the findings of other researchers (Kramer, 1996; Vucetic and Dobry, 1991; Ishibashi, 1992).

Based on the results obtained using the procedure described above, we suggest that Table 4-3 be replaced with the following table. The recommended values are discussed in more detail below. The new table is consistent with the site amplification tables (Tables 2-4 and 2-5) in two important ways. First, the layout and level of complexity is identical. Second, the indication of problem soils that require site-specific investigation (Site Class E with strong shaking and all of Site Class F) is consistent. It should be noted that the new table does not provide ratios of effective shear wave velocity because 1) such values are not used in subsequent calculations, and 2) the user may recreate this information using Equation 4-6.

Effective Shear Modulus Ratio (G/G_0)				
Site Class	Effective Peak Acceleration, $S_{XS}/2.5$			
	$S_{XS}/2.5 = 0$	$S_{XS}/2.5 = 0.1$	$S_{XS}/2.5 = 0.4$	$S_{XS}/2.5 = 0.8$
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	*
F	*	*	*	*

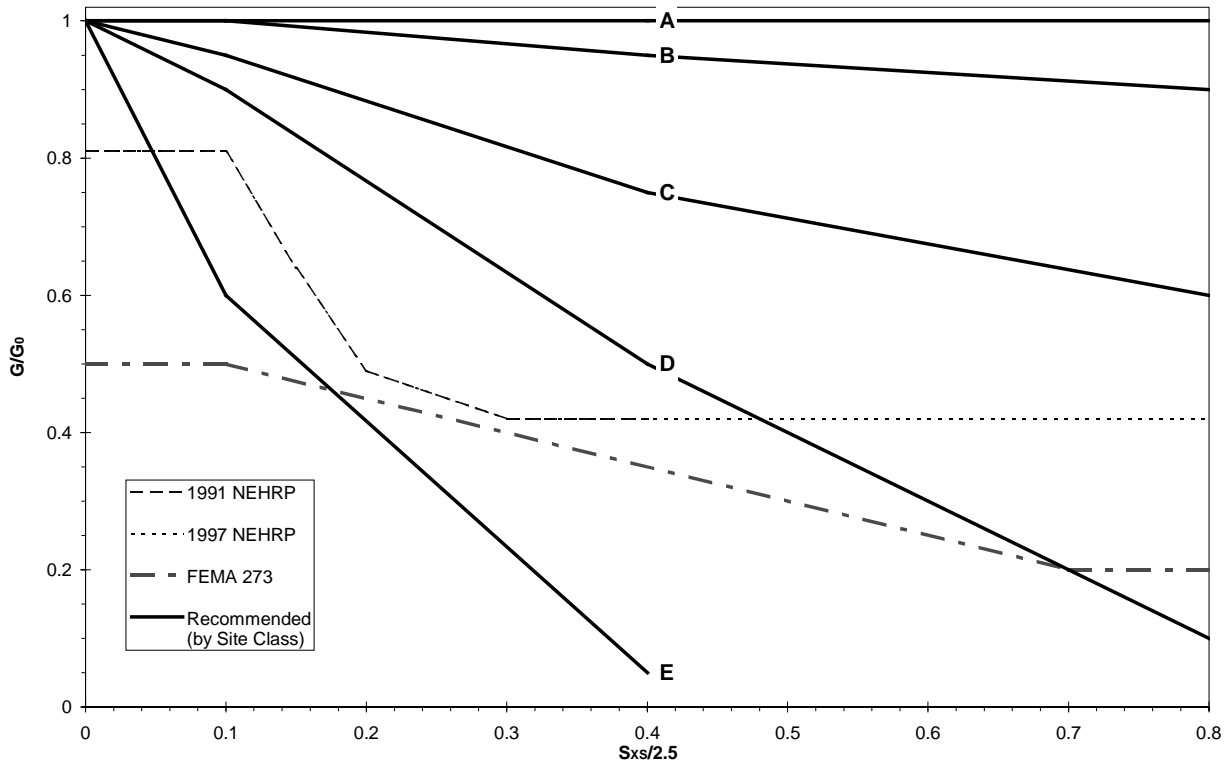
NOTE: Use straight-line interpolation for intermediate values of $S_{XS}/2.5$.

*Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

The recommended shear modulus reduction curves are compared with the values currently specified in both FEMA 273 and the NEHRP *Recommended Provisions* in the figure below. The following observations may be made.

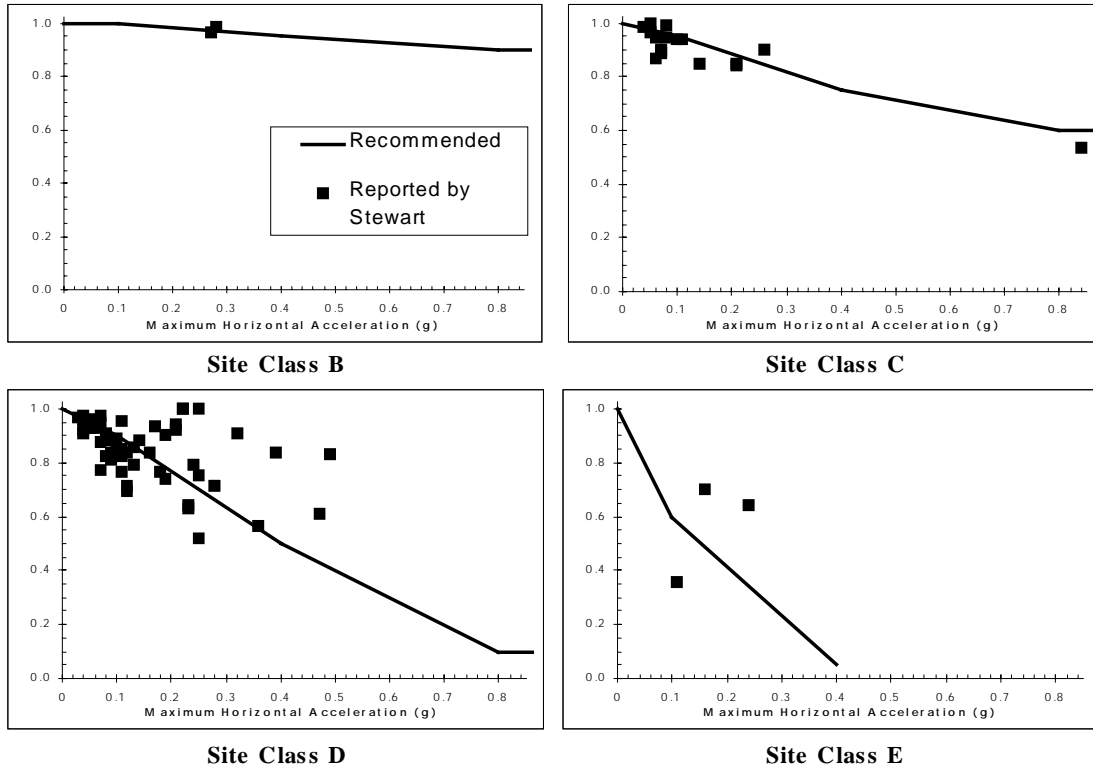
- as the peak ground acceleration approaches zero, the modulus reduction factor approaches unity,

Shear Modulus Reduction Factor v. Peak Ground Acceleration



- modulus reduction effects are significantly more pronounced for softer soils, and
- the modulus reduction factors given in both FEMA 273 and the NEHRP *Recommended Provisions* overestimate the modulus reduction effects for Site Classes A, B, and C.

Representative results from the parameter variation studies are provided in Appendix C. The variability increases as the initial shear wave velocity decreases; that is, wider variations should be expected for softer soils. For the ranges of parameters considered, the variation in the final result is generally within a factor of two of the recommended values. The figure below compares the recommended values with measured results reported by Stewart (Stewart, 1998).



Uncertainties in SSI Analysis

There are several sources of uncertainty in the soil-structure interaction analyses outlined in this prestandard. The current approach is to vary the calculated foundation strength and stiffness between upper and lower bound estimates based on twice and half the values defined as “expected.” This approach is intended to account for variations in response due to

- rate of loading,
- assumed elasto-plastic soil behavior,
- level of strain,
- cyclic loading, and
- variability of soil properties.

Rate of Loading

According to Gazetas (1991a), "For all soils, cohesionless and cohesive, the frequency, or the rate of loading, has no practical effect on G_{max} . This means that soil is basically not a viscous, but rather a hysteretic, material."

Liquefaction is a special case of strength loss due to rate of loading, cyclic response, and other characteristics. However, it is treated separately in FEMA 273.

There is published research concerning the rate dependence of soil strength. Some of this research as it applies to the bearing capacity of soils is summarized by Das (Das, 1999). The following observations are made concerning the dynamic bearing capacity versus the static bearing capacity:

- for dry sands, varies between 0.67 and 1.0
- for submerged sands, varies between 0.7 and about 1.4
- for cohesive soils, varies between 1.0 and 1.5

Assumed Elasto-Plastic Behavior

This is discussed in considerable detail in FEMA 274.

Level of Strain

As calculated in FEMA 273, the effective shear modulus could vary from the expected value by a factor of 5 and the ultimate passive resistance could be overestimated by a factor of 20. In the context of the upper and lower bounds prescribed by FEMA 273, this amount of variability is unacceptable. Therefore, changes are proposed above (for the calculation of effective shear modulus and passive pressure resistance) that produce results that are within the defined bounds.

Cyclic Loading

Silty soils may degrade and loose sands may densify due to cyclic loading. However, these effects are not generally significant. Some discussion of these effects is already provided in Section C4.4.

Variability of Soil Properties

Soil is not an engineered product. Natural variability of soil characteristics is one of the most significant sources of uncertainty. However, this variation is best considered along with all other uncertainties.

Each of the sources of variability considered above produce results that are generally within a factor of two above or below the expected value. It is conceivable that certain conditions will fall outside the bounds prescribed in FEMA 273. However, it is not the objective to guarantee that the answer is always within the applied factor. Instead, the intent is that 1) solution sensitivity be identified, and 2) that the bounds considered reasonably capture the expected behavior. Current practice (both conventional and within the nuclear industry) has suggested that variation by a factor of two is generally appropriate. Geotechnical engineers often use a safety factor of two to establish lower bound values for use in design. Another good measure of overall variability is provided by ASCE 4. This standard for the seismic analysis of nuclear structures says in Section 3.3.1.7,

The uncertainties in the SSI analysis shall be considered. In lieu of a probabilistic evaluation of uncertainties, an acceptable method to account for uncertainties in SSI analysis is to vary the soil shear modulus. Soil shear modulus shall be varied between the best estimate value times $(1 + C_v)$ and the best estimate value divided by $(1 + C_v)$, where C_v is a factor that accounts for uncertainties in the SSI analysis and soil properties. The minimum value of C_v shall be 0.5.

It is recommended that this prestandard continue to prescribe variation by a factor of two. The commentary could note (consistent with the ASCE 4 approach) that if additional testing is performed, the range could be narrowed to that defined by multiplying and dividing by $(1 + C_v)$, but not less than 1.5. The coefficient of variation, C_v , would be defined as the standard deviation divided by the mean.

The commentary should caution geotechnical engineers that truly average results should be reported and that the actual factor of safety applied to arrive at design values be reported. The design values recommended by geotechnical engineers are generally consistent with the lower bound. If such reduced values are used by the structural engineer as expected values, the application of the prescribed upper and lower bound variations will not achieve the intended aim.

Foundation/Soil Stiffness Classification

Equation 4-8 in FEMA 273 provides a transition point between foundation behavior that may be considered rigid and that which should include explicit consideration of foundation flexibility. Unfortunately, in the translation of FEMA 273 to the prestandard, this equation was mistakenly applied as a transition point between methods 2 and 3. Regarding Equation 4-8, it should be noted that it applies to a very specific case, and it is more stringent than traditional recommendations (NAVFAC, 1986b; Bowles, 1988).

The shears and moments in foundation elements are conservative when such elements are considered rigid. However, soil pressures may be significantly underestimated when foundation flexibility is ignored. In resolving this issue, the text of the standard should not isolate one specific case. Instead the approach should be performance driven. The flexibility and nonlinear response of soil and of foundation structures should be considered when the acceptability (results) would change. The following two specific cases could be included in the commentary.

For beams on elastic supports (for instance, strip footings and grade beams) with a point load at midspan, the beam may be considered rigid when

$$\frac{EI}{L^4} > \frac{2}{3} k_{sv} B$$

The above equation is generally consistent with traditional beam-on-elastic foundation limits (NAVFAC, 1986b; Bowles, 1988). The resulting soil bearing pressures are within 3% of the results including foundation flexibility.

For rectangular plates (with plan dimensions L and B , and thickness t , and mechanical properties E_f and ν_f) on elastic supports (for instance, mat foundations or isolated footings) subjected to a point load in the center, the foundation may be considered rigid when

$$4k_{sv} \sum_{m=1}^5 \sum_{n=1}^5 \frac{\sin^2\left(\frac{m \cdot \pi}{2}\right) \sin^2\left(\frac{n \cdot \pi}{2}\right)}{\left[\pi^4 D_f \left(\frac{m^2}{L^2} + \frac{n^2}{B^2}\right)^2\right] + k_{sv}} < 0.03$$

where,

$$D_f = \frac{E_f t^3}{12(1 - \nu_f)^2}$$

The above equation is based on Timoshenko's solutions for plates on elastic foundations (Timoshenko, 1959). The general solution has been simplified by restriction to a center load. Only the first five values of m and n (in the infinite series) are required to achieve reasonable accuracy.

Rocking Behavior

Motivated by observations following the Chilean earthquake in May of 1960, George Housner undertook a theoretical study of the behavior of rocking structures (Housner, 1963). Housner addressed

- free vibration response (period, amplitude, and energy reduction),
- overturning due to a constant acceleration,
- overturning due to a half-sine acceleration pulse, and
- overturning due to earthquake motion.

A later study (Priestley, 1978) provided experimental verification of Housner's work and extended Housner's study by relating the energy reduction factor, r , with equivalent viscous damping. This paper also presented a response spectrum design approach for rocking structures. The experimental results indicate that

- Housner's theoretical equations for frequency and amplitude are correct, and
- Housner's assumption of perfectly inelastic collisions during rocking overestimates the actual energy reduction (and corresponding viscous damping).

A recent study (Makris, 1998) focused on the rocking response of equipment to impulsive horizontal accelerations. The motions addressed include

- half-sine pulses (an error in Housner's solution is identified),
- one-sine pulses,
- one-cosine pulses,
- various other cycloidal pulses, and
- seismic excitation.

Based on these findings, a design approach to the rocking problem is outlined below. Consideration of two types of behavior are recommended. First, a response spectrum design method that may be used to predict the displacement response of the rocking system is outlined. Second, users are referred to Makris's work to investigate the response to acceleration pulses.

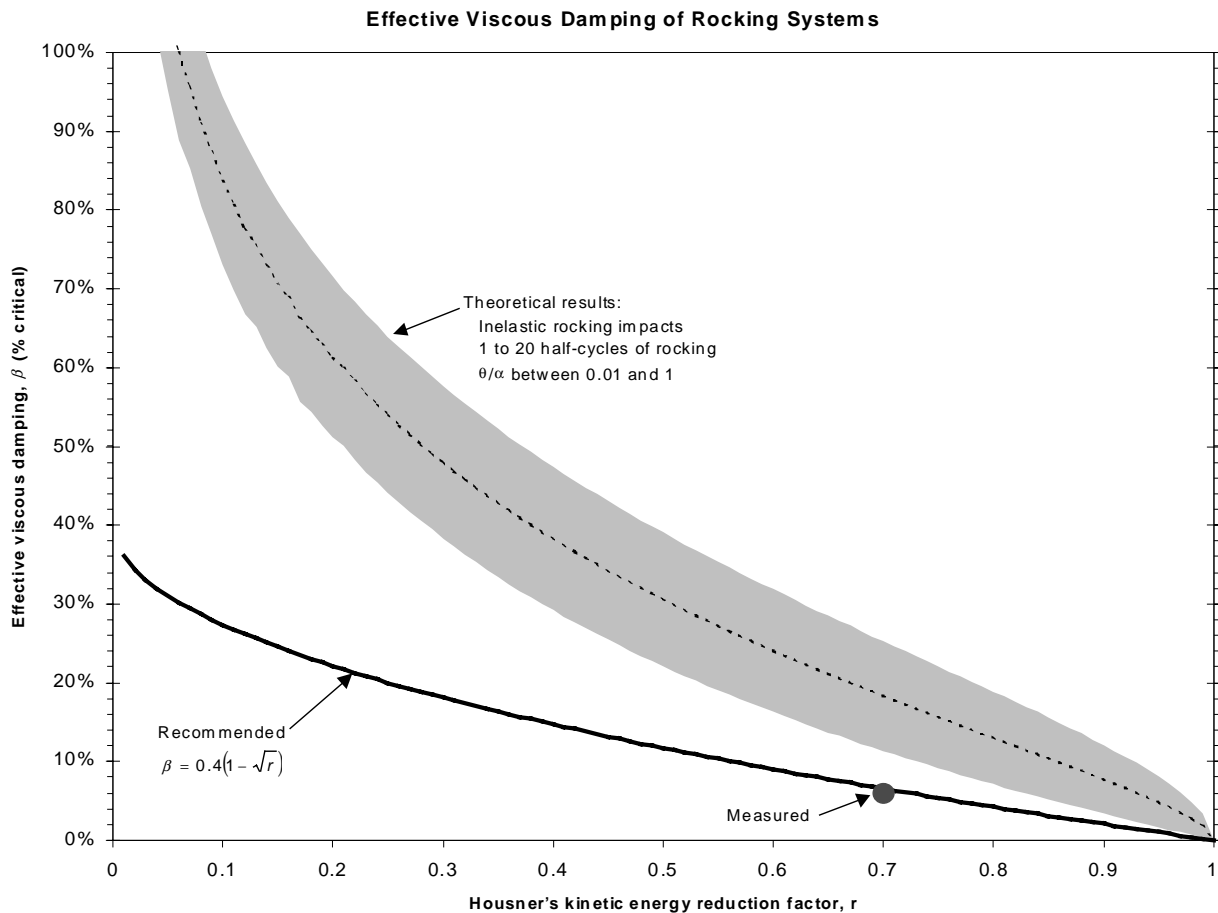
The response spectrum design method involves the following steps:

- calculate the mass, weight, and center of gravity for the rocking system (or subsystem);
- calculate the soil contact area, center of contact, and rocking system dimension, R ;
- determine whether rocking will initiate,
- calculate the effective viscous damping of the rocking system (and the corresponding design displacement spectrum);
- calculate (graphically or iteratively) the period and amplitude of rocking (the solution will not converge if overturning will occur--that is, when $\theta > \alpha$).

A one-page outline of the response spectrum design approach is provided in Appendix C. Priestley's experimental work demonstrated that Housner's approach can overestimate the energy reduction of the system. The figure below shows the relationship between Housner's kinetic energy reduction factor, r , and the effective viscous damping of the system, β (as a fraction of critical damping). For the range of system properties considered, Housner's approach produces values in the shaded region. The results measured by Priestley are also shown. The simple recommended equation has no theoretical basis. Instead, it was chosen because it:

- matches Priestley's experimental results;
- reflects low levels of damping, as expected, for slender structures (Hadjian, 1998),
- corresponds to about half the damping from Housner's approach,
- provides less pronounced increases in damping for very squat structures which have not been thoroughly investigated (Priestley, 1978), and

- produces values within the range of Table 2-6 of FEMA 273.



The procedure outlined above can be adapted for the determination of the target displacement for NSP analyses of rocking structures. The results also indicate period elongation and effective damping that may be included in LSP analyses, although it should be noted that the above-described solutions for the inclusion of soil flexibility and structural rocking are mutually exclusive; these two forms of soil-structure interaction should not be considered to occur simultaneously.

An example of both portions of the recommended rocking design approach is provided in Appendix C. This example is an adaptation of Priestley's example, modified as follows:

- uses U.S. Customary units,
- the design spectrum is based on $S_{XS} = 0.9$ and $S_{XI} = 0.4$ (instead of the El Centro spectra), and
- the viscous damping is calculated using the recommended equation (rather than an arbitrary increase in Housner's r value).

Summary of Recommendations

Two items that require clarification by the Chapter 3 author(s) have been identified. First, in the translation of FEMA 273 into the prestandard, clarity of the guidelines for selection of appropriate SSI methods (for the various analysis procedures) has been lost. Second, additional discussion of the limitations placed on the beneficial effects of soil-structure interaction should be provided. In particular, it is recommended that the 25% rule of Section 3.2.6 not be applied to the nonlinear procedures and the increased conservatism for the LSP of FEMA 273 versus the ELF of the NEHRP *Recommended Provisions* should be explained.

No additional limitations on the beneficial effects of soil-structure interaction are needed for the Nonlinear Static Procedure. The parameter variations required in Chapters 3 and 4 are expected to bound the response.

New stiffness solutions for shallow foundations that are applicable to all rectangular foundations (including strip footings) are presented. This revision requires the replacement of Figures 4-2 and 4-3.

Revisions to the calculation procedure for the force-deformation response of lateral soil springs are proposed. These changes produce results for which the stiffness and strength of shallow and deep foundations are consistent with accepted procedures of soil mechanics. In particular, the calculation of lateral strength due to passive pressure and base traction are revised.

New effective shear modulus factors are proposed. These new factors are based on research conducted during the course of this project. A revised version of Table 4-3 is presented. The revised table is consistent with recent research and the soil amplification tables found in Chapter 2 of FEMA 273.

The range of variation required in Chapter 4 for strength and stiffness parameters (multiplication and division by a factor of two) is reaffirmed. A provision for a slightly relaxed range of parameters is recommended when additional testing is provided.

Recommendations are made regarding the classification of the relative stiffnesses of foundations and the supporting soils. Additional guidance (for inclusion in the Commentary) is provided for two-way foundation components.

Rocking behavior is examined and a rocking design approach suitable for incorporation into commentary is proposed.

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Appendix A
Revised Text and Commentary for FEMA 273 Prestandard

Underscored and struck text is not included here for brevity.

Appendix B
Revised Figures for FEMA 273 Prestandard

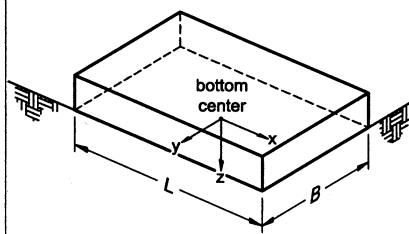
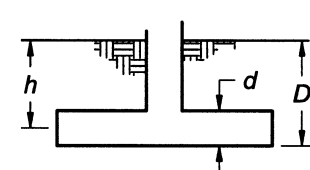
Degree of freedom	Stiffness of foundation at surface	Note:
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-\nu} \left[3.4 \left(\frac{L}{B} \right)^{0.65} + 1.2 \right]$	 <p>Orient axes such that $L \geq B$</p>
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-\nu} \left[3.4 \left(\frac{L}{B} \right)^{0.65} + 0.4 \frac{L}{B} + 0.8 \right]$	
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-\nu} \left[1.55 \left(\frac{L}{B} \right)^{0.75} + 0.8 \right]$	
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-\nu} \left[0.4 \left(\frac{L}{B} \right) + 0.1 \right]$	
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-\nu} \left[0.47 \left(\frac{L}{B} \right)^{2.4} + 0.034 \right]$	
Torsion about z-axis	$K_{zz,sur} = GB^3 \left[0.53 \left(\frac{L}{B} \right)^{2.45} + 0.51 \right]$	
Degree of freedom	Correction factor for embedment	Note:
Translation along x-axis	$\beta_x = \left(1 + 0.21 \sqrt{\frac{D}{B}} \right) \cdot \left[1 + 1.6 \left(\frac{hd(B+L)}{BL^2} \right)^{0.4} \right]$	 <p>d = height of effective sidewall contact (may be less than total foundation height) h = depth to centroid of effective sidewall contact</p> <p>For each degree of freedom, calculate $K_{emb} = \beta K_{sur}$</p>
Translation along y-axis	$\beta_y = \beta_x$	
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21} \frac{D}{B} \left(2 + 2.6 \frac{B}{L} \right) \right] \cdot \left[1 + 0.32 \left(\frac{d(B+L)}{BL} \right)^{2/3} \right]$	
Rocking about x-axis	$\beta_{xx} = 1 + 2.5 \frac{d}{B} \left[1 + \frac{2d}{B} \left(\frac{d}{D} \right)^{-0.2} \sqrt{\frac{B}{L}} \right]$	
Rocking about y-axis	$\beta_{yy} = 1 + 1.4 \left(\frac{d}{L} \right)^{0.6} \left[1.5 + 3.7 \left(\frac{d}{L} \right)^{1.9} \left(\frac{d}{D} \right)^{-0.6} \right]$	
Torsion about z-axis	$\beta_{zz} = 1 + 2.6 \left(1 + \frac{B}{L} \right) \left(\frac{d}{B} \right)^{0.9}$	

Figure 4-2 Elastic Solutions for Rigid Footing Spring Constants

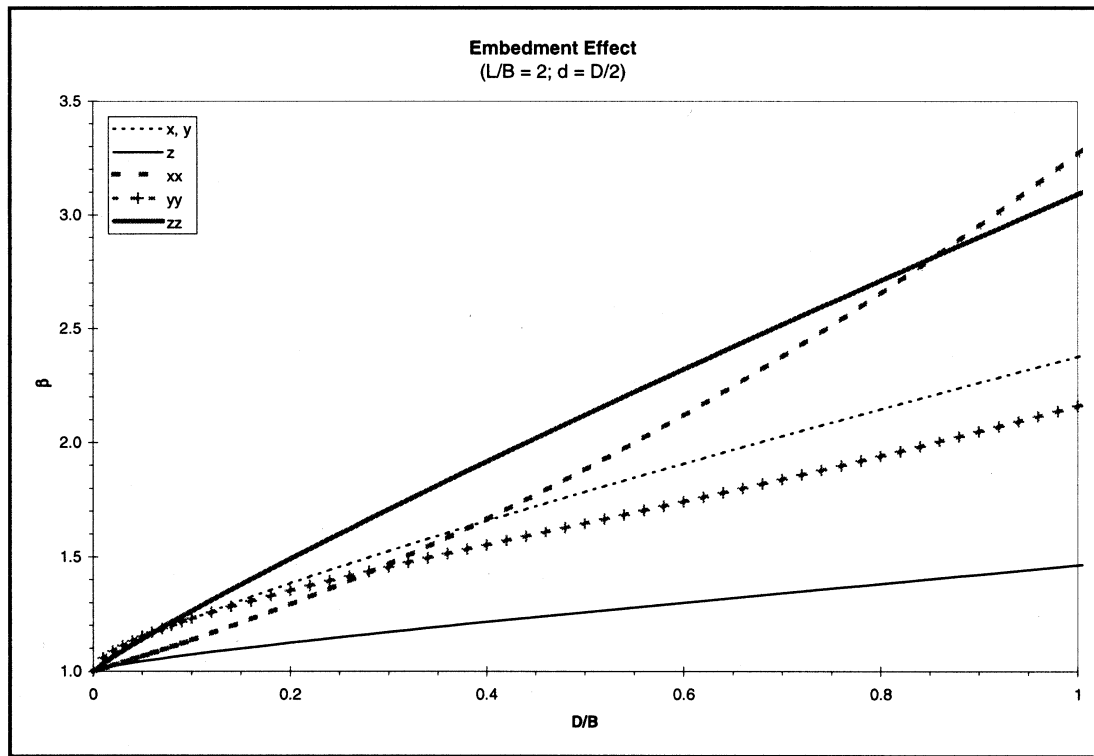
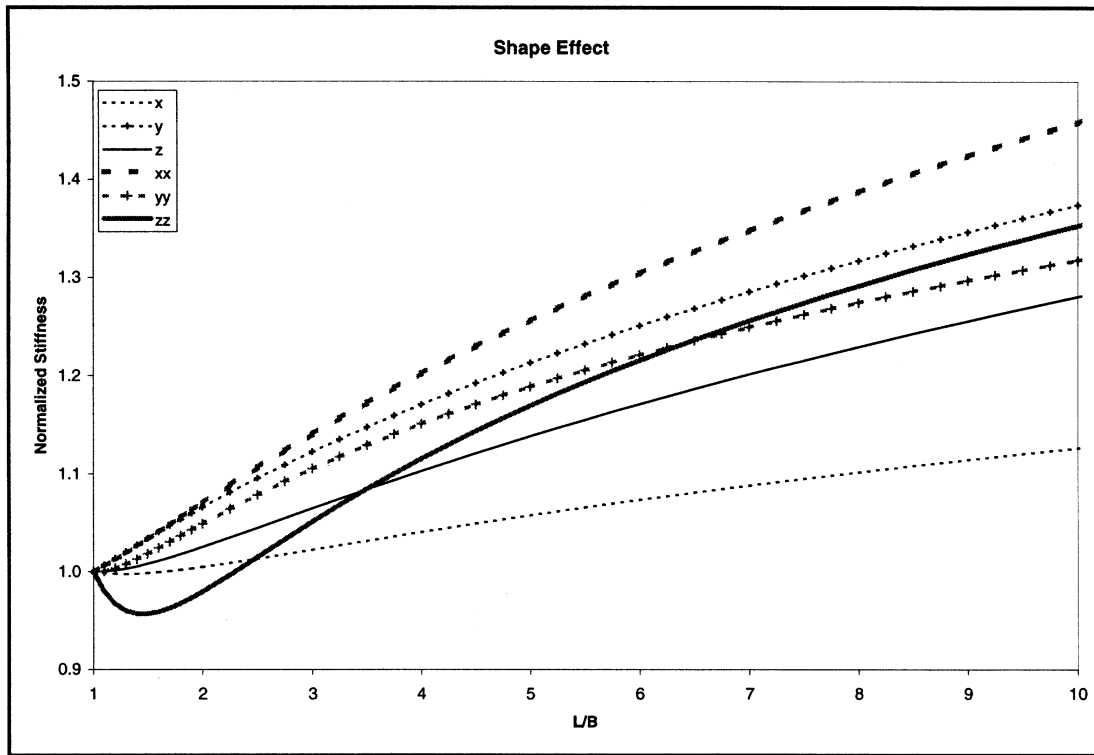


Figure 4-3 (a) Foundation Shape Effect (b) Foundation Embedment Effect

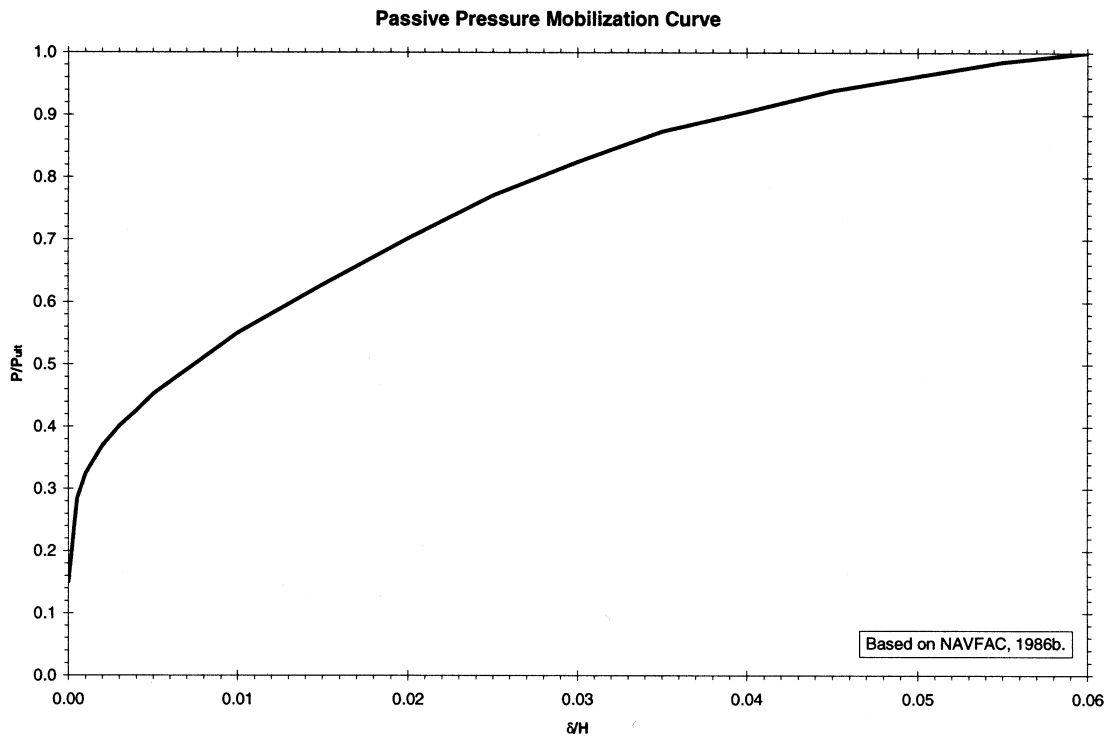


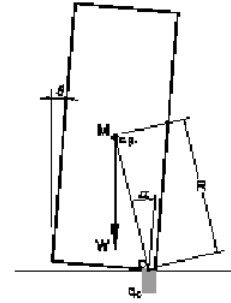
Figure 4-4 Passive Pressure Mobilization Curve

Appendix C

Revised Figures for FEMA 273 Prestandard

Mass, weight, and center of gravity:

Note that, in general, the mass and weight will not be consistent with each other. The mass, M , is the total seismic mass tributary to the wall. The weight, W , is the vertical gravity load reaction. For the purposes of these calculations, the vertical location of the center of gravity is taken at the vertical center of the seismic mass and the horizontal location of the center of gravity is taken at the horizontal center of the applied gravity loads.



Soil contact area and center of contact:

The soil contact area is taken as W/q_c . The wall rocks about point O located at the center of the contact area.

Wall rocking potential:

Determine whether the wall will rock by comparing the overturning moment to the restoring moment. For this calculation, S_a is based on the fundamental, elastic (no-rocking) period of the wall. The wall will rock if $S_a > (W/Mg)\tan \alpha$. If rocking is not indicated, discontinue these calculations.

Rocking calculations:

Calculate I_O , the mass moment of inertia of the rocking system about point O.

Calculate the effective viscous damping, β , of the rocking system as follows:

$$\beta = 0.4(1 - \sqrt{r}) \quad \text{where} \quad r = \left[1 - \frac{MR^2}{I_O}(1 - \cos(2\alpha)) \right]^2$$

Construct the design response spectrum at this level of effective damping using the procedure defined in Section 2.6.1.5 of FEMA 273. By iteration or graphical methods, solve for the period and displacement that simultaneously satisfy the design response spectrum and the following rocking period equation:

$$T = \frac{4}{\sqrt{\frac{WR}{I_O}}} \cosh^{-1} \left(\frac{1}{1 - \frac{\theta}{\alpha}} \right) \quad \text{where}$$

$$\theta = \frac{\delta_{rocking}}{R \cos \alpha}$$

Also recall that

$$S_d = S_a g \frac{T^2}{4\pi^2}$$

At the desired solution,

$$\delta_{rocking} = S_d$$

**G. Special Study 5—
Report on Multidirectional Effects and P-M
Interaction on Columns**

Jack P. Moehle
Consulting Civil Engineer

RECEIVED
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12 August, 1999

Jon Heintz
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225 Bush Street, Suite 1000
San Francisco, CA 94104

Subject: Special Studies Report
Project: FEMA 273 Prestandard

Jon:

Please find enclosed my special studies report. I feel reasonably successful with two out of three. The third was even more successful, as I could not bring myself to make any changes to our current draft.

I skewed my efforts toward the acceptance criteria for concrete columns, which I understand have been crippling many rehab efforts. I think I have made some real progress. I would appreciate, however, if you would send a copy of the report to James Wight with a note that I requested the copy be sent to him - if he has concerns I would like to hear them.

Regards,



Jack P. Moehle

Enclosure: Paper and Zip disk version of report

Global Issue 3-4 – Multi-directional effects.

Section: 3.2.7

Classification: Technical Revision

Discussion: When a structure is displaced to its limit state in one direction, there is no reserve capacity to resist additional demands caused by displacements in the perpendicular direction. Also, the addition of displacements in the perpendicular direction is not intuitive and requires further explanation. It is unclear how to combine the acceptance criteria to elements receiving demands from multiple directions, particularly in the case of nonlinear pushover analyses.

Proposed Approach to Resolution: The requirements of all of 3.2.7 will be reconsidered from a technical and practical perspective. The following are the primary areas for focus:

- 1) How much demand is realistic? Available studies of bi-directional response will be reviewed to identify trends related to bi-directional response. Results will be organized for presentation to the project team. A summary answer to the question of how much demand is realistic will be provided.
- 2) How can this be analyzed reasonably with the current analysis technologies? Most computer packages do not readily (or at all) allow for bi-directional loading, and component acceptance criteria generally are not provided in FEMA 273 for bi-directional loading. It is likely that three options will be proposed: (a) full bi-directional loading, (b) uni-directional loading with increase in the loading amplitude, or (c) penalized acceptance criteria for some critical components to be used with non-amplified uni-directional loading. The final recommendation also might be to ignore multi-axial loading for all but a few critical components, in which case guidance will be provided for identifying when it is critical.
- 3) How will vertical effects be included? The recommendation probably will be parallel to that of the NEHRP 97 or IBC 2000 provisions.
- 4) How to express this in the prestandard? The resolved procedures must be presented in efficient and unambiguous language.

Summary of Findings:

1) *Range of demands*

I examined a sampling of research studies on this subject, including:

Pecknold, Inelastic structural response to 2D ground motion, J. EM, ASCE Oct 1974

Cheong, Varying axial load effects on inelastic behavior of a symmetric RC building subjected to earthquake motions, Structural Engineering Worldwide, 1998

Menun, Response spectrum method for interacting seismic responses, 6NCEE, 1998

Oliva, Biaxial seismic response of RC columns, JSE, ASCE, June 1987

De Stefano, Biaxial inelastic response of systems under bi-directional ground motions, 1ECEE, 1995

De Stefano, An evaluation of the inelastic response of systems under biaxial seismic excitations, Engineering Structures, Sept 1996.

The general conclusion of all these is that bi-directional loading increases demands. However, none of them provided any results that could be quickly and usefully assimilated in FEMA 273.

Perhaps the easiest idealization, that approximates mean response, is that response occurs in elliptical orbits, as suggested in Figure 1. This idealization suggests that appreciable deformation demands can occur in the orthogonal direction while the structure is responding essentially at full amplitude in the primary direction. For a structure that is yielding in both directions with moderate to large ductility demand, this means that maximum forces could occur in both directions simultaneously for some types of loadings (e.g., axial load on corner columns of frames).

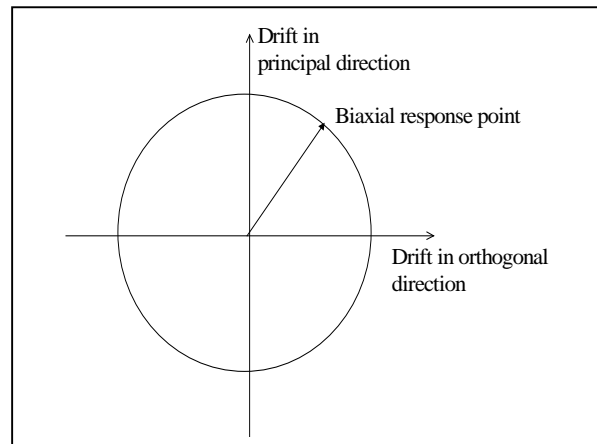


Figure 1 Idealized biaxial response envelope

In other cases (e.g., exterior columns other than corner columns where axial load varies significantly only for one direction of loading), biaxial effects are less important. In such cases, the component usually can accommodate *nearly* maximum drift in the principal direction while the orthogonal direction has moderate drift. At least within the accuracy of FEMA 273 acceptance criteria, I think this assumption is reasonable.

Some studies show that, because of the biaxial loading reduces component resistance along each principal axis, biaxial response amplitudes tend to be larger than uniaxial response amplitudes. This adds to the overall problem, but certainly would be beyond the scope of FEMA 273. I think we can assign this problem to the NEHRP/IBC writers, who might solve it one or two generations after our time.

For vertical accelerations, the approach expressed in FEMA 273 need not be different from that in the NEHRP provisions, and can be equally as vague.

Beyond these generalities, I did not find definitive solutions. We still can help the designer through some reasonable specifications. For example, rather than require the designer to *consider multidirectional effects in all cases*, knowing full well that this is just shoveling liability on the designer, we could put the onus on the ASCE standards committee to point out those specific cases where multidirectional effects need to be considered. If we set the problem up in this fashion, we are accomplishing our prestandard assignment.

2) Modifications to FEMA 273 to account for bidirectional and vertical loading

I have no substantial technical contribution to add. Instead, I suggest a simple modification to the PT draft to separate the approaches for linear and nonlinear analysis. Also, the effect of vertical acceleration is revised to more closely match the NEHRP provisions.

3.2.7 Multidirectional Excitation Effects

[(3.2.7.i) Buildings shall be designed for seismic forces in any horizontal direction. For regular buildings for which components do not form part of two or more intersecting elements, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of the building. For buildings ~~Buildings~~ with plan irregularity as defined in Section 3.2.3 and buildings in which one or more primary columns ~~components~~ form part of two or more intersecting frame or braced frame elements, multidirectional excitation effects shall be considered as follows:

Where required to consider multidirectional effects, and where Linear Static Procedure or Linear Dynamic Procedure is used as the basis for design, the following approach shall be permitted. Horizontally oriented orthogonal X and Y axes shall be established for the building. The elements and components of buildings shall be designed (a) for the forces and deformations associated with 100% of the design forces in the X direction plus the forces and deformations associated with 30% of the design forces in the perpendicular horizontal direction, and (b) for the forces and deformations associated with 100% of the design forces in the Y direction plus the forces and deformations associated with 30% of the design forces in the X direction. Other combination rules shall be permitted where verified by experiment or analysis.

Where required to consider multidirectional effects, and where Nonlinear Static Procedure or Nonlinear Dynamic Procedure is used as the basis for design, the following approach shall be permitted. Horizontally oriented orthogonal X and Y axes shall be established for the building. The elements and components of buildings shall be designed (a) for the forces and deformations associated with 100% of the design displacement in the X direction plus the forces (not deformations) associated with 30% of the design displacement in the perpendicular horizontal direction, and (b) for the forces and deformations associated with 100% of the design displacements in the Y direction plus the forces (not deformations) associated with 30% of the design displacement in the X direction. Other combination rules shall be permitted where verified by experiment or analysis.

~~[(3.2.7.ii) Where required to consider multidirectional effects, the following approach shall be permitted. Horizontally oriented X and Y axes shall be established for the building. The elements and components of buildings shall be designed (a) for the forces and deformations associated with 100% of the design displacements in the X direction plus the forces associated with 30% of the design displacements in the perpendicular horizontal direction, and (b) for the forces and deformations associated with 100% of the design displacements in the Y direction plus the forces associated with 30% of the design displacements in the Y direction. Other combination rules shall be permitted where verified by experiment or analysis. analysis performed for excitation in two orthogonal directions as follows: (1) 100% of the forces and deformations from the first analysis in one horizontal direction plus 30% of the forces and deformations from the second analysis in the orthogonal horizontal direction, and (2) 30% of the forces and deformations from the first analysis and 100% of the forces and deformations from the second analysis. Alternatively, it is acceptable to use SRSS to combine multidirectional effects where appropriate. Multidirectional effects on components shall include both torsional and translational effects.]~~

~~[(3.2.7.iii) All other buildings shall be either evaluated for multidirectional excitation effects as specified above or evaluated for seismic forces and displacements acting noneconcurrently in the direction of two orthogonal axes. For regular buildings the two orthogonal axes shall be the principal axes.]~~

~~[(3.2.7.iv) The effects of vertical excitation on horizontal cantilevers and horizontal prestressed elements shall be evaluated by static or dynamic response methods designed to resist the vertical component of earthquake ground motion. Vertical earthquake shaking shall be characterized by a ground shaking response spectrum with ordinates equal to 67% of those of the horizontal earthquake shaking spectrum specified in Section 2.6.1.5 unless alternative vertical response spectra developed using site specific analysis are approved by the authority having jurisdiction.]~~

In chapter 6, make the following modifications:

For concrete columns under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the design axial load P_{UF} shall be calculated as a force-controlled action in accordance with 3.4. The design moments M_{UD} shall be calculated about each principal axis in accordance with 3.4. Acceptance shall be based on the following equation:

$$\left(\frac{M_{UDx}}{m_x \kappa M_{CEx}} + \frac{M_{UDy}}{m_y \kappa M_{CEy}} \right)^2 \leq 1$$

where:

M_{UDx} = design bending moment about x axis for axial load P_{UF} , kip-in.,

M_{UDy} = design bending moment about y axis for axial load P_{UF} , kip-in.,

M_{CEx} = expected bending moment strength about x axis, kip-in.,

M_{CEy} = expected bending moment strength about y axis, kip-in.,

m_x = m factor for column for bending about x axis,

m_y = m factor for column for bending about y axis.

Alternative approaches based on principles of mechanics shall be permitted.

Global Issue 6-13- Acceptance criteria for P-M interaction in concrete columns.

Section: 6.5.x.4, 6.6.x.4, 6.7.x.4, 6.8.x.4, 6.9.2.4, 6.10.5,

Classification: Technical Revision

Discussion: This issue was raised at the 3/3/99 Standards Committee meeting. Flexure in concrete columns is treated as deformation-controlled, while axial loads are force-controlled. For concrete braced frames, section 6.10.5, both flexure and axial actions are considered deformation-controlled. It is unclear how to combine actions and compare with capacities represented on P-M interaction curves.

Proposed Approach to Resolution: The requirements of the relevant sections will be reconsidered from a technical and practical perspective. The following are the primary areas for focus:

- 1) How to treat P-M interaction using the LSP? Is it reasonable to increase the moment capacity by m while not increasing axial capacity similarly? What kinds of solutions result? Examination of this issue may lead to improved technical approach, or it may turn out that the technical approach cannot be readily improved. In either case, guidance needs to be improved. The guidance will be of two types: a) how to combine P and M and use m factors when limit analysis is not applied, and b) improved guidance on how to conduct limit analysis for this case to reduce the axial loads.
- 2) How to treat P-M interaction in the NSP? Treatment in the NSP is much easier, as the nonlinear behavior is tracked directly. A difficulty in practice lies in trying to establish the modeling parameters, which differ depending on the level of axial load. Also, in terms of acceptance criteria, a key question here is when is it deformation-controlled and when is it force-controlled?
- 3) When is it important to track P-M interaction? It may be appropriate to identify components for which P-M interaction need not be tracked, and provide guidance to this effect in the pre-standard.
- 4) How does P affect moment-curvature relation? This will be examined for a few typical cases, and may lead to guidance on treating P-M interaction, and may also lead to modifications of acceptance criteria.
- 5) Examine approaches to dealing with P-M interaction, including modifying the loads or modifying the acceptance criteria.
- 6) How to express this in the prestandard? The resolved procedures must be presented in efficient and unambiguous language.

Summary of Findings:

This special study topic proved too slippery for much progress. Instead, some of the time originally allocated to this topic was spent on other topics. General conclusions are described below. No FEMA 273 changes are recommended.

1) How to treat P-M interaction using the LSP?

The usual conclusion governs the response here - the LSP simply cannot be made completely rational for nonlinear response. The only rational approach to supplement the LSP is limit analysis to estimate maximum axial force demands. The commentary already provides guidance on how this can be done.

2) How to treat P-M interaction in the NSP?

Treatment in the NSP is straightforward.

3) When is it important to track P-M interaction?

I did not discover any special limits to when it is important to track PM interaction. At low axial loads, where the effect usually is considered less important, is where PM interaction has the largest impact on curvature capacity. At higher axial loads, the effect on curvature capacity is less, but the consequence might be more. All these aspects are accounted for reasonably in the current FEMA 273 or in revisions recommended in the next special study.

4) How does P affect moment-curvature relation?

See next special study report.

5) Examine approaches to dealing with P-M interaction

My opinion is that the acceptance criteria, modified in the next special study report, are the best way to handle the issue.

6) How to express this in the prestandard?

No new revisions, other than those reported in other special studies reports.

Global Issue 6-10 – Acceptance criteria for concrete columns.

Section: 6.4.4, 6.5, Tables 6-7 and 6-11.

Classification: Technical Revision

Discussion: Several building evaluations have shown that designs are controlled by concrete column acceptance criteria, and in several of these it has not been feasible to retrofit the building to eliminate the column deficiencies. Some engineers have developed the opinion that the acceptance criteria are too conservative, both for primary and secondary columns.

Proposed Approach to Resolution: The requirements of all of 6.4.4 and 6.5 will be reconsidered from a technical and practical perspective. The following are the primary areas for focus:

- 1) Shear strength provisions for concrete columns – Can the equation for shear strength be improved? One area for focus is whether it is necessary for the shear strength contribution of concrete to degrade to zero at moderate to high ductility demands. The approach to this problem will be to re-examine test data for columns that are typical of those that are resulting in acceptance problems in practice. Additional data now are available for this purpose.
- 2) Acceptance criteria – Are the acceptance criteria of Chapter 6 for columns consistent with the approach defined in Chapter 2? Special attention here will be paid to both primary and secondary columns, considering available test data. If the acceptance criteria are consistent with the tests and with Chapter 2, revisit the overall approach to determine if there is excessive conservatism resulting from accumulation of factors of safety applied independently in multiple parts of the process.
- 3) Express the resolved procedures/criteria in efficient and unambiguous language.

Summary of Findings:

1) Shear strength provisions for concrete columns

Note: This work was carried out in coordination with Abe Lynn, Acting Assistant Professor, Cal Poly, San Luis Obispo.

Test Data Review:

Data were gathered for reinforced concrete columns falling in the following range of parameters:

$$0.5 < b/h < 2$$

$$1.5 < a/d < 4$$

with or without lap splices

longitudinal steel ratio at least 0.01

transverse steel ratio less than 0.004

$$0.09 < P/Agf'_c < 0.5$$

h and b not much less than 8 inches

Shear failure before or after flexural yielding.

Twenty-eight tests were identified, and data were organized for study. The attached table summarizes parameters for the tests.

Specimen	h (=b) (in)	d (in)	a (in)	Splice length (in)	Ast (in ²)	pl	Asw (in ²)	s (in)	pw	hoop conf	hook bend	Fc (psi)	fy (psi)	hoop fy (psi)	P (k)	P/ Agfc	Fail mode	Vu (k)	Vu/(Ag rfc)	δy (in)	δu (in)	jud
Lynn and Moeble																						
3CLH18	18.0	15.0	58.0	none	3.81	0.031	0.22	18.0	0.00068	rectangle 90	rectangle 90	3709	48000	58000	113	0.094	SCF	61	3.09	0.76	1.20	1.6
3SLH18	18.0	15.0	58.0	25	3.81	0.031	0.22	18.0	0.00068	rectangle 90	rectangle 90	3709	48000	58000	113	0.094	SCF	60	3.04	0.68	1.15	1.7
2CLH18	18.0	15.0	58.0	none	2.37	0.020	0.22	18.0	0.00068	rectangle 90	rectangle 90	4795	48000	58000	113	0.073	FSCF	54	2.41	0.72	3.00	4.2
2SLH18	18.0	15.0	58.0	20	2.37	0.020	0.22	18.0	0.00068	rectangle 90	rectangle 90	4795	48000	58000	113	0.073	FSCF	52	2.32	0.68	1.80	2.6
2CMH18	18.0	15.0	58.0	none	2.37	0.020	0.22	18.0	0.00068	rectangle 90	rectangle 90	3734	48000	58000	340	0.281	STF	71	3.59	0.31	0.60	1.9
3CMH18	18.0	15.0	58.0	none	3.81	0.031	0.22	18.0	0.00068	rectangle 90	rectangle 90	4011	48000	58000	340	0.262	SCF	76	3.70	0.28	0.60	2.1
3CMD12	18.0	15.0	58.0	none	3.81	0.031	0.38	12.0	0.00174	diamond 90	diamond 90	4011	48000	58000	340	0.262	SCF	80	3.90	0.36	0.90	2.5
3SMD12	18.0	15.0	58.0	25	3.81	0.031	0.38	12.0	0.00174	diamond 90	diamond 90	3734	48000	58000	340	0.281	SCF	85	4.29	0.33	0.90	2.7
Bett, Klingner and Jirsa [Bett 1985]																						
I-1	12.0	10.4	18.0	none	1.32	0.024	0.20	8.0	0.00205	diamond 130	diamond 130	4333	67000	60000	65	0.104		47	4.96	0.19	0.57	3.0
Ikeida [Ikeida 1968]																						
H-3	7.9	6.8	19.7	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	62877	81471	18	0.100	FSCF	17	5.03	0.13	0.52	4.2
H-4	7.9	6.8	19.7	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	62877	81471	18	0.100	FSCF	17	5.19	0.12	0.41	3.5
H-5	7.9	6.8	19.7	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	62877	81471	18	0.100	FSCF	18	5.59	0.12	0.28	2.3
L-4	7.9	6.8	19.7	none	0.55	0.018	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	50387	68981	35	0.200	FSCF	13	3.93	0.10	0.52	5.3
L-5	7.9	6.8	19.7	none	0.55	0.018	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	50387	68981	35	0.200	FSCF	15	4.66	0.09	0.55	5.8
L-6	7.9	6.8	19.7	none	0.55	0.018	0.09	3.9	0.00283	rectangulih80	rectangulih80	2839	50387	68981	35	0.200	FSCF	15	4.66	0.08	0.66	8.0
Umemura and Endo [Umemura 1970]																						
6701	7.9	7.1	23.6	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2555	67135	47264	35	0.222	STF	16	5.11	0.17	0.51	3.1
6703	7.9	7.1	15.7	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2555	67135	47264	35	0.222	STF	24	7.60	0.13	0.25	1.9
6704	7.9	7.1	15.7	none	0.59	0.019	0.09	3.9	0.00283	rectangulih80	rectangulih80	2555	67135	47264	35	0.222	FSCF	30	9.69	0.10	0.31	3.2
6710	7.9	7.1	23.6	none	0.59	0.019	0.09	7.9	0.00141	rectangulih80	rectangulih80	2555	67135	47264	88	0.556	SCF	19	5.93	0.15	0.42	2.9
6804	7.9	7.1	15.7	none	0.33	0.011	0.04	4.7	0.00105	rectangulih80	rectangulih80	4769	55213	94245	35	0.119	FSCF	18	4.11	0.09	0.94	10.0
E3	7.9	7.1	15.7	none	0.30	0.010	0.04	3.9	0.00126	rectangulih80	rectangulih80	2143	46555	76077	35	0.265	FSCF	11	3.99	0.11	0.64	5.8
E4	7.9	7.1	15.7	none	0.30	0.010	0.04	3.9	0.00126	rectangulih80	rectangulih80	1902	46555	76077	35	0.299	FSCF	13	4.84	0.11	0.94	8.9
E5	7.9	7.1	15.7	none	0.31	0.010	0.04	3.9	0.00126	rectangulih80	rectangulih80	2015	53793	76077	35	0.282	FSCF	16	5.58	0.11	0.54	4.9
E6	7.9	7.1	15.7	none	0.31	0.010	0.04	3.9	0.00126	rectangulih80	rectangulih80	1902	53793	76077	35	0.299	FSCF	15	5.57	0.11	0.63	5.7
Kokusho [Kokusho 1964]																						
2C12	7.9	6.7	19.7	none	0.39	0.013	0.10	3.9	0.00332	rectangulih80	rectangulih80	2881	75510	51097	35	0.197	FSCF	17	5.02	0.11	0.41	3.6
2C22	7.9	6.7	19.7	none	0.61	0.020	0.10	3.9	0.00332	rectangulih80	rectangulih80	2959	75510	51097	35	0.192	FSCF	20	5.87	0.15	0.37	2.5
Kokusho and Fukuhara [Kokusho 1965]																						
4C32	7.9	6.7	19.7	none	0.88	0.029	0.10	3.9	0.00332	rectangulih80	rectangulih80	3179	51877	45703	88	0.446	FSCF	25	7.08	0.09	0.29	3.1
4C42	7.9	6.7	19.7	none	1.20	0.039	0.10	3.9	0.00332	rectangulih80	rectangulih80	3179	51877	45703	88	0.446	FSCF	25	7.08	0.10	0.20	2.0

When FEMA 273 was developed, the number of identified tests with parameters falling in the range of most interest (identified above) was more limited. Instead, at that time, shear strength equations were developed to fit data covering a much broader range of parameters. The resulting equations may be more generally applicable, but may not be particularly good at representing behavior of columns more typical of older existing buildings. Note that some of the columns in the database have cross section as small as 8 inches.

Application of FEMA 273 to the Test Data:

Shear strength equations of FEMA 273 were used to calculate shear strength, V_n . Ratios of experimental shear strength to V were calculated, and plotted as a function of displacement ductility achieved in the test. Results are plotted in Figure 2. Values exceeding unity are cases where the column developed strength exceeding the strength calculated by FEMA 273. FEMA 273 tends to be excessively conservative, especially for cases where displacement ductility exceeds 2, because it sets V_c equal to zero.

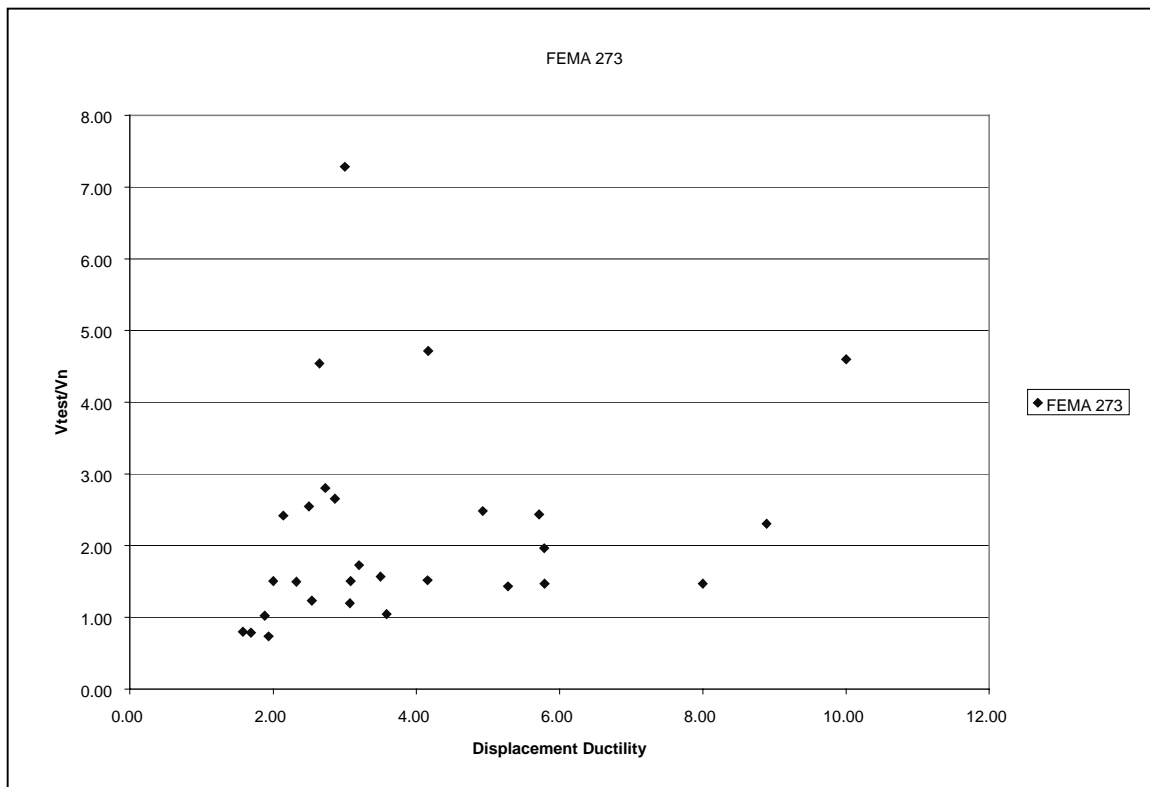


Figure 2 Comparison of FEMA 273 and Test Data

Derivation of a revised shear strength model:

An alternative shear strength model was developed considering strength of materials theory and test data. The starting point was the assumption that shear strength was composed of two parts, as follows:

$$V_n = V_c + V_s$$

where:

V_c is the concrete contribution

V_s is the transverse steel contribution

(Approaches including an axial load contribution in the form of a diagonal strut represented by V_p were considered but deemed inappropriate.)

Concrete and steel components were set up as follows:

A. Concrete Contribution

Shear strength was assumed to be related to the calculated nominal principal tension stress in the column. Tension stress capacity was set equal to $f_{tc} = 6\sqrt{f'_c}$. The principal stress relationship is

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

$$\sigma_y = 0 \text{ within the span}$$

Letting: $\sigma_x = -\frac{P}{A_g}$, (negative for compression)

And substituting $f_{tc} = 6\sqrt{f'_c}$, gives:

$$\tau_{xy} = 6\sqrt{f'_c} \sqrt{1 + \frac{P}{6A_g\sqrt{f'_c}}}$$

As with ACI Committee 426, concrete strength is affected by the inverse of the aspect ratio:

$$v_c = \frac{\tau_{xy}}{a/d} = 6\frac{\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6A_g\sqrt{f'_c}}}$$

The aspect ratio effect accounts in part for the fact that flexure produces additional tension stresses and in part for the ability of a low-aspect-ratio section to redistribute internal actions after initial cracking.

The aspect ratio is limited to the following range:

$$2 \leq \frac{a}{d} \leq 3$$

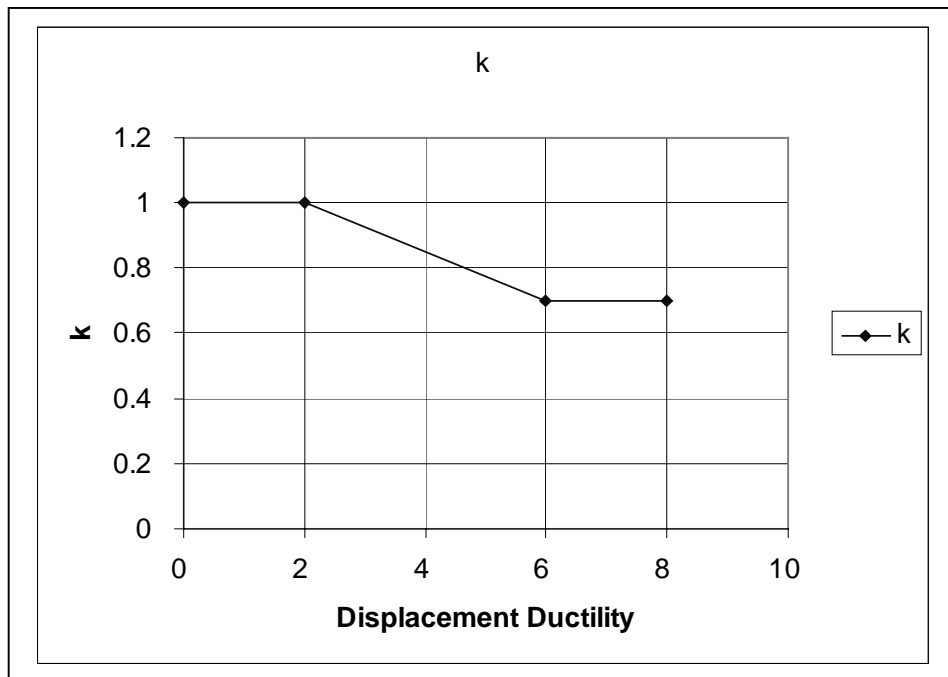
Note that this range is essentially the same as that used by ACI 318 for calculation of shear strength of low-rise walls.

Shear strength attributed to concrete is obtained by multiplying the nominal shear stress capacity times an effective area, as follows:

$$V_c = v_c(0.8A_g)$$

Finally, shear strength contribution of concrete was assumed to vary with displacement ductility using modifier k , resulting in the final expression for V_c .

$$V_c = k \left(\frac{6\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6\sqrt{f'_c} A_g}} \right) (0.8A_g)$$



B. Steel Contribution

The transverse steel contribution, V_s , is represented in the usual way:

$$V_s = \frac{A_v f_{wy} d}{s \tan \theta}$$

where θ is the angle of the crack from the longitudinal axis of the column.

In comparing the calculated versus actual shear strength, the conventional assumption of $\theta = 45^\circ$ was found to be adequate, rather than smaller angles as has been suggested in some recent publications.

Also, FEMA 273 penalizes the effectiveness of transverse reinforcement when it is spaced widely, the penalty depending also on the displacement ductility demand. To simplify this relation, it is proposed that shear strength contribution from steel V_s be taken as half that given by the equation above if $s > d/2$. This modifier applies to all the data collected as part of this study.

Figure 3 compares test strength with strength from the proposed equation. The correlation is much improved compared with that for FEMA 273.

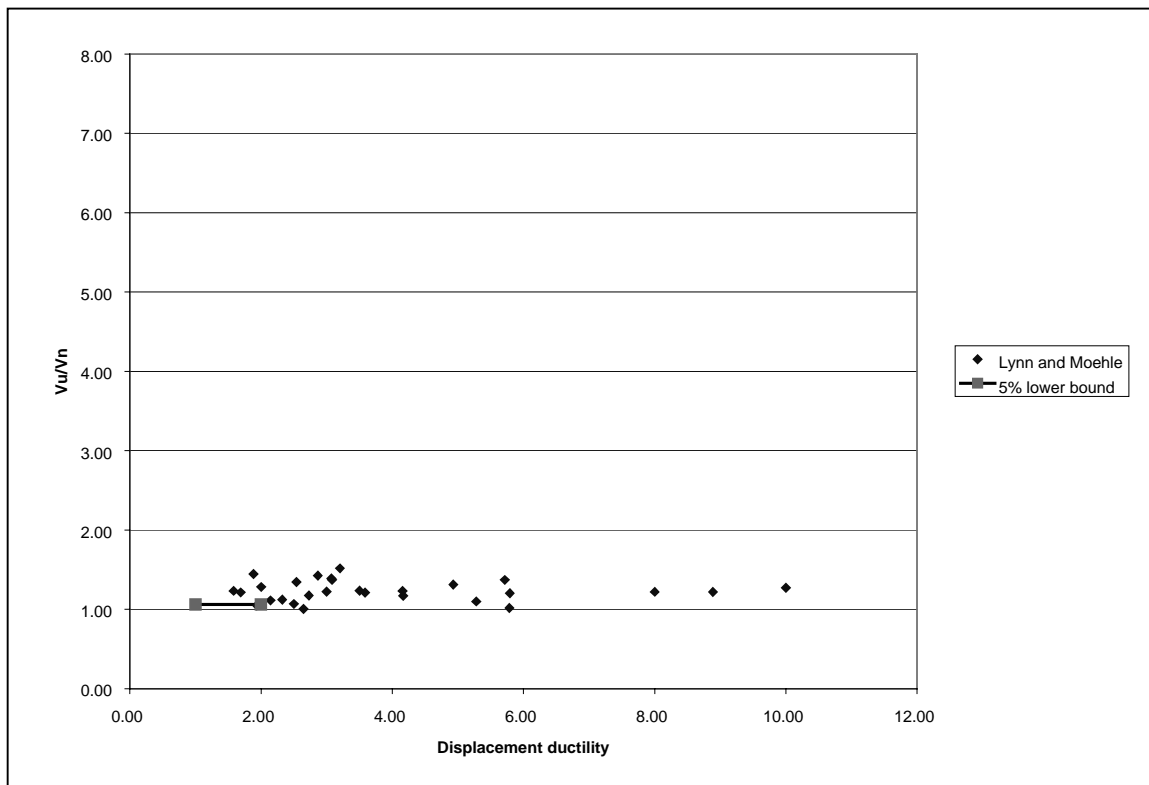


Figure 3 Comparison of proposed equation and test data

Also shown in Figure 3 is a line corresponding to 5% lower bound, assuming normal distribution of the data. This line suggests that $\phi = 1$ might be appropriate. As shown in Figure 4, however, the correlation with test data is a function of column size, with less conservative results indicated for larger columns. Therefore, I recommend to use $\phi = 0.85$ as is the convention for reinforced concrete.

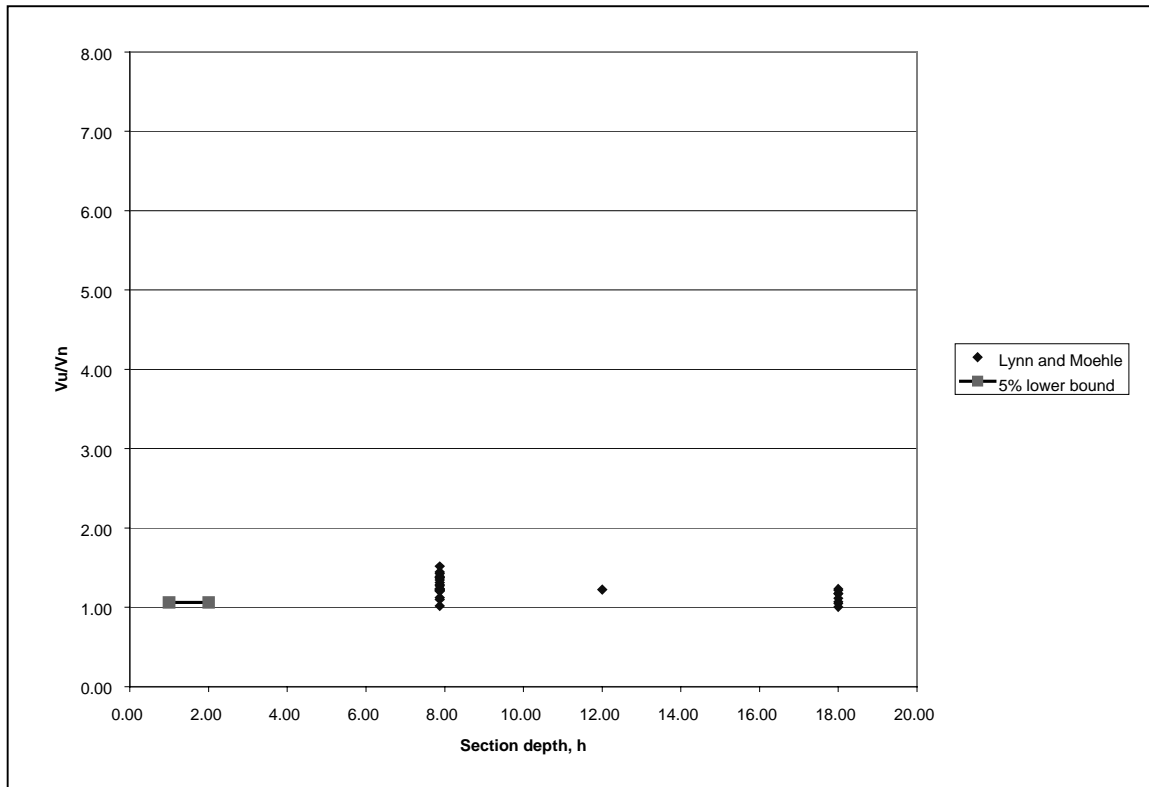


Figure 4 Comparison of proposed shear strength and test data as a function of column size

2) Acceptance criteria for concrete columns

I did not gather significant amounts of data for lightly confined reinforced concrete columns controlled by flexure. Instead, recommendations are made primarily on the basis of theoretical considerations, tempered by the test database described previously.

Calculated moment-curvature relations

Figure 5 plots relations between moment and curvature, calculated using assumptions of FEMA 273. Limiting compression strain is 0.005. Theoretical effects of axial load and reinforcement ratio are evident. On the basis of tests on bridge columns, I tend to not believe the trend of decreasing curvature capacity with increasing longitudinal reinforcement ratio.

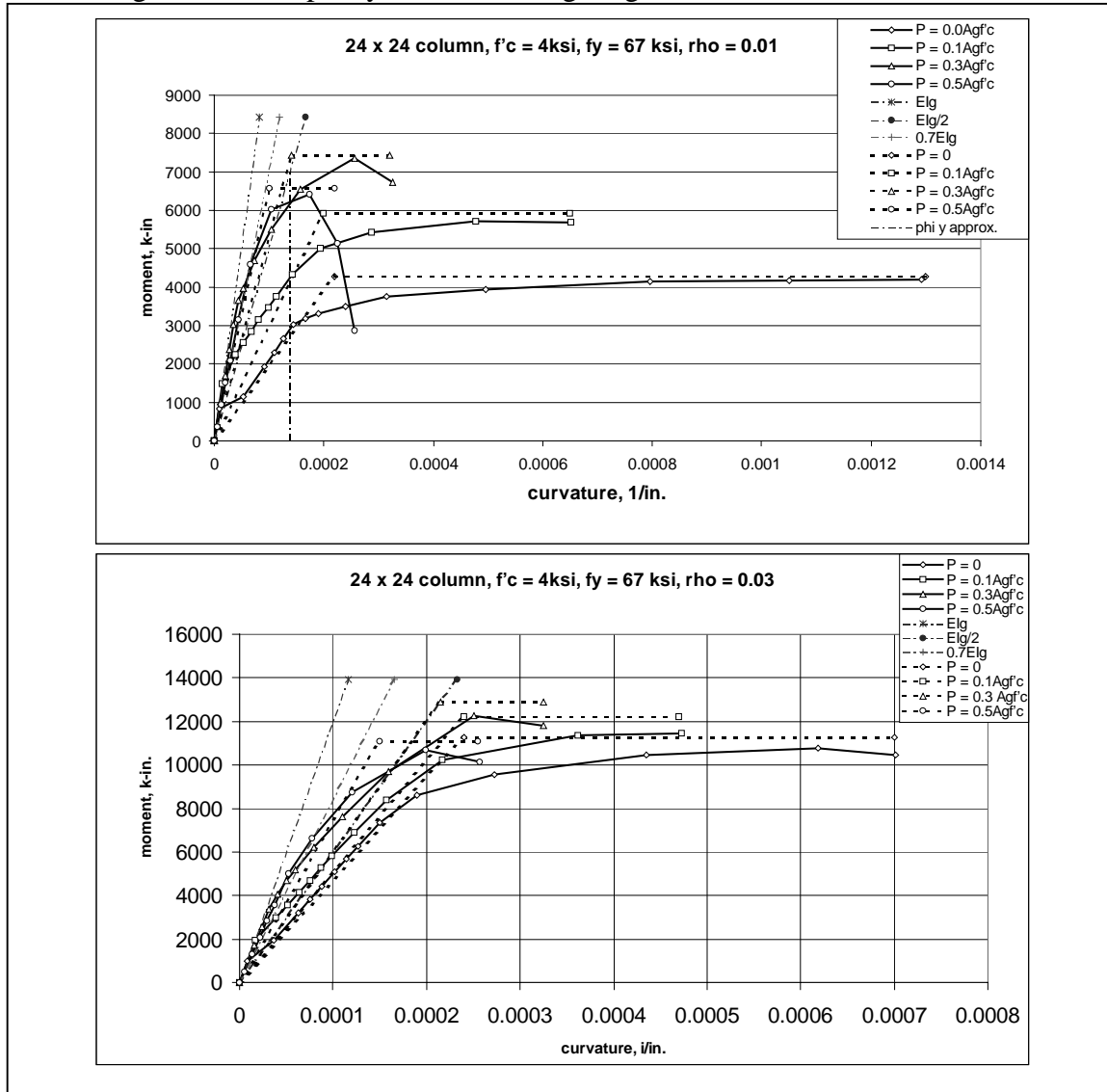


Figure 5 Calculated moment-curvature relations

Figure 5 also contains idealized bilinear moment-curvature relations. The strength was based on calculations assuming yield strength of longitudinal reinforcement was 1.25 times the nominal value, per FEMA 273. Effective stiffness was taken as a secant through the calculated curve at moment equal to three-quarters of the calculated strength. Also shown are EI_g , $0.7EI_g$, and $0.5EI_g$.

FEMA 273 recommends to use $EI_{eff} = 0.7 EI_g$ for columns in compression. On the basis of the data in Figure 5, it appears more reasonable to use a value closer to $0.5EI_g$ for columns with axial load less than $0.3f'_cA_g$. Note that the calculated EI value assumes the section is fully cracked, whereas the column likely is not cracked at all locations along the length. This is offset by slip of reinforcement from connections, which typically is ignored in the calculation of member stiffness. For axial loads higher than $0.5f'_cA_g$, $0.7EI_g$ approximates the stiffness. Interpolation would be appropriate between these limits. The reduced effective stiffness will reduce calculated column demands in many cases.

Displacement ductility values were calculated using the bilinear relations in Figure 5 and a simple plastic hinge model as described in FEMA 273. Results are shown in Figure 6. Values are a function of aspect ratio (and theoretically also a function of longitudinal reinforcement ratio, which I discount as noted previously). Values are relatively flat for axial loads greater than $0.1f'_cA_g$, with values ranging from 1.5 to 2. Review of the experimental data presented previously for columns failing in shear indicated displacement ductility capacities were never less than 2. Therefore, for this axial load range, I propose a minimum value of $m = 2$ for primary columns. Larger values of m probably could be proposed for axial loads approaching 0, but I don't think this is a practical consideration.

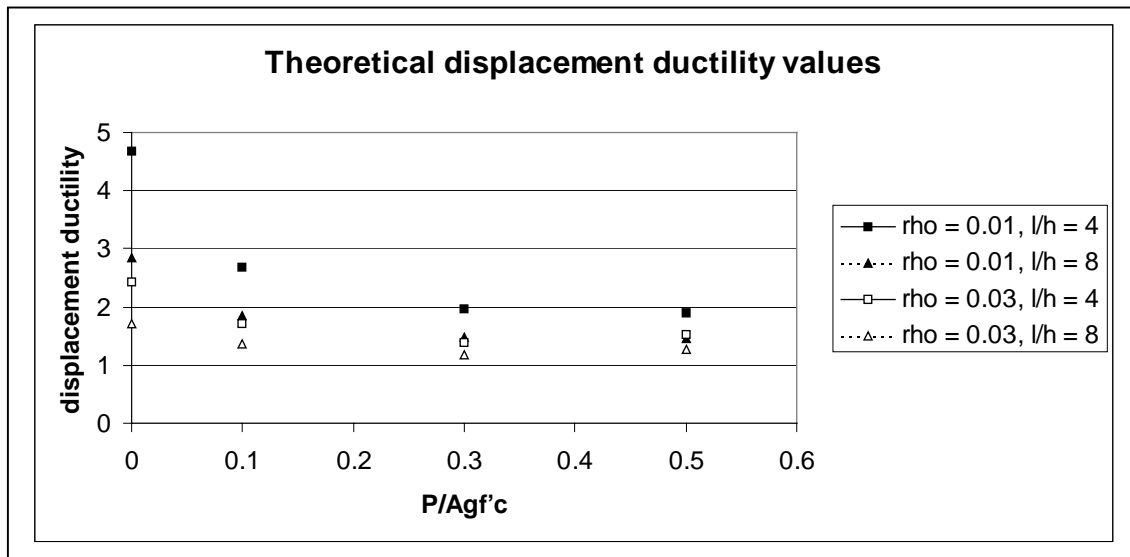


Figure 6 Calculated displacement ductility capacities

Theoretical plastic rotation capacities were calculated using the bilinear relations in Figure 5 using the plastic hinge model of FEMA 273. Results are shown in Figure 7. For large axial load ratios ($<0.4f'_cA_g$), it is difficult to justify more than 0.002rad for primary columns. For smaller axial loads, values as high as 0.006 are justifiable. Note that these are less than the FEMA 273 acceptance values for primary columns for CP.

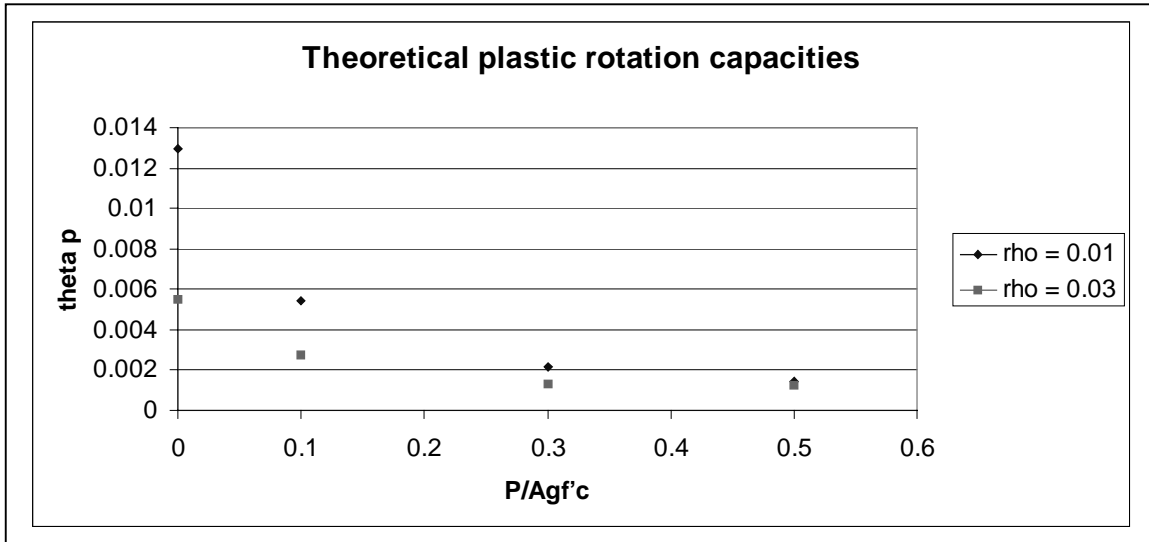


Figure 7 Theoretical plastic hinge rotation capacities

Given a good model for shear strength, and after having examined the data for different shear levels in the database shown previously, I cannot justify decreasing the acceptance criteria as the shear force increases, as FEMA 273 current does. I recommend to eliminate this effect in FEMA 273.

FEMA 273 recommends strict acceptance criteria for columns controlled by shear. Given an appropriate shear model, however, as long as the shear demand is limited to less than or equal to the shear capacity, these acceptance criteria can be modified.

- When using linear methods – Shear strength is calculated on the basis of the ductility demand (as imputed from flexural D/C ratio). Where $D/C > 2$, assume $k = 0.7$ in the shear strength calculation; otherwise $k = 1$. Where the shear demand is calculated directly from the linear analysis, assume column shear to be force-controlled and use the appropriate equations of chapter 3 with the J factor. Where the shear demand is calculated directly from plastic analysis, use that demand directly.
- When using nonlinear methods – Shear demand is calculated directly from the analysis. Shear strength is calculated on the basis of ductility demand. Alternatively, to save effort, estimate the shear strength using $k = 0.7$. If the shear strength exceeds the demand, the acceptance is based on flexural limits. If the shear strength is less than the demand, then the engineer can estimate the flexural ductility level at which the shear strength is reached, which will provide a limit for acceptance. It might turn out that the resulting ductility is 1.

Data for columns considered secondary components

Most tests of columns are terminated shortly after loss of lateral load capacity. Two test series continue lateral deformation cycles until loss of vertical capacity. The first comprises eight columns tested by Lynn and Moehle (Earthquake Spectra, see table). The second is an ongoing test program by Sezen and Moehle at PEER, for which results of two column tests currently are available. Both series are for columns with 18-in. cross section and light transverse reinforcement. Data for the tests are in Figure 8. For each test, the lower point corresponds to nominal loss of lateral capacity (20% reduction in resistance) and the upper point corresponds to loss of gravity capacity.

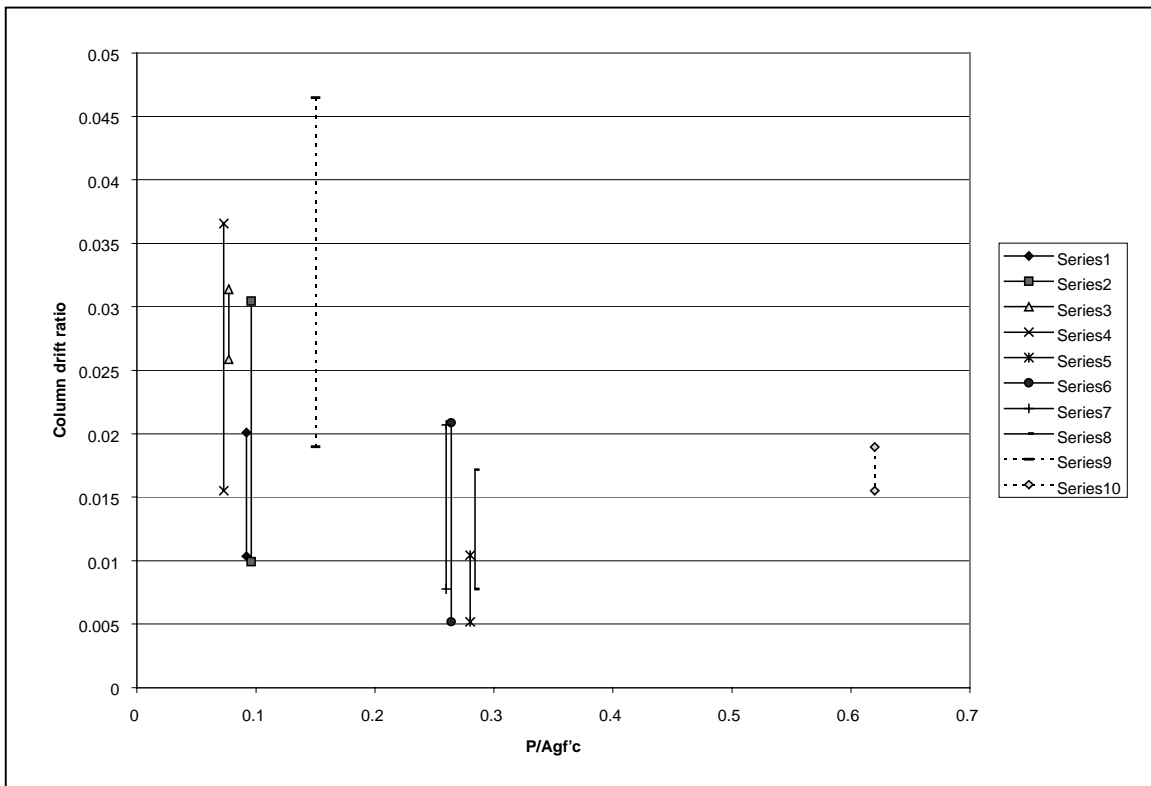


Figure 8 Drift at loss of lateral capacity and at collapse as a function of axial load for ten tests

The data in Figure 8 suggest one conclusion - columns with lower axial load tend to have larger reserve deformation capacity. Note that data for one series (continuous lines) do not align with those for the other series (broken lines), though the trends within a series are similar.

Figure 9 plots plastic rotation angles (drift at failure minus drift at yield), for the same ten columns, as a function of axial load. Note that these columns all eventually failed in shear, after flexural yielding.

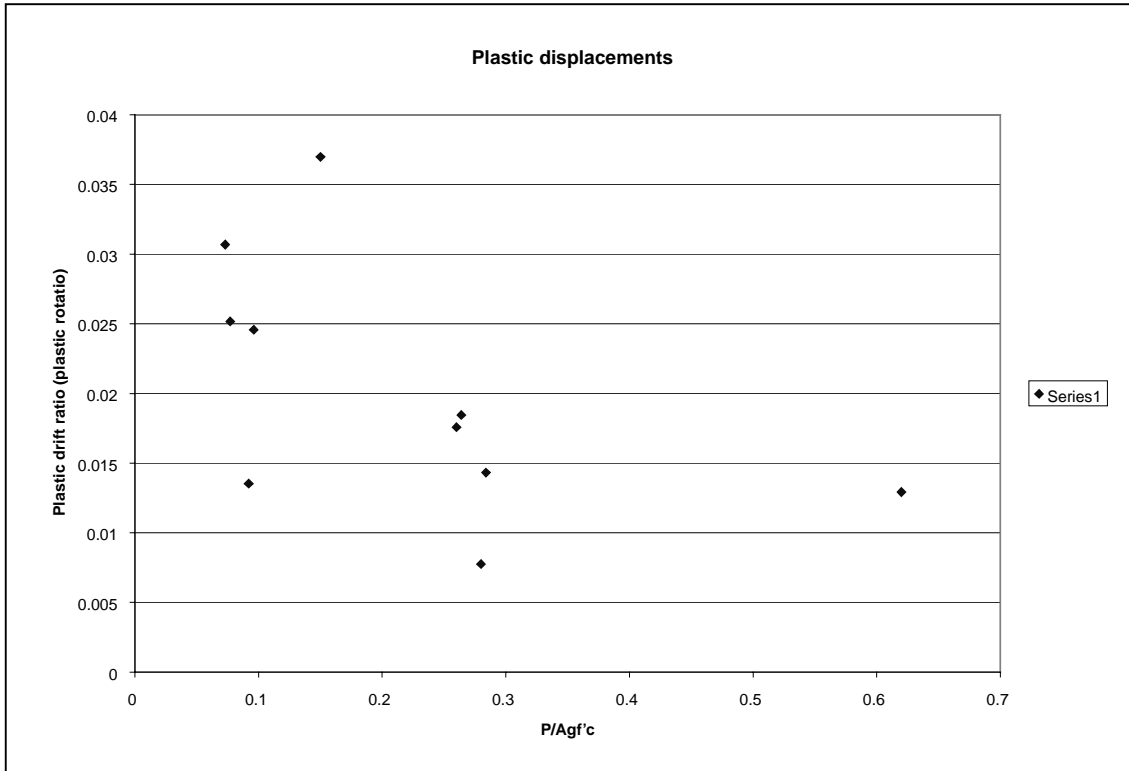


Figure 9 Plastic rotation capacity at collapse as a function of axial load

3) Recommended changes to FEMA 273

A) Effective stiffness

Modify Table 6-4 as follows:

Table 6-4 Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns in compression with compression due to design gravity loads $\geq 0.5A_gf'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.3A_gf'_c$ or with Columns in tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: It shall be permitted to take I_g for T-beams as twice the value of I_g of the web alone. Otherwise, I_g , shall be based on the effective width as defined in Section 6.4.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. Alternatively, the more conservative effective stiffnesses shall be used.

B) Modify the following paragraph to provide better directions on how to modify flexural deformability as a function of design shear.

[(6.4.3.iv) Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear shall be taken into consideration. When using analytical models for flexural deformability that do not directly consider effect of shear, and where design shear equals or exceeds $6\sqrt{f'_c} A_g$, where f'_c is in psi and A_g is gross area of web in inches, the design value shall not exceed eighty percent of the value calculated using the analytical model.]

C) Accounting for light transverse reinforcement

Modify 6.4.4.3 as follows:

6.4.4.3 Transverse Reinforcement

[(6.4.4.ii) Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion. ~~Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Within yielding regions of components with moderate or high ductility demands, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where: (1) longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, or (2) For columns and beams in which perimeter hoops are either lap spliced or have hooks that are not adequately anchored in the concrete core, transverse reinforcement shall be assumed not more than 50% effective in regions of moderate ductility demand and shall be assumed ineffective in regions of high ductility demand. Within yielding regions of components with low ductility demands, and outside yielding regions for all ductility demands, transverse reinforcement shall be assumed ineffective in resisting shear or torsion where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear.~~]

D) Shear strength of concrete columns

Replace 6.5.2.3.3 with a revised section, as follows:

6.5.2.3.3 Columns

[(6.5.2.3.iii) For columns, the contribution of concrete to shear strength, V_c , calculated according to Equation 6-3 shall be permitted.

$$V_c = k \left(\frac{6\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{N_u}{6\sqrt{f'_c} A_g}} \right) (0.8A_g) \quad (6-3)$$

in which $k = 1.0$ in regions of low ductility demand, 0.7 in regions of high ductility demand, and varies linearly between these extremes in regions of moderate ductility demand; $\phi = 0.75$ for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete; N_u = axial compression force in pounds ($= 0$ for tension force); M/V is the largest ratio of moment to shear under design loadings for the column but shall not be taken greater than 3 or less than 2 ; d is the effective depth; and A_g is the gross cross-sectional area of the column. It shall be permitted to assume $d = 0.8h$, where h is the dimension of the column in the direction of shear. Where axial force is calculated from the linear procedures of Chapter 3, the maximum compressive axial load for use in Equation 6-3 shall be taken as equal to the value calculated considering design gravity load only, and the minimum compression axial load shall be calculated according to Equation (3-15). Alternatively, limit analysis as specified in 3.4.2.1B shall be permitted to be used to determine design axial loads for use with the linear analysis procedures of Chapter 3. Alternative formulations for column strength that consider effects of reversed cyclic, inelastic deformations and that are verified by experimental evidence shall be permitted.]

[(6.5.2.3.v) For columns satisfying the detailing and proportioning requirements of Chapter 21 of ACI 318, the shear strength equations of ACI 318 shall be permitted to be used.]

E) Modeling and Acceptance Criteria

Modify the following paragraph, as noted:

[(6.5.2.2.B.iv) For beams and columns, the generalized deformation in Figure 6-1 shall be either the chord rotation or the plastic hinge rotation. For beam-column joints, the generalized deformation shall be shear strain. Values of the generalized deformation at points B, C, and D shall be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternately, where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities shall be as defined by Tables 6-6 and 6-7, and where. ~~Where~~ the generalized deformation is shear distortion of the beam-column joint, shear angle capacities shall be as defined by Table 6-8. For columns designated as primary components and for which calculated design shears exceed design shear strength as defined by Equation (6-3), the permissible deformation for the collapse prevention performance level shall not exceed the deformation at which shear strength is calculated to be reached; the permissible deformation for the life safety performance level shall not exceed three quarters of that value.]

Modify Tables 6-7 and 6-11 as follows:

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Columns (continued)

Conditions			Modeling Parameters ⁴			Acceptance Criteria ⁴				
			Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians				
						Component Type				
						Primary		Secondary		
						Performance Level				
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure¹										
	Trans. Reinf. ²									
_ 0.1	C	3	0.02	0.03	0.2	0.005	0.040.015	0.02	0.040.02	0.03
0.1	C	6	0.0150.016	0.0250.024	0.020.2	0.0050.005	0.040.012	0.0150.016	0.040.016	0.0250.024
_ 0.4	C	3	0.015	0.025	0.2	0.00.003	0.0050.012	0.015	0.040.018	0.025
0.4	C	6	0.040.012	0.0150.02	0.020.2	0.00.003	0.0050.01	0.040.012	0.040.013	0.0150.02
_ 0.1	NC	3	0.040.006	0.015	0.2	0.005	0.005	0.040.006	0.0050.01	0.015
0.1	NC	6	0.0050.005	0.0050.012	0.2	0.0050.005	0.0050.004	0.0050.005	0.0050.008	0.0050.012
_ 0.4	NC	3	0.0050.003	0.0050.01	0.2	0.00.002	0.00.002	0.0050.003	0.000.006	0.000.01
0.4	NC	6	0.00.002	0.00.008	0.2	0.00.002	0.00.002	0.00.002	0.00.005	0.00.008
ii. Columns controlled by shear										
See paragraph 6.5.2.2.B.iv										
Hoop spacing $\leq d/2$, or ≤ 0.1			0.0	0.015	0.2	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height^{1,3}										
Hoop spacing $\leq d/2$			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing $> d/2$			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$^{1,3}										
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
<p>1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.</p> <p>2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at $\leq d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.</p> <p>3. To qualify, hoops shall not be lap spliced in the cover concrete, and hoops shall have hooks embedded in the core or other details to ensure that hoops are adequately anchored following spalling of cover concrete.</p> <p>4. Linear interpolation between values listed in the table is permitted.</p>										

Table 6- 11 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions		<i>m</i> factors ⁴					
		Primary			Secondary		
		Performance Level					
		IO	LS	CP	LS	CP	
i. Columns controlled by flexure¹							
	Trans. Reinf. ²						
_ 0.1	C	3	2	3	4	<u>3</u> ₄	<u>4</u> ₅
0.1	C	6	2	<u>3</u> _{2.4}	<u>3</u> _{3.2}	<u>3</u> _{3.2}	<u>3</u> ₄
_ 0.4	C	3	1	2	<u>2</u> ₃	<u>2</u> ₃	<u>2</u> ₄
0.4	C	6	1	<u>4</u> _{1.6}	<u>2</u> _{2.4}	<u>4</u> _{2.4}	<u>2</u> _{3.2}
_ 0.1	NC	3	2	2	3	2	3
0.1	NC	6	2	<u>2</u> _{1.6}	<u>2</u> _{2.4}	<u>2</u> _{1.6}	<u>2</u> _{2.4}
_ 0.4	NC	3	1	<u>4</u> _{1.5}	2	<u>4</u> _{1.5}	2
0.4	NC	6	1	<u>4</u> ₁	<u>4</u> _{1.6}	<u>4</u> ₁	<u>4</u> _{1.6}
ii. Columns controlled by shear^{1,3}							
Hoop spacing _ <i>d</i> /2, or _ 0.1		-	-	-	-	2	3
Other cases		-	-	-	-	<u>4</u> _{1.5}	<u>4</u> ₂
iii. Columns controlled by inadequate development or splicing along the clear height^{1,3}							
Hoop spacing _ <i>d</i> /2		-	-	-	-	3	4
Hoop spacing > <i>d</i> /2		-	-	-	-	2	3
iv. Columns with axial loads exceeding 0.70<i>P</i>_o^{1,3}							
Conforming reinforcement over the entire length		1	1	2	2	2	2
All other cases		-	-	-	-	1	1
<ol style="list-style-type: none"> When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at _ <i>d</i>/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (<i>V</i>_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming. To qualify, hoops shall not be lap spliced in the cover concrete, and shall have hooks embedded in the core or other details to ensure that hoops are adequately anchored following spalling of cover concrete. Linear interpolation between values listed in the table is permitted. 							

H. Special Study 6— Acceptability Criteria (Anomalous *m*-values)

ACCEPTABILITY CRITERIA (ANOMALOUS m -VALUES)

by

Mike Mehrain
10/11/99

GENERAL COMMENTS

1. There are two general areas in which values of m - appear to be on the conservative side.

- a. Values for Immediate Occupancy (I.O.) performance level**

Immediate Occupancy performance level does not have a well defined point on the force deflection curve. The intended performance for Immediate Occupancy is not “damage-free” structure; it is a structure which would be damaged but would not be shut down for evaluation or strengthening. Thus, minor post-earthquake damage is accepted. It appears that the values selected in FEMA 273 correspond to a lower level of damage than intended for Immediate Occupancy.

As an example, a fully ductile code-conforming reinforced concrete or steel moment frame (with “good” connections) in which beams undergo plastic deformation has m - value of 2.0. This would result in no observable damage in the structure. It is interesting to compare FEMA 273 Immediate Occupancy with the requirement of the State of California for Hospitals. The ratio between strength demand of hospitals to ordinary buildings is 1.5 ($I = 1.5$). This same ratio in FEMA 273 for fully ductile steel or concrete frames is 3 ($m = 2$ vs. 6). A more severe case is steel braced frames in which the ratio is about 7 ($m = 0.8$ vs. 6.0).

Recommendation: the m - values for Immediate Occupancy be increased, so the ratio to LS would be in the order of 2.0.

- b. Materials with low ductility**

Materials and actions with low ductility have often an $m = 1.0$ or lower in the present document. Review of the test data for these non-ductile components indicate that ductility in the range of approximately 2 is available even in brittle structural components (except in very few cases, such as shear in unreinforced masonry construction). As a frame of reference, FEMA 178 permits the use of $R = 2.0$ for nonductile concrete construction with any source of brittleness including shear failure, premature bond failure, etc.) A recent study by Professor Jack Moehle for concrete columns also resulted in mostly an increase in the m - values for brittle behavior.

Recommendation: use a minimum value of $m = 1.25, 1.5, \text{ and } 1.75$ for IO, LS and CP for primary elements and 1.75 and 2.0 for secondary elements (exception URM and a few other highly brittle cases). For nonlinear analysis, use minimum plastic rotation angle or plastic shear angle of .0015, .0020, .003 for primary and .003, .004 for secondary elements.

2. For flexural elements in bending, the appropriate nonlinear parameter to be used is plastic rotation angle. This is the case in the concrete section. However, the steel section uses ductility or ratio of total chord rotation to yield chord rotation. I believe that this is an error and can result in significant problems. I strongly recommend that the flexural actions in steel chapter (moment frames and link beam in eccentric braced frames) be modified to use plastic rotation angle.

3. SECTION 2.4.4.2

Definition of deformations and force controlled actions. These appear to be complicated and in some instances may not be completely correct. In general, deformation controlled actions are actions that produced the overall plastic displacement of the structure. The components that are responsible for the inelastic actions may or may not be ductile. When buildings have a combination of ductile and brittle actions, structural deformations are originated from the nonlinear behavior of ductile actions. However, when buildings are constructed of non-ductile elements, the small plastic deformation of the structure is produced from small nonlinear action of nonductile elements. These nonductile elements are “deformation controlled”.

4. SECTION 2.8.3.5

The definition of lower bound strength is average -1 sigma, and not as defined in this section.

5. SECTION 2.8.3.6

Two new equations 2-6 and 2-7 have been introduced into the second PT draft that did not exist in the FEMA 273 document. In all acceptability tables, the criteria for Immediate Occupancy for primary and secondary elements are the same. However, these two equations are not the same. Furthermore, if such a cap is necessary, it should probably be $g + .25a$.

6. SECTION 3.4.2.1.2 – THE VALUE OF J FOR FORCE CONTROLLED ACTIONS

7. The presently specified maximum value of $J = 2$ has caused some controversy. Minor modification of this value is warranted. Note that there are two other approaches for calculation of force in force controlled actions, which are more accurate. They are:
 - (a) From a rational analysis using limit analysis.
 - (b) J to be taken as the smallest DCR for components in the load path delivering force to the component in question.

Both of the above “more accurate” procedures suggest a larger force in buildings designed for Immediate Occupancy as compared to Life Safety. Therefore, it seems appropriate to set the maximum value of J in equation 3-17 to be also a function of performance level.

Recommendation: J in equation 3-17 need not exceed 2.5 for Collapse Prevention, 2.0 for Life-Safety, and 1.5 for Immediate Occupancy.

There has also been concern about the value of J being a function of spectral acceleration. An alternate to the existing formulation is to make equation 3-15 applicable only to regions of high seismicity. For other regions of seismicity use equations 3-16.

ACCEPTABILITY TABLES

For reference, items discussed below are shown on the attached acceptability tables.

CHAPTER 5 (STEEL)

The acceptability criteria tables of second PT draft are somewhat different from those of FEMA 273. They include some typographical errors as well as changes that may not be appropriate. As an example, for partially restrained connections, FEMA 273 provided different acceptability criteria depending on which connection piece within the connection reaches its ultimate load. The modified tables in PT draft provides for yielding of the angles only and consider action of the bolts and rivets as a force controlled.

ATC is presently in the process of updating the acceptability tables of FEMA 273. Their preliminary proposed changes are attached. I will use the new revised tables as the basis for my comments as they appear to be more reasonable and represent the latest changes.

TABLE 5.3

1. The values for m provided in this table for linear procedure in comparison with those indicated in Table 5.4 for nonlinear analysis do not conform with section 2.8.3.7 (requiring that the m – values for linear analysis is .75 times deformations used for nonlinear analysis.)
2. For fully restrained moment connections, the values for secondary elements are below 1.0. As secondary elements, these connections only need to have the shear tab continue to resist gravity loads. Failure of welded flange connections is acceptable. Should we not permit a much higher m – value?
3. Footnote 6 refers to a condition where one-half of the indicated m – values should be used. A lower limit $m = 1$ should be added to this footnote [probably a better approach is to cut the plastic deformation component by one-half rather than the entire m – value, therefore, the new m - value would be equal to $(old\ m + 1)/2$].

-
4. It is not clear how to consider both moment and axial force as force controlled and what interaction equation to use. Suggest treating this case same as for columns with lower axial load, but use lower m – values, if necessary.

TABLE 5-4

1. As discussed above, the acceptance criteria for this table should be changed to plastic rotation angles.
2. Two missing ductility values under “columns (b)” should be added. It appears that the values of 5 and 7 for (d) and (e) are appropriate.
3. It is not clear how footnote 1 would be used. This approach appears to be a difficult process that can be significantly simplified by providing acceptance criteria in the form of plastic rotation angle.
4. Footnote 2 has several issues. Columns in moment frames are not designed for maximum force that can be delivered. Instead, they are designed for maximum axial force that can be delivered plus unreduced moments using equation 5-19 or 5-20. Therefore this footnote is applicable when axial load alone is present, such as brace frames. The next issue is reference to the “maximum force that can be delivered”. This should be replaced by “as force controlled component” in order to use lower bound for capacity.
5. Footnote 7: see comment 4 on Table 5-3.
6. The value c for panel zone is given as 1.0. By definition the value of c must be less than 1.
7. If there is no panel zone yielding what are these nonlinear deformation parameters referring to?
8. What is the difference between values under “b. panel zone yield” and those given under “panel zones” four rows above?

TABLE 5-6

1. For clarity, the heading should refer to “connection type and weakest link within the connection”.
2. Footnote 1 does not say what should be done if there is no web plate to carry shear.
3. See comment item 3 for Table 5-3.

TABLE 5-8

1. The conservatism associated with the m – values for braces in compression for Immediate Occupancy was discussed before. It is important to note that with the m – values indicated, it will be extremely difficult to design a brace frame for Immediate Occupancy.
2. In an eccentric brace frame, columns under tension are force controlled. Also the value of $c = 1.0$ is inconsistent with definitions in Chapter 2.
3. As indicated before, plastic deformation of link beam should be represented by plastic rotation angle. This will also eliminate the use of Footnote 3, which is rather awkward and probably not accurate in all cases.

CHAPTER 6 – CONCRETE

TABLES 6-6 AND 6-10

1. Sections ii and iii allow no inelastic action in beams, even though the new modified Table 6-7 does allow this in columns.

Recommendation: permit minimum inelastic action for “beams controlled by shear” and “inadequate splicing”.

TABLES 6-7 AND 6-11

1. In the new modified Table 6-7, “column controlled by shear” refers to paragraph 6.5.2.2.Biv. It is unclear what the reader is supposed to do.
2. There appears to be a typographical error for “conforming reinforcement over the entire length” under “collapse prevention”.
3. Section iii value are related to moment in the column and not axial or shear force. If this is correct, it should be specified.
4. Missing values indicate “force control” action. It should be indicated as a footnote.

TABLES 6-8 AND 6-12

1. In a nonlinear analysis, how could the design shear force exceed the shear capacity.
2. Joint shear deformation is permitted in secondary elements. It appears that the drift angles permitted are quite high when axial force in column is relatively large.

-
3. Recommendation: increase the level of axial force ratio from .4 to .7 or .8 with the associated plastic rotation angle of 0 (with interpolation in between).
 4. Footnote 3, the second sentence should be “design strength”, not “design shear force”.
 5. In Table 6-12 for linear analysis, definite “design shear force” in Footnote 3.
 6. Note that in Table 6-12, column axial force has no effect on joint m values (see comments in item 2 above).
 7. 6. Table 6-12, this is an interesting situation, the joint shear is first checked as a force controlled action in order to calculate V/V_n . The joint shear is then checked as deformation controlled action using m – values. Is this really what is intended? Isn’t joint ductility and strength a function of ductility demand of the connecting beam in flexure?

TABLE 6-14

- (1) m values for IO and LS cannot have the same numerical values. The reason is as follows: the coefficient C2 in calculation of pseudo-lateral load is higher for LS than IO. If the acceptability criteria is the same, certain buildings that pass IO would not pass LS because of the larger C2 coefficient!

TABLE 6-18

1. The headings are “drift ratio in percentage or chord rotation in radians”. The values should not be percent, but ratios or radians, therefore, the values under Section i should modify accordingly.
2. The deformation parameter for secondary element appears to be very low. There is typically no loss of gravity resistance associated with short coupling beams. Criteria for coupling beams cannot be the determining factor for strengthening or stiffening an existing building with short coupling beams.

TABLE 6-20

1. See item 2, Table 6-18.

CHAPTER 7 – MASONRY BUILDINGS

TABLE 7-4

1. The numerical values are given in percentage. For consistency, they should be changed to ratios.

TABLE 7-6, 7-7, 7-8 & 7-9

1. Indicate in these tables what should be done if the numerical value of the variables is outside of the range provided.

CHAPTER 8 – WOOD AND LIGHT METAL FRAMING

1. Change Figure 8 for conformity with other chapters. Delete the backbone curve and reference to V^u and V_y .
2. Section 8.4iv and 8.4v refers to connections developing 1.2 times the yield capacity of the wall. This is similar but not exactly the same as checking force controlled actions.

Recommendation: change the sentence to require treatment as a force controlled action.

TABLE 8-3

1. The number of significant figures or rounding off is different from other chapters. Recommendation: combine rows such as 1 x 6 and 1 x 10 sheathing, and round off m – values to the nearest 0.25 or 0.5.
2. For “structural panels” and “stucco on studs”, it appears that for secondary components, the taller the element, the easier it can accommodate the displacement of primary elements. Thus, the lower m – value for taller elements is questionable.
3. Footnote 1 -- When element height is more than the value indicated, the walls are not effective and therefore are secondary elements. Apparently, these secondary elements do not need to be checked for acceptability. (i.e. – no m – values as secondary elements provided).
4. Typographical error: For double diagonal sheathing for I.O. m – should be 1.25.
5. Under “connections”, for connection assemblies such as Simpson hold-downs, m – values should be provided.

TABLE 8-4

1. Footnote provides acceptability criteria for primary and secondary components. The equations for Life Safety, as provided, is not consistent with Chapter 2.

TABLE 8-5

1. This table provides ultimate capacity of structural systems. Equation 8-4 defines expected capacity to be equal to the ultimate strength. However, in Section 8-4vi, expected capacity is defined to be equal to yield strength, and in equation 8.3 yield strength is defined as being 80% of ultimate strength. This is inconsistent.

Since this document uses “expected capacity” throughout, it might be appropriate to delete references to yield strength and ultimate strength and change Table 8-5 to represent values for “expected capacity”. For all other cases, expected strength is defined as 80% of the maximum resistance provided by the element as determined from laboratory testing.

CONCLUDING RECOMMENDATIONS

A. As a result of the project team deliberations, the following conclusions were reached:

1. **Acceptability Criteria for IO**

Section 2.8.3.6 defines the acceptability criteria for LS & CP but not for IO within line 1-2 of the force-deformation curve. Plastic deformation limit for IO. should often be between 25 and 50 percent of plastic deformation limit for LS.

2. **Materials with Low Ductility**

In the Second Draft of ASCE 356, actions with low ductility often had an $m = 1.0$. Review of the test data for these non-ductile components indicate that limited ductility in the range of approximately 2 is often available, except in very few cases. Therefore, m -values should be increased accordingly.

The following table should be used as a guide for minimum acceptable values:

Primary / Secondary		
	Linear (m -value)	Nonlinear (plastic deformation)
IO	1.25	.0015
LS	1.5 / 1.75	.0020 / .003
IP	1.75 / 2.0	.003 / .004

3. Secondary Actions

m -values can be increased for secondary actions when gravity load resistance is provided.

Example: short coupling beams between shear walls.

These considerations were applied to the concrete and wood chapters as shown on the attached tables.

B. The masonry tables did not need change to address the above issues but may be modified under a separate study of this chapter.

C. The steel chapter was not changed because a major modification of this chapter is being implemented by another project team.

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions	Modeling Parameters ³			Acceptance Criteria ³			
	Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians			
				Performance Level			
			IO	Component Type			
				Primary		Secondary	
a	b	c	LS	CP	LS	CP	

i. Beams controlled by flexure¹

$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf. ²	$\frac{V}{b_w d \sqrt{f_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01

ii. Beams controlled by shear¹

Stirrup spacing ≤ d/2	0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2	0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01

iii. Beams controlled by inadequate development or splicing along the span¹

Stirrup spacing ≤ d/2	0.0	0.02	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2	0.0	0.01	0.0	0.0	0.0	0.005	0.01

iv. Beams controlled by inadeq

- When more than one of the conditio
- Under the heading "Transverse Rein is conforming if, within the flexural strength provided by the stirrups (V_u ...

0.0030 0.0015 0.0020 0.0030 0.03

3. Linear interpolation between values listed in the table shall be permitted.

6.5.2.4 Acceptance Criteria

6.5.2.4.1 Linear Static and Dynamic Procedures

[(6.5.2.4.A.i)] All actions shall be classified as being either deformation-controlled or force-controlled, as defined in Section 2.4.4. In primary components,

deformation-controlled actions shall be restricted to flexure in beams (with or without slab) and columns. In secondary components, deformation-controlled actions shall be restricted to flexure in beams (with or without slab), plus restricted actions in shear and reinforcement

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Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

Conditions	Modeling Parameters ⁴					Acceptance Criteria ⁴				
	Plastic Rotation Angle, radians		Residual Strength Ratio			Plastic Rotation Angle, radians				
						Performance Level				
	a		b			c			Component Type	
									Primary	
IO		LS		CP		LS		CP		
i. Columns controlled by flexure¹										
$\frac{P}{A_g f_c}$	Trans. Reinf. ²	$\frac{V}{b_w d \sqrt{f_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear^{1,3}										
All cases ⁵	—	—	—	—	—	—	—	—	—	—
iii. Columns controlled by inadequate development or splicing along the clear height^{1,3}										
Hoop spacing ≤ d/2	0.01	0.02	0.4	0.005	0.005	0.0				
Hoop spacing > d/2	0.0	0.01	0.2	0.0	0.0	0.0				
iv. Columns with axial loads exceeding 0.70P_o^{1,3}										
Conforming reinforcement over the entire length	0.015	0.025	0.02	0.0	0.005	0.0				
All other cases	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
- Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
- To qualify, hoops shall not be lap spliced in the cover concrete, and hoops shall have hooks embedded in the core or other details to ensure that hoops are adequately anchored following spalling of cover concrete.
- Linear interpolation between values listed in the table shall be permitted.
- For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.

Handwritten notes: $\frac{P}{A_g f_c}$ and $\frac{V}{b_w d \sqrt{f_c}}$ are circled. Next to them are handwritten values: .0030 and .0040.

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Table 6-9 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

Conditions	Modeling Parameters ⁴				Acceptance Criteria ⁴			
	Shear Angle, radians <i>Plastic</i>		Residual Strength Ratio		Plastic Rotation Angle, radians			
					Performance Level			
	Component Type		Primary		Secondary			
IO	LS	CP	LS	CP				

i. Interior joints

$\frac{P}{A_g f_c}$ ²	Trans. Reinf. ¹	$\frac{V}{V_n}$ ³								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.01	0.015

ii. Other joints

$\frac{P}{A_g f_c}$ ²	Trans. Reinf. ¹	$\frac{V}{V_n}$ ³								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0075	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	—	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	—	0.0	0.005	0.0	0.0	0.0

- Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming, respectively. A joint is conforming if closed hoops are spaced at $\leq h/3$ within the joint. Otherwise, the joint is nonconforming. Also, to qualify as conforming details under ii, hoops shall not be lap spliced in the column core, and shall have hooks embedded in the core or other details to ensure that hoops remain adequately anchored following spalling of cover concrete.
- This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force shall be calculated using limit analysis procedures as described in Chapter 3.
- This is the ratio of the design shear force to the shear strength for the joint. The design shear force shall be calculated according to Section 6.5.4.3.
- Linear interpolation between values listed in the table shall be permitted.

**Chapter 6: Concrete
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Table 6-11 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

Conditions			<i>m</i> -factors ³				
			Performance Level				
			Component Type				
			Primary		Secondary		
			IO	CP	LS	CP	
i. Beams controlled by flexure¹							
$\frac{p-p'}{p_{bal}}$	Trans. Reinf. ²	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.0	C	≤ 3	2	6	7	6	10
≤ 0.0	C	≥ 6	2	3	4	3	5
≥ 0.5	C	≤ 3	2	3	4	3	5
≥ 0.5	C	≥ 6	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	3	4	3	5
≤ 0.0	NC		1	2	3	2	4
≥ 0.5	NC		2	3	3	3	4
≥ 0.5	NC		1	2	2	2	3
ii. Beams controlled by s							
Stirrup spacing						3	4
Stirrup spacing						2	3
iii. Beams controlled by i							
Stirrup spacing						3	4
Stirrup spacing						2	3
iv. Beams controlled by i							
ment into be:			2			3	4

- When more than one of the conditions i, ii, iii, and iv occurs for a
- Under the heading "Transverse Reinforcement," "C" and "NC" a is conforming if, within the flexural plastic region, closed stirrups strength provided by the stirrups (V_s) is at least three-fourths of th
- Linear interpolation between values listed in the table shall be permitted.

numerical value from the table. details, respectively. A component rate and high ductility demand, the red nonconforming.

6.5.3 Post-Tensioned Concrete Beam-Column Moment Frames

described in Section 6.5.2.1, the analysis model shall consider potential failure of tendon anchorages.]

6.5.3.1 General Considerations

[(6.5.3.1.ii)The analysis procedures described in Chapter 3 shall apply to frames with post-tensioned beams satisfying the following conditions:

[(6.5.3.1.i)The analytical model for a post-tensioned concrete beam-column frame element shall be established following the criteria specified in Section 6.5.2.1 for reinforced concrete beam-column moment frames. In addition to potential failure modes

- The average prestress, f_{pc} , calculated for an area equal to the product of the shortest cross-sectional

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Table 6-12 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions			m-factors ⁴				
			Performance Level				
			IO	Component Type			
				Primary		Secondary	
		LS	CP	LS	CP		
I. Columns controlled by flexure¹							
$\frac{P}{A_g f'_c}$	Trans. Reinf. ²	$\frac{V}{b_w d \sqrt{f'_c}}$					
≤ 0.1	C	≤ 3	2	3	4	4	
≤ 0.1	C	≥ 6	2	2.4	3.2	3.2	
≥ 0.4	C	≤ 3	1	2	3	3	
≥ 0.4	C	≥ 6	1	1.6	2.4	2.4	
≤ 0.1	NC	≤ 3	2	2	3	2	
≤ 0.1	NC	≥ 6	2	1.6	2.4	1.6	
≥ 0.4	NC		1	1.5	2	1.5	
≥ 0.4	NC		1	1	1.6	1	
ii. Columns controlled by s							
Hoop spacing ≤ d/2.						2	
or $\frac{P}{A_g f'_c} \leq 0.1$						3	
Other cases						1.5	
iii. Columns controlled by ... development or splicing along the clear height^{1,3}							
Hoop spacing ≤ d/2			-	-	-	3	
Hoop spacing > d/2			-	-	-	2	
iv. Columns with axial loads exceeding 0.70P_o^{1,3}							
Conforming reinforcement over the entire length			1	1	2	2	
All other cases			-	-	-	1	

- When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
- Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
- To qualify, hoops shall not be lap spliced in the cover concrete, and shall have hooks embedded in the core or other details to ensure that hoops are adequately anchored following spalling of cover concrete.
- Linear interpolation between values listed in the table shall be permitted.

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-13 Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam-Column Joints

Conditions			m-factors ⁴				
			Performance Level				
			IO	Component Type			
				Primary ⁵		Secondary	
	LS	CP	LS	CP			
i. Interior joints							
$\frac{P}{A_g f_c}$ ²	Trans. Reinf. ¹	$\frac{V}{V_n}$ ³					
≤ 0.1	C	≤ 1.2	-	-	-	3	4
≤ 0.1	C	≥ 1.5	-	-	-	2	3
≥ 0.4	C	≤ 1.2	-	-	-	3	4
≥ 0.4	C	≥ 1.5	-	-	-	2	3
≤ 0.1	NC	≤ 1.2	-	-	-	2	3
≤ 0.1	NC	≥ 1.5	-	-	-	2	3
≥ 0.4	NC	≤ 1.2	-	-	-	2	3
≥ 0.4	NC	≥ 1.5	-	-	-	2	3
ii. Other joints							
$\frac{P}{A_g f_c}$ ²	Trans. Reinf. ¹	$\frac{V}{V_n}$ ³					
≤ 0.1	C	≤ 1.2	-	-	-	3	4
≤ 0.1	C	≥ 1.5	-	-	-	2	3
≥ 0.4	C	≤ 1.2	-	-	-	3	4
≥ 0.4	C	≥ 1.5	-	-	-	2	3
≤ 0.1	NC	≤ 1.2	-	-	-	2	3
≤ 0.1	NC	≥ 1.5	-	-	-	2	3
≥ 0.4	NC	≤ 1.2	-	-	-	1	1
≥ 0.4	NC	≥ 1.5	-	-	-	1	1

- Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming if closed hoops are spaced at $\leq h_c/3$ within the joint. Otherwise, the component is considered nonconforming; under it, hoops shall not be lap spliced in the cover concrete, and shall have hooks embedded in the core to be adequately anchored following spalling of cover concrete.
- This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area and the concrete compressive strength. The design axial force shall be calculated using limit analysis procedures as described in C.
- This is the ratio of the design shear force to the shear strength for the joint. The design shear force shall be calculated using limit analysis procedures as described in C.
- Linear interpolation between values listed in the table shall be permitted.
- All interior joints shall be force-controlled; m-factors shall not apply.

1.5 2.0 ag ill

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-18 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Flexure

Conditions	Plastic Hinge Rotation (radians)		Residual Strength Ratio		Acceptable Plastic Hinge Rotation (radians)					
					Performance Level					
					Component Type					
					Primary		Secondary			
	a	b	c	IO	LS	CP	LS	CP		
i. Shear walls and wall segments										
$\frac{(A_s - A'_s)f_y + P}{I_w I_w \sqrt{f'_c}}$	$\frac{\text{Shear}}{I_w I_w \sqrt{f'_c}}$	Confined Boundary ¹								
≤ 0.1	≤ 3	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.001	0.003	0.005	0.005	0.010

- Requirements for a confined boundary are the same as those given in ACI 318.
- Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ d/2, and (b) strength of closed stirrups $V_s \geq$ required shear strength of column.
- Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ d/2, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

0.0015

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-18 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Flexure (continued)

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation (radians)					
			a	b		c	Performance Level				
							IO	Component Type			
								Primary		Secondary	
LS	CP	LS	CP								
≤ 0.1	≤ 3	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015	
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010	
≥ 0.25	≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005	
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.002	0.003	0.003	0.004	

ii. Columns supporting discontinuous shear walls

Transverse reinforcement ²	a	b	c
Conforming	0.010	0.015	0.20
Nonconforming	0.0	0.0	0.0

plastic hinge

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation (radians)					
			a	b		c	Performance Level				
							IO	Component Type			
								Primary		Secondary (4)	
LS	CP	LS	CP								

plastic hinge
shear
rotation

iii. Shear wall coupling beams

Reinforcement	a	b	c	IO	LS	CP	LS	CP
Longitudinal reinforcement and transverse reinforcement ³		0.050		0.010	0.02			0.050
Conventional longitudinal reinforcement with conforming transverse reinforcement	0.025	0.040		0.005	0.010	0.025	0.025	0.040
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	0.020	0.035		0.005	0.012	0.020	0.020	0.035
Diagonal reinforcement	0.030	0.050	0.60	0.006	0.018	0.030	0.030	0.050

0.02 → *0.015* *0.030* ← *0.04*

- Requirements for a confined boundary are the same as those given in ACI 318.
- Requirements for closed stirrups: reinforcement are: (a) closed stirrups over the entire length of the column at a spacing $\leq d/2$, and (b) strength of $V_c \geq 3/4$ of required.
- Conventional for reinforcement or shear strength of *coupling* consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_c \geq 3/4$ of required.

(4) Secondary beams spanning $\angle 8'-0"$ with bottom reinforcement continuous into the supporting walls shall be permitted to

exceed these values.

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-19 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Shear

Conditions	Drift Ratio (%), or Chord Rotation (radians) ¹	Residual Strength Ratio	Acceptable Drift (%) or Chord Rotation (radians) ¹					
			Performance Level					
			Component Type					
			Primary			Secondary		
	d	e	c	IO	LS	CP	LS	CP
I. Shear walls and wall segments								
All shear walls and wall segments ²	0.75	2.0	0.40	0.40	0.80	0.75	0.75	1.5
II. Shear wall coupling beams (4)								
Longitudinal reinforcement and transverse reinforcement ³	$\frac{\text{Shear}}{f_w w \sqrt{f_c}}$							
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.018	0.030	0.60	0.006	0.012	0.015	0.024
	≥ 6	0.012 16	0.020 24	0.30	0.004 5	0.008 12	0.010 16	0.018 24
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.008	0.008	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.006	0.007	0.012

- For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-3 and 6-4.
- For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15 A_g f_c$; otherwise, the member must be treated as a force-controlled component.
- Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

given in Section 6.4.5. Reduced flexural strengths shall be evaluated at locations where splices govern the usable stress in the reinforcement. The need for confinement reinforcement in shear wall boundary members shall be evaluated by the procedure in ACI 318 or other approved procedure.]

[(6.8.2.3.vii)The nominal flexural and shear strengths of coupling beams shall be evaluated using the principles and equations contained in Chapter 21 of ACI 318. The strength of longitudinal or diagonal reinforcement shall be taken equal to 125% of the specified yield strength.]

[(6.8.2.3.viii)The nominal shear and flexural strengths of columns supporting discontinuous shear walls shall be evaluated as defined in Section 6.5.2.3.]

6.8.2.3 Strength
[(6.8.2.3.ii)For an uncracked member (as defined in Section 6.5.2.3.1) that is subjected to shear, the required transverse reinforcement shall be provided with a steel ratio of 0.0015.]

(4) Use same sentence as in table 6-18

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-20 Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Flexure

Conditions	m-factors						
	Performance Level						
	IO	Component Type					
		Primary		Secondary			
	LS	CP	LS	CP			
i. Shear walls and wall segments							
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	Shear (5) $\frac{V}{t_w l_w f'_c}$	Confined Boundary ¹					
≤ 0.1	≤ 3	Yes	2	4	6	6	8
≤ 0.1	≥ 6	Yes	2	3	4	4	6
≥ 0.25	≤ 3	Yes	1.5	3	4	4	6
≥ 0.25	≥ 6	Yes	1	2	2.5	2.5	4
≤ 0.1	≤ 3	No	2	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	2	2.5	2.5	4
≥ 0.25	≤ 3	No	1	1.5	2	2	3
≥ 0.25	≥ 6	No	1	1	1.5	1.5	2
ii. Columns supporting discontinuous shear walls							
Transverse reinforcement ²							
Conforming			1	1.5	2	n.a.	n.a.
Nonconforming			1	1	1	n.a.	n.a.
iii. Shear wall coupling beams							
Longitudinal reinforcement and transverse reinforcement ³		Shear (5) $\frac{V}{t_w l_w f'_c}$					
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	4	6	6	9
		≥ 6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	3.5	5	5	8
		≥ 6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	5	7	7	10

- Requirements for a confined boundary are the same as those given in ACI 318.
- Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ d/2, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
- Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ d/3, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

(4) Same as in table 6-18

(5) Shear calculated as force controlled action

**Chapter 6: Concrete
(Systematic Rehabilitation)**

Table 6-21 Numerical Acceptance Criteria for Linear Procedures—Members Controlled by Shear

Conditions	ID	m-factors				
		Performance Level				
		Component Type				
		Primary		Secondary		
	LS	CP	LS	CP		
i. Shear walls and wall segments						
All shear walls and wall segments ¹	2	2	3	2	3	
ii. Shear wall coupling beams (3)						
Longitudinal reinforcement and transverse reinforcement ²						
		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$				
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	3	4	4	6
	≥ 6	1.2	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2.5	3	3	4
	≥ 6	1.2	1.5	1.5	1.5	2.5

- For shear walls and wall segments where inelastic behavior is governed by shear, reinforcement must be symmetrical, and the maximum shear demand must be ≤ 6_v action.
- Conventional longitudinal reinforcement consists of top and bottom steel parallel to reinforcement consists of: (a) closed stirrups over the entire length of the beam at a shear strength of beam.

er must be ≤ 0.15 Ag f_c, the longitudinal shall be considered to be a force-controlled
e beam. Conforming transverse gth of closed stirrups V_t ≥ 3/4 of required

1.25

requirements of Section 6.4.7 and other provisions of this standard.

6.8.2.5 Rehabilitation Measures
(6.8.2.5) The following measures may be effective in rehabilitating reinforced shear walls, wall segments, coupling beams, and reinforced concrete columns supporting discontinuous shear walls:
Addition of wall boundary members. Addition of boundary members may be an effective measure in strengthening shear walls or wall segments that have insufficient flexural strength. These members may be either cast-in-place reinforced concrete elements or steel sections. In both cases, proper connections should be made between the existing wall and the added members. The shear capacity of the rehabilitated wall should be re-evaluated.

Addition of confinement jackets at wall boundaries. Increasing the confinement at the wall boundaries by the addition of steel or reinforced concrete jackets may be an effective measure to improve the flexural deformation capacity of a shear wall. For each type of jacket, the longitudinal reinforcement should be continuous from story to story unless the jacket is also being used to increase the flexural capacity. The minimum thickness for a concrete jacket should be 2 inches. Carbon fiber wrap should be permitted for improving the confinement of concrete in compression.
Reduction of flexural strength. Reduction in the flexural capacity of a shear wall to change the governing failure mode from shear to flexure may be an effective rehabilitation measure. It may be accomplished by providing a specific number of longitudinal bars along the edges of the shear wall.

(3) See footnote (4) table 6-18

**Chapter 8: Wood and Light Metal Framing
(Systematic Rehabilitation)**

Table 8-3 Numerical Acceptance Factors for Linear Procedures—Wood Components

	Height/Length Ratio (h/L) ¹	m-factors for Linear Procedures ²				
		IO	Primary		Secondary	
			LS	CP	LS	CP
Shear Walls						
Horizontal 1" x 6" Sheathing	h/L < 1.0	1.8	4.2	5.0	5.0	5.5
Horizontal 1" x 10" Sheathing	h/L < 1.0	1.6	3.4	4.0	4.0	5.0
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	h/L < 1.5	1.4	2.6	3.0	3.1	4.0
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	h/L < 1.5	1.3	2.3	2.6	2.8	3.0
Diagonal 1" x 6" Sheathing	h/L < 1.5	1.5	2.9	3.3	3.4	3.8
Diagonal 1" x 8" Sheathing	h/L < 1.5	1.4	2.7	3.1	3.1	3.6
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	h/L < 2.0	1.3	2.2	2.5	2.5	3.0
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	h/L < 2.0	1.3	2.0	2.3	2.5	2.8
Double Diagonal 1" x 6" Sheathing	h/L < 2.0	1.2	1.8	2.0	2.3	2.5
Double Diagonal 1" x 8" Sheathing	h/L < 2.0	1.2	1.7	1.9	2.0	2.5
Vertical 1" x 10" Sheathing	h/L < 1.0	1.0			3.6	4.1
Structural Panel or Plywood Panel Sheathing or Siding	h/L < 1.0*	1.0			4.5	5.5
	h/L > 2.0* h/L < 3.5	1.0	6.0		3.0	4.0 7.0
Stucco on Studs	h/L < 1.0*	1.0			3.6	4.0
	h/L = 2.0*	1.0	5.0		2.5	3.0 6.0
Stucco over 1"-x Horizontal Sheathing	h/L < 2.0	1.0			3.5	4.0
Gypsum Plaster on Wood Lath	h/L < 2.0	1.0			4.6	5.1
Gypsum Plaster on Gypsum Lath	h/L < 2.0	1.8	4.2	5.0	4.2	5.5
Gypsum Plaster on Metal Lath	h/L < 2.0	1.7	3.7	4.4	3.7	5.0
Gypsum Sheathing	h/L < 2.0	1.9	4.7	5.7	4.7	6.0
Gypsum Wallboard	h/L < 1.0*	1.9	4.7	5.7	4.7	6.0
	h/L = 2.0*	1.6	3.4	4.0	3.8	4.5
Horizontal 1" x 6" Sheathing with Cut-In Braces or Diagonal Blocking	h/L < 1.0	1.7	3.7	4.4	4.2	4.8
Fiberboard or Particleboard Sheathing	h/L < 1.5	1.6	3.2	3.8	3.8	5.0

- For ratios greater than the maximum listed values, the component shall be considered not effective in resisting lateral loads.
- Linear interpolation shall be permitted for intermediate values if an asterisk appears next to the h/L or L/b value.

(i.e. treated as Secondary Component with m = ∞)

**Chapter 8: Wood and Light Metal Framing
(Systematic Rehabilitation)**

Table 8-4 Normalized Force-Deflection Curve Coordinates for Nonlinear Procedures—Wood Components

Shear Wall Type—Types of Existing Wood and Light Frame Shear Walls	Height/Length Ratio h/L ¹	d	e	c
Horizontal 1" x 6" Sheathing	$h/L < 1.0$	5.0	6.0	0.3
Horizontal 1" x 10" Sheathing	$h/L < 1.0$	4.0	5.0	0.3
Horizontal Wood Siding Over Horizontal 1" x 6" Sheathing	$h/L < 1.5$	3.0	4.0	0.2
Horizontal Wood Siding Over Horizontal 1" x 10" Sheathing	$h/L < 1.5$	2.6	3.6	0.2
Diagonal 1" x 6" Sheathing	$h/L < 1.5$	3.3	4.0	0.2
Diagonal 1" x 8" Sheathing	$h/L < 1.5$	3.1	4.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 6" Sheathing	$h/L < 2.0$	2.5	3.0	0.2
Horizontal Wood Siding Over Diagonal 1" x 8" Sheathing	$h/L < 2.0$	2.3	3.0	0.2
Double Diagonal 1" x 6" Sheathing	$h/L < 2.0$	2.0	2.5	0.2
Double Diagonal 1" x 8" Sheathing	$h/L < 2.0$	2.0	2.5	0.2
Vertical 1" x 10" Sheathing	$h/L < 1.0$	3.6	4.0	0.3
Structural Panel or Plywood Panel Sheathing or Siding	$h/L < 1.0^2$	4.5	5.5	0.3
	$h/L > 2.0^2$ $h/L < 3.5$	3.0	4.0	0.2
Stucco on Studs	$h/L < 1.0^2$	3.6	4.0	0.2
	$h/L = 2.0^2$	2.5	3.0	0.2
Stucco Over 1" x Horizontal Sheathing	$h/L < 2.0$	3.5	4.0	0.2
Gypsum Plaster on Wood Lath	$h/L < 2.0$	4.6	5.0	0.2
Gypsum Plaster on Gypsum Lath	$h/L < 2.0$	5.0	6.0	0.2
Gypsum Plaster on Metal Lath	$h/L < 2.0$	4.4	5.0	0.2
Gypsum Sheathing	$h/L < 2.0$	5.7	6.3	0.2
Gypsum Wallboard	$h/L < 1.0^2$	5.7	6.3	0.2
	$h/L = 2.0^2$	4.0	5.0	0.2

- For ratios greater than the maximum listed values, the component shall be considered not effective in resisting lateral loads. (A)
- Linear interpolation shall be used for intermediate values.

Notes: (a) Acceptance criteria for primary components 0.25

$$\begin{aligned} (\Delta/\Delta_c) IO &= 1.0 + 0.2(d - 1.0) \\ (\Delta/\Delta_c) LS &= 1.0 + 0.8(d - 1.0) \\ (\Delta/\Delta_c) CP &= d \end{aligned}$$

(b) Acceptance criteria for secondary components 0.75

$$\begin{aligned} (\Delta/\Delta_c) LS &= d \\ (\Delta/\Delta_c) CP &= c \end{aligned}$$

(c) Linear interpolation shall be permitted for intermediate values if asterisks appear next to the h/L or L/b values.

(i.e. Secondary Component
With $e = \infty$) (13) of (13)

**I. Special Study 7—
Report on Study of C-Coefficients**

ASCE/FEMA 273 Prestandard Project Report on Study of C-Coefficients

William T. Holmes

October 7, 1999

Purpose

The purpose of the study was to review several perceived issues concerning the C-coefficients to see if any changes or clarifications are justified at this time. The issues reviewed were:

- A. The interaction of C_2 and C_3 . To a degree, both are increasing displacements due to negative post-yield stiffness. Are they “double-counting” for this effect?

The value of C_2 based on performance level. In the linear procedures, C_2 increases pseudo lateral loads (displacements) with declining performance levels, to account for increased importance of poor hysteretic behavior. In conventional, force-based design, loads are increased for superior performance levels (to decrease nonlinearity and damage). The effect of C_2 thus has been confusing to some users.

Inadequate displacement demand in the nonlinear static procedure for very weak buildings or for buildings with brittle “secondary-type” elements.

- B. In the nonlinear static procedure, C_1 is a measure of both period (increases with decreasing period) and strength of the structure (increases with decreasing strength—as measured by $1/R$). However, C_1 is capped by the value used in the linear static method, which was set at 1.5 and is proportional only to period. There was concern that the capping was minimizing or eliminating the intended penalty for weak structures.
- C. In the nonlinear static procedure, all elements must be modeled—including those that might be classified as secondary in the linear static procedure. This increases the elastic period and decreases the target displacement—even if weak and brittle elements, such as spandrels, fail at low loading.

Summary of Recommended Changes

Although all of the issues describe above are valid to some degree, there is currently no justification to make significant changes to the prestandard document.

A valid argument could be made to address issue B by making C_2 a function of the DCRs (demand capacity ratios) developed in Section 2.4.1, but a translation of Table 3-1 to equivalent DCRs would be compounding the judgmental nature of C_2 in the first place (see FEMA 274, pages 3-14, 3-15). This rather arbitrary change is not recommended at this time.

It is recommended to add a footnote to Table 3-1 referenced to the title Performance Level in column 1:

Footnote 3. Performance Level used for C_2 is not necessarily the performance level designated by the Performance Objective, but may be taken as the level actually achieved as judged by the performance of the components. Linear interpolation may be used to estimate values of C_2 for intermediate performance levels.

Issue A. Interaction of C_2 and C_3

FEMA 274 suggests that both coefficients are considering post-elastic negative stiffness, but C_2 is primarily measuring pinched hysteretic behavior (which often implies stiffness or strength degradation) and C_3 is triggered when post-elastic negative stiffness is probably caused by P- Δ effects (related to θ , the stability coefficient). Although the formulation of C_3 in the nonlinear procedures is not directly (numerically) related to P- Δ , its value is limited by the values obtained in the linear static procedure—which is directly related to θ .

In addition, C_2 is not assigned based on post-elastic negative stiffness, but based on use of certain systems known to exhibit pinched hysteretic behavior. Since C_2 was assigned largely by judgement (FEMA 274, pages 3-14, 3-15), the appropriate interaction with C_3 is not apparent.

No changes or clarifications are recommended at this time. A more systematic study to test the interaction between C_2 and C_3 , perhaps using statistical methods similar to those originally used for C-coefficients, is recommended.

Issue B. C_2 Related to performance level

C_2 is measuring the effect of pinched hysteretic behavior on inelastic displacement. For buildings designed at or near limiting component acceptability limits, more inelastic behavior would occur in buildings designed to *Collapse Prevention* than when designed to *Immediate Occupancy* and therefore C_2 should be larger for the CP case. Although counter to elastic-force based procedures, this increase in C_2 is correct.

Almost as a separate issue from the confusion described above, the performance level is not a direct measure of the extent of inelastic behavior, particularly in zones of moderate and low seismicity. For example, a building assigned a desirable performance level of *Life Safety* could meet the acceptability criteria of that level and actually remain nearly elastic—more closely associated with *Immediate Occupancy*. A more direct measure of nonlinear behavior would be some combination of the DCRs defined in Section 2.4.1, or some other measure of the extent of actual nonlinear behavior.

The values of C_2 contained in Table 3-1 could be translated for each performance level into equivalent DCRs or a system of weighted DCRs. However, this translation would be compounding the judgmental nature of C_2 in the first place (see FEMA 274, pages 3-14, 3-15). This rather arbitrary change is not recommended at this time.

However, a clarification is recommended to allow an engineer to use a lower C_2 if the structure under consideration meets or nearly meets a superior performance level to the targeted performance level.

It is recommended to add a footnote to Table 3-1 referenced to the title Performance Level in column 1:

Footnote 3. Performance Level used for C_2 is not necessarily the performance level designated by the Performance Objective, but may be taken as the level actually achieved as judged by the performance of the components. Linear interpolation may be used to estimate values of C_2 for intermediate performance levels.

Issue C. Inadequate displacement demand for weak buildings or buildings with brittle secondary elements

In the nonlinear static procedure, C_1 is a measure of both period (increases with decreasing period) and strength of the structure (increases with decreasing strength-as measured by $1/R$). However, C_1 is capped by the value used in the linear static method, which was set at 1.5 and is proportional only to period. There was concern that the capping was minimizing or eliminating the intended penalty for weak structures.

The relationship between R , T and C_1 is shown in Table 1 below. Capping of C_1 at a maximum of 1.5 affects shaded values. It can be seen that capping, in general, only affects buildings with periods less than 0.3 seconds. The additional effects of small strengths (high R s) for realistic building periods of 0.2 seconds and above reduces displacement demands to values 60% to 80% of the value yielded by the formula. This inconsistency is one of several created by the rule used during development of 273 to the effect that the nonlinear procedures should not be more conservative than the linear procedures.

Table 1: Relationship between T, R, and C₁

0.5		Characteristic Period of response Spectrum (T ₀)								
		T Period (sec)								
		0.1	0.15	0.2	0.25	0.3	0.35	0.4	0.45	0.5
R factor	12	4.67	3.14	2.38	1.92	1.61	1.39	1.23	1.10	1.00
	10	4.60	3.10	2.35	1.90	1.60	1.39	1.23	1.10	1.00
	8	4.50	3.04	2.31	1.88	1.58	1.38	1.22	1.10	1.00
	6	4.33	2.94	2.25	1.83	1.56	1.36	1.21	1.09	1.00
	4	4.00	2.75	2.13	1.75	1.50	1.32	1.19	1.08	1.00
	2	3.00	2.17	1.75	1.50	1.33	1.21	1.13	1.06	1.00

This underestimation of displacement demand will affect only a small number of structures that obviously will be short and stiff. Use of the nonlinear procedure will probably be rare with this type of structure. The small number of buildings affected, coupled with the lack of damage generally noted in these buildings and the complexity of the relationship with the linear procedure, indicates that a “fix” for this condition is not justified at this time.

In the nonlinear static procedure, all elements must be modeled—including those that might be classified as secondary in the linear static procedure. This increases the elastic period and decreases the target displacement—even if weak and brittle elements, such as spandrels, fail at low loading.

The method used to create an equivalent bilinear model of the buildings, as shown in Figure 3-1, is intended to take care of this structure. The effective period will be lengthened from the elastic period by the redefinition of the elastic slope portion of the curve. There are many shapes of pushover curves possible and this technique will work better on some than others. For example, failure of spandrels above 0.6V_y will not be well represented. However, no systematic underestimation of displacement is apparent.

No change or clarifications associated with Issue C has been identified.

**J. Special Study 8—
Incorporation of Selected Portions of
Recent Related Documents**

**ASCE/FEMA-273 PRESTANDARD PROJECT
Special Study Report**

**INCORPORATION OF SELECTED PORTIONS OF
RECENT RELATED DOCUMENTS**

FEMA-306: *Evaluation of Earthquake Damaged Concrete and Masonry Wall
Buildings - Basic Procedures Manual*

FEMA-307: *Evaluation of Earthquake Damaged Concrete and Masonry Wall
Buildings - Technical Resources*

FEMA-308: *Repair of Earthquake Damaged Concrete and Masonry Wall
Buildings*

ATC-40: *Seismic Evaluation and Retrofit of Concrete Buildings*

prepared by

**APPLIED TECHNOLOGY COUNCIL
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October 7, 1999

1. Introduction

The purpose of this report is to present and discuss certain modifications to FEMA-273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. The modifications are proposed for incorporation as the document evolves into a prestandard. These modifications result from the coordination of selected portions of four recent related documents (ATC-40, FEMA-306, FEMA-307, and FEMA-308). As FEMA-273 has been applied in practice, issues have arisen regarding application of certain procedures, interpretation of some provisions, and results stemming from portions of the document. These issues have been formulated into issue statements and assembled in this report for reference during the prestandard process (ASCE, 1999a). In addition, it is also expected that anecdotal experiences from user groups and reports from the Building Seismic Safety Council (BSSC) case study projects, when completed, will identify issues that will need to be addressed further.

Basic information on ATC-40: *Seismic Evaluation and Retrofit of Concrete Buildings* is presented in Section 2 of this report. Similarly, Section 3 summarizes the ATC-43 project on the evaluation and repair of earthquake-damaged concrete and masonry wall buildings. The ATC-43 project resulted in the preparation of three documents, namely, FEMA-306: *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, FEMA-307: *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Technical Resources*, and FEMA-308: *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*. A general overview of the changes proposed for FEMA-273 is contained in Section 4. The proposed modifications augment FEMA-273 with readily available excerpts and cross-references to enhance the technical quality of the document and facilitate its use by the practitioner in the short term. Further in the future, changes may be supported by more detailed incorporation of the information in the related documents. Recommendations for this process are summarized in Section 5. References are listed in Section 6. Finally, the modifications proposed for incorporation into the first draft of the FEMA-273 prestandard are contained in the Appendix.

2. ATC-40: Seismic Evaluation and Retrofit of Concrete Buildings

Background

Proposition 122, passed by California's voters in 1990, created the Earthquake Safety and Public Buildings Rehabilitation Fund of 1990, supported by a \$300 million general obligation bond program for the seismic retrofit of state and local government buildings. As a part of the program, Proposition 122 authorizes the Seismic Safety Commission (SSC) to spend up to 1% of the proceeds of the bonds, or approximately \$3 million, to carry out a range of activities that will capitalize on the seismic retrofit experience in the private sector to improve seismic retrofit practices for government buildings. The purpose of California's Proposition 122 research and development program is to develop state-of-the-practice recommendations to address current needs for seismic retrofit provisions and seismic risk decision tools. The program is focused specifically on vulnerable concrete structures consistent with the types of concrete buildings that make up a significant portion of California's state and local government inventories.

In 1994, as part of the Proposition 122 Seismic Retrofit Practices Improvement Program, the Commission awarded the Applied Technology Council (ATC) a contract to develop a recommended methodology and commentary for the seismic evaluation and retrofit of existing concrete buildings (Product 1.2). In 1995 the Commission awarded a second, related contract to ATC to expand the Product 1.2 effort to include effects of foundations on the seismic performance of existing concrete buildings (Product 1.3). The results of the two projects have been combined and are presented in the ATC-40 Report (also known as SSC-96-01).

Two other reports recently published by the California Seismic Safety Commission, the *Provisional Commentary for Seismic Retrofit* (1994) and the *Review of Seismic Research Results on Existing Buildings* (1994), are Products 1.1 and 3.1 of the Proposition 122 Program, respectively. These two reports provide the basis for the development of the recommended methodology and commentary contained in the ATC-40 document.

The ATC-40 document is organized into two volumes. Volume One contains the main body of the evaluation and retrofit methodology, presented in 13 chapters, with a glossary and a list of references. This volume contains all of the parts of the document required for the evaluation and retrofit of buildings. Volume Two consists of appendices containing supporting materials related to the methodology: four example building case study reports, a cost-effectiveness study related to the four building studies, and a review of research on the effects of foundation conditions on the seismic performance of concrete buildings.

The ATC-40 project was conducted under the direction of ATC Senior Consultant Craig Comartin, who served as Principal Investigator, and Richard W. Niewiarowski, who served as Co-Principal Investigator and Project Director. Fred Turner served as SSC Project Manager. Overview and guidance were provided by the Proposition 122 Oversight Panel consisting of Frederick M. Herman (Chair), Richard Conrad, Ross Cranmer, Wilfred Iwan, Roy Johnston, Frank McClure, Gary McGavin, Joel McDonald, Joseph P. Nicoletti, Stanley Scott, and Lowell Shields. The Product 1.2 methodology and commentary were prepared by Sigmund A. Freeman, Ronald O. Hamburger, William T. Holmes, Charles Kircher, Jack P. Moehle, Thomas A. Sabol, and Nabih Youssef (Product 1.2 Senior Advisory Panel). The Product 1.3 Geotechnical/Structural Working Group consisted of Sunil Gupta, Geoffrey Martin, Marshall Lew, and Lelio Mejia. William T. Holmes, Yoshi Moriwaki, Maurice Power and Nabih Youssef served on the Product 1.3 Senior Advisory Panel. Gregory P. Luth and Tom H. Hale served as the Quality Assurance Consultant and the Cost Effectiveness Study Consultant, respectively.

Key Features

The ATC-40 document is a comprehensive, technically sound methodology and supporting commentary for the seismic evaluation and retrofit design of existing concrete buildings. The document applies to the overall structural system and its seismic elements (concrete frames, shear walls, diaphragms, foundations) and components (stiffness, strength, and deformability of columns, beams, walls, slabs, and joints). Consideration of nonstructural systems and components is also included.

The methodology is performance-based: the evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Acceptable performance is measured by the level of structural and nonstructural damage expected from the earthquake shaking. Damage is expressed in terms of post-yield, inelastic, deformation limits for various seismic elements and structural components found in concrete buildings. The analytical procedure incorporated in the methodology accounts for postelastic deformations of the structure by using simplified nonlinear static analysis methods.

The information is presented in the form of a step-by-step procedure for both evaluation and retrofit of existing buildings. The procedure recognizes, however, that some steps may be de-emphasized or performed in a different order on a case-by-case basis.

The primary components of the procedure used in various steps of the evaluation and retrofit procedure include:

- ◆ definitions of seismic performance levels and seismic demand criteria for establishing seismic performance objectives,
- ◆ guidance for the review of existing conditions, preliminary determination of deficiencies, formulation of a retrofit strategy, and for establishing an appropriate quality assurance program,

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- ◆ analytical methods or techniques for detailed investigations to assess seismic capacity and expected seismic performance of existing buildings and for verification of retrofit performance, and
 - ◆ materials characteristics rules and assumptions for use in modeling, assignment of capacities, and assessment of acceptable performance.

Modeling rules and acceptance limits are provided for a variety of reinforced, cast-in-place, concrete seismic elements and components, including beam-column frames; slab-column frames; solid, coupled, and perforated shear walls; concrete diaphragms; and foundations. Unreinforced masonry infill and precast concrete components are not considered in the document. These rules, assumptions, and limits are included for existing, non-complying seismic elements and components, and for new, complying, seismic elements and components used in retrofits.

The methodology includes guidelines for the consideration of foundation-soil effects. Detailed modeling rules and acceptance limits for various types of foundations and foundation-structure combinations in various soil conditions are included.

The analytical procedure used in the document is simplified nonlinear static analysis. Several methods of performing nonlinear static analyses are presented, although the capacity spectrum method is emphasized. Other analytical methods are also noted and discussion is provided to assist the retrofit professional in the selection of an analytical procedure appropriate for use in the detailed analysis of a particular building.

Relationship to FEMA-273

From a basic technical perspective ATC-40 is very similar to and compatible with FEMA-273/274. The ATC-40 characterization of seismic hazard, including ground shaking, focuses on California, but is consistent with the technical procedures of FEMA-273. There are relatively minor differences between the documents in the nomenclature used for performance objectives. The modeling rules and acceptability criteria for generating a “pushover” curve for a concrete building for use in a nonlinear static analysis procedure (NSP) are essentially the same.

There are several specific differences between the two documents. Although the basic procedures of ATC-40 are applicable to other building types, the materials information is limited to concrete buildings. FEMA-273/274 provides information on a wider range of structural materials. ATC-40 recommends the use of the NSP for the analysis of concrete buildings. It documents the detailed development of the capacity spectrum method for determining displacement demand, but states that the coefficient method is an acceptable alternative. It recognizes the efficacy of other analysis alternatives in some cases, but does not provide detailed guidance on their application. FEMA 273/274 documents several alternatives for analysis including the NSP. It provides a detailed development of the coefficient method while allowing the capacity spectrum method as an alternative.

The most fundamental difference is the tone and purpose of the documents. FEMA-273/274, in its original form, is written to provide specific requirements for engineers. The scope is limited to technical details. The language is generally prescriptive. The basic purpose of the FEMA-273 *Guidelines* is to serve as a framework for the development of future codes and standards. In fact, the current re-writing as a Prestandard is another step in that direction. In contrast, the objective of ATC-40 is to provide technical guidance within the broader context of the evaluation and retrofit process. Consequently, the language is expansive and explanatory. The intention is that ATC-40 be an application manual that covers a wide range of activities and technical alternatives.

3. ATC-43: Evaluation and Repair of Earthquake Damaged Concrete and Masonry Wall Buildings

Background

Following the two damaging California earthquakes in 1989 (Loma Prieta) and 1994 (Northridge), many concrete wall buildings and masonry wall buildings were repaired using federal disaster-assistance funding. The repairs were based on inconsistent criteria, giving rise to substantial controversy regarding criteria for the repair of cracked concrete and masonry wall buildings. To help resolve this controversy, the Federal Emergency Management Agency (FEMA) initiated in 1996 a project on evaluation and repair of earthquake-damaged concrete wall buildings and masonry wall buildings. The project was conducted through the Partnership for Response and Recovery (PaRR), a joint venture of Dewberry & Davis of Fairfax, Virginia, and Woodward-Clyde Federal Services of Gaithersburg, Maryland. The Applied Technology Council (ATC), under subcontract to PaRR, was responsible for criteria and procedures development (the ATC-43 project).

The ATC-43 project addressed the investigation and evaluation of earthquake damage and policy issues relating to the repair and upgrade of earthquake-damaged buildings. The project dealt with buildings whose primary lateral-force-resisting systems consist of concrete or masonry bearing walls, or whose vertical-load-bearing systems consists of concrete or steel frames with concrete or masonry infill panels. The intended audience consists of design engineers, building owners, building regulatory officials, and government agencies.

The project results are reported in three documents:

FEMA-306: *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedures Manual*, provides guidance on evaluation of damage and analysis of performance. Included in the document are component damage classification guides, and test and inspection guides.

FEMA-307, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Technical Resources*, contains supplemental information, including results from a theoretical analysis of the effects of prior damage on single-degree-of-freedom mathematical models, additional background information on the component guides, and an example of the application of the basic procedures.

FEMA-308, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, discusses the technical and policy issues pertaining to the repair of earthquake-damaged buildings.

The project included a workshop to provide an opportunity for the user community to review and comment on the proposed evaluation and repair criteria. The workshop, open to the profession at large, was held in Los Angeles on June 13, 1997 and was attended by 75 participants.

The ATC-43 project was conducted under the direction of ATC Senior Consultant Craig Comartin, who served as Co-Principal Investigator and Project Director. Technical and management direction were provided by a Technical Management Committee consisting of Christopher Rojahn (Chair), Craig Comartin (Co-Chair), Daniel Abrams, Mark Doroudian, James Hill, Jack Moehle, Andrew Merovich (ATC Board Representative), and Tim McCormick. The Technical Management Committee created two Issue Working Groups to pursue directed research to document the state of the knowledge in selected key areas as follows: (1) an Analysis Working Group, consisting of Mark Aschheim (Group Leader) and Mete Sozen (Senior Consultant); and (2) a Materials Working Group, consisting of Joe Maffei (Group Leader and Reinforced Concrete Consultant), Greg Kingsley (Reinforced Masonry Consultant), Bret Lizundia (Unreinforced Masonry Consultant), John Mander (In-Filled Frame Consultant), Brian Kehoe and other consultants from Wiss, Janney, Elstner and Associates (Tests, Investigations, and Repairs Consultant). A Project Review Panel provided technical overview and guidance. The Panel members were Gregg Borchelt, Gene Corley, Edwin Huston, Richard Klingner, Vilas Mujumdar, Hassan Sassi, Carl Schulze, Daniel Shapiro, James Wight, and Eugene Zeller.

Key Features

The basic premise of the documents is that when an earthquake causes structural damage to a building, the anticipated performance of the building during a future earthquake may change. The difference, if any, in the performance of the damaged building versus the undamaged building is a loss resulting from the structural damage caused by the damaging earthquake. The direct and indirect costs of hypothetical measures to restore the anticipated future performance to that of the building in its undamaged state represent the magnitude of this loss. The procedures and criteria documented in the three volumes address:

- a. the investigation and documentation of damage caused by earthquakes,
- b. the classification of the damage for building components according to the mode of structural behavior and the severity of the damage,
- c. the evaluation of the effects of the damage on the anticipated performance of the building during future earthquakes,
- d. the development of hypothetical measures that would restore the anticipated performance to that of the undamaged building,
- e. a policy framework to facilitate decisions on acceptance of damage, restoration to pre-event conditions, or upgrade of performance, and
- f. procedures for the repair of damaged components.

The evaluation procedure assumes that when an earthquake causes damage to a building, a competent engineer can assess the effects, at least partially, through visual inspection augmented by investigative tests, structural analysis, and knowledge of the building construction. The documents provide detailed guidance on the documentation of damage. By determining how the structural damage has changed structural properties, it is possible to compare analytically the future performance of the damaged building with that for undamaged conditions. This is accomplished using component modification factors selected on the basis of the observed damage. It is also feasible to develop potential actions (Performance Restoration Measures) to restore the damaged building to a condition such that its future earthquake performance would be essentially equivalent to that of the undamaged building. The documents contain outline specifications for these measures.

Relationship to FEMA 273

FEMA-306/307/308 essentially extend the nonlinear static procedures of FEMA-273 to address the evaluation and repair of earthquake-damaged buildings. Nonlinear static analysis procedures are used to evaluate the capability of the building in its undamaged condition to meet a selected performance objective. The components of the structural model are then modified to reflect the effects of the damage and the model is re-analyzed for the same performance objective. The change in performance capability is a measure of the effects of the damage. The effectiveness of repair measures may be evaluated similarly by modifying the structural components to reflect the repair measures and analyzing performance capability once again.

FEMA-306/307/308 focuses on concrete wall buildings and masonry wall buildings, although the basic approach could be applied to buildings in general. Component strength and acceptability criteria for wall components rely on FEMA-273 recommendations as a starting point, but other formulations are recognized as acceptable to reflect observed behavior and properties. Also, component behavior modes are delineated and discussed much more extensively.

4. Overview of Proposed Modifications to FEMA-273

There is much in ATC-40 and FEMA-306/307/308 that can enhance the prestandard version of FEMA-273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. The approach that has been taken at this initial coordination stage has been to incorporate information directly, and by reference, into FEMA-273 from the other documents where possible. Thus changes that would require extensive re-writing or re-structuring of the prestandard have not been developed. In the future some of these type of enhancements might be considered as outlined below in Section 5.

The general areas of modification fall into the following general categories:

- a. The broader perspective of ATC-40 with respect to the overall rehabilitation process is referenced where appropriate to provide an expanded context and discussion for the prescriptive requirements of the prestandard.
- b. The information in ATC-40 on presumptive capacity for piles and drilled piers is incorporated.
- c. The expanded procedures of FEMA-306/307/308 for investigating the condition of concrete wall buildings are included.
- d. The detailed treatment of concrete and masonry wall components and behavior modes in FEMA-306/307/308 is referenced.
- e. The information in FEMA-306/307/308 regarding damaged components is recognized.
- f. The techniques of FEMA-306/307/308 for component repair are referenced.

Each of the proposed modifications is specified in detail in Appendix A of this report. They are numbered consecutively as ATC1 through ATC38 and each is assigned a short descriptive title. Where appropriate there is a *Global issue reference* to designate the specific issue in the *Global Topics Report-1* (ASCE, 1999b) that the modification would address in whole or part. The *Revision classification* (Technical, Commentary, Editorial, Application of Current Research, for example) also refers to the designations of the *Global Topics Report-1* (ASCE, 1999b). The *Section in First Draft* identifies the location of the proposed change in the first prestandard draft (ASCE, 1999a). The *Suggested change* specifies the actual modification and the *Discussion/justification* provides supplemental information where appropriate.

5. Recommendations for Future Enhancement

The modifications proposed in this report are relatively simple and can be implemented without major impact on the prestandard. In the longer term, however, the overall quality of the procedures for existing buildings could be significantly enhanced by a greater degree of consolidation of the material with that available in ATC-40 and FEMA-306/307/308. The following are some of the more comprehensive changes that might be made at a later time:

- a. ATC-40 could serve as a starting point for the development of an implementation manual for seismic rehabilitation in general and a user's guide for FEMA-273 in particular. This would allow the further development of the prestandard as a code without extraneous material that might be considered ambiguous. The manual would be for discussion, explanation, and illustration and might subsume the existing commentary. The manual would address the current need for greater understanding of performance based design and nonlinear static procedures. Development would require the expansion of the scope of ATC-40 to all materials, the incorporation of FEMA-273 required commentary, and the inclusion of examples other than those for reinforced concrete.
- b. The direct incorporation of the evaluation and repair procedures of FEMA-306/307/308 would expand the scope of FEMA-273 to a general purpose standard for seismic issues related to existing buildings. Municipalities and other jurisdictions could adopt a single standard governing pre-event rehabilitation and post event repair requirements. Although the basic procedures are generally applicable to all type of structures, this would require the expansion of the material data of FEMA-306/307/308 to include all possibilities. It would also require adoption of specific recommendations for the performance goals for post earthquake repairs and their relationship to the pre-event performance capability of the building.
- c. The detailed treatment of FEMA-306/307/308 for components and modes of behavior for concrete and masonry walls could be incorporated into the standard directly. This would require substantial re-writing of both the masonry and concrete chapters of FEMA-273. The benefit would be a much more comprehensive compendium of the actual characteristics of these types of buildings. Although the current document mentions the possibility of numerous modes of behavior, the information is not explicit or complete, and can lead to erroneous results if the wrong component type or behavior mode is selected.
- d. The basic approach of reducing component strength, stiffness, and displacement acceptability based on damage and prior ductility demand is proposed by FEMA-306/307/308. Currently, it is assumed that the damage is from a prior earthquake. This concept might be extended to apply during a single event, effectively specifying component degradation parameters for inelastic analyses. The advantage would be to eliminate the confusing concept of "secondary components" currently included in FEMA-273. This change would require a significant amount of development and the generation of modification factors for a more comprehensive list of components and materials.

6. References

- ASCE, 1999a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, ASCE/FEMA-273 Prestandard first draft, prepared by the American Society of Civil Engineers, funded by the Federal Emergency Management Agency, Washington, DC.
- ASCE, 1999b, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Global Topics Report-1, Identification of Global Issues*, ASCE/FEMA 273 Prestandard first draft, prepared by the American Society of Civil Engineers, funded by the Federal Emergency Management Agency, Washington, DC.
- ATC, 1996, *Seismic Evaluation and Retrofit of Concrete Buildings*, prepared by the Applied Technology Council (Report ATC-40) for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.
- ATC, 1997, *NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council (ATC-33 project) for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Reports FEMA 273 and 274, Washington, DC.
- ATC, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-306), Washington, DC.
- ATC, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Technical Resources*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-307), Washington, DC.
- ATC, 1998c, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-308), Washington, DC.
- BSSC, 1999, *Case Studies Project Report*, prepared by A.T. Merovich for the Building Seismic Safety Council, Washington, DC.

Appendix

Proposed Modifications to FEMA 273

ATC1. Augment commentary on performance levels and rehabilitation objectives.

Global issue reference: 2.5, 2.14

Revision classification: Commentary

Section in First Draft: C1.2.1

Suggested change: Add a second paragraph as follows:

“Additional discussion on this subject may be found in ATC-40 (ATC, 1996).”

Discussion/justification: ATC 40 provides more discussion on the setting of performance objectives. It also provides guidance on the appropriate roles of the architect, owner, engineer, and building official.

ATC2. Augment commentary on quantitative specifications of component behavior.

Global issue reference: 6.1, 6.3, 6.4, 7.1, 7.4, 7.5

Revision classification: Commentary

Section in First Draft: C1.2.4

Suggested change: Add a sentence at the end of the first paragraph as follows:

“Additional information on quantitative data on component behavior for concrete and masonry wall buildings may be found in FEMA 306 and 307 (ATC, 1998a, b).”

Discussion/justification: FEMA 306 and 307 provide extensive compatible information on specific behavior modes for components of concrete wall buildings and masonry wall buildings.

ATC3. Add the repair of damaged concrete and masonry wall buildings to scope.

Global issue reference: N/A

Revision classification: Application of Published Research

Section in First Draft: 1.3.1

Suggested change: Add a sentence at the end of the first paragraph (1.3.1.i) as follows:

“Concrete and masonry wall buildings previously damaged by earthquakes may be rehabilitated using the nonlinear analysis methods of this standard in conjunction with FEMA 306, 307, and 308 (ATC, 1998a, b, c).”

Discussion/justification: FEMA 306, 307, and 308 address the evaluation and repair of earthquake-damaged concrete and masonry wall buildings. These guidelines are compatible with FEMA 273.

ATC4. Augment commentary on activities and policies associated with seismic repair and rehabilitation.

Global issue reference: N/A

Revision classification: Commentary

Section in First Draft: C1.3.2

Suggested change: Add a sentence at the end of paragraph C1.3.2.ii as follows:

“Guidance on policy for the repair and upgrading of earthquake-damaged buildings may be found in FEMA 308 (ATC, 1998c).”

Add a sentence at the end of paragraph C1.3.2.iii as follows:

“ATC-40 (ATC, 1996) provides further guidance on the selection of Rehabilitation Objectives.”

Discussion/justification: FEMA 308 presents a framework for making decisions on whether to accept earthquake damage, repair it, or upgrade building performance. ATC-40 includes extensive general discussion of issues involved with rehabilitation.

ATC5. Augment commentary on relationship the other documents.

Global issue reference:

Revision classification: Commentary

Section in First Draft: C1.4

Suggested change:

Add the following to paragraph C1.4.i:

“FEMA 306: *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual* (ATC, 1998a) presents practical criteria and guidance for the evaluation of earthquake damage to buildings with primary lateral-force-resisting systems consisting of concrete and masonry walls and infilled frames. These procedures classify damage according to mode of behavior and severity. An analysis method similar to the nonlinear static procedure of FEMA 273 is used to evaluate the change in the anticipated performance of a building caused by the observed damage. The document contains extensive information on the properties and behavior modes of wall components that is supplemental to, and compatible with, that in FEMA 273. It also contains outline specifications for test and inspection procedures to document existing structural properties.

FEMA 307: *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Technical Resources* (ATC, 1998b) contains extensive data that forms the basis of the procedures of FEMA 306, particularly on the interpretation of previous tests of component behavior. An example application of the procedure is included.

FEMA 308: *The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings* (ATC, 1998c) provides a framework for implementing policy on the repair and upgrading of buildings for seismic performance. This framework relies on the basic evaluation procedures of FEMA 306. The framework could be readily applied to buildings other than concrete and masonry wall buildings. The document also contains outline specifications for typical repair techniques for concrete and masonry wall components.”

Change the description of ATC-40 in paragraph C1.4.i as follows:

“ATC-40: *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996) is technically similar to FEMA 273. Modeling and acceptability criteria are provided only for concrete buildings. The document, however, presents a broad perspective of the rehabilitation process that is applicable to any building type. The recommended analysis method is the nonlinear static procedure. The document covers in detail the capacity spectrum method of calculating displacement demand.”

Discussion/justification: This modification updates the list of related documents in FEMA 273.

ATC6. Augment commentary on characteristics of existing buildings.

Global issue reference:

Revision classification: Commentary

Section in First Draft: C1.5.1.2

Suggested change: Add the following sentence to the end of paragraph C1.5.1.2.i:

“FEMA 306 (ATC, 1998a) includes outline specifications for test and inspection procedures to document existing structural properties of earthquake-damaged concrete and masonry wall buildings (see Table 1-x). ATC-40 (ATC, 1996) discusses general procedures for investigating concrete buildings.”

Table 1-x: Summary of inspection and test procedures (from FEMA 306)

Structural Or Material Property	Material			Test ID	Test Type
	Reinf. Conc.	Reinf. Mas.	URM		
<u>Crack Location and Size</u>	√	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 1</u>	<u>Visual Observation</u>
<u>Spall Location and Size</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 1</u>	<u>Visual Observation</u>
	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 2</u>	<u>Sounding</u>
<u>Location of Interior Cracks or Delaminations</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 6</u>	<u>Impact Echo</u>
	<input type="checkbox"/>			<u>NDE 7</u>	<u>SASW</u>
	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 1</u>	<u>Selective Removal</u>
<u>Reinforcing Bar Buckling or Fracturing</u>	<input type="checkbox"/>	<input type="checkbox"/>		<u>NDE 1</u>	<u>Visual Observation</u>
	<input type="checkbox"/>	<input type="checkbox"/>		<u>IT 1</u>	<u>Selective Removal</u>
<u>Relative Age of Cracks</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 2</u>	<u>Petrography</u>
<u>Relative Compressive Strength</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 3</u>	<u>Rebound Hammer</u>
<u>Compressive Strength</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 3</u>	<u>Material Extraction and Testing</u>
<u>Reinforcing Bar Location and Size</u>	<input type="checkbox"/>	<input type="checkbox"/>		<u>NDE 4</u>	<u>Rebar Detector</u>
	<input type="checkbox"/>	<input type="checkbox"/>		<u>NDE 8</u>	<u>Radiography</u>

Structural Or Material Property	Material			Test ID	Test Type
	Reinf. Conc.	Reinf. Mas.	URM		
	<input type="checkbox"/>	<input type="checkbox"/>		<u>NDE 9</u>	<u>Penetrating Radar</u>
	<input type="checkbox"/>	<input type="checkbox"/>		<u>IT 1</u>	<u>Selective Removal</u>
<u>Strength of Reinforcing Bar</u>	<input type="checkbox"/>	<input type="checkbox"/>		<u>IT 3</u>	<u>Material Extraction and Testing</u>
<u>Wall Thickness</u>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 1</u>	<u>Visual Observation</u>
	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 6</u>	<u>Impact Echo</u>
	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 1</u>	<u>Selective Removal</u>
<u>Presence of Grout in Masonry Cells</u>		<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 2</u>	<u>Sounding</u>
		<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 6</u>	<u>Impact Echo</u>
		<input type="checkbox"/>	<input type="checkbox"/>	<u>NDE 7</u>	<u>SASW</u>
		<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 1</u>	<u>Selective Removal</u>
<u>Strength of Masonry Units</u>		<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 3</u>	<u>Material Extraction and Testing</u>
			<input type="checkbox"/>	<u>IT 4, 5</u>	<u>In Situ Testing</u>
<u>Mortar Strength</u>		<input type="checkbox"/>	<input type="checkbox"/>	<u>IT 1</u>	<u>Petrography</u>
			<input type="checkbox"/>	<u>IT 4, 5</u>	<u>In Situ Testing</u>

Discussion/justification: This modification provides additional sources of information on documenting existing building conditions.

ATC7. Augment commentary on social, economic, and political considerations.

Global issue reference: 2.10

Revision classification: Commentary

Section in First Draft: C1.6

Suggested change: Add the following sentence to the end of the first paragraph of Table C1-1:

“FEMA 308 (ATC, 1998c) includes discussion of non-engineering issues related to the repair of earthquake-damaged buildings.”

Discussion/justification: This modification provides an additional source of information on the subject.

ATC8. Augment commentary on rehabilitation triggers.

Global issue reference: N/A

Revision classification: Commentary

Section in First Draft: C1.6.2.1

Suggested change: Add the following sentence to the end of paragraph C1.6.2.1.i:

“Another trigger for rehabilitation in the past has been earthquake damage. FEMA 308 discusses experiences in recent earthquakes and presents a framework for post-earthquake triggers for repair and upgrading of damaged buildings.”

Discussion/justification: This modification provides additional source of information on the subject.

ATC9. Add references to Chapter 1.

Global issue reference:

Revision classification: Editorial

Section in First Draft: C1.7

Suggested change: Add the following references:

ATC, 1996, *Seismic Evaluation and Retrofit of Concrete Buildings*, prepared by the Applied Technology Council (Report ATC-40) for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.

ATC, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-306), Washington, DC.

ATC, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Technical Resources*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-307), Washington, DC.

ATC, 1998c, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-308), Washington, DC.

Discussion/justification: This addition simply provides references for the proposed modifications.

ATC10. Augment commentary on foundation strength and stiffness.

Global issue reference: 4.1, 4.2, 4.4

Revision classification: Commentary

Section in First Draft: C4.4

Suggested change: Add the following paragraph C4.4.iii:

“This chapter provides procedures to estimate foundation stiffness and strength. ATC-40 (ATC, 1996) incorporates technically similar procedures. ATC-40 also includes discussion and extensive commentary on typical issues encountered in the modeling of foundations for structural analyses.”

Discussion/justification: This addition provides an additional resource for the engineer.

ATC11. Add presumptive capacities for piers and piles.

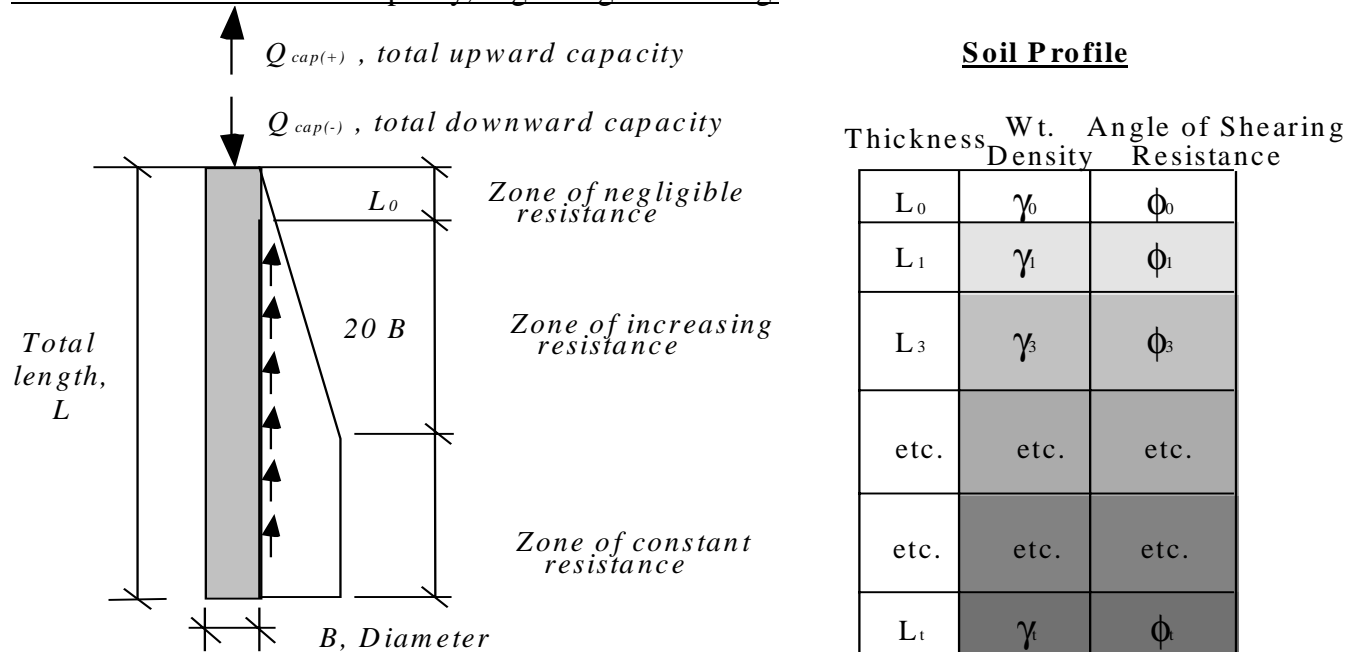
Global issue reference: 4.4

Revision classification: Technical

Section in First Draft: 4.4.1.1

Suggested change: Add the following paragraph 4.4.1.1.ii:

“In the absence of specific design data, the procedures illustrated in Figures 4-x and 4-y may be used to calculate presumptive capacities, for granular and cohesive materials, respectively, to calculate preliminary estimates of capacity of piers and piles. Ranges of typical values of parameters for use with these procedures are presented Tables 4-x, -y, -z and -zz. These all have been adapted from ATC-40 (ATC, 1996) and NAVFAC (1986). For friction resistance in granular materials the top of the pier or pile, for a length of 3 to 5 diameters, shall be neglected. The upward frictional capacity of a pile or pier in cohesive materials shall be assumed to be equal to the downward frictional capacity, neglecting end bearing.”



Downward Capacity

$$Q_{cap(-)} = P_t N_q A_t + \sum_{i=1}^{t-1} F_{di} P_i \tan \delta_i a_s L_i$$

Where P_t = Effective vert. stress at tip

$$P_t = \sum_{i=0}^{t-1} L_i \gamma_i \leq P @ L_0 + 20B$$

N_q = Bearing capacity factor (see Table 4-x)

A_t = Bearing area at tip

F_{di} = Effective horiz. stress factor for downward load (see Table 4-y)

P_i = Effective vert. stress at depth i

$$P_i = \sum_{j=0}^i L_j \gamma_j \leq P @ L_0 + 20B$$

δ = Friction angle between pile/pier and soil at depth i (see Table 4-z)

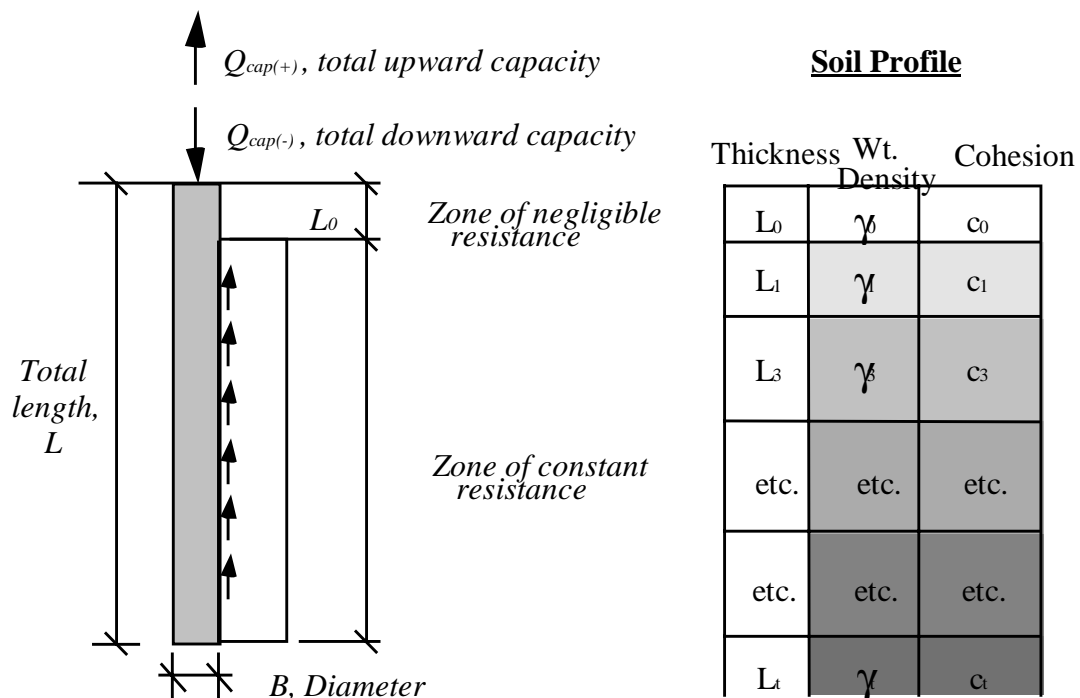
a_s = Surface area of pile/pier per unit length

Upward Capacity

$$Q_{cap(+)} = \sum_{i=1}^{t-1} F_{ui} P_i \tan \delta_i a_s L_i$$

Where F_{ui} = Effective horiz. stress factor for upward load (see Table 4-y)
 other parameters as for downward capacity

Figure 4-x. Pile or Pier Capacities for Granular Soils (adapted from NAVFAC, 1986)



Downward Capacity $Q_{cap(-)} = c_t N_c A_t + \sum_{i=1}^{t-1} c_{ai} a_s L_i$

Where c_t = Cohesion strength of soil (see Table 4-zz) at tip
 N_c = Bearing capacity factor 9.0 for depths greater than 4B
 A_t = Bearing area at tip
 c_{ai} = Cohesion strength of soil (see Table 4-zz) at depth i
 a_s = Surface area of pile/pier per unit length

Upward Capacity $Q_{cap(+)} = \sum_{i=1}^{t-1} c_{ai} a_s L_i$

Where parameters are as for downward capacity

Figure 4-y. Pile or Pier Capacities for Cohesive Soils (adapted from NAVFAC, 1986)

Table 4-x. Typical Pile and Pier Capacity Parameters: Bearing Capacity Factors, N_q
(adapted from NAVFAC, 1986)

<i>Placement</i>	<i>Angle of Shearing Resistance for Soil, ϕ (degrees)</i>												
	<u>26</u>	<u>28</u>	<u>30</u>	<u>31</u>	<u>32</u>	<u>33</u>	<u>34</u>	<u>35</u>	<u>36</u>	<u>37</u>	<u>38</u>	<u>39</u>	<u>40+</u>
Driven Pile	10	15	21	24	29	35	42	50	62	77	86	120	145
Drilled Pier	5	8	10	12	14	17	21	25	30	38	43	60	72

Table 4-y. Typical Pile and Pier Capacity Parameters: Effective Horizontal Stress Factors, F_{dj} and F_{uj} (adapted from NAVFAC, 1986)

<i>Pile or Pier Type</i>	<i>Downward</i> F_{dj}		<i>Upward</i> F_{uj}	
	<i>low</i>	<i>high</i>	<i>low</i>	<i>high</i>
Driven H-pile	0.5	1.0	0.3	0.5
Drive straight prismatic pile	1.0	1.5	0.6	1.0
Drive tapered pile	1.5	2.0	1.0	1.3
Driven jetted pile	0.4	0.9	0.3	0.6
Drilled pier	0.7		0.4	

Table 4-z. Typical Pile or Pier Capacity Parameters: Friction Angle, δ (degrees)
(adapted from NAVFAC, 1986)

<i>Pile or pier</i>	δ
<i>Material</i>	
Steel	20
Concrete	0.75 ϕ
Timber	0.75 ϕ

Table 4-zz. Typical Pile/Pier Capacity Parameters: Cohesion, c_t and Adhesion, c_a (psf) (adapted from NAVFAC, 1986)

Pile Material	Consistency of Soil (approx. STP blow count)	Cohesion, c_t		Adhesion, c_a	
		low	high	low	high
Timber and Concrete	Very soft (<2)	0	250	0	250
	Soft (2-4)	250	500	250	480
	Med. Stiff (4-8)	500	1000	480	750
	Stiff (8-15)	1000	2000	750	950
	Very Stiff (>15)	2000	4000	950	1300
Steel	Very soft (<2)	0	250	0	250
	Soft (2-4)	250	500	250	460
	Med. Stiff (4-8)	500	1000	460	700
	Stiff (8-15)	1000	2000	700	720
	Very Stiff (>15)	2000	4000	720	750

Discussion/justification: This addition relies on previously published procedures for calculating presumptive capacities for piles and piers.

ATC12. Augment commentary on foundation strength and stiffness.

Global issue reference: 4.1, 4.2, 4.3, 4.4

Revision classification: Commentary

Section in First Draft: C4.4

Suggested change: Add the following paragraph C4.4.iii:

“This chapter provides procedures to estimate foundation stiffness and strength. ATC-40 (ATC, 1996) incorporates technically similar procedures. ATC-40 also includes discussion and extensive commentary on typical issues encountered in the modeling of foundation for structural analyses.”

Discussion/justification: This addition provides an additional resource for the engineer.

ATC13. Add references to Chapter 4.

Global issue reference: 4.1, 4.2, 4.3, 4.4

Revision classification: Editorial

Section in First Draft: C4.9

Suggested change: Add the following references:

ATC, 1996, *Seismic Evaluation and Retrofit of Concrete Buildings*, prepared by the Applied Technology Council (Report ATC-40) for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.

NAVFAC, 1986, *Foundations and Earth Structures*, NAVFAC DM-7.02, Naval Facilities Engineering Command, Department of the United States Navy, Alexandria, Virginia.

Discussion/justification: This addition provides references for the proposed modifications.

ATC14. Add provision and commentary on concrete and masonry infilled steel frames.

Global issue reference: 5.1, 5.7

Revision classification: Technical and Commentary

Section in First Draft: 5.7

Suggested change: Add a paragraph 5.7.iii as follows:

“Potential failure any point in concrete and masonry infilled frames shall be considered to identify component types and critical modes of component behavior.”

Add a paragraph C5.7.iii as follows:

“The identification of components in concrete and masonry infilled frames depends on the relative strengths of the infill panels and surrounding frame. Openings in panels affect panel strength and component identification. This concept is covered in FEMA 306 (ATC, 1998a) and is summarized in Section C6.8.1.1.iii of this document. Typical infill components are summarized in Table 5-x from FEMA 306 (ATC, 1998a). Infill components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. FEMA 306 and 307 (ATC, 1998a, b) contain extensive information on the behavior modes of infill components. These are summarized in Table 5-y from FEMA 306.

Table 5-x: Component Types for Infilled Frames (from FEMA 306)

Component Type		Description/Examples	Materials/Details
INPS	Solid infill panel	Space within frame components completely filled	Concrete Reinforced Unreinforced Masonry (clay brick, hollow clay tile, concrete block) Reinforced Unreinforced
INPO	Infill panel with openings	Doors and windows Horizontal or vertical gaps Partial height infill Partial width infill	Same as solid
	Sub-components similar to:		
	INP1 Strong pier	RC1 RM1 URM1	Concrete Reinf. masonry URM
	INP2 Weak pier	RC2 RM2 URM2	Concrete Reinf. masonry URM
	INP3 Weak spandrel (lintel)	RC3 RM3 URM3	Concrete Reinf. masonry URM
INP4 Strong spandrel (lintel)	RC4 RM4 URM4	Concrete Reinf. masonry URM	
INF1	Frame column	Vertical, gravity load carrying	Concrete Steel
INF2	Frame beam	Horizontal, gravity load carrying	Concrete Steel
INF3	Frame joint	Connection between column and beam components Rigid moment resisting Partially rigid Simple shear	Monolithic concrete Precast concrete Bolted steel Riveted steel Welded steel

Note: References to components are from FEMA 306.

Table 5-y: Behavior modes for solid infilled panel components (from FEMA 306)

<u>Behavior Mode</u>	<u>Description/Likelihood of Occurrence</u>	<u>Ductility</u>	<u>Fig. within FEMA 306</u>	<u>Paragraph within Section 8.2.3a of FEMA 306</u>
<u>Bed-joint sliding</u>	<u>Occurs in brick masonry, particularly when length of panel is large relative to height aspect ratio is large.</u>	<u>High</u>	<u>8-2</u>	<u>i</u>
<u>Corner compression</u>	<u>Crushing generally occurs with stiff columns.</u>	<u>Moderate</u>	<u>8-1c</u>	<u>iii</u>
<u>Diagonal cracking</u>	<u>Likely to occur in some form.</u>	<u>Moderate</u>	<u>8-1,8-4</u>	<u>ii</u>
<u>General shear failure</u>	<u>Is the limiting case and will generally occur for large drifts.</u>	<u>Limited</u>	<u>8-1,8-3,8-4</u>	
<u>Out-of-plane failure</u>	<u>More likely to occur in upper stories of buildings. However, out-of-plane "walking" is likely to occur in the bottom stories due to concurrent in-plane loading.</u>	<u>Low</u>	<u>8-5</u>	<u>iv</u>

Discussion/justification: These modifications will expand the scope of the discussion of behavior modes and enhance the understanding of the user on infill behavior. This should be back-referenced in Chapters 6 and 7.

ATC15. Add references to Chapter 5.

Global issue reference:

Revision classification: Editorial

Section in First Draft: C5.12

Suggested change: Add the following references:

ATC, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-306), Washington, DC.

ATC, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Technical Resources*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-307), Washington, DC.

Discussion/justification: This addition provides references for the proposed modifications.

ATC16. Add commentary on the general assessment of existing conditions of concrete buildings.

Global issue reference:

Revision classification: Commentary

Section in First Draft: 6.3.3

Suggested change: Add a section of commentary C6.3.3 as follows:

“FEMA 306 (ATC, 1998a) provides guidance on the documentation and evaluation of earthquake damage to concrete bearing wall and infilled frame buildings. Procedures in FEMA 306 may be used in conjunction with FEMA 273 for the rehabilitation of such buildings.”

Discussion/justification: This commentary provides resources for including damaged concrete wall buildings within the scope of FEMA 273.

ATC17. Add commentary on the visual inspection of existing conditions of concrete buildings.

Global issue reference: 6.3

Revision classification: Commentary

Section in First Draft: C6.3.3.2.A.i

Suggested change: Add a sentence at the end of Section C6.3.3.2.A.i as follows:

“FEMA 306 (ATC, 1998a) and ATC-40 (ATC, 1996) also provide procedures and discussion on the visual inspection of concrete buildings.”

Discussion/justification: This commentary provides additional resources on the subject.

ATC18. Modify commentary on the documentation of damage to concrete buildings.

Global issue reference: 6.3

Revision classification: Commentary

Section in First Draft: C6.3.3.2.A.ii

Suggested change: Change Section C6.3.3.2.A.ii to read as follows:

“The damage should be quantified using supplemental methods cited in this chapter, the FEMA 274 Commentary, and FEMA 306 (ATC, 1998a).”

Discussion/justification: FEMA 306 provides extensive guidance on the quantification of damage to concrete wall buildings.

ATC19. Add commentary on testing to determine existing conditions for concrete buildings.

Global issue reference: 6.3

Revision classification: Commentary

Section in First Draft: C6.3.3.2.B.ii

Suggested change: Add a sentence at the end of Section C6.3.3.2.B.ii as follows:

“As noted in Chapter 1 (see Table 1-x), FEMA 306 (ATC, 1998a) also provides extensive guidance and outline specifications for tests and inspections of existing concrete buildings.”

Discussion/justification: This commentary provides additional resources on the subject.

ATC20. Add provision and commentary on concrete and masonry infilled concrete frames.

Global issue reference: 6.4

Revision classification: Technical and Commentary

Section in First Draft: **6.7.1**

Suggested change: Add a paragraph 6.7.1.iv as follows:

“Potential failure at any point in concrete and masonry infilled concrete frames shall be considered in the identification of component types and critical modes of component behavior.”

Add a paragraph C6.7.1.iv as follows:

“The identification of components in concrete and masonry infilled frames depends on the relative strengths of the infill panels and surrounding frame. Openings in panels affect panel strength and component identification. This concept is covered in *FEMA 306* (ATC, 1998a) and is summarized in Section C6.8.1.1.iii of this document. Typical infill components are summarized in Table 5-x from *FEMA 306* (ATC, 1998a). Infill components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. *FEMA 306 and 307* (ATC, 1998a, b) contain extensive information on the behavior modes of infill components. These are summarized in Table 5-y from *FEMA 306*.”

Discussion/justification: These modifications will expand the scope of the discussion of behavior modes and enhance the understanding of the user on infill behavior.

ATC21. Augment commentary on rehabilitation measures for concrete frames with masonry infills.

Global issue reference: 6.4

Revision classification: Commentary

Section in First Draft: 6.7.2.5

Suggested change: Add a paragraph C6.7.2.5.ii as follows:

“FEMA 308 (ATC, 1998c) contains extensive data on repair techniques that may be applicable to the rehabilitation of concrete frames with masonry infill (see Table 6-x). These techniques are summarized in repair guides containing the following information:

<u>Repair Name and ID</u>	<u>For reference and identification</u>
<u>Repair Category</u>	<u>Cosmetic Repair, Structural Repair, or Structural Enhancement</u>
<u>Materials</u>	<u>Applicability to reinforced concrete, reinforced masonry, or unreinforced masonry</u>
<u>Description</u>	<u>Basic overview of the objectives and scope of the repair procedure</u>
<u>Repair Materials</u>	<u>Typical products used for the repair</u>
<u>Equipment</u>	<u>A summary of the tools, instrumentation, or devices required</u>
<u>Execution</u>	<u>General sequence of operations</u>
<u>Quality Assurance</u>	<u>Measures required to achieve satisfactory installation</u>
<u>Limitations</u>	<u>Restrictions on the effectiveness of the repair</u>
<u>Standards and References</u>	<u>Applicable sources of further information”</u>

Table 6-x: Summary of repair procedures contained in FEMA 308 (ATC, 1998c)

<u>Repair Category</u>	<u>Material</u>			<u>Repair ID</u>	<u>Repair Type</u>
	<u>Reinf. Conc.</u>	<u>Reinf. Mas.</u>	<u>URM</u>		
<u>Cosmetic Repair</u>	<u>x</u>	<u>x</u>	<u>x</u>	<u>CR 1</u>	<u>Surface coating</u>
			<u>x</u>	<u>CR 2</u>	<u>Repointing</u>
	<u>x</u>	<u>x</u>		<u>CR 3</u>	<u>Crack injection with epoxy</u>
<u>Structural Repair</u>			<u>x</u>	<u>SR 1</u>	<u>Crack injection with cementitious grout</u>
		<u>x</u>		<u>SR 2</u>	<u>Crack injection with epoxy</u>
	<u>x</u>	<u>x</u>		<u>SR 3</u>	<u>Spall repair</u>
	<u>x</u>			<u>SR 4</u>	<u>Rebar replacement</u>
	<u>x</u>	<u>x</u>	<u>x</u>	<u>SR 5</u>	<u>Wall replacement</u>
<u>Structural Enhancement</u>	<u>x</u>	<u>x</u>	<u>x</u>	<u>SE 1</u>	<u>Concrete overlay</u>
	<u>x</u>	<u>x</u>	<u>x</u>	<u>SE 2</u>	<u>Composite Fibers</u>
	<u>x</u>			<u>SE 3</u>	<u>Crack Stitching</u>

Notes: Repairs for concrete walls can also be used for concrete frames in infill systems

Repairs for steel frames of infill systems are described in the component repair guides

Discussion/justification: The referenced guides provide greater detail on certain techniques than currently contained in FEMA 273.

ATC22. Augment commentary on rehabilitation measures for concrete frames with concrete infills.

Global issue reference: 6.4

Revision classification: Commentary

Section in First Draft: **C6.7.3.5**

Suggested change: Add a paragraph C6.7.3.5.ii as follows:

“FEMA 308 (ATC, 1998c) contains extensive data on repair techniques that may be applicable to the rehabilitation of concrete frames with concrete infill (see Section C6.7.2.5 and Table 6-x)”

Discussion/justification: The referenced guides provide greater detail on certain techniques than currently contained in FEMA 273.

ATC23. Augment commentary on reinforced concrete shear walls, wall segments, and coupling beams.

Global issue reference: 6-6

Revision classification: Commentary

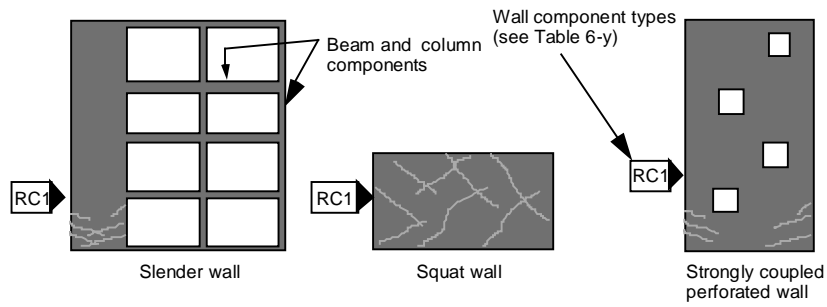
Section in First Draft: **C6.8.1.1 and C6.8.1.3**

Suggested change: Add a paragraph C6.8.1.1.iii as follows:

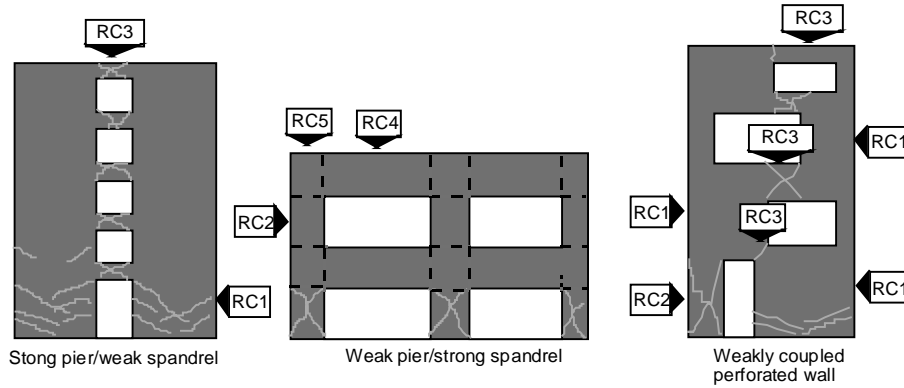
“The identification of components in concrete shear wall elements depends on the relative strengths of the wall segments. Vertical segments are often termed piers, while horizontal deep beam segments are called coupling beams or spandrels. This concept is covered in FEMA 306 (ATC, 1998a) and ATC-40 (ATC, 1996). Typical components for concrete walls are summarized in Table 6-y from FEMA 306. A plastic analysis of the entire wall element as it is displaced laterally to form a mechanism can help to identify component types as shown in Figure 6-x.”

Table 6-y: *Component types and descriptions for reinforced concrete walls (from FEMA 306).*

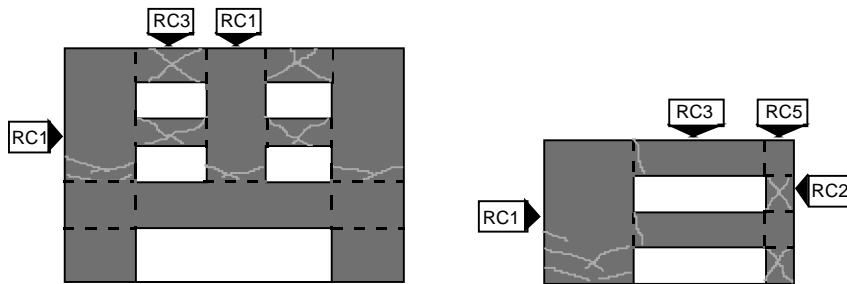
<u>Component Type</u> <u>(per FEMA 306)</u>		<u>Description</u>	<u>FEMA 273 Designation</u>
<u>RC1</u>	<u>Isolated Wall or Stronger Wall Pier</u>	<u>Stronger than beam or spandrel elements that may frame into it so that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge, shear failure, etc. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior.</u>	<u>Monolithic reinforced concrete wall or vertical wall segment</u>
<u>RC2</u>	<u>Weaker Wall Pier</u>	<u>Weaker than the spandrels to which it connects, characterized by flexural hinging top and bottom, or shear failure, etc.</u>	
<u>RC3</u>	<u>Weaker Spandrel or Coupling Beam</u>	<u>Weaker than the wall piers to which it connects, characterized by hinging at each end, shear failure, sliding shear failure, etc.</u>	<u>Horizontal wall segment or coupling beam</u>
<u>RC4</u>	<u>Stronger Spandrel</u>	<u>Should not suffer damage because it is stronger than attached piers. If this component is damaged, it should probably be re-classified as RC3.</u>	
<u>RC5</u>	<u>Pier-Spandrel Panel Zone</u>	<u>Typically not a critical area in RC walls.</u>	<u>Wall segment</u>



Cantilever Wall Mechanisms



Pier/Spandrel Mechanisms



Mixed Mechanisms

Figure 6-x: Identification of component types in wall elements (from FEMA 306(ATC, 1998a))

Discussion/justification: This commentary can help to dispel confusion regarding wall components, wall segments, coupling beams etc.

ATC24. Add provision and commentary for expanded range of concrete wall component behavior modes.

Global issue reference: 6-1, 6-6

Revision classification: Technical and Commentary

Section in First Draft: **6.8.2.1**

Suggested change: Modify the second sentence of paragraph 6.8.2.1.i as follows:

“Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered to identify critical modes of behavior for components.”

Add a paragraph to the beginning of Section C6.8.2.1 as follows:

“Concrete shear wall components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. FEMA 306 and 307 (ATC, 1998a, b) contain extensive information on the behavior modes of concrete wall components. These are summarized in Table 6-y from FEMA 306. The likelihood of occurrence of the behavior modes for different component types (see para. C6.8.1.1.iii) is summarized in Table 6-z.”

Table 6-z Likelihood of earthquake damage to reinforced concrete walls according to wall component and behavior modes (from FEMA 306).

Behavior Mode	Wall Component Type		
	Isolated Wall or Stronger Wall Pier (RC1)	Weaker Wall Pier (RC2)	Weaker Spandrel or Coupling Beam (RC3)
A. Ductile Flexural Response	common in well-designed walls <i>See Guide RC1A</i>	may occur <i>See Guide RC2A</i>	may occur, particularly if diagonally reinforced <i>Similar to Guide RC2A</i>
B. Flexure/Diagonal Tension	common <i>See Guide RC1B</i>	common <i>Similar to Guide RC1B</i>	common <i>See Guide RC3B</i>
C. Flexure/Diagonal Compression (Web Crushing)	common (frequently observed in laboratory tests) <i>See Guide RC1C</i>	may occur	may occur
D. Flexure/Sliding Shear	may occur, particularly for squat walls <i>See Guide RC1D</i>	may occur	common <i>See Guide RC3D</i>
E. Flexure/ Boundary-Zone Compression	common <i>See Guide RC1E</i>	may occur	unlikely
F. Flexure/Lap-Splice Slip	may occur	may occur	may occur
G. Flexure/Out-of-Plane Wall Buckling	may occur (observed in laboratory tests)	unlikely	unlikely
H. Preemptive Diagonal Tension	common <i>Similar to Guide RC2H</i>	common <i>See Guide RC2H</i>	common <i>Similar to Guide RC2H</i>
I. Preemptive Web Crushing	may occur in squat walls (observed in laboratory tests)	may occur	may occur
J. Preemptive Sliding Shear	may occur in very squat walls or at poor construction joints.	may occur in very squat walls or at poor construction joints.	unlikely
K. Preemptive Boundary Zone Compression	may occur in walls with unsymmetric sections and high axial loads	may occur in walls with unusually high axial load	unlikely
L. Preemptive Lap-Splice Slip	may occur	may occur	may occur
M. Global foundation rocking of wall	common	n/a	n/a
N. Foundation rocking of individual piers	may occur	may occur	n/a

Notes: • Shaded areas of table indicate behavior modes for which a specific Damage Classification and Repair Guide is provided in FEMA 306. The notation *Similar to Guide...* indicates that the

behavior mode can be assessed by using the guide for a different, but similar, component type or behavior mode.

- *common* indicates that the behavior mode has been evident in post-earthquake field observations and/or that experimental evidence supports a high likelihood of occurrence.
- *may occur* indicates that the behavior mode has a theoretical or experimental basis, but that it has not been frequently reported in post-earthquake field observations.
- *unlikely* indicates that the behavior mode has not been observed in either the field or the laboratory.

”
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Discussion/justification: FEMA 273 does not explicitly recognize modes of behavior other than flexure or shear. This modification will expand the scope of the concrete wall section and enhance the understanding of the user on wall behavior.

ATC25. Add provisions for damaged concrete shear walls.

Global issue reference:

Revision classification: Technical and Commentary

Section in First Draft: 6.8.2.1

Suggested change: Add a paragraph 6.8.2.1.zz as follows:

“Approved applicable alternatives or supplemental information to the provisions of this section may be used to evaluate concrete shear walls previously damaged by earthquakes.”

Add a paragraph C6.8.2.1.zz as follows:

“FEMA 306 and 307 (ATC, 1998a, b) contain extensive information on the properties and behavior of damaged concrete shear walls. Although developed for the purpose of evaluating earthquake damage, these documents can provide expanded information to address specific conditions encountered for individual buildings. The effects of damage on component behavior are modeled generically in Figure 6-y. Acceptability criteria for damaged components are illustrated in Figure 6-z. The factors used to modify component properties for observed damage range from 0.0 to 1.0 and are defined as follows:

$\lambda_K \equiv$ modification factor for idealized component force-deformation curve accounting for change in effective initial stiffness resulting from earthquake damage.

$\lambda_Q \equiv$ modification factor for idealized component force-deformation curve accounting for change in expected strength resulting from earthquake damage.

$\lambda_D \equiv$ modification factor applied to component deformation acceptability limits considering earthquake damage.

$RD \equiv$ absolute value of the residual deformation in a structural component, resulting from earthquake damage.

The values of the modification factors depend upon the behavior mode and the severity of damage to the individual component. They are tabulated in the Component Guides in FEMA 306. Component stiffness is most sensitive to damage. Reduction in strength implies a higher significance of damage. After relatively severe damage, the magnitude of acceptable displacements is reduced.

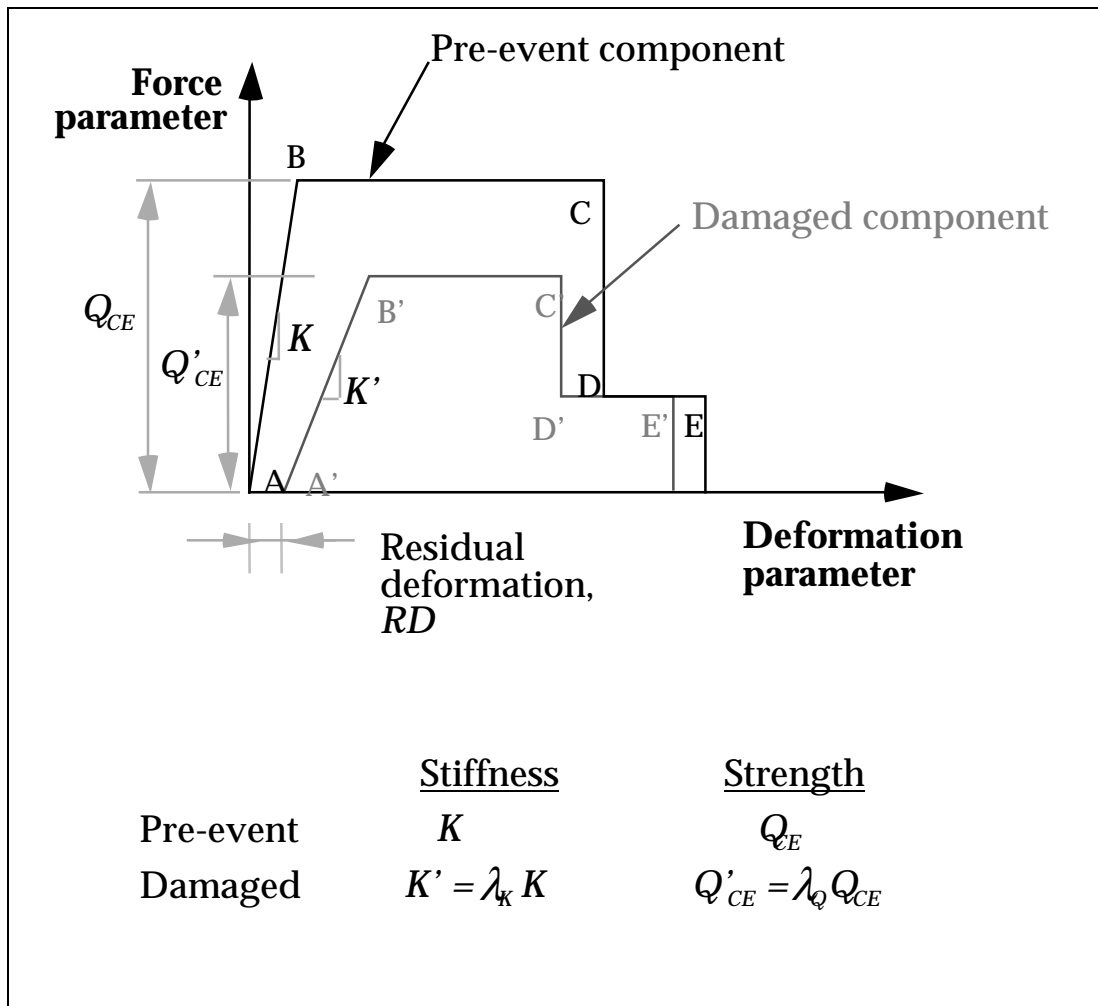


Figure 6-y: Component Modeling Criteria (from FEMA 306)

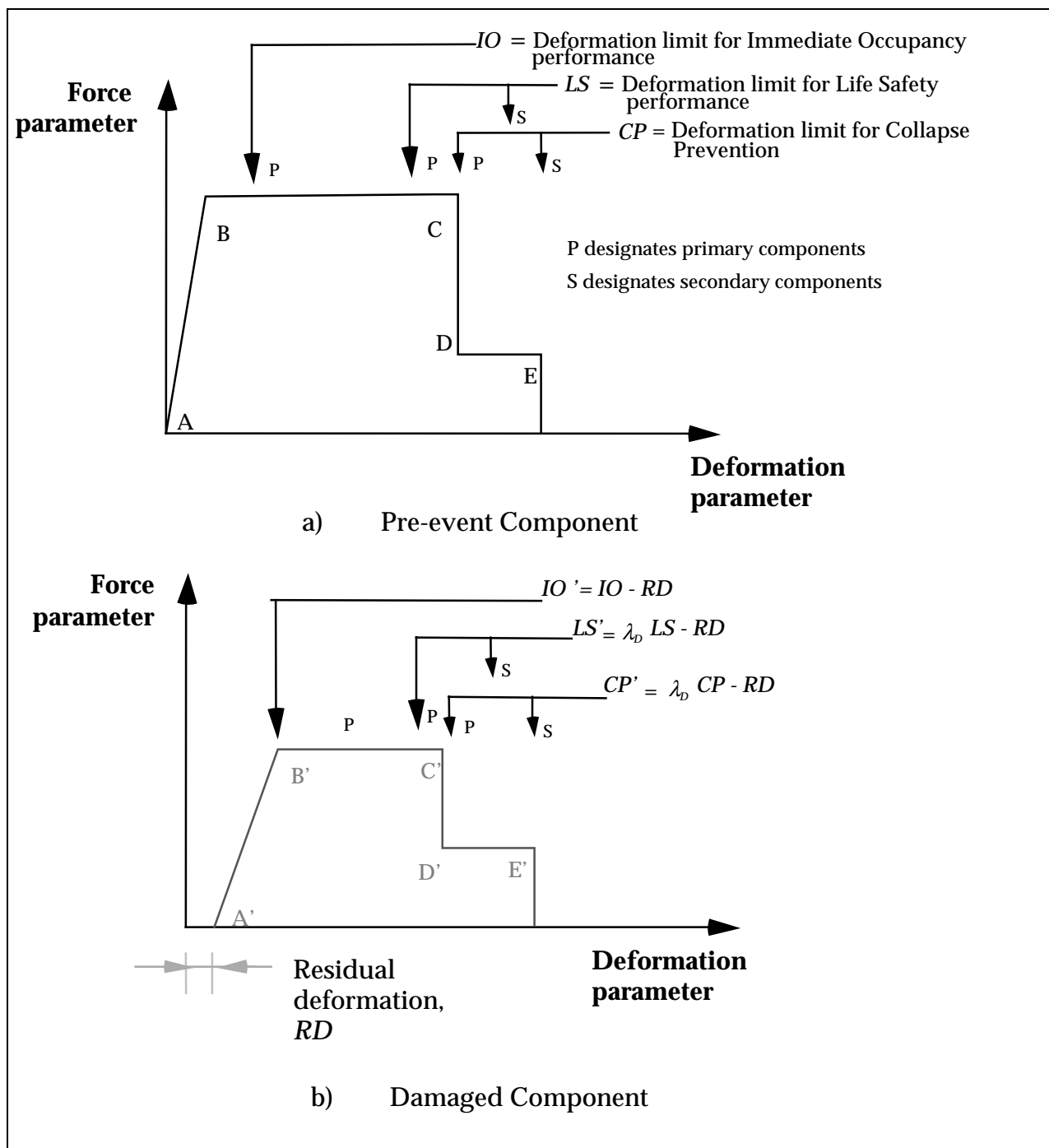


Figure 6-z: Component Acceptability Criteria (from FEMA 306)

Discussion/justification: Since they are not consensus standards, FEMA 306 and 307 cannot be explicitly cited as alternative provisions, and their use must be subject to approval. Nonetheless, the data in the documents provide an opportunity to expand the scope of FEMA 273 to include damaged concrete shear walls.

ATC26. Add allowance for alternative provisions for determination of stiffness of reinforced concrete wall components.

Global issue reference: 6-1, 6-6

Revision classification: Technical and Commentary

Section in First Draft: **6.8.2.2**

Suggested change: Add a third sentence to paragraph 6.8.2.2.i as follows:

“Other approved methods for determining component stiffness may be used if justified by specific conditions.”

Add a paragraph C6.8.2.2.i as follows:

“FEMA 306 and 307 (ATC, 1998a, b) provide expanded discussion of component behavior.”

Discussion/justification: FEMA 306 and 307 explain component behavior in more detail than FEMA 273.

ATC27. Add allowance for alternative provisions for determination of strength of reinforced concrete wall components.

Global issue reference:

Revision classification: Technical and Commentary

Section in First Draft: **6.8.2.3**

Suggested change: Add a third sentence to paragraph 6.8.2.3.i as follows:

“Other approved methods for determining component strength may be used if justified by specific conditions.”

Add a paragraph C6.8.2.3.i as follows:

“FEMA 306 and 307 (ATC, 1998a, b) provide expanded discussion of component behavior and alternative procedures for calculation of component strengths.”

Discussion/justification: FEMA 306 and 307 explain component behavior in more detail than FEMA 273.

ATC28. Augment commentary on rehabilitation measures for concrete walls.

Global issue reference:

Revision classification: Commentary

Section in First Draft: **6.8.2.5**

Suggested change: Add a paragraph C6.8.2.5.iii as follows:

“FEMA 308 (ATC, 1998c) contains extensive data on repair techniques that may be applicable to the rehabilitation of concrete walls. These are summarized in Section 6.7.2.5 and Table 6-x of this document.

Discussion/justification: The referenced guides provide greater detail on certain techniques than currently contained in FEMA 273.

ATC29. Add references to Chapter 6.

Global issue reference:

Revision classification: Editorial

Section in First Draft: C6.16

Suggested change: Add the following references:

ATC, 1996, *Seismic Evaluation and Retrofit of Concrete Buildings*, prepared by the Applied Technology Council (Report ATC-40) for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.

ATC, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-306), Washington, DC.

ATC, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Technical Resources*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-307), Washington, DC.

ATC, 1998c, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-308), Washington, DC.

Discussion/justification: This addition simply provides references for the proposed modifications.

ATC30. Augment commentary on history of masonry.

Global issue reference:

Revision classification: Commentary

Section in First Draft: C7.2

Suggested change: Add a sentence at the end of paragraph C7.2.i as follows:

“FEMA 306 (ATC, 1998a) contains further information on the history of reinforced and unreinforced masonry construction

Discussion/justification: FEMA 306 provides useful supplemental background.

ATC31. Add allowance for alternative provisions for determination of material properties and condition assessment of masonry.

Global issue reference: N/A

Revision classification: Technical and Commentary

Section in First Draft: 7.3

Suggested change: Add a second sentence to paragraph 7.3.1.i as follows:

“Other approved methods for determining existing component properties may be used if justified by specific conditions.”

Add a sentence to paragraph C7.3.i as follows:

“FEMA 306 and 307 (ATC, 1998a, b) provide expanded discussion of component behavior and alternative procedures for calculation of component properties.”

Discussion/justification: FEMA 306 and 307 explain component behavior in more detail than FEMA 273. They also provide extensive information on alternative methods of determining properties based on actual conditions.

ATC32. Modify definition of assigned of masonry condition designations.

Global issue reference: 7-5

Revision classification: Technical and Commentary

Section in First Draft: 7.3.1 and 7.8

Suggested change:

Add the following to Section 7.3.2.1:

“For purposes of obtaining component material properties, the condition of the masonry shall be classified as good, fair, or poor in accordance with the definitions in Section 7.8.”

Add the following commentary as Section C7.3.2.1:

“The classification of the condition of masonry components requires consideration of the type of component, the anticipated mode of inelastic behavior, and the nature and extent of damage or deterioration. This is discussed and explained for earthquake-damaged components in *FEMA 306*, *307* and *308*. These documents also contain extensive information of the effects of damage on the strength, stiffness and displacement acceptability of masonry components. Included are damage classification guides with visual representations of typical earthquake-related damage to masonry components. These may be very useful in classifying the condition of masonry in accordance with Section 7.3.2.1. The severity of damage to components in *FEMA 306*, *307* and *308* is categorized as Insignificant, Slight, Moderate, Heavy, and Extreme. Masonry in good condition would be comparable to a *FEMA 306* severity of damage of Insignificant or Slight. Fair condition is comparable to Moderate. Poor condition is comparable to Heavy or Extreme.”

Discussion/justification: FEMA 306 and 307 provide procedures to quantify the effects of observed damage and can help to eliminate qualitative and arbitrary assignment of masonry condition.

ATC33. Augment commentary on rehabilitated masonry walls.

Global issue reference:

Revision classification: Commentary

Section in First Draft: 7.4.1.3

Suggested change: Modify paragraph C7.4.ii to read as follows:

“Possible rehabilitation methods are described in Sections C7.4.1.3.A-C7.4.1.3.J. Additional information may be found in FEMA 308 (ATC, 1998c) as summarized in Section C6.7.2.5.ii of this document.”

Discussion/justification: FEMA 308 provides greater detail on certain techniques than currently contained in FEMA 273.

ATC34. Add commentary on URM component types.

Global issue reference: 7-1, 7-4

Revision classification: Commentary

Section in First Draft: 7.4.2

Suggested change: Add a paragraph C7.4.2.i as follows:

“The identification of components in URM wall elements depends on the relative strengths of the wall segments. Vertical segments are often termed piers, while horizontal deep beam segments are called coupling beams or spandrels. This concept is covered in FEMA 306 (ATC, 1998a) and is summarized in Section C6.8.1.1.iii of this document.”

Discussion/justification: This commentary clarifies the need to determine component types based on relative strengths.

ATC35. Add provision and commentary for expanded range of URM component behavior modes.

Global issue reference: 7-1, 7-4

Revision classification: Technical and Commentary

Section in First Draft: 7.4.2

Suggested change: Add a paragraph 7.4.2.iii as follows:

“Potential failure at any point in the URM wall shall be considered in the identification of critical modes of behavior for URM components.”

Add a paragraph C7.4.2.iii as follows:

“URM wall components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. FEMA 306 and 307 (ATC, 1998a, b) contain extensive information on the behavior modes of URM wall components. These are summarized in Table 7-y from FEMA 306.

Table 7-y: Behavior Modes for URM Walls (from *FEMA 306*)

Ductility Category	Behavior Mode	Likelihood of Occurrence and Damage Guide Reference		
		Solid Wall (URM1)	Weak Piers (URM2)	Weak Spandrels (URM3)
Higher Ductility	Foundation Rocking	Common in field; no experiments; see text	NA	NA
	A. Wall-Pier Rocking	Possible; similar to URM2A Guide	Common in field; has experiments; see URM2A Guide	NA
	B. Bed Joint Sliding	Common in field; has experiments; similar to URM2B	Common in field; has experiments; see URM2B Guide	Unlikely; no guide
	C. Bed Joint Sliding at Wall Base	Possible; similar to URM2B Guide	NA	NA
	D. Spandrel Joint Sliding	NA	NA	Common in field; no experiments; see URM3D Guide
Moderate Ductility	E. Rocking/Toe Crushing	Seen in experiments; similar to URM2A Guide	Possible; similar to URM2A Guide	NA
	F. Flexural Cracking/Toe Crushing/Bed Joint Sliding	Seen in experiments; see URM1F Guide	Possible; similar to URM1F Guide	Unlikely; no guide
	G. Flexural Cracking/Diagonal Tension	Possible	Seen in experiments; similar to URM2K Guide	Unlikely
	H. Flexural Cracking/Toe Crushing	Seen in experiments; see URM1H Guide	Possible; similar to URM1H guide	Possible; no guide
	I. Spandrel Unit Cracking	NA	NA	Common in field; see URM3I guide
Little or No Ductility	J. Corner Damage	Common in field; no experiments; no specific guide; see text	NA	Common in outer pier of upper stories; no specific guide; see text
	K. Preemptive Diagonal Tension	Possible; similar to URM2K guide	May be common in field; seen in experiments; see URM2K Guide	May be common in field; no experiments; similar to URM2K Guide
	L. Preemptive Toe Crushing	Theoretical; similar to URM1H Guide	Theoretical; similar to URM1H Guide	Unlikely; no guide
	M. Out-of-Plane Flexural Response	Common in field; see URM1M Guide	Possible; similar to URM1M Guide	Unlikely; no guide

Note • Shaded areas of table indicate behavior modes for which a specific Damage Classification and Repair Guide is provided in FEMA 306. The notation *Similar to Guide...* indicates that the behavior mode can be assessed by using

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- the guide for a different, but similar, component type or behavior mode.
 - common indicates that the behavior mode has been evident in post-earthquake field observations and/or that experimental evidence supports a high likelihood of occurrence.
 - may occur indicates that the behavior mode has a theoretical or experimental basis, but that it has not been frequently reported in post-earthquake field observations.
 - unlikely indicates that the behavior mode has not been observed in either the field or the laboratory.

Discussion/justification: This modification will expand the scope of the discussion of behavior modes and enhance the understanding of the user on URM wall behavior.

ATC36. Add provision and commentary on reinforced masonry wall component types and behavior modes.

Global issue reference: 7-1, 7-4

Revision classification: Technical and Commentary

Section in First Draft: 7.4.3

Suggested change: Add a paragraph 7.4.3.i as follows:

“Potential failure at any point in the reinforced masonry wall shall be considered in the identification of component types and critical modes of component behavior.”

Add a paragraph C7.4.3.i as follows:

“The identification of components in reinforced masonry wall elements depends on the relative strengths of the wall segments. Vertical segments are often termed piers, while horizontal deep beam segments are called coupling beams or spandrels. This concept is covered in FEMA 306 (ATC, 1998a) and is summarized in Section C6.8.1.1.iii of this document. Reinforced masonry wall components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. FEMA 306 and 307 (ATC, 1998a, b) contain extensive information on the behavior modes of reinforced masonry wall components. These are summarized in Table 7-z from FEMA 306.

Table 7-z: Likelihood of earthquake damage to RM components according to component and behavior mode (from *FEMA 306*).

<u>Ductility</u>		<u>Behavior Mode</u>	<u>Wall Component Type</u>			
			<u>RM1</u>	<u>RM2</u>	<u>RM3</u>	<u>RM4</u>
			<u>Stronger Pier</u>	<u>Weaker Pier</u>	<u>Weaker Spandrel</u>	<u>Stronger Spandrel</u>
High ductility	A	Flexure	Common <i>See Guide RM1A</i>	Unlikely	Common <i>See Guide RM3A</i>	N/A
		Foundation rocking	May occur <i>See FEMA 273 or ATC-40</i>	May occur <i>See FEMA 273 or ATC-40</i>	N/A	N/A
Moderate ductility	B	Flexure / diagonal shear	Common <i>See Guide RM1B</i>	Common <i>See Guide RM2B</i>	May occur <i>Similar to Guide RM3A</i>	N/A
	C	Flexure / sliding shear	May occur <i>See Guide RM1C</i>	May occur <i>Similar to Guide RM1C</i>	Unlikely	N/A
	D	Flexure / out-of-plane instability	May occur following large displacement cycles <i>See Guide RM1D</i>	Unlikely	Unlikely	N/A
	E	Flexure / lap splice slip	May occur <i>See Guide RM1E</i>	Unlikely	May occur	N/A
	F	Pier rocking	May occur <i>Similar to Guide RM1E</i>	May occur <i>Similar to Guide RM1E</i>	N/A	N/A
Little or no ductility	G	Preemptive diagonal shear	Common <i>Similar to Guide RM2G</i>	Common <i>See Guide RM2G</i>	Common <i>See Guide RM3G</i>	N/A
	H	Preemptive sliding shear	May occur in poorly detailed wall <i>Similar to Guide RM1C</i>	May occur in poorly detailed wall <i>Similar to Guide RM1C</i>	N/A	N/A

Notes: Shaded areas of the table with notation “See Guide ...” indicate behavior modes for which a specific Component Guide is provided in **FEMA 306**. The notation “Similar to Guide ...” indicates that the behavior mode can be assessed by using the guide for a different, but similar component type or behavior mode.

Common indicates that the behavior mode has been evident in post-earthquake field observations and/or that experimental evidence supports a high likelihood of occurrence.

May occur indicates that a behavior mode has a theoretical or experimental basis, but that it has not been frequently reported in post-earthquake field observations.

Unlikely indicates that the behavior mode has not been observed in either the field or the laboratory.

N/A indicates that the failure mode cannot occur for that component.

Discussion/justification: This modification will expand the scope of the discussion of behavior modes and enhance the understanding of the user on reinforced masonry wall behavior.

ATC37. Add provision and commentary on masonry infill.

Global issue reference: 7-1, 7-4

Revision classification: Technical and Commentary

Section in First Draft: 7.5

Suggested change: Add a paragraph 7.5.iii as follows:

“Potential failure at any point in masonry infilled frames shall be considered in the identification of component types and critical modes of component behavior.”

Add a paragraph C7.5.iii as follows:

“The identification of components in masonry infilled frames depends on the relative strengths of the infill panels and surrounding frame. Openings in panels affect panel strength and component identification. This concept is covered in *FEMA 306* (ATC, 1998a) and is summarized in Section C6.8.1.1.iii of this document. Typical infill components are summarized in Table 5-x from *FEMA 306* (ATC, 1998a). Infill components can exhibit a number of inelastic behavior modes. The strength and ductility of the component is dependent on its behavior mode. *FEMA 306 and 307* (ATC, 1998a, b) contain extensive information on the behavior modes of infill components. These are summarized in Table 5-y from *FEMA 306*.

Discussion/justification: These modifications will expand the scope of the discussion of behavior modes and enhance the understanding of the user on infill behavior.

ATC38. Add references to Chapter 7.

Global issue reference:

Revision classification: Editorial

Section in First Draft: C7.10

Suggested change: Add the following references:

ATC, 1998a, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-306), Washington, DC.

ATC, 1998b, *Evaluation of Earthquake-Damaged Concrete and Masonry Wall Buildings - Technical Resources*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-307), Washington, DC.

ATC, 1998c, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*, prepared by the Applied Technology Council (ATC-43 project), published by the Federal Emergency Management Agency (Report FEMA-308), Washington, DC.

Discussion/justification: This addition provides references for the proposed modifications.

**K. Special Study 8—
Incorporating Results of the SAC Joint Venture Steel
Moment Frame Project**

ASCE/FEMA 273 Prestandard Project

Special Study Report **Incorporating Results of the SAC Joint Venture Steel Moment Frame Project**

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Executive Summary

This report covers the following four tasks related to a general review of the Prestandard draft with respect to the FEMA/SAC Joint Venture publications on moment-resisting steel frames. A summary of these tasks is included below. The review of Chapter 5 of the Prestandard has revealed some areas, beyond the scope of this Special Study project, for which revisions and/or editing are recommended. These are briefly summarized as item #5 below.

1. *Review Chapter 5 of the Prestandard for general agreement with approaches developed for acceptance criteria by the FEMA/SAC Steel Project. Develop recommended changes to the Prestandard.*

Chapter 5 (and portions of Chapters 2 and 3, where applicable) of the Prestandard draft was reviewed for its general agreement with the approaches developed by the SAC project. Significant differences between the two publications are noted and recommendations for revisions are made. Appendix A includes proposed revisions to the text and commentary, which will bring the Prestandard into general agreement with the FEMA/SAC publications as well as provide some commentary on the differences. The commentary is also used for directing the design engineer to the FEMA/SAC publications, which provide a wealth of background information on moment frame connections.

Action Items:

- Revise Chapter 5 of the Prestandard based on the underline/strike-through revisions contained in Appendix A of this report.
- Add Commentary for Section 1.3 to introduce the FEMA/SAC documents.

2. *Review particular values for acceptance criteria for moment frames for agreement with those contained in the FEMA/SAC recommendations. Develop recommended changes to the Prestandard.*

The acceptance criteria for moment frames were reviewed based on the findings of the SAC project. Appendix A contains revised values for specific acceptance criteria. A methodology was developed for adapting the reliability-based acceptance criteria contained in the FEMA/SAC documents into the framework of the Prestandard. This methodology accounts for the demand factors, resistance factors, and confidence indices assigned to the various connection configurations.

Action Items:

- Add connection types and associated modeling parameters and acceptance criteria to Tables 5-4 and 5-5 of the Prestandard based on the modified tables contained in Appendix B of this report.
- Include additional revisions to beam and column acceptance criteria in Tables 5-4 and 5-5 as discussed in this report.
- For partially restrained connections, reinstate the categorization of limit states and acceptance criteria from FEMA 273, which better correlate with the findings of the SAC project. (the PR section, contained in Appendix A was also edited for clarity and internal consistency.)

3. *Review SAC testing and investigations for input to acceptance criteria for other steel systems, connections or joints (e.g., gravity connections, welds, bolted connections). Develop recommended changes to the prestandard.*

The SAC testing and investigations described in the various “State of Art Reports” were reviewed for their applicability to other steel systems, connections, or joints. Since the SAC testing tended to be system-based rather than component-based, there is little applicability to steel systems other than moment connections. Simple beam-to-column gravity connections were tested as an assembly, and therefore, acceptance criteria for these elements have been proposed for the Prestandard along with the recommendations in item #2.

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4. Review the FEMA/SAC reliability framework to assess its future application to the Prestandard. Make recommendations regarding conversion of FEMA 273 to this or another reliability framework and outline development work that may be required to achieve this. Develop appropriate commentary for the Prestandard to address issues of reliability.

The applicability of the FEMA/SAC reliability framework for future inclusion into other sections of the Prestandard was considered. With respect to the FEMA/SAC publications on moment frames, the reliability framework is appropriate given the amount of testing and investigation that has been performed to substantiate the results. The methodology used for developing the acceptance criteria of the Prestandard from the reliability-based SAC framework could be used for other systems and materials provided that the background testing data exists. A systematic and comprehensive program like the SAC project has not been performed for other systems or materials, and therefore the data needed to adequately develop a reliability framework for these is not readily available. There is likely to exist a substantial base of data for some systems (reinforced concrete moment frames for example) that, if compiled, could form the basis of a reliability framework.

The primary recommendation for this task is to first perform a comprehensive search into the availability of research data for other systems and materials. This would need to be followed by systematic, reliability-based analytical research (similar to that reported in FEMA 355F) to determine appropriate factors for the bias, uncertainty, etc.

Adding commentary language to the Prestandard that addresses issues of reliability was considered. However, since no explicit mention on the subject of reliability is currently contained in the Prestandard, it was judged to be inappropriate and potentially misleading to include such language since it pertains to only one specific material type and system.

5. The following is a list of areas requiring future revision and/or editing but are beyond the scope of this Special Study.

Action Items:

- Include information for steel shapes not currently covered in Prestandard Table 5-2, “Default Lower-Bound Material Strengths,” as described in this report.
- References to the AISC *Seismic Provisions for Structural Steel Buildings* should be changed to the 1997 edition. Currently, the chapter refers to the 1992 edition of the *Provisions* by way of reference to the 1997 AISC *Manual of Steel Construction*. In addition, the Prestandard should refer only to specifications produced by AISC and not to the *Manual of Steel Construction* (either volume) as it is not a consensus document.
- Several equations are incorrectly referenced in Chapter 5 (e.g. the reference to Equation (3-21) in section 5.5.2.4.2, No. 1 should be to Equation (3-22). A thorough review of all references is recommended.
- In Section 5.5.2.4.2, No. 4, the operator in Equation (5-14) should be changed to “greater than or equal to” from “less than”.
- In Section 5.5.2.4.3, Item No. 3 and No. 4 should be reversed to maintain consistency in the ordering of these subsections.
- Convert all nonlinear acceptance criteria in Chapter 5 from ductility factors to plastic rotations or deformations. The results of a simple conversion (without changes of substance) were transmitted to the Project Team under separate cover.

INTRODUCTION

This report contains proposed modifications to the Second Draft of the FEMA 356 *Prestandard for the Seismic Rehabilitation of Buildings* (referred to herein as FEMA 356) based on publications produced in the course of the SAC Joint Venture's steel moment-resisting frame project. The proposed revisions are based on 100% Draft versions of the following publications:

- FEMA 350 – Recommended Seismic Design Criteria for Moment-Resisting Steel Frame Structures
- FEMA 351 – Recommended Seismic Evaluation and Upgrade for Existing Welded Steel Moment-Resisting Frame Structures
- FEMA 355c – State of Art Report on Systems Performance
- FEMA 355d – State of Art Report on Connection Performance
- FEMA 355f – State of Art Report on Performance Prediction and Evaluation

These reports, which will be referenced in C5.14, Reference Commentary, are herein referred to collectively as the SAC documents. The SAC-developed FEMA publications not listed here were not consulted for these revisions.

SUMMARY OF FINDINGS

Chapter 1: Rehabilitation Requirements

Section C1.3 Design Basis

Add commentary describing FEMA 351 and its applicability to the evaluation and rehabilitation of steel moment frames. Also include a very brief introduction to the state of art reports to the extent that they are applicable to Chapter 5 of FEMA 356. Add the following section after item 9 in the commentary. (FEMA 351 should precede ATC 40 as steel precedes concrete in the standard). The five SAC references that are added to Section C5.14 should be added to Section C1.9 as well.

“10. FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Moment-Resisting Steel Frame Structures*, (SAC, 2000b), and companion documents are the product of a major project undertaken by the SAC Joint Venture to address the issue of the seismic performance of this structural system. The publications are based on the findings of a multi-year program of investigation and research, and FEMA 351 provides a complete and very detailed methodology for the evaluation and rehabilitation of steel moment frames. The methodology addresses two of the performance levels shown in Table C1-2 - immediate occupancy and collapse prevention - and by the application of explicit reliability methods produces a confidence level associated with achieving the specified performance objective. Acceptance criteria are generally related to building drift levels and were established by calibrating the results of extensive testing of various connection configurations with nonlinear analyses. The SAC publications include “State of Art Reports” presenting the results of research into analytical modeling (FEMA 355c, SAC, 2000c), connection performance (FEMA 355d, SAC, 2000d), and performance prediction and reliability (FEMA 355f, SAC 2000e). These publications provide detailed information that could be useful to the design engineer engaged in systematic evaluation and rehabilitation of these structures.

The SAC methodology differs in many ways from the approach outlined in this standard. However, the overall results are intended to be compatible because the acceptance criteria for steel moment frames were developed by adapting the SAC results to the framework of this standard.

11. FEMA 350, *Recommended Seismic Design Criteria for New Moment-Resisting Frame Structures* (SAC, 2000a) is a SAC Joint Venture publication that provides recommendations for the design new moment-resisting steel frame components.”

Chapter 2: General Requirements

Section 2.4.1.2 Limitations on Use of the Linear Static Procedure

There is a slight difference in applicability requirements for the LSP between the two documents. FEMA 356 limits the use of the LSP to buildings of 100 feet or less. FEMA 351, Section 3.4.2 and Table 3-3 limits the use of LSP to buildings with periods equal to or less than $3.5T_s$. In practice, these limits will be nearly the same for most building configurations and seismic regions. For 100-foot steel moment frame, the FEMA 356 Method 2 period is 1.4 sec, therefore when T_s is greater than $1.4 / 3.5 = 0.4$ sec, the height limitation will govern. Note the procedure selection is based on $3.5T_s$ in the 2000 Edition of the NEHRP Recommended Provisions. This method would allow the use of linear procedures under a wider range of site conditions. However, the current height limit is conservative. No change to text.

Chapter 3: Analysis Procedures for Systematic Rehabilitation

Section 3.3.1.2.2 Method 2 (Period Determination)

According to FEMA 351, Section 3.4.3.2, C_t is 0.028 for steel moment-resisting frames as opposed to 0.035 in FEMA 356. To produce lower period estimates, the FEMA 351 value is based on the mean minus standard deviation reported in the literature. Since FEMA 356 is based on mean levels, $C_t = 0.035$ is appropriate. No change to text. Provide brief commentary discussion in Section C5.5.1.

Section 3.3.1.3.1 Pseudo Lateral Load (LSP)

Computation of the C_3 factor is slightly different in the two documents. FEMA 351, Section 3.4.3.3.1 and Table 3-4 provides tabulated values that are dependent on performance level and connection type. We consider this table too specific to be placed in the FEMA 356 general analysis procedure. No change to text. Provide brief commentary discussion in Section C5.5.1.

FEMA 351, Section 3.4.3.3.1 contains a C_4 factor that is not included in FEMA 356, and the C_m factor in FEMA 356 is not contained in the FEMA 351 procedure. The C_4 factor relates to frame overstrength and depends on frame detailing (special, intermediate, ordinary) and performance level. It is considered too specific to be placed in the FEMA 356 general analysis procedure, especially since FEMA 356 does not contain these frame detailing definitions. Though the C_m factor is not in the FEMA 351 procedure, its effects are roughly similar to those produced by the C_4 factor for CP performance (and LS by interpolation), and there is no compelling reason to eliminate it from FEMA 356 for steel moment frames. No change to text. Provide brief commentary brief discussion in Section C5.5.1.

Section 3.3.3.3.2 Target Displacement (NSP)

Computation of the C_2 factor is slightly different in the two documents. According to FEMA 356, Table 3-3, the C_2 factor can be greater than 1.0 for steel frames with non-ductile or non-rigid connections. While FEMA 351, Section 3.4.5.3.1 references FEMA 273 without modification, FEMA 355f, Section 4.2.6.2.C states that $C_2= 1.0$ for all steel moment frames. Since FEMA 351 does not include the FEMA 355f approach to C_2 , we find no compelling reason to revise FEMA 356. No change to text. Provide brief commentary discussion in Section C5.5.1.

Chapter 5: Steel (Systematic Rehabilitation)

Section 5.3.2.5 Default Properties

We suggest revision of the tabulated default values to reflect SAC research. For consistency with Section 5.3.2.3 and the rest of the document, the expected and lower-bound values should be based on the mean and mean minus one standard deviation values, respectively. Revisions to the table are attached.

This table still has problems that are beyond the scope of this special study to resolve. Material properties are not recommended for pipes, TS, HSS, and other sections. One solution would be to take the lower-bound values as the nominal values. (Table 5-3 already has entries for “not listed” items.) The tabulated material properties are for samples taken from the flanges of wide flange shapes (by tensile group). Either these properties (by tensile group) should also be applied to other rolled shapes (as is implied by omission of a note to the contrary) or the line item for “Other rolled shapes” should be provided. The current item in Table 5-3 for “ASTM A36, Rolled Shapes” is incorrect as it would be applied to values from Table 5-2 that are already well above the nominal yield.

Section 5.5 Steel Moment Frames

Section 5.5.1 General

Add discussion that, in general, steel moment frame behavior is highly dependent on connection configuration and detailing. Provide Table 5-X, which includes description of the various connection types for which modeling procedures and acceptance criteria are provided. The table includes most of the connections contained in FEMA 351 and FEMA 355d as well as a brief description to allow the user to determine which connection criteria to use. Connections are also defined as FR or PR.

Section C5.5.1 General

Add commentary on the FEMA 351 approach to evaluation of moment frames. In general, note that there are differences in the analysis procedures (LSP and NSP as described above) between the FEMA 351 and the standard but do not discuss the differences. State that the procedures in Chapter 3 of the standard should be used without modification. Also state that procedure for assessing column strength is different in FEMA 351.

Add commentary describing the purpose of the connection definitions and directing the user to FEMA 351 for more detailed discussion of various connection types.

Delete reference to “nominally unrestrained” connections and revise definition of PR connections to include simple shear or pinned connections. FEMA 351 defines these connections as PR and since we are providing acceptance criteria for these connections, we do not want to add an entire category pertaining to simple connections. Commentary on simple connections is added to Section 5.5.3.

Section 5.5.2 Fully Restrained Moment Frames

Section 5.5.2.1 General

Indicate that connections are fully restrained where indicated in Table 5-X. Revise definition to state that connection types not included in the table shall be considered fully restrained if the joint is stronger than the weaker member being joined and deformations do not contribute more than 10% to total lateral deflection of the frame. This definition is consistent with FEMA 355f, Section 8.5.2.1.

Section C5.5.2.1 General

Add commentary explaining that SAC contains 2 types of FR connections – Type 1 (ductile) and Type 2 (brittle). These definitions are not used in FEMA 356, as the distinction is reflected in the acceptance criteria for the two types. Change reference from FEMA 267 to FEMA 351.

Section 5.5.2.2 Stiffness

Section 5.5.2.2.1 Linear Static and Dynamic Procedures

5. Connections

Revise section to reference connections in Table 5-X. Remove references to the 3 specific connections that are currently included in this section. Note that per the SAC definitions, a flange plate connection is FR for plates welded to the beam and PR for plates bolted to the beam, and an end plate connection is PR. A footnote in Table 5-X will indicate that the flange plate and end plate connections may be considered FR, for the purposes of modeling, if they meet the strength and stiffness requirements of Section 5.5.2.1.

Section 5.5.2.2.2 Nonlinear Static Procedure

Parameters appear to be consistent with SAC procedures for constructing load-deformation curves. No change to text.

Section C5.5.2.2.2 Nonlinear Static Procedures

Add commentary directing the user to consider the behavior of various tested connection configurations that are contained in FEMA 355d.

Section C5.5.2.2.3 Nonlinear Dynamic Procedures

Add commentary directing the user to consider the behavior of various tested connection configurations that are contained in FEMA 355d.

Section 5.5.2.3 Strength

Section 5.5.2.3.2 Linear Static and Dynamic Procedures

4. FR Beam-Column Connections

Add text indicating that connection strength shall be based on the governing element. Revise commentary to direct the user to FEMA 351 for computation of connection strengths.

Section 5.5.2.3.4 Nonlinear Dynamic Procedures

Add commentary directing the user to consider the behavior of various tested connection configurations that are contained in FEMA 355d.

Section 5.5.2.4 Acceptance Criteria

Add commentary stating that, in general, the strength and behavior of the moment frame will be governed by the connections.

The acceptance criteria for linear and nonlinear procedures has been developed based on FEMA 351 and FEMA 355d.

Section 5.5.2.4.2 Linear Static and Dynamic Procedures

1. Beams

Strike paragraph 5.4.2.3.A.v from this section since modifiers have been moved to Table 5-4. No additional changes to text. Beam acceptance criteria in Table 5-4 are not revised numerically, but additional modifiers to include effects of web slenderness (based on FEMA 355d, Section 4.6) have been added to the table as indicated below.

In Table 5-4,

change Beams-flexure item a to $\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} < \frac{418}{\sqrt{F_{ye}}}$

change Beams-flexure item b to $\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} > \frac{640}{\sqrt{F_{ye}}}$

change Beams-flexure item c to $\frac{52}{\sqrt{F_{ye}}} \leq \frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{418}{\sqrt{F_{ye}}} \leq \frac{h}{t_w} \leq \frac{640}{\sqrt{F_{ye}}}$

2. Columns

No change to text. Add commentary in Section C5.5.1 noting that the SAC procedure for determining seismic demand for column axial compression and splice tension is different, as are the acceptance criteria. Specifically, the SAC procedure does not require consideration of column flexural demands. Column acceptance criteria in Table 5-4 are not revised numerically, but additional modifiers to include effects of web slenderness (based on FEMA 355d, Section 4.6) have been added to the table as indicated below.

In Table 5-4,
For $P/P_{CL} < 0.20$,

change Columns-flexure item a to $\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} < \frac{300}{\sqrt{F_{ye}}}$

change Columns-flexure item b to $\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} > \frac{460}{\sqrt{F_{ye}}}$

change Columns-flexure item c to $\frac{52}{\sqrt{F_{ye}}} \leq \frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{300}{\sqrt{F_{ye}}} \leq \frac{h}{t_w} \leq \frac{460}{\sqrt{F_{ye}}}$

For $0.2 < P/P_{CL} < 0.50$,

change Columns-flexure item a to $\frac{b_f}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} < \frac{260}{\sqrt{F_{ye}}}$

change Columns-flexure item b to $\frac{b_f}{2t_f} > \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} > \frac{400}{\sqrt{F_{ye}}}$

change Columns-flexure item c to $\frac{52}{\sqrt{F_{ye}}} \leq \frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{260}{\sqrt{F_{ye}}} \leq \frac{h}{t_w} \leq \frac{400}{\sqrt{F_{ye}}}$

3. Panel Zone

There are no changes to the text or Table 5-4 concerning panel zone acceptance criteria resulting from the SAC project. However, for consistency with the nonlinear acceptance criteria, we have added m-factors for panel zones in secondary elements. Note also that panel zone strength is now a criterion for FR connection acceptance criteria as discussed below.

4. FR Beam-Column Connections

Indicate that connection behavior is dependent on adequacy of continuity plates, balanced strength conditions in the panel zone, beam L/d ratio, and the slenderness of the beam flanges and web. Provide, in the text, modifications to m-factors based on specified continuity plate, panel zone, beam L/d limitations, and beam flange/web slenderness based on the SAC publications. Also provide commentary on these modifiers.

Although slenderness of the beam flanges and web does not have an effect on the connection itself, it does affect the performance of the connection assembly. Therefore, a modifier based on the slenderness equations in Table 5-4 per item #1 above have been used. The modifier varies from 0.5 for slenderness above the upper limit (equation item b, above) to 1.0 for slenderness below the lower limit (equation item a, above) with straight line interpolation, based on the worst case of flange or web, used in between. This 0.5 modifier is based on the current values in Table 5-4 and is considered a rough approximation. The background data for this modifier are by no means rigorous as evidenced by the closing sentence of FEMA 355d, Section 4.6 which reads “it is recommended that further research be made to address the minimum unsupported length issue and the maximum slenderness issues, since these appear to be areas where further economy and improved seismic performance are possible.” Research on the combined effects of local flange and web buckling and lateral-torsional buckling has been performed using monotonic loading, but the behavior under cyclic loading is not well understood at this time.

Add various SAC-tested configurations (corresponding to those defined in Table 5-X) to Table 5-4.

Change “less than” to “greater than or equal to” in equation 5-14. (Error in text, not SAC related).

The results of SAC-sponsored testing were directly incorporated in the nonlinear modeling and acceptance criteria as described in the next section of this report. The linear acceptance criteria were developed from the nonlinear acceptance criteria in a manner consistent with the framework of FEMA 356 but based on the reliability information developed in the course of the SAC project. Rules for the development of acceptance criteria from test results are provided in Section 2.8.3 of FEMA 356. Item 7 of that section indicates that m values should be assigned such that the ductility capacity for linear procedures is 0.75 times that permitted for nonlinear procedures. This is consistent with the notion that linear analysis results are less accurate than are nonlinear analysis results. The SAC project explicitly identified the bias and uncertainty inherent in the various analytical procedures (as applied to steel moment frames). This level of bias and uncertainty is reflected by γ_a factors for various procedures as a function of performance level and system characteristics. In order to reflect the relative accuracy of the linear and nonlinear procedures in the FEMA 356 steel moment frame acceptance criteria (that is, to achieve similar reliability) we calculated the average ratios of $\gamma_{a,NSP}$ to $\gamma_{a,LSP}$ or $\gamma_{a,LDP}$ for IO and CP (0.97 and 0.86, respectively). We then assigned m values such that the ductility capacity for linear procedures is 1 and 0.86 times that permitted for nonlinear procedures (for IO and CP, respectively).

Section 5.5.2.4.3 Nonlinear Static and Dynamic Procedures

1. Beams

No change to text. Beam modeling parameters and acceptance criteria in Table 5-5 are not revised numerically, but additional modifiers to include effects of web slenderness (based on FEMA 355d, Section 4.6) have been added to the table. These changes are the same as for linear procedures as indicated previously.

2. Columns

No change to text. Add commentary in Section C5.5.1 noting that the SAC procedure for determining seismic demand for column axial compression and splice tension is different, as are the acceptance criteria. Specifically, the SAC procedure does not require consideration of column flexural demands. Column modeling parameters and acceptance criteria in Table 5-5 are not revised numerically, but additional modifiers to include effects of web slenderness (based on FEMA 355d, Section 4.6) have been added to the table. These changes are the same as for linear procedures as indicated previously.

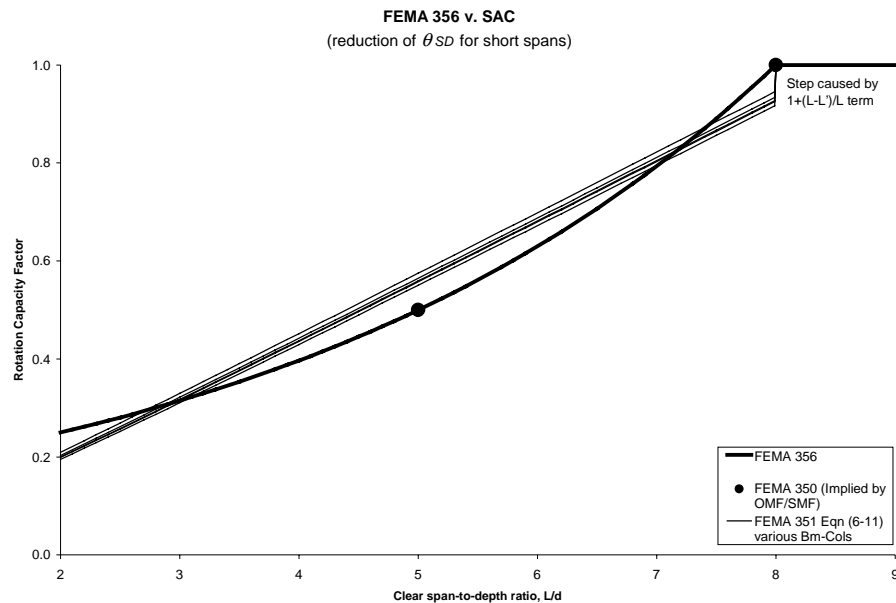
3. FR Beam-Column Connections

Suggest changing to #4 to be consistent.

Indicate that connection behavior is dependent on adequacy of continuity plates, balanced strength conditions in the panel zone, beam L/d ratio and the slenderness of the beam flanges and web. Provide, in the text, modifications to plastic rotation criteria based on specified continuity plate, panel zone, beam L/d limitations, and beam/web slenderness contained in the SAC publications. Also provide commentary on these modifiers.

The beam flange and web slenderness modifier is the same as for linear procedures as indicated previously.

We considered two options for incorporating the reduction of plastic rotation capacity for small span-to-depth ratios based on the FEMA 350 recommendations, namely, subtract 0.02 as L/d goes from 8 to 5 or multiply by $1/2$ as L/d goes from 8 to 5. We chose the latter. The following figure compares the proposed FEMA 356 modifier with the FEMA 350 and FEMA 351 recommendations.



Add various SAC-tested configurations to Table 5-5.

The results of SAC-sponsored testing were directly incorporated in the nonlinear modeling and acceptance criteria. The test results summarized in FEMA 355d were used to define the modeling criteria items a and b in Table 5-5. Where FEMA 355d contained data to define the residual strength ratio, c , such data were used; otherwise, c was taken as 0.2. IO acceptance criteria were calculated using FEMA 356 equation (2-9). It should be noted that equation (2-10) conflicts with the basis of FEMA 356 as indicated by the definitions of IO in items 6.1.1 and 6.2.1 of Section 2.8.3 and all of the tables of acceptance criteria. As defined in FEMA 356, IO performance is not related to the classification as primary or secondary. Therefore, Equation (2-10) is not appropriate.

The nonlinear acceptance criteria were developed from the modeling criteria in a manner consistent with the framework of FEMA 356 but based on the reliability information developed in the course of the SAC project. Rules for the development of acceptance criteria from test results are provided in Section 2.8.3 of FEMA 356. Items 6.1.2 and 6.2.2 of that section define the intended relationship between LS and CP acceptance criteria. Because LS performance does not appear in the SAC documents, we have used the Section 2.8.3 rules to develop LS acceptance criteria. The SAC documents also do not differentiate primary and secondary elements. However, the SAC reliability studies (reported in FEMA 355f) are based on buildings with steel moment frame lateral systems; this is most closely related to primary elements in the FEMA 356 framework.

Item 6.1.3 of FEMA 356 Section 2.8.3 indicates that the ductility capacity for primary elements should be taken as 0.75 times that permitted for secondary elements. This modification is intended to produce more reliable performance for primary elements. The SAC project explicitly identified the uncertainty and variability of performance related to structural modeling assumptions and the predicted character of ground shaking. This uncertainty is reflected by γ factors as a function of connection type, performance level, and building height. In order to reflect the relative reliability of primary and secondary performance in the FEMA 356 steel moment frame acceptance criteria (that is, to provide consistently more reliable performance for primary elements), we calculated the average values of $1/\gamma$ for CP performance of the SAC connection types 1 and 2 (0.76 and 0.66, respectively). We used these values to develop primary acceptance criteria from the secondary acceptance criteria taken directly from FEMA 355d.

The SAC guidelines provide a method to calculate the confidence level associated with achieving a specified performance objective. The method is based on the correlation of calculated demand/capacity ratios including demand factors γ and γ_a (discussed above) and a resistance factor ϕ (< 1) with confidence levels that are sensitive to an uncertainty coefficient, β_{UT} , and the slope of the hazard curve, k . In an average sense (based on the results of the SAC project), the application of the FEMA 356 criteria developed as described above are expected to result in confidence levels of 50% to 60% for CP performance of primary elements.

4. Panel Zone

Suggest changing to #3 to be consistent.

No changes to text or Table 5-5 concerning panel zone acceptance criteria resulting from SAC project. However, for consistency with the modeling criteria, we have added deformation limits for panel zones in secondary elements. Note also that panel zone strength is now a criterion for FR connection acceptance criteria as discussed above.

Section 5.5.2.5 Rehabilitation Measures

Section C5.5.2.5 Rehabilitation Measures

Revise commentary to include SAC references.

Section 5.5.3 Partially Restrained Moment Frames

Section 5.5.3.1 General

Connections are partially restrained where indicated in Table 5-X. Revise the definition to state that connection types not included in the table shall be considered partially restrained if the strength of the connection is less than the weaker of the two members being joined or if joint deformations contribute more than 10% to total lateral deflection of the frame. This is consistent with FEMA 355f, Section 8.5.2.1.

FEMA 356 currently defines and provides evaluation guidance for four PR connections as described in Section 5.5.3.3. These are 1) riveted or bolted clip angle, 2) riveted or bolted T-stub, 3) flange plate (welded or bolted to beam), and 4) end plate. A fifth type – composite partially restrained connections – is listed as a general type without guidance. Acceptance criteria for these connections are based on a limit state analysis of the various components of the connection assembly.

The SAC documents define and provide acceptance criteria for six PR connections – 1) bolted, unstiffened end plate, 2) bolted, stiffened end plate, 3) bolted flange plates, 4) double split tee, 5) shear tab with slab, and 6) shear tab without slab. The welded flange plate connection is defined as FR. Definitive recommendations for the clip angle connection are not included in the SAC documents. The plastic rotation capacities for PR connections reported in FEMA 355d are not explicitly tied to specific limit states, although some discussion of controlling limit states is provided. In general, the FEMA 273 acceptance criteria are in agreement with the test results summarized in FEMA 355d. However, it appears that the FEMA 273 writers examined the available test results with an eye to distinguishing the effect of controlling limit state on the resulting connection performance.

Integration into FEMA 356 of the SAC material for PR connections is complicated by the fact that the two documents contain a different suite of connections with somewhat different evaluation methodologies. This integration is even further complicated by changes made as FEMA 356 was developed from FEMA 273. We have considered three methods for integration:

- 1) Maintain the FEMA 356 limit state methodology and add the SAC methodology with acceptance criteria for the available connections, giving the user an option of either method for the overlapping connection types. This is not the most appropriate method for use in a standard as conflicts are sure to result.
- 2) Replace the FEMA 356 limit state methodology with the SAC rotation-based methodology for all connections included in SAC. If this approach were taken, the meaningful differences in rotation capacity due to controlling limit state would be lost.
- 3) Maintain the FEMA 356 limit state methodology (with FEMA 273 values) and do not use the limit-state-independent SAC acceptance criteria. Shear tab connection criteria would be added based on the results of SAC testing.

We recommend that the approach in option 3 be taken, and the revisions contained below and in Appendix A reflect this approach.

Section 5.5.3.2 Stiffness

Section 5.5.3.2.1 Linear Static and Dynamic Procedures

Add commentary that points to FEMA 274 for more detailed PR rotational stiffness information.

Section 5.5.3.2.2 Nonlinear Static Procedure

Add commentary that points to FEMA 355d for nonlinear behavior of PR connections.

Section 5.5.3.2.3 Nonlinear Dynamic Procedure

Add commentary that points to FEMA 355d for nonlinear behavior of PR connections.

Section 5.5.3.3 Strength

In the development of FEMA 356, the clarity of this section has suffered. As presently organized the capacity calculations seem to apply only when the nonlinear static procedure is used. Although the organization of this information in FEMA 273 was less than perfect, it was clear that the limit state calculations applied to both linear and nonlinear procedures. We have proposed revisions to clarify the capacity calculations. Specifically, the connection types and limit state calculations are moved to the section on linear procedures and the nonlinear static procedure section refers to that section.

A significant conceptual change was introduced to this section in the second draft of FEMA 356; many PR connection limit states were redefined as force-controlled. The connection performance criteria reported in FEMA 355d do not support this change. Consistent with these changes to the text, the corresponding entries in Table 5-5 were revised to read “force-controlled behavior” where plastic rotations were previously provided. In Table 5-4 these same connections and limit states have m values as large as 8. Classification as force-controlled for nonlinear procedures and m values of 8 for linear procedures is clearly in conflict with the philosophy of this standard. If a compelling case (based on technical information beyond that used in the SAC project) can be made for reclassifying these limit states as force-controlled, the acceptance criteria for linear procedures must be revised for consistency. Otherwise, we would recommend that the FEMA 273 values (and classifications) be reinstated, as indicated on our revised Table 5-5. As noted above, the PR connection test results reported in FEMA 355d are generally consistent with the FEMA 273 values.

For consistency with the SAC connection names, the new Table 5-X, and the classification of welded flange plates as an FR connection, we suggest that the following changes be made to the headings and table entries (Tables 5-4 and 5-5). “Top and Bottom T-Stub” should become “Double Split Tee.” “Composite Top Angle Bottom” should become “Composite Top and Clip Angle Bottom.” “Flange Plates Welded to Column Bolted or Welded to Beam” should become “Bolted Flange Plates.” “End Plate Bolted to Column Welded to Beam” should become “Bolted End Plate.”

The captions for Figures 5-3 through 5-6 should be revised to be consistent with the definitions used in the text. Figure 5-7 should be deleted or moved to the commentary since it represents only two of several different types of “other partially restrained connections.”

Section 5.5.3.4 Acceptance Criteria

Section 5.5.3.4.2 Linear Static and Dynamic Procedures

Add entries to Table 5-4 for shear connections with and without slab.

Unless additional technical information is available, we recommend that the FEMA 273 values continue to be used. However, all m -values are to be equal to or greater than 1.0 consistent with Global Topic 5-9. Therefore, IO m values for Flange Plate item b. and End Plate item c. will not be changed back to the values of 0.5 in FEMA 273.

We have an editorial suggestion that we believe would improve the usability of Table 5-4. Where acceptance criteria are a function of the limit state (expressed as a sub-item) we suggest that the limit state number in the text be noted (e.g., “Limit State 4 (angle flexure)”).

For completeness, we have added m values for two limit states for which they do not currently exist. Top and Bottom Clip Angle Limit State 2 (tension in horizontal leg) and Double Split Tee Limit State 3 (tension in tee stem) are considered deformation controlled in Section 5.5.3.3.2, yet no m values were provided in FEMA 273. We propose to use the m values for tensile yielding of the Flange Plate connection, which is judged to be similar limit state.

We also propose editorial revisions to Table 5-4 to make the order and wording of each component/element consistent with the text and with Table 5-5.

Section 5.5.3.4.3 Nonlinear Static and Dynamic Procedures

Add entries to Table 5-5 for shear connections with and without slab.

Unless additional technical information is available, we recommend that the FEMA 273 values be reinstated for all limit states that are currently defined as force-controlled. Also, the acceptance criteria for the Top and Bottom Clip Angle, Limit State 4 (angle flexure) appear to have been entered incorrectly. The FEMA 273 values will be reinstated for this limit state.

We have an editorial suggestion that we believe would improve the usability of Table 5-5. Where modeling and acceptance criteria are a function of the limit state (expressed as a sub-item) we suggest that the limit state number in the text be noted (e.g., “Limit State 4 (angle flexure)”).

For completeness, we have added acceptance criteria for two limit states for which they do not currently exist. Top and Bottom Clip Angle Limit State 2 (tension in horizontal leg) and Double Split Tee Limit State 3 (tension in tee stem) are considered deformation controlled in Section 5.5.3.3.2, yet no m values were provided in FEMA 273. We propose to use the acceptance criteria for tensile yielding of the Flange Plate connection, which is judged to be similar limit state.

We also propose editorial revisions to Table 5-5 to make the order and wording of each component/element consistent with the text and with Table 5-4.

(Note that in the 3rd SC draft, modeling criteria for PR connections in Table 5-5 are incorrectly identified as “d” and “e”. Consistent with FEMA 273 and previous drafts of FEMA 356, these should indicate plastic rotations “a” and “b” as the do in Appendix B.)

Section 5.5.3.5 Rehabilitation Measures

Section C5.5.3.5 Rehabilitation Measures

Revise commentary to include SAC references.

Section 5.6.3 Eccentric Braced Frames (EBF)

Section 5.6.3.4.1 General (Acceptance Criteria)

Add commentary stating that the acceptance criteria for FR connections was based on typical moment frame proportioning and configuration. The L/d modifier in Tables 5-4 and 5-5 was not tested for the relatively short link beams in EBFs.

ASCE/FEMA 273 Prestandard Project

**Special Study Report: Incorporating Results of the SAC Joint Venture
Steel Moment Frame Project**

Appendix A: FEMA 356 Edits

Table 5-2 Default Lower-Bound Material Strengths ¹

Properties based on ASTM and AISC Structural Steel Specification Stresses

Date	Specification	Remarks	Tensile Strength ² , Ksi	Yield Strength ² , Ksi
1900	ASTM, A9	Rivet Steel	50	30
	Buildings	Medium Steel	60	35
1901–1908	ASTM, A9	Rivet Steel	50	25
	Buildings	Medium Steel	60	30
1909–1923	ASTM, A9	Structural Steel	55	28
	Buildings	Rivet Steel	46	23
1924–1931	ASTM, A7	Structural Steel	55	30
		Rivet Steel	46	25
	ASTM, A9	Structural Steel	55	30
		Rivet Steel	46	25
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	60	33
		Eyebar flats unannealed	67	36
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct. 30, 1933	Structural Steel	55	30
		Structural Steel	60	33
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings)	Rivet Steel	52	28
	ASTM, A141-32T adopted as a standard			
1934 on	ASTM, A9	Structural Steel	60	33
	ASTM, A141	Rivet Steel	52	28
1961 – 1990	ASTM, A36	Structural Steel		
		Group 1	62	44
		Group 2	59	41
		Group 3	60	39
		Group 4	62	37
			70	41

	Group 5			
1961 on	ASTM, A572, Grade 50	Structural Steel		
	Group 1		65	50
	Group 2		66	50
	Group 3		68	51
	Group 4		72	50
	Group 5		77	50
1990 on	A36 & Dual Grade	Structural Steel		
	Group 1		66	49
	Group 2		67	50
	Group 3		70	52
	Group 4		70	49

1. Lower-bound values for material prior to 1960 are based on minimum specified values. Lower-bound values for material after 1960 are mean -1 standard deviation values from statistical data.

2. The indicated values are representative of material extracted from the flanges of wide flange shapes.

Table 5-3 Factors to Translate Lower-Bound Steel Properties to Expected Strength Steel Properties

Property	Year	Specification	Factor
Tensile Strength	Prior to 1961		1.10
Yield Strength	Prior to 1961		1.10
Tensile Strength	1961-1990	ASTM A36	1.10
	1961-present	ASTM A572, Group 1	1.10
		ASTM A572, Group 2	1.10
		ASTM A572, Group 3	1.05
		ASTM A572, Group 4	1.05
		ASTM A572, Group 5	1.05
	1990-present	ASTM A36 & Dual Grade, Group 1	1.05
		ASTM A36 & Dual Grade, Group 2	1.05
		ASTM A36 & Dual Grade, Group 3	1.05
		ASTM A36 & Dual Grade, Group 4	1.05
Yield Strength	1961-1990	ASTM A36	1.10
	1961-present	ASTM A572, Group 1	1.10

		ASTM A572, Group 2	1.10
		ASTM A572, Group 3	1.05
		ASTM A572, Group 4	1.10
		ASTM A572, Group 5	1.05
	1990-present	ASTM A36, Rolled Shapes	1.50
		ASTM A36, Plates	1.10
		Dual Grade, Group 1	1.05
		Dual Grade, Group 2	1.10
		Dual Grade, Group 3	1.05
		Dual Grade, Group 4	1.05
Tensile Strength	All	Not Listed ¹	1.10
Yield Strength	All	Not Listed ¹	1.10

1. For materials not conforming to one of the listed specifications.

5.5 Steel Moment Frames

5.5.1 General

[(5.4.1.ii)The behavior of steel moment-resisting frames is generally dependent on the connection configuration and detailing. Table 5-X identifies the various connection types for which acceptance criteria are provided. Modeling procedures, acceptance criteria, and rehabilitation measures for Fully Restrained (FR) Moment Frames and Partially Restrained (PR) Moment Frames shall be as defined in Sections 5.5.2 and 5.5.3, respectively.]

5 -X Steel Moment Frame Connection Types		
Connection	Description ^{1,2}	Type
Welded Unreinforced Flange (WUF)	Full-penetration welds between beam and columns flanges, bolted or welded web, designed prior to code changes following the Northridge earthquake	FR
Bottom Haunch in WUF w/ Slab	Welded bottom haunch added to existing WUF connection with composite slab ³	FR
Bottom Haunch in WUF w/o Slab	Welded bottom haunch added to existing WUF connection without composite slab ³	FR
Welded Cover Plate in WUF	Welded cover plates added to existing WUF connection ³	FR
Improved WUF-Bolted Web	Full-penetration welds between beam and column flanges, bolted web ⁴	FR
Improved WUF-Welded Web	Full-penetration welds between beam and column flanges, welded web ⁴	FR
Free Flange	Web is coped at ends of beam to separate flanges, welded web tab resists shear and bending moment due to eccentricity due to coped web ⁴	FR
Welded Flange Plates	Flange plate with full-penetration weld at column and fillet welded to beam flange ⁴	FR
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face ⁴	FR
Welded Bottom Haunch	Haunched connection at bottom flange only ⁴	FR
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges ⁴	FR
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange ⁴	FR
Top and Bottom Clip Angles	Clip angles bolted or riveted to beam flange and column flange	PR
Double Split Tee	Split Tees bolted or riveted to beam flange and column flange	PR

Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab	PR
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange ⁴	PR ⁵
Bolted End Plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange	PR ⁵
Shear Connection w/ Slab	Simple connection with shear tab, composite slab	PR
Shear Connection w/o Slab	Simple connection with shear tab, no composite slab	PR

1. Where not indicated otherwise, definition applies to connections with bolted or welded web.
2. Where not indicated otherwise, definition applies to connections with or without composite slab.
3. Full-penetration welds between haunch or cover plate to column flange conform to the requirements of the AISC *Seismic Provisions for Structural Buildings* (AISC, 1997c)
4. Full-penetration welds conform to the requirements of the AISC *Seismic Provisions for Structural Buildings* (AISC, 1997c)
5. For purposes of modeling, connection may be considered FR if it meets strength and stiffness requirements of Section 5.5.2.1.

ASCE/FEMA 273 Prestandard Project

**Special Study Report: Incorporating Results of the SAC Joint Venture Steel Moment Frame
Project**

Appendix B: Revisions to Tables 5-4 and 5-5

Additional linear acceptance criteria (add to Table 5-4)

Connection	IO	Primary		Secondary	
		LS	CP	LS	CP
FR Connections					
WUF	1.0	4.3 - 0.083d	3.9 - 0.043d	4.3 - 0.048d	5.5 - 0.064d
Bottom haunch in WUF with slab	1.6	2.7	3.4	3.8	4.7
Bottom haunch in WUF without slab	1.3	2.1	2.5	2.8	3.3
Welded cover plate in WUF	2.4 - 0.030d	4.3 - 0.067d	5.4 - 0.090d	5.4 - 0.090d	6.9 - 0.118d
Improved WUF-bolted web	1.4 - 0.008d	2.3 - 0.021d	3.1 - 0.032d	4.9 - 0.048d	6.2 - 0.065d
Improved WUF-welded web	2.0	4.2	5.3	5.3	6.7
Free flange	2.7 - 0.032d	6.3 - 0.098d	8.1 - 0.129d	8.4 - 0.129d	11.0 - 0.172d
Reduced beam section	2.2 - 0.008d	4.9 - 0.025d	6.2 - 0.032d	6.5 - 0.025d	8.4 - 0.032d
Welded flange plates					
Flange plate net section	1.7	3.3	4.1	5.7	7.3
Other limit state					
Welded bottom haunch	1.6	3.1	3.8	4.6	5.9
Welded top and bottom haunches	1.6	3.1	3.9	4.7	6.0
Welded cover-plated flanges	1.7	2.8	3.4	3.4	4.2
PR Connections					
Shear connection with slab	1.6 - 0.005dbg	----	----	13.0 - 0.290dbg	17.0 - 0.387dbg
Shear connection without slab	4.9 - 0.097dbg	----	----	13.0 - 0.290dbg	17.0 - 0.387dbg

d is the depth of the beam.

d_{bg} is the depth of the bolt group.

Tabulated values shall be modified as indicated in Sec. 5.5.2.4.2, item 4.

Additional nonlinear modeling and acceptance criteria (add to Table 5-5)

Connection	a	b	c	IO	Primary		Secondary	
					LS	CP	LS	CP
FR Connections								
WUF	0.051 - 0.0013d	0.043 - 0.0006d	0.2	0.0128 - 0.0003d	0.0337 - 0.0009d	0.0284 - 0.0004d	0.0323 - 0.0005d	0.043 - 0.0006d
Bottom haunch in WUF with slab	0.026	0.036	0.2	0.0065	0.0172	0.0238	0.0270	0.036
Bottom haunch in WUF without slab	0.018	0.023	0.2	0.0045	0.0119	0.0152	0.0180	0.023
Welded cover plate in WUF	0.056 - 0.0011d	0.056 - 0.0011d	0.2	0.0140 - 0.0003d	0.0319 - 0.0006d	0.0426 - 0.0008d	0.0420 - 0.0008d	0.056 - 0.0011d
Improved WUF-bolted web	0.021 - 0.0003d	0.050 - 0.0006d	0.2	0.0053 - 0.0001d	0.0139 - 0.0002d	0.0210 - 0.0003d	0.0375 - 0.0005d	0.050 - 0.0006d
Improved WUF-welded web	0.041	0.054	0.2	0.0103	0.0312	0.0410	0.0410	0.054
Free flange	0.067 - 0.0012d	0.094 - 0.0016d	0.2	0.0168 - 0.0003d	0.0509 - 0.0009d	0.0670 - 0.0012d	0.0705 - 0.0012d	0.094 - 0.0016d
Reduced beam section	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.0125 - 0.0001d	0.0380 - 0.0002d	0.0500 - 0.0003d	0.0525 - 0.0002d	0.07 - 0.0003d
Welded flange plates								
Flange plate net section	0.03	0.06	0.2	0.0075	0.0228	0.0300	0.0450	0.06
Other limit state	force-controlled							
Welded bottom haunch	0.027	0.047	0.2	0.0068	0.0205	0.0270	0.0353	0.047
Welded top and bottom haunches	0.028	0.048	0.2	0.0070	0.0213	0.0280	0.0360	0.048
Welded cover-plated flanges	0.031	0.031	0.2	0.0078	0.0177	0.0236	0.0233	0.031
PR Connections								
Shear connection with slab	0.029 - 0.0002dbg	0.15 - 0.0036dbg	0.4	0.0073 - 0.0001dbg	----	----	0.1125 - 0.0027dbg	0.15 - 0.0036dbg
Shear connection without slab	0.15 - 0.0036dbg	0.15 - 0.0036dbg	0.4	0.0375 - 0.0009dbg	----	----	0.1125 - 0.0027dbg	0.15 - 0.0036dbg

d is the depth of the beam.

dbg is the depth of the bolt group.

Tabulated values shall be modified as indicated in Sec. 5.5.2.4.3, item 4.

L. Ballot Comment Resolution Report

Unofficial Letter Ballot on the Second Draft of FEMA 356 Prestandard for the Seismic Rehabilitation of Existing Buildings

Ballot Comment Resolution Report

Introduction

On March 22, 2000 the Second Draft of the FEMA 356 Prestandard for the Seismic Rehabilitation of Buildings was published. This document was submitted to the ASCE Standards Committee (SC) for Seismic Rehabilitation for informal letter ballot. The purpose of this ballot was to receive and consider written comments from the SC on specific technical issues while there was still time to make revisions during the funded portion of the development of the Prestandard. The ballot was unofficial, so formal ASCE rules on balloting were suspended.

This report represents the ASCE/FEMA 356 Prestandard Project Team (PT) response to comments received from the unofficial letter ballot on the Second Draft of the Prestandard, and serves as a record of that ballot. While formal ASCE rules were suspended, every effort was made to respond in a manner consistent with those rules whenever possible.

Every written comment received as of June 1, 2000, is listed in this report by author name and ballot number as follows:

Editorial:

The text of comments judged editorial is not reproduced in this report, although an indication of the PT's acceptance of the editorial comments is included for each item.

Affirm w/comment:

Affirmative comments that are more substantive may have a brief paraphrased summary of the comment followed by a ruling of the PT's acceptance of the comment.

Negative:

All negative comments are documented individually in this report with a brief paraphrased summary of the negative comment, a classification of editorial, persuasive, or non-persuasive, and a brief discussion of the resolution. Negative comments judged non-persuasive have a response explaining the reason for the non-persuasive finding.

In response to comments received, the PT may have taken one of the following actions:

Editorial - Accepted:

Comments judged editorial in which the suggested changes have been incorporated into the text of the Prestandard

Editorial - Accepted with revisions:

Comments judged editorial in which the suggested changes have been revised in some way and then incorporated into the text of the Prestandard.

Editorial - Not accepted:

Comments judged editorial in which the verbiage was judged inconsistent or otherwise not appropriate for inclusion into the text of the Prestandard.

Persuasive:

Comments judged to be technically substantive and valid, and the suggested changes have been incorporated into the text of the Prestandard

Persuasive - No change made:

Comments judged to be technically substantive and valid, however, further study of information or additional research is required before the suggested changes can be incorporated into the text of the Prestandard

Non-persuasive:

Comments judged to be technically substantive but not appropriate for inclusion into the text of the Prestandard

GENERAL:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Lawver	n/a	n/a	n/a
Refers to McClure's comments on overturning. See response to McClure Item 22, Section 3.2.10.			
Misovec	all	all	n/a
General comment that labeling codes (paragraph numbers) in commentary sections is unclear. These numbers are intended to track where the information came from during the draft process and will be deleted in the final document.			

CHAPTER 1:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Hess	1	1.1	negative
Document not ready for ballot. Should establish standing committees to look at each chapter.			
<i>Non-persuasive</i> — It is the opinion of FEMA, ASCE, and the PT that the profession is best served by the development of standards for use in practice, which can then be improved over time as research information becomes available. This is especially true in the case of rehabilitation of existing buildings, where there are no nationally accepted standards governing prevailing practice. Just as building codes for new construction evolve over time, so is the vision for FEMA 356, which at this point in time represents the best current knowledge with regard to seismic rehabilitation.			
Lundeen	1, 2	1.1, 1.2	affirm w/comment
<i>Editorial</i> — Accepted with revised changes. See revisions.			
Misovec	1, 3	1.1, 1.3	affirm w/comment
<i>Editorial</i> — Suggested change for 1.1 is accepted. The confusion about paragraph numbering is addressed in the response to his general comment, above. His comment for C1.3 asks whether HTML technology can be used to "quickly modify the document." This ASCE consensus standard can only be modified by a consensus standard approval process.			
Trahern	1	1.1	affirm w/comment
<i>Editorial</i> — Accepted with revised changes. See revisions.			

Turner	1	1.1	negative
<p>1. Relocate operative requirement that defines who is responsible for selecting an objective to Section 1.2.2.</p> <p><i>Persuasive</i> — See revisions. Scoping sections of other chapters have been similarly revised.</p> <p>2. Delete the definition of Code Official in this section, which is redundant with Section 1.7, definitions.</p> <p><i>Persuasive</i> — See revisions. Scoping sections of other chapters have been similarly revised.</p>			
Yusuf	1, 4, 5, 6	1.1, 1.4, 1.5, 1.6	affirm w/comment
<p><i>Editorial</i> — Accepted with revised changes. See revisions.</p>			
Fallgren	2, 6, 9	1.2, 1.6, 1.8	affirm w/comment
<p><i>Editorial</i> — Comment in C1.2.6.2 is accepted. The maps referred to in 1.6.1 will be properly referenced in the final Prestandard. The suggestion to define "X" in Equations 1-4 and 1-5 and in Section 1.8 is not accepted because it does not represent selected hazard level which is determined by the selected values of S_s and S₁. The suggestion to delete from Figure 1-1 the Equation 1-8 is not accepted but the figure will be corrected to match Equation 1-8. Suggested correction in C1.6 is accepted.</p>			
Hom	2	1.2.1	affirm w/comment
<p>Suggests making evaluation using FEMA 310 mandatory in 1.2.1.</p> <p><i>Non-persuasive</i> — See Kehoe Comment 2, Item 2. The requirement to perform a prior seismic evaluation will be clarified in Section 1.2, but it is the opinion of the PT that other approved evaluation methods should be permitted in addition to FEMA 310.</p>			
Kehoe	2	1.2	negative
<p>1. The verbiage in the document should be general to both evaluation and rehabilitation.</p> <p><i>Non-persuasive</i> — The stated intent of the document is rehabilitation, which is a higher criteria than evaluation, and it was intended to keep that distinction clear. Direction on use of FEMA 356 as an evaluation tool covered in FEMA 310. It is conceivable to use FEMA 356 as a “zero-rehab” evaluation tool, but that implies the building meets the performance level at a higher level of reliability than a FEMA 310 Tier 3 evaluation at 0.75 times the demands. It is the opinion of the PT that the verbiage is sufficiently general and can be applied in cases of evaluation when needed.</p> <p>2. No reference is made to the step of evaluation prior to rehabilitation.</p> <p><i>Persuasive</i> — Reference to evaluation will be made more explicit in Section 1.2. See revisions.</p>			

Trahern	2	1.2.3	negative
As-built information is impossible to obtain in many instances.			
<i>Non-persuasive</i> — Section 2.2 explains what is intended by as-built information and how to obtain it.			
Turner	2	1.2	negative
Relocate operative requirements from Section 1.1, Scope, to this section.			
<i>Persuasive</i> — See revisions.			
Kehoe	3	1.3	negative
<i>Editorial</i> — Accepted. Remove the words “performance based.”			
Nicoletti	3, 6	C1.3, C1.6	affirm w/comment
<i>Editorial</i> — Suggested change to include the Vision 2000 document in C1.3 is not accepted as the list of documents is intended to include only those "generically related" documents FEMA developed prior to FEMA 273. The suggested correction in C1.6 is made.			
Turner	3	C1.3	affirm w/comment
<i>Editorial</i> — Not accepted. Reference to ATC 40 should remain because it is generally related to FEMA 273 regardless of the current opinion of its validity, which could change.			
Fantozzi	4	1.4.1, 1.4.3	affirm w/comment
<i>Persuasive</i> — See Kehoe Comment 3, Item 4. Editorial comment on Section 1.4.3 is accepted with revised changes.			
Hess	4	1.4	negative
Establish a separate track of rehabilitation objectives for nonstructural elements.			
<i>Non-persuasive</i> — The PT does not have technical justification to revise the requirements for nonstructural rehabilitation objectives at this time.			
Johnson	4	1.4	affirm w/comment
<i>Editorial</i> — Not accepted. The character is not stray. See response to Misovec General Comment.			
Kehoe	4	1.4	negative
1. The verbiage in the document should be general to both evaluation and rehabilitation.			
<i>Non-persuasive</i> — See response to Kehoe Comment 1, Item 2.			

2. References to performance levels 3-C and 5-E in Section 1.4.1 occur before the terms are defined.

Editorial —

Not accepted. Reference to definitions of required terms are provided in the preceding section (1.4).

3. Section 1.4.1 references building codes that are deemed to meet the BSO. This implies the standard is being used for evaluation. This also implies that buildings evaluated and judged to meet the requirements of one of the cited codes are then deemed to meet the BSO. This means the cited codes are being used as evaluation criteria.

Persuasive —

References to building codes deemed to meet the BSO will be removed from the standard. This issue is more appropriately addressed in the FEMA 310 evaluation document benchmark buildings provisions.

4. Section 1.4.1 is not clear.

Editorial —

Accepted. See revisions.

5. Restrictions on limited rehabilitation do not permit measures that might reduce the strength of some components but improve overall performance of the building (i.e., remove infill in frame/infill buildings).

Editorial —

Accepted with revised change. See revisions.

6. The statement that partial rehabilitation shall be designed to allow for completion of the Rehabilitation Objective should be deleted. The phrase “to allow for” is open to interpretation.

Editorial —

Accepted with revised change. See revisions.

7. References to performance levels 3-C and 5-E in Section 1.4.3.2 occur before the terms are defined.

Editorial —

See response to Kehoe Comment 2, Item 4.

Lundeen	4	1.4	negative
See Kehoe Comment 3, Item 4 (similar).			
Trahern	4	1.4.1	negative
Buildings designed to recent codes may not be acceptable due to changes in detailing practices or seismicity of the region.			
Persuasive —			
See Kehoe Comment 3, Item 4.			
Turner	4	1.4	negative
1. See Kehoe Comment 3, Item 4.			
2. Replace “collapse prevention” with softer term such as “near collapse” since prevention could be construed as a guarantee of performance.			

Non-persuasive —

This recommendation should be considered further in relation to GT 2-14 regarding performance levels implying a guarantee of performance.

Kehoe	5	1.5	negative
1.	The life safety performance level cannot be quantified as a definite level and should be considered as a range.		
	<i>Persuasive</i> — No change made. This issue is already identified in GT 2-24 and recommended for basic research.		
2.	The definition of the Immediate Occupancy Performance Level is not attainable, since any observed cracking can be claimed to have diminished the stiffness of the building beyond its pre-earthquake condition.		
	<i>Persuasive</i> — It is the opinion of the PT that stiffness is an important component of IO performance and that meeting acceptance criteria of this standard will essentially preserve the pre-earthquake strength and stiffness of the structure. Adding a permissible reduction in strength or stiffness in the definition of IO performance will create a secondary acceptance criteria that may conflict the rest of the standard. This issue was discussed at the 8/23/00 Standards Committee meeting. It was decided that the definition could be revised to state that IO performance is safe to occupy after an earthquake and the structure essentially retains the pre-earthquake strength and stiffness.		
3.	Sections 1.5.1.1 through 1.5.1.6 should discuss performance levels and ranges in numeric order.		
	<i>Editorial</i> — Accepted. See revised changes.		
4.	Operational performance of nonstructural components should have input from building owner as well as code official.		
	<i>Editorial</i> — Accepted with revised changes. See revisions.		
5.	Six foot maximum dimension criteria for Hazards Reduced Level not appropriate.		
	<i>Editorial</i> — Not accepted. The proposed change does not address the intent of the provision.		
6.	The definition of the Collapse Prevention Target Building Performance Level does not explicitly discuss nonstructural components.		
	<i>Editorial</i> — Accepted. See revisions.		
7.	Quantitative values in Tables C1-3 through C1-5 should be deleted.		
	<i>Non-persuasive</i> — Values occur in the commentary are non-binding. This information is considered useful in describing the difference between performance levels, and can be useful to engineers in understanding the new concepts of the prestandard.		

Turner	5	1.5	negative
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Replace “meet the requirements” with “meet or exceed the requirements” throughout.

Editorial —

Not accepted. It is implied that exceeding the requirements still meets the requirements.

Kehoe	6	1.6	negative
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1. Need definition of active fault.

Persuasive —

The definition of active fault has been taken from the 1997 NEHRP Provisions and included in Section 1.7.

2. Need references to Figures xx-yy.

Editorial —

Accepted. See Fallgren comments, Item 6.

3. Remove the 10%/50 year earthquake from the definition of the BSE-1 hazard level.

Non-persuasive —

This issue is addressed in GT 2-16 and the reason for inclusion of the 10%/50 year earthquake is described in the discussion.

4.a. Requirements for vertical seismic effects should be clarified.

Editorial —

Not accepted. Requirements are specified in Section 2.6.11.

4.b. Use of 2/3 horizontal for vertical spectra should be revised.

Persuasive —

No change made. This issue should be considered further as a new global issue.

5. More guidance on damping values should be provided in Section 1.6.1.5.3.

Persuasive —

No change made. This issue should be considered further as a new global issue.

Lundeen	6	1.6	negative
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1. Define active fault

Persuasive —

See Kehoe Comment 1, Item 6.

2. Provide values for Type E soils in the highest ground shaking columns of Tables 1-4 and 1-5.

Persuasive —

The tables will be revised to match the 2000 NEHRP Provisions during the 3rd draft cycle.

3. Clarify the intent of Section 1.6.2.1.4 regarding the use of site specific spectra.

Editorial —

Accepted. “Constructed” has been removed from the section to improve clarity.

4. Editorial comment on C1.6.2.1 accepted. See revisions.

5. Revise Section 1.6.3 to base zones of seismicity on 2/3 MCE instead of 10%/50 hazard levels.

Non-persuasive —

The current formulation has been retained for consistency with the BSO.

McClure	6	1.6	negative
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Global Issue 2-2 regarding ground motion pulses has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.

Non-persuasive —

It is the consensus of FEMA, ASCE and the PT that global issues identified during the Prestandard process and left unresolved pending future research are locations where the document can be improved, but do not constitute a fundamental flaw in the application of the FEMA 273 methodology to the rehabilitation of buildings. While the FEMA 343 Case Studies report identified a number of technical and usability recommendations for further study, the stated intent of these this recommendations was “to improve the ease with which engineers can apply the Guidelines provisions and the efficiency of the designs that result.” This opinion is confirmed by the summary conclusion presented in FEMA 343, Section 2.2 Technical Adequacy, which states that “In summary, the case studies results support the conclusion that the Guidelines provides a technically adequate approach to seismic rehabilitation that is fundamentally sound but that, for some aspects of design, may be more stringent than is necessary to achieve the targeted building performance.” It is the opinion of ASCE policy makers and the PT that the profession is best served by the development of standards for use in practice, which can then be improved over time as research information becomes available. This is especially true in the case of rehabilitation of existing buildings, where there are no nationally accepted standards governing prevailing practice. Just as building codes for new construction evolve over time, so is the vision for FEMA 356, which at this point in time represents the best current knowledge with regard to seismic rehabilitation.

McConnell	6	1.6	negative
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The MCE hazard level results in unreasonable increases in seismic force values for some areas of the nation.

Non-persuasive —

The PT does not have technical justification to revise the basis of the MCE hazard level at this time.

Pappas	6	1.6	affirm w/ comment
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Editorial —

Accepted with revised changes. Proper reference to USGS design map information will be provided prior to publication of the Prestandard.

Paruvakat	6, 8	1.6, 1.8	affirm w/comment
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Persuasive —

S_s and S_1 are inconsistently defined as "acceleration" with units of g. Since they are multiplied by weight to obtain force, they are dimensionless coefficients of acceleration divided by g. They will all be consistently called "acceleration parameters" in the Prestandard. Suggested correction in 1.6.1.4 is made.

Turner	6	1.6	negative
<p>1. Revise Tables 1-4 and 1-5 for Site Class Fa and Fv values to be consistent with proposed changes to the NEHRP Provisions in BSSC Proposal 3-18 for consistency with the 2000 NEHRP Provisions.</p> <p><i>Persuasive</i> — The tables will be revised to match the 2000 NEHRP Provisions during the 3rd draft cycle.</p> <p>2. Symbols S₁ and S_S should be revised to emphasize differences from similar symbols in the NEHRP Provisions.</p> <p><i>Non-persuasive</i> — This issue was addressed and discussed in GT 2-15.</p>			
Kehoe	7	1.7	negative
<p><i>Persuasive</i> — Change accepted to include the definition of active fault.</p>			
Turner	7	1.7	negative
<p>Revise the definition of “Rehabilitation Method” so that it does not conflict with a definition of the same term used in another standard (Secretary of the Interior Standards for the Treatment of Historic Properties).</p> <p><i>Editorial</i> — Accepted with revised changes. See revisions.</p>			
Turner	8	1.8	negative
<p>See Turner Comment 2, Item 6.</p>			
Turner	9	1.9	negative
<p>See Turner Comment on Ballot Item 3, section C1.3.</p>			

CHAPTER 2:

Author	Item	Section	Vote
Yusuf	10	2.1	affirm w/comment
<i>Editorial</i> — Reference to Chapter 4 for the simplified rehabilitation has been corrected to Chapter 10.			
Fantozzi	11	2.2.5	affirm w/comment
<i>Editorial</i> — Accepted. Format of table locations, section breaks and page breaks will be addressed in the final draft.			
Iqbal	11	2.2.4.1	affirm w/comment
The 4% separation requirement is too stringent and should be reduced for buildings in lower regions of seismicity or for buildings that have matching floor levels.			
<i>Non-persuasive</i> — Section 2.6.10.2 already exempts buildings with matching diaphragm levels and similar heights for LS Performance Levels and lower.			
Kehoe	11	2.2	negative
<ol style="list-style-type: none">1. Clarify engineer's responsibility when a subsurface investigation must be performed in Section 2.2.3.			
<i>Persuasive</i> — See revisions.			
<ol style="list-style-type: none">2. Clarify notification procedures of Section 2.2.4 when insufficient information is available on adjacent structures.			
<i>Persuasive</i> — See revisions.			
<ol style="list-style-type: none">3. Clarify references to adjacent building in Section 2.2.4.3.			
<i>Persuasive</i> — See revisions.			
<ol style="list-style-type: none">4. Clarify requirements on chemical, fire, or explosion hazards from adjacent buildings.			
<i>Persuasive</i> — Accepted with revised changes. See revisions. Intent of this section is to consider the appropriateness of the selected rehabilitation objective in light of the potential for these types of hazards.			
<ol style="list-style-type: none">5. Clarify application of Table 2-1 with explanation in Section 2.2.6.4.1.			
<i>Editorial</i> — Accepted with revised changes. Table 2-1 has been edited for additional clarity. Much of the information proposed for Section 2.2.6.4.1 is already included in Sections 2.2.6.1 through 2.2.6.3. See revisions.			

6. Data collection requirements on adjacent buildings in 2.2.6.1 should be coordinated with 2.2.4.

Persuasive —
See revisions.

7. Clarify how κ values are substantiated.

Persuasive —
Accepted with revised changes. See revisions.

Lundeen	11	2.2	negative
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1. Delete the requirement to document as-built information (and source of such information) in the rehabilitation design.

Persuasive —
See revisions.

2. Delete the requirement to notify the code official when the owner cannot provide information on adjacent structures.

Persuasive —
See Kehoe Comment 2, Item 11.

3. Revise the requirements in Section 2.2.4.3 regarding fire, chemical leakage or explosion.

Persuasive —
See Kehoe Comment 4, Item 11.

Yusuf	11	2.2	affirm w/comment
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Use of the term “exposed” implies this condition must be observable in its existing condition.

Editorial —
Not accepted. The term “exposed” can also apply to conditions which are observed through necessary removal of finished or destructive investigation if required in Chapters 4 through 8.

Pappas	11, 14	2.2, 2.5	affirm w/comment
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Editorial —
Accepted. See revisions.

Paruvakat	11	2.2.3	affirm w/comment
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Persuasive —
See Kehoe Comment 1, Item 11.

Trahern	11	2.2	negative
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See Trahern Comment on Ballot Item 2, Section 1.2.3.

Kehoe	12	2.3	negative
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1. Reference evaluation step prior to rehabilitation in this section.

Non-persuasive —
Prior evaluation has been referenced in Section 1.2.

2. Clarify who selects analysis procedure in Section 2.3.2.

Editorial —
Accepted. See revisions.

Chang	13, 19, 20	2.4, 2.10, 2.11	affirm w/comment
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Editorial comments in above noted sections accepted. See revisions.

Fantozzi	13	2.4.1.2	negative
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Provide an exception to the limitation on the use of the LSP for buildings over 100 feet when the building is regular and located in a region of low seismicity.

Persuasive —

This issue was discussed at the 8/23/00 Standards Committee meeting. The limitation will be revised to state that structures with $T < 3.5T_s$ may use the LSP, consistent with changes proposed for the 2000 NEHRP Provisions.

Kehoe	13	2.4	negative
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1. Clarify the steps for determining the presence of irregularities.

Editorial —

Not accepted. Section 2.4.1.1 specifies the procedure.

2. Substitute “configuration” for “condition” in Section 2.4.1.1.

Editorial —

Accepted. See revisions

3. Specify Code Official for approving alternative analyses.

Editorial —

Not accepted. The Code Official is the implied entity for making all approvals.

4. The 2V:1H slope criteria for classifying component behavior is extraneous information and should be deleted from Section 2.4.4.3.1.

Persuasive —

See revisions.

Johnson	13	2.4	negative
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Acceptance criteria for linear procedures, particularly force-controlled elements are unusable in their present form. Component m-factors of 2 to 3 cannot be justified in comparison against code criteria for new buildings. Reduce applied forces to some multiple of the resisting moment and increase m-factors commensurate with observed performance of buildings.

Persuasive —

No change made. The ideas behind these comments have been considered in multiple issues documented in the Global Topics Report, including 2-1, 2-6, 3-10, 3-13, 3-27, 5-1, 6-1, 7-1, 8-1 and in Special Study 6 – Acceptability Criteria (Anomalous m-values). Some progress on conservatism in the LSP has been made with the revisions published in the Prestandard, but further research is recommended.

McClure	13	2.4.1, 2.4.4	negative
<p>Global Issues 2-19 regarding upper limits on DCRs for linear analysis and 2-6 regarding baseline adjustments to acceptance criteria have been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
Lundeen	13	2.4	affirm w/comment
<p>1. Remove the requirement for the second response spectrum analysis considering only the first mode response in Section 2.4.2.1.</p> <p><i>Non-persuasive</i> — Commercial software can be set to consider as many (or as few) modes as desired.</p> <p>2. Consider a different approach for addressing higher mode effects with the NSP in Section 2.4.2.1 (i.e., increase the target displacement by some factor).</p> <p><i>Non-persuasive</i> — The requirements of this section do not combine m-factor concepts with nonlinear acceptance. There are really two separate analyses performed. The concern about higher mode effects is a dynamic response of the structure that is significantly different from the first mode response. Amplifying the target displacement does not adequately account for higher mode effects because this just pushes the building farther with the same first mode displaced shape and does not check the building for other shapes.</p> <p>3. Move statistical definitions of QCL, QCE, and material properties in 2.4.4.4 and 2.4.4.5 to the commentary. Actual definitions are in Chapters 5 through 8.</p> <p><i>Non-persuasive</i> — These definitions are required for the generic definition of the quantities. Chapters 5 through 8 supplement these definitions with specific capacity calculations for specific actions.</p>			
Misovec	13	C2.4	affirm w/comment
<p><i>Editorial</i> — Accepted with revised changes. See revisions.</p>			
Nicoletti	13	C2.4.4.3.1	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			
Trahern	13	2.4	negative
<p>1. Use of the terminology “all” is onerous. Delete “all” in Section 2.4.</p> <p><i>Persuasive</i> — See revisions.</p> <p>2. Add “primary elements” as a qualifier to the requirement for one continuous load path.</p> <p><i>Non-persuasive</i> — Nonlinear analyses modeled using components including full degrading backbone curves may utilize elements performing at secondary limits of response in the lateral force resisting system.</p>			

Kehoe	14	2.5	negative
1.	Specify how to determine unacceptable performance for performance levels not covered by FEMA 310.		
	<i>Persuasive</i> — This issue was discussed at the 8/23/00 Standards Committee meeting. The commentary was judged to provide sufficient information in this regard.		
2.	Define “generally acceptable overall performance” or move to commentary.		
	<i>Editorial</i> — Accepted with revised changes. Sections 2.5.1 through 2.5.7 have been editorially revised to state permissible rehabilitation strategies and omit qualifiers of “acceptable performance.”		
3.	Excessive mass in Section 2.5.5 needs to be related to strength and stiffness.		
	<i>Editorial</i> — See response to Kehoe Comment 2, Item 14.		
4.	Define “unacceptable performance” or move to commentary.		
	<i>Editorial</i> — See response to Kehoe Comment 2, Item 14.		

Johnson	15	2.6.7.1, 2.6.9	affirm w/comment
1.	The wall anchorage force amplification factor $X=3.0$ should only apply in the central portion of the diaphragm.		
	<i>Non-persuasive</i> — Analyses reported by Mehraïn, et al., “Dynamic Analysis of Tilt-up Buildings,” 4 th NCEE, 1990, conclude that dynamic amplification occurs over the entire length of the diaphragm.		
2.	The requirements for buildings with shared common elements are excessively restrictive when the only shared elements are in the foundation. In this case separation is impractical.		
	<i>Persuasive</i> — The requirement to separate or tie buildings sharing foundation elements together was softened for the case when the superstructures meet the separation requirements of Section 2.6.10. See revisions.		

Kehoe	15	2.6	negative
1.	Provide direction on estimating deflection of adjacent buildings.		
	<i>Persuasive</i> — See revisions.		
2.	Specify Code Official for approving analysis procedure.		
	<i>Editorial</i> — Not accepted. See Kehoe Comment 3, Item 13.		
3.	Define “permanent live loads” in Section 2.6.11.		
	<i>Editorial</i> — Accepted with revised changes. Cross-reference to gravity loads specified in Section 3.2.8 provided. See revisions.		

Lundeen	15	2.6	affirm w/ comment
<p>Include performance level adjustment factors in Equations 2-3, 2-4, and 2-5 to vary force with performance (similar to factors included in 2-6 and 2-7).</p> <p><i>Non-persuasive</i> — Equations 2-3, 2-4 and 2-5 are connection requirements intended to make sure the components of a building are tied together. The forces are set at a minimum level intended to apply to all performance levels.</p>			
Trahern	15	2.6	affirm w/comment
<p><i>Editorial</i> — Comment to add a column in Table 2-4 for flexible diaphragms is accepted.</p>			
Breiholz	16	2.7	affirm w/comment
<p><i>Editorial</i> — Accepted with revised changes. See revisions.</p>			
Hui	16	2.7.1	affirm w/comment
<p><i>Editorial</i> — Not accepted. "Code Official" is defined in Chapter 1 and could mean the local building official if that entity has the legal charge to enforce the standard.</p>			
Kehoe	16	2.7	negative
<p>1. Include requirements for structural observation in the standard.</p> <p><i>Persuasive</i> — See revisions.</p> <p>2. Replace “employ” with “engage the services of.”</p> <p><i>Editorial</i> — Accepted. See revisions.</p>			
Lundeen	16	2.7	negative
<p>1. Move the Quality Assurance Plan to the Commentary. Mandatory quality assurance is the responsibility of the code official and should be beyond the scope of this document.</p> <p><i>Non-persuasive</i> — The intent of the Prestandard is to specify that quality assurance shall be performed and who should perform it; and the PT judges this to be an important part of the document. The sections have been editorially clarified to avoid conflicting with code requirements.</p> <p>2. Remove all references to ASCE 7-98 from this section, as it is inconsistent with adopted building code requirements.</p> <p><i>Non-persuasive</i> — See Lundeen Comment 1, Item 16.</p> <p>3. Move reporting and compliance procedures to the commentary.</p> <p><i>Non-persuasive</i> — See Lundeen Comment 1, Item 16.</p>			

Trahern	16	2.7	negative
See Lundeen Comment 1, Item 16.			
McClure	17	2.8	negative
Global Issue A-12 regarding acceptance criteria for archaic materials has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.			
<i>Non-persuasive</i> —			
See response to McClure Comment on Ballot Item 6, Section 1.6.			
Kehoe	18	2.9	negative
Define “permanent live loads.”			
<i>Editorial</i> —			
See Kehoe comment 3, item 15.			

CHAPTER 3:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Kehoe	21	3.1	affirm w/ comment

Editorial —

Accepted. All scoping sections have been revised accordingly to refer to “Definitions.”

Kehoe	22	3.2	negative
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1. Including amplification of torsion in evaluation of torsional effects is inconsistent with Item 4, Section 3.2.2.2.2.

Persuasive —

Accepted with revised changes. See revisions.

2. Actual torsion should be considered for all structures.

Persuasive —

See revisions.

3. Consideration of accidental torsion should be keyed to actual torsion.

Persuasive —

See revisions.

4. Elements included in a mathematical model should be left to the discretion of the engineer and not keyed to primary or secondary classifications.

Non-persuasive —

Modeling requirements were set with the limitations of the analysis procedures in mind. Inclusion or exclusion of primary or secondary elements is intended to generally provide conservative results for overall response of the structure using the given procedure (i.e., exclusion of secondary in linear procedures will maximize force demands on primary elements; inclusion of secondary in nonlinear will maximize degradation effects).

5. Nonstructural components should also be included in linear and nonlinear dynamic analyses.

Persuasive —

See revisions.

6. In Section 3.2.4.2, replace “story” with “vertical lateral force resisting elements.”

Editorial —

Accepted. See revisions.

7. Diaphragm deflection should consider forces due to offsets in the vertical lateral force resisting system above the diaphragm.

Editorial —

Not accepted. The existing verbiage is judged appropriate.

8. Modeling of diaphragm flexibility should consider stiffness based on the structural characteristics of the diaphragm.

Persuasive —

See revisions.

9. Correct subscript “i” in Section 3.2.5.1.1.

Editorial —

Accepted. See revisions.

10. Specify Code Official for approving alternate methods.

Editorial —

Not accepted. See Kehoe Comment 3, Item 13.

11. Section 3.2.7.2 should refer to when vertical effects need to be considered.

Editorial —

Accepted with revised changes. See revisions.

12. Omit requirement to evaluate for post EQ residual gravity capacity in Section 3.2.9.

Non-persuasive —

Verification of design assumptions and post EQ gravity capacity are considered important aspects of the original guidelines. The section has been editorially revised and commentary added to clarify the intent.

13. GT 2-1 regarding overturning is recommended for basic research.

Non-persuasive —

See response to McClure Item 22, Section 3.2.10.

14. Clarify resistance to overturning provided by dead load.

Persuasive —

Accepted with revisions. See revisions.

15. Revise equations 3-5 and 3-6; omit C_2 ; increase R_{OT} for immediate occupancy.

Persuasive —

Accepted with revisions. See revisions.

Lundeen	22, 24	3.2, 3.4	affirm w/comment
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Editorial —

Accepted with revised changes. See revisions.

Johnson	22, 23	3.2, 3.3	affirm w/comment
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1. Replace QW with QL in Equation 3-3.

Editorial —

Accepted.

2. Clarify the extent to which Equations 3-5 and 3-6 should be applied to structural elements supported by foundations that rock.

Persuasive —

No change made. This issue has been addressed and should be considered further in GT 4-8 regarding rocking.

Malik	22	3.2	affirm w/comment
1.	Add a requirement in Section 3.2.2.1 for connections to be modeled if they have more strength but less ductility than the connected components.		
	<i>Non-persuasive</i> — If the connection is stronger than the connected parts, low ductility does not matter because ductility demands will be limited by the weaker connected elements yielding sooner.		
2.	Requirements in Section 3.2.2.3 to include or exclude secondary components in linear models are confusing and more difficult to apply than to simply model them. The decision on what to include should be left to the judgement of the engineer.		
	<i>Non-persuasive</i> — See Kehoe Comment 4, Item 22.		
3.	It is not sufficient to consider the fundamental period alone in deciding if SSI effects will result in an increase in spectral accelerations. Revise to consider an increase in period of significant modes (i.e., modal mass > 15%).		
	<i>Non-persuasive</i> — The fundamental period is appropriate and has been used as the basis for many analysis aspects of the Prestandard. Use of significant modes is an unnecessary complication.		
4.	It is difficult to explicitly model damping of individual footings, but conservative to ignore damping effects. In Section 3.2.6.2, the engineer should be allowed to ignore damping unless the effort to include it is deemed acceptable.		
	<i>Persuasive</i> — See revisions.		
5.	Ignoring vertical seismic forces in combination with horizontal forces is unconservative for overturning.		
	<i>Non-persuasive</i> — Multidirectional effects were considered in GT 3-4. The referenced Special Study 5 – Report on Multidirectional Effects and P-M Interaction on Columns concluded that vertical need not be combined with horizontal.		

McClure	22	3.2.2.2	affirm w/ comment
	Global Issues 3-22, 3-30 and 3-31 were identified as needing resolution, which is expected, but not yet developed. These issues should be resolved.		
	Accepted – The PT intends to develop resolutions during the third draft cycle.		

McClure	22	3.2.10	negative
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Section 3.2.10 does not properly address the FEMA 343 Technical Issue T-1 regarding overconservative treatment of overturning in linear procedures. Sample calculation provided.

Non-persuasive —

While FEMA 343 identifies overturning as an issue with regard to FEMA 273, inconsistencies between calculated and observed results for building response to seismic ground motion has been inherent in engineering practice since the inception of seismic design. FEMA 356 includes an alternative linear procedure for evaluation of overturning that is consistent with building codes for new construction. Thus FEMA 356 is no different than prevailing practice. For the Immediate Occupancy Performance Level, it was the consensus of the authors of the original overturning sidebar in FEMA 273, as well as the PT, that this higher performance level warranted the reduced displacements expected with higher levels of overturning stability. However, $R_{OT} = 1.0$ for IO was judged to be overconservative in comparison to current code. Considering requirements for new essential facilities, an R_{OT} of 4.0 was conservatively created as discussed in GT 2-23. The actual response of a given structure is a complicated nonlinear soil-structure interaction problem that is only approximated with linear analysis methods. It is considered acceptable practice to err on the side of conservatism when simplified procedures are used. When linear procedures are used and dead loads are not sufficient to resist calculated uplift forces, alternative solutions such as mobilizing adjacent columns or installation of pile foundations may be feasible. Within the context of the FEMA 356, more advanced analysis procedures are available that can be used to explicitly evaluate the effects of rocking and uplift to reduce this conservatism. For higher performance, this additional effort may be warranted.

McClure	22	3.2.2.3, 3.2.10	negative
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Global Issues 11-4 regarding effects of nonstructural on structural response and 2-1 regarding overturning have been classified as unresolved pending future research and should be resolved before development of the Prestandard document.

Non-persuasive —

See response to McClure comment on Ballot Item 6, Section 1.6.

Nicoletti	22	3.2.2.2.2	affirm w/comment
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See Kehoe comments 1, 2 and 3, Item 22.

Pappas	22, 23, 24	3.2, 3.3, 3.4	affirm w/comment
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Editorial —

Accepted. See revisions.

Turner	22	3.2	negative
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1. R_{OT} values in Section 3.2.9 appear to be arbitrarily based on current building codes.

Non-persuasive —

Values are entirely based on current building codes. The stated intent of the procedure is to provide an alternative that is consistent with current code.

2. The term “full lateral forces” is not defined.

Persuasive —

See revisions.

Breiholz	23	3.3	affirm w/comment
<i>Editorial</i> — Accepted. Figures will be legible in the final draft.			
Gould	23	3.3	affirm w/ comment
See Breiholz, Ballot Item 23.			
Hom	23	3.3	affirm w/comment
See Kehoe Comment 4, Item 23.			
Lundeen	23	3.3	negative
<p>1. Remove the reference to ASCE 7 in the definition of snow load for W in Equation 3-10, and replace with the text of the definition.</p> <p><i>Editorial</i> — Accepted with revised changes. Other codes (IBC) reference ASCE 7 for the calculation of snow loads, so the reference is retained here. See revisions.</p> <p>2. Delete the phrase “an approved” in Sections 3.3.1.3.4 and 3.3.3.2.3.</p> <p><i>Editorial</i> — Accepted. See revisions.</p> <p>3. Include J or omit C_1, C_2, C_3 in the denominator of Equation 3-13 to coordinate diaphragm requirements in Sections 6.11 and 8.5 with the calculated force level.</p> <p><i>Persuasive</i> — Suggested change accepted with revisions. Force- versus deformation-controlled nature of diaphragms and diaphragm components will be coordinated between Chapters 3, 5, 6, 7, and 8.</p> <p>4. Confirm that the approach in Section 3.3.1.3.5 produces similar results to that of the UCBC or FEMA 178.</p> <p><i>Non-persuasive</i> — The section was created as a result of Special Study 2 – Analysis of Special Procedure Issues to investigate the possibility of incorporating the UCBC Special Procedure into the Prestandard. The Special Procedure in its entirety was judged not applicable to the Prestandard in general, although certain concepts were considered appropriate for inclusion. The procedure in Section 3.3.1.3.5 is not intended to be equivalent to the Special Procedure, but is judged appropriate for general analysis of URM buildings.</p>			
Kehoe	23	3.3	negative
<p>1. The Method 1 calculation of period should permit the use of the Rayleigh Method.</p> <p><i>Persuasive</i> — See revisions.</p> <p>2. The Method 3 calculation of period should be simplified to $T=C_{td} (L)^{1/2}$ where L is the diaphragm span and C_{td} is a materials based coefficient.</p> <p><i>Persuasive</i> — No change made. This issue will be considered further as a new global issue.</p> <p>3. Use of the terms “actions” and “deformations” is redundant.</p>			

Editorial —

Accepted with revised changes. See revisions.

4. Omit C_2 factor from all sections.

Persuasive —

The C_2 factor was considered in GT 3-27 and set equal to 1.0 for linear procedures. At the 2/15/00 SC meeting, the committee voted to omit the C_2 factor. Recent research from SAC seems to support that C_2 can be eliminated. For nonlinear procedures the definition of C_2 has been revised to permit the use of $C_2 = 1.0$. Global Issue 3-33 was created to study this issue further.

5. Provide specific direction to explicitly model out-of-plane offsets in the vertical lateral force resisting system.

Editorial —

Accepted with revised changes. Direction added in Section 3.2.2.1. See revisions.

6. See Kehoe Comment 4, Item 23.
7. Provide guidance on how to account for crosswalls in the calculation of diaphragm deflection in Section 3.3.1.3.5.

Non-persuasive —

The special procedure is not applicable to the general analysis provisions of this document. There is no method of explicitly calculating the effect of crosswalls. Benefits of crosswalls, however, can be indirectly considered through increased damping permitted in Section 1.6.1.5.3.

8. Provide guidance on modeling stiffness in Section 3.3.2.2.1.

Editorial —

Accepted with revised changes. Direction added in Section 3.2.2.1. See revisions.

9. Pairs of earthquake ground motions for time history analyses should be consistent.

Editorial —

Accepted. See revisions.

10. See Kehoe Comment 4, Item 23.
11. Add alternative to calculate diaphragm forces using Equation 3-13 in Section 3.3.2.3.2.

Editorial —

Not accepted. The second sentence of this section already says that these forces should not be less than 85% of Equation 3-13.

12. Editorial comment on Section 3.3.3.2.1 accepted. See revisions.
13. Engineers should be permitted to determine which secondary elements should be included in the model.

Non-persuasive —

The use of secondary acceptance criteria for nonlinear analyses as specified in Section 3.4.3.2.1 requires that all components be modeled so that overall degradation of the structure can be captured and accounted for by the C_3 factor. An engineer always has the option to demonstrate that any particular secondary component would not significantly affect results and could therefore be ignored.

14. Add a new section on ground motion characterization to reference S_a for the NSP.

Editorial —

Not accepted. Ground motion characterization sections occur in the dynamic procedures (LDP and NDP), but not in the static procedures (LSP and NSP). The proposed section would not add any clarity.

15. Miscellaneous editorial comments on Section 3.3.3.2.2 accepted with revised changes. See revisions.

16. It may not be possible to balance areas above and below the pushover curve; requiring the bilinear curve to pass through the actual curve at the target may result in bilinear curves that do not closely resemble the actual behavior.

Non-persuasive —

The construction of the bilinear curve is somewhat subjective and approximate due to its graphical procedure. The concern will be partially addressed by the addition of “approximate” to qualify the balancing of areas. The referenced provisions were added to provide more uniformity in the construction of the curve. It was the opinion of the PT that it was important the idealized curve match the actual curve at the target displacement. The procedures have been tested and appear to work satisfactorily on actual building analyses.

17. Provide guidance on modeling stiffness in Section 3.3.3.2.5.

Editorial —

Accepted with revised changes. Guidance added to Section 3.2.2.1. See revisions.

18. See Kehoe Comment 3, Item 23.

19. See Kehoe Comment 4, Item 23.

20. Replace $1/C_0$ with effective modal mass in Equation 3-16.

Persuasive —

See revisions.

21. Editorial comments on Section 3.3.4.1 accepted. See revisions.

22. Recreate applicable portions of the referenced section in Section 3.3.4.2.1.

Editorial —

Not accepted. In the interest of brevity, the PT decided not recreate sections when a reference would suffice.

23. See Kehoe Comment 9, Item 23.

McClure	23	3.3	negative
Global Issues 3-18, 3-14, 3-13, 3-23, 3-1, 3-10, 3-6, 3-17, and 3-20 have been classified as unresolved pending future research and should be resolved before development of the Prestandard document.			

Non-persuasive —

See response to McClure comment on Ballot Item 6, Section 1.6.

McClure	23	3.3.1.2, 3.3.1.3	affirm w/ comment
Global Issues 3-32 and 3-29 were identified as needing resolution, which is expected, but not yet developed. These issues should be resolved.			

Accepted – The PT intends to develop resolutions during the third draft cycle.

Strand	23	3.3.1.2.2	negative
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The coefficient $C_t=0.018$ for empirical calculation of period for concrete moment frames appears to be too low, especially if cracked sections are considered.

Non-persuasive —

This value was installed as resolution to GT 3-3. It comes directly from the referenced Goel and Chopra research of measured concrete frame periods using strong motion records. Measured periods include the “real” (cracked or uncracked) condition of the components at the time of the earthquake. It is the opinion of the PT that this coefficient represents the most appropriate empirical estimate for concrete frames.

Kehoe	24	3.4	negative
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1. Equation 3-21 relating the J-factor to the spectral response coefficient is not appropriate.

Persuasive —

See response to McClure Item 24, Section 3.4.2

2. Section 3.4.2.2.3 provisions for prohibiting the formation of plastic hinges when using linear procedures is not required. Plastic hinging is not explicitly evaluated in linear procedures.

Editorial —

Accepted. See revisions.

McClure	24	3.4.2	negative
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There is no rational engineering basis for Equation 3-21 relating the J-factor to the spectral response coefficient S_{xs} . An alternate equation should be developed that is more rational.

Persuasive —

The relationship between the J-factor and S_{xs} , and the reason it was included in original FEMA 273, is described in Global Issue 3-26 and has been added in FEMA 356 as commentary. The PT concurs that the relation between the J-factor and S_{xs} is questionable. However, it is the opinion of the PT that the concept of a force reduction factor is appropriate, and a conservative formulation of it should remain in the Prestandard. The section has been revised to remove Equation 3-21 and replaces it with an emphasis on DCR values in the load path, which is more rational.

McClure	24	3.4.2	negative
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Global Issue 3-19 regarding gravity load capacity has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.

Non-persuasive —

See response to McClure comment on Ballot Item 6, Section 1.6.

CHAPTER 4:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Paruvakat	29, 31	4.2, 4.4	affirm w/comment
<p>1. Geosynthetics should be included in Section 4.2.1.1.1.</p> <p><i>Editorial</i> — Comment withdrawn. Information covered in Item 3 of that section.</p> <p>2. Specify how the “foundation area” is to be defined in the case of deep foundations in Section 4.2.1.1.2.</p> <p><i>Persuasive</i> — See revisions.</p> <p>3. Replace “soil shear strength” with “soil cohesion” in Section 4.2.1.1.2.</p> <p><i>Persuasive</i> — See revisions.</p> <p>4. Commentary C4.2.2.2 on evaluating increased lateral earth pressures on retaining walls due to liquefaction is too simplified.</p> <p><i>Persuasive</i> — See revisions.</p> <p>5. The term “geologic materials” in Section 4.2.2.3, Item 2 should be replaced with “geologic deposits.”</p> <p><i>Persuasive</i> — See revisions.</p> <p>6. Geotechnical reports usually include a larger factor of safety than the 1.5 to 2.0 implied by Equations 4-1 and 4-2 in Section 4.4.1.2. Using lower than actual strength in NDP models will overestimate material damping and underestimate demands on the structure.</p> <p><i>Persuasive</i> — See revisions. Increase factors to 3.0 and reduce m-factors in Section 4.4.3.2.1 for fixed base foundation from 4 to 3.</p>			
Basu	31, 34	4.4, 4.7	affirm w/ comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			

Lundeen	31	4.4	negative
<p>Much of this section is textbook type information. The scope of the Prestandard needs to be more consistent from chapter to chapter.</p> <p><i>Non-persuasive</i> — Much of the information contained in this section has been studied and refined as a result of Special Study 4 – Foundation Issues. It is the opinion of the PT that this type of information is very relevant to the scope of the document, and that the level of detail is appropriate. Section 4.4 can be viewed as analogous to Section 6.5.2 (and other material sections) because it outlines strength, stiffness, and acceptance criteria for a system (R/C moment frames, for example). In the case of Section 4.4, the system is the foundation.</p>			
McClure	31	4.4	affirm w/ comment
<p>Global Issue 4-8 regarding rocking behavior was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Pappas	31	4.4	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			
Breiholz	32	4.5	affirm w/comment
<p>Application of seismic earth pressures of Equation 4-11 is too conservative. Justify a reduced pressure, or eliminate it.</p> <p><i>Persuasive</i> — Accepted with revised changes. Equation 4-11 is intended to be a conservative simplification of research which demonstrates these pressures exist. Reference has been added to site-specific geotechnical investigation to obtain seismic pressures in lieu this equation. While observed damage may be rare, there are circumstances (listed in the commentary) where it would be appropriate to rehabilitate a building wall for seismic earth pressures. Therefore the PT has decided to retain the requirement. The commentary has been expanded to clarify that these earth pressures are intended to check local acceptability of wall components, and should not be used to increase the overall base shear on a building.</p>			
Johnson	32	4.5	affirm w/comment
<p><i>Editorial</i> — Not accepted. In the judgment of the PT, consideration of lateral pressures on the uphill side of a building on a sloping site is a matter of engineering practice and should be considered by the engineer in the application of the procedures of the Prestandard.</p>			
Paruvakat	32	4.5	negative
<p>1. The title of Section 4.5 is misleading with regard to buildings and should be revised to Earth Pressure on Building Walls.</p> <p><i>Editorial</i> — Accepted.</p>			

2. The use of uniform pressure on basement walls, which are most likely fixed at both ends, is unconservative. Actual pressures are closer to parabolic.

Persuasive —

No change made. The PT studied this issue using references provided by Paruvakat. While the research shows that the distribution is approximately parabolic, the resulting change in total demands on the wall is very small (within 8% for cases studied). It is the opinion of the PT that the existing uniform pressure be retained for simplicity. Commentary C4.5 has been revised to state the complexity of the pressure distribution.

3. “Mononobe” is misspelled.

Editorial —

Accepted.

CHAPTER 5:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
McClure	n/a	Ch's 5, 6, 7, 8	negative
<p>Global Issue A-6 regarding behavior of rehabilitated elements has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
McClure	40	5.4-5.9	affirm w/ comment
<p>Global Issue 5-11 regarding expected strength of anchor bolts was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Pappas	41	5.5	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			
McClure	42	5.6	affirm w/ comment
<p>Global Issue 5-12 regarding braced frame connection requirements was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Misovec	43	5.7	affirm w/comment
<p>Provide direction on how to consider stiffened wall plates.</p> <p><i>Persuasive</i> — The provisions of the Prestandard consider that the plates are sufficiently stiffened to prevent buckling of the plates. A reference has been added to the commentary to refer to further information on the design of steel plate shear walls.</p>			
McClure	44	5.8.X.3	negative
<p>Global Issue 5-1 regarding conservative m-factors has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
Nicoletti	44, 45	C5.8, C5.9.4.5	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			

CHAPTER 6:

Author	Item	Section	Vote
McClure	n/a	Ch. 6	affirm w/ comment
<p>Global Issue 6-14 regarding guidance for lightweight concrete was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
McClure	n/a	6.5-6.13	negative
<p>Global Issue 6-1 regarding conservative m-factors has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
Fantozzi	53, 66	6.3, 6.16	affirm w/comment
<p>Add reference to ACI 437</p> <p><i>Persuasive</i> — Reference will be added during the third draft cycle.</p>			
Johnson	53, 58	6.3, 6.8	affirm w/comment
<p>1. <i>Editorial</i> — Accepted. See revisions.</p> <p>2. Shear stiffness for rectangular sections should permit use of $A_w=5/6A_g$ in Section 6.3.2.2.</p> <p><i>Non-persuasive</i> — Effective stiffness values are provided in Table 6-5.</p>			
McClure	53	6.3.2.4.4	affirm w/ comment
<p>Global Issue 6-19 regarding sampling of prestressing steel was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Pappas	53, 54, 55, 57	6.3, 6.4, 6.5, 6.7	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			
McClure	54	6.4	negative
<p>Global Issues 6-17 regarding concrete columns in tension and 6-20 regarding concrete flange provisions have been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			

Iqbal	55	6.5.3.1	negative
<p>Average prestress limited to 350 psi on the cross section is too low and should be raised to 700 psi as in the 1994 NEHRP Provisions.</p> <p><i>Persuasive</i> — See revisions.</p>			
McClure	58	6.8.2	affirm w/ comment
<p>Global Issues 6-6 regarding shear wall component definitions and 6-18 regarding shear wall yield moment were identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Nicoletti	59	C6.9.1.3	affirm w/comment
<p>Shear in tilt-up panels should be deformation-controlled and connections should be force-controlled.</p> <p><i>Persuasive</i> — Commentary will be revised to be consistent with acceptance criteria specified in Section 6.9.2.4, which references Section 6.8.2.4 for monolithic shear walls, and specifies shear and flexure as deformation controlled actions.</p>			
McClure	61	6.11	affirm w/ comment
<p>Global Issue 6-16 regarding diaphragm m-factors was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			

CHAPTER 7:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
McClure	n/a	Ch. 7	affirm w/ comment
<p>Global Issue 7-6 regarding use of 1.25 fy for masonry was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Kariotis	69	7.3.2.6	Negative
<p>Revise the definition of v_{te} in Equation 7-1 from average bed-joint shear strength to the second decile of test values obtained in accordance with Equation 7-2.</p> <p><i>Persuasive</i> — No change made. The calculation of V_{me} is intended to be an expected strength. While the comment makes an important point about test variability, for consistency with the rest of the Prestandard, the definition of V_{te} has been left as average shear strength used for the calculation of V_{me}. New global issue 7-10 has been created for further consideration of this issue.</p>			
Kehoe	69	7.3	negative
<ol style="list-style-type: none"> Editorial comment on Section 7.3.1 accepted. See revisions. Reference ASTM standards for testing the strength and modulus of masonry. <p><i>Persuasive</i> — See revisions.</p> <ol style="list-style-type: none"> For determining elastic modulus in Section 7.3.2.4, reference the same ASTM standard used for prism testing in Section 7.3.2.3. <p><i>Non-persuasive</i> — The referenced ASTM standard test procedure does not apply to determining elastic modulus.</p> <ol style="list-style-type: none"> Revise the title of Section C7.3.3.2.4 Radiography, which does not match the contents. <p><i>Editorial</i> — Accepted with revised changes. The subject of the section is intended to be radiographic (x-ray) devices. The content has been clarified.</p>			
Kehoe	70	7.4	negative
<p>Create a classification of partially reinforced walls to address walls with less reinforcement than minimum specified for reinforced walls. Walls with some reinforcement cannot rock and should not be treated as unreinforced masonry.</p> <p><i>Persuasive</i> — Revised changes. Existing Table 7-6 contains acceptance criteria for walls with reinforcement ratios as low as .0002 depending on material properties. The definition of reinforced masonry in Section 7.8 has been revised to remove limits on reinforcing ratios.</p>			

McClure	70, 71, 73	7.4, 7.5, 7.7	negative
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Global Issues 7-1 regarding conservative m-factors and 7-4 regarding guidance on infill panels has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.

Non-persuasive —

See response to McClure comment on Ballot Item 6, Section 1.6.

Paruvakat	73	7.7	affirm w/comment
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Editorial —

Accepted. See revisions.

Kehoe	74	7.8	negative
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See Kehoe comment Ballot Item 70, Section 7.4.

Kehoe	76	7.10	negative
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1. Provide applicable year for referenced codes and standards.

Editorial —

Accepted. This change will be incorporated throughout the document in the 3rd draft cycle.

2. Include references for ASTM standard test procedures for compressive strength and modulus.

Editorial —

Accepted. See revisions.

CHAPTER 8:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
McClure	n/a	Ch. 8	negative
<p>Global Issue 8-3 regarding wood values based on judgement has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
Fantozzi	79	8.3.2.5	affirm w/comment
<p>A yield capacity of 120 plf for single straight sheathed diaphragms appears too low in comparison with the 1997 UCBC allowable value of 100 plf.</p> <p><i>Persuasive</i> — No change made. The PT is investigating the source of the value and will resolve during the 3rd draft cycle.</p>			
Johnson	80	8.4	affirm w/comment
<p>Clarify how to convert ASD capacity of proprietary hardware connectors (i.e., hold-downs) to yield (expected) capacity. Provide values in Table 8-3 Connections.</p> <p><i>Persuasive</i> — Revised changes. Because factors of safety on allowable values can vary between manufacturers, Q_{CE} will be defined based on average ultimate test values provided by manufacturers.</p>			
McClure	80	Table 8-1	negative
<p>Global Issue 8-1 regarding conservative m-factors has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
McClure	80	8.4	affirm w/ comment
<p>Global Issue 8-8 regarding guidance for wood posts was identified as needing resolution, which is expected, but not yet developed. This issue should be resolved.</p> <p>Accepted – The PT intends to develop resolutions during the third draft cycle.</p>			
Nicoletti	80, 82	C8.4.3.1, C8.6.1.1	affirm w/comment
<p><i>Editorial</i> — Accepted. Section 8.6 has been editorially revised to provide missing information. See revisions.</p>			

CHAPTER 9:

Author	Item	Section	Vote
McClure		Ch. 9	negative
<p>Global Issue 9-4 regarding Chapter 9 controls has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
Lundeen	86	9.1	negative
<p>Much of Chapter 9 is textbook type information. The scope of the standard needs to be more consistent from chapter to chapter.</p> <p><i>Persuasive</i> — No change made. This issue is partially covered by GT 9-4 and was raised by the Project Advisory Committee. The PT judges that while Chapter 9 is very long, the information is useful and relevant to performing analyses using isolation or energy dissipation techniques. The PT lacks sufficient information to reduce the content of Chapter 9 at this time.</p>			
McClure	88	9.3	negative
<p>Global Issue 9-1 regarding validation of procedures has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			

CHAPTER 10:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Hom	93, 95	10.1, 10.3	affirm w/comment
<p>1. Section 10.1 should be a continuation of the requirement to perform a seismic evaluation prior to rehabilitation.</p> <p><i>Non-persuasive</i> — This information is not appropriate in Section 10.1, which is a scoping section. Section 10.2 already explicitly states that a FEMA 310 evaluation must be performed.</p> <p>2. Reorganize Table C10-20 by the sequence in FEMA 310 rather than FEMA 178. Omit the FEMA 178 column from the table.</p> <p><i>Persuasive</i> — See revisions.</p>			
Pappas	95	C10.3.1.3	affirm w/comment
<p><i>Editorial</i> — Accepted. See revisions.</p>			

CHAPTER 11:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Hess	99	11.1	negative

Chapter 11 has been so modified from what was produced by the ATC 33 subcommittee as to be unrecognizable. A standing subcommittee should be formed within the ASCE Standards committee to develop this chapter in tandem with FEMA 310.

Non-persuasive —

The PT does not consider the chapter unrecognizable, however, it does not disagree with the idea of forming committees for future improvement of the document.

Hattis	100	11.2	negative
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Add a row to Table 11-1 in Item 2 Partitions to reflect Section 11.9.2.1.1 on glazed partitions.

Persuasive —

See revisions.

Hess	100	11.2	negative
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Add requirement to identify a list of nonstructural components to be considered by performing an initial evaluation using FEMA 310 or FEMA 178 checklists.

Non-persuasive —

Prior evaluation is now covered in Section 1.2. See response to Hom comment, Item 2.

Kehoe	100	11.2	negative
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1. Include a building walkthrough and condition assessment of nonstructural components in the procedure list of Section 11.2

Editorial —

Accepted. See revisions.

2. Requirements for performance levels other than LS and IO need to be added to Table 11-1.

Non-persuasive —

This issue was addressed and resolved in GT 11-7. Operational Performance is not addressed by this document and Hazards Reduced Performance is evaluated using LS criteria, for a subset of components (falling hazards) identified in Section 1.5.2.4. This issue was discussed at the 8/23/00 Standards Committee meeting. Further study of this issue is recommended.

3. Provide a 10 psf weight limit for classifying heavy versus light partitions.

Non-persuasive —

This issue was addressed and resolved in GT 11-9. Original FEMA 273 included a 5 psf weight limit, but this value was too low and inconsistent with the original notion that masonry partitions are “heavy.” Heavy and light partitions are defined in Section 11.9.2.1.

4. Define what is meant by applied ceilings.

Non-persuasive —

Applied ceilings are defined in Section 11.9.4.1, category a.

5. Combine canopies and marquees with parapets and appendages in Table 11-1.

Non-persuasive —

The requirements are not identical. Canopies and marquees must be designed for vertical acceleration as specified in Section 11.9.6.3.1.

6. Combine vibration isolated equipment and non-vibration isolated equipment in Table 11-1.

Non-persuasive —

The requirements are not identical. Values for the component amplification factor a_p in Table 11-2 are different depending on vibration isolation.

7. Define what constitutes a “type” of nonstructural component.

Non-persuasive —

The PT considers the term “type” to be self-explanatory in this context.

8. Specify what constitutes a deviation in samples.

Non-persuasive —

The PT considers the term “deviation” to be self-explanatory in this context.

Hess	101	11.3	negative
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Add requirement to identify nonstructural components that are at risk by performing an initial evaluation using FEMA 310.

Non-persuasive —

See response to Hess comment, Item 100.

Kehoe	101	11.3	negative
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1. The content of Section 11.3.1 Historical and Component Evaluation Considerations does not match the title.

Editorial —

Not accepted. The commentary contains considerable historical information.

2. Commentary Section C11.3.1 is unnecessarily long.

Editorial —

Accepted. The content will be edited during the 3rd draft cycle.

3. The criteria for LS and Hazards Reduced Performance are the same. There should be a distinction.

Non-persuasive —

This issue was partially addressed by GT 11-7. While the criteria is the same, there is a difference between LS and HR in that only a certain subset of components (falling hazards) are addressed for the HR defined in 1.5.2.4.

4. Provide guidance on where to find acceptance criteria for Operational Performance and who approves it.

Non-persuasive —

Operational performance is outside the current scope of the document. The intent is for other resources to be used to work in cooperation with the local jurisdiction to establish criteria and obtain approval for a specific rehabilitation project. This issue will be considered further as a new global issue.

Hess	102	11.4	negative
<p>This section should be expanded back to what it was in ATC33. Performance of nonstructural is not always parallel to that of structural. This section should spell out applicable criteria for different elements.</p> <p><i>Non-persuasive</i> — Information from original ATC 33 Section 11.4 is redundant with, and included in Section 1.5.</p>			
Kehoe	102	11.4	negative
<p>Delete Section 11.4, which provides no other function than to refer to Section 1.4.</p> <p><i>Editorial</i> — Not accepted. The section is judged to have value in that it emphasizes the need to select a nonstructural goal as part of the Rehabilitation Objective, which determines how the rest of the chapter is to be used.</p>			
Hess	103	11.5	negative
<p>Provide guidance on how different categories of nonstructural elements affect structural response.</p> <p><i>Persuasive</i> — No change made. This issue was identified in GT 11-4 and recommended for basic research.</p>			
Kehoe	103	11.5	negative
<p>Provide commentary that discusses ways in which nonstructural components may affect structural response.</p> <p><i>Persuasive</i> — No change made. See response to Hess comments on Ballot Item 103.</p>			
McClure	103	11.5.1	negative
<p>Global Issue 11-4 regarding effects of nonstructural on structural has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>			
McClure	103	11.5	negative
<p>Seconds Kehoe comments on this ballot item.</p> <p>See response to Kehoe comments on this ballot item.</p>			
Hess	104	11.6	negative
<p>The content of this section should state when the existing element is acceptable and how to evaluate it to be consistent with the title.</p> <p><i>Persuasive</i> — See Kehoe Comment 1, Item 104. Acceptability and evaluation criteria are spelled out in Sections 11.9, 11.10, and 11.11.</p>			

Kehoe	104	11.6	negative
			<p>1. The content of Section 11.6 does not match the title, and is redundant with information in Section 11.7.</p> <p><i>Editorial</i> — Accepted with revised changes. See revisions.</p> <p>2. Provide definitions of rigid and flexibly mounted equipment within the text in addition to the definitions in Section 11.12. Provide a reference to the Tri-Services Manual for evaluating IO performance of flexible nonstructural components.</p> <p><i>Editorial</i> — Not accepted. The definitions are judged acceptable. Currently amplification of forces for flexible mounted equipment is addressed by coefficients in Table 11-2. Section 11.7.6 permits the use of other methods.</p>
McClure	104	11.6	negative
			<p>Global Issue 11-5 regarding sensitivity of nonstructural to deformation has been classified as unresolved pending future research and should be resolved before development of the Prestandard document.</p> <p><i>Non-persuasive</i> — See response to McClure comment on Ballot Item 6, Section 1.6.</p>
McClure	104	11.6	negative
			<p>Seconds Kehoe comments on this ballot item.</p> <p>See response to Kehoe comments on this ballot item.</p>
Hattis	105	11.7	negative
			<p>Add a row to Table 11-2 in Item 2 Partitions to reflect Section 11.9.2.1.1 on glazed partitions.</p> <p><i>Persuasive</i> — See revisions.</p>
Hess	105	11.7	negative
			<p>Sections 11.6 and 11.7 are redundant and should be combined.</p> <p><i>Persuasive</i> — See Kehoe Comment 1, Item 104.</p>
Kehoe	105	11.7	negative
			<p>1. Provide guidance on approved codes for prescriptive procedures and specify the Code Official as the approving authority.</p> <p><i>Editorial</i> — Not accepted. C11.7.2 provides guidance. The Code Official is the default approving authority and need not be specified.</p>

2. Equations for F_p in Section 11.7.3 are based on 97 NEHRP and 97 UBC, but differ in coefficients selected. Research does not justify the inverted triangular distribution over height. This is more than needed for LS performance, but not sufficient for IO performance.

Persuasive —

No change made. This issue was considered in GT 11-8. The PT decided to remain consistent with NEHRP and UBC provisions. The classification of GT 11-8 has been revised and this issue should be considered further in relation to further study of available information.

3. To resolve Comment 2, replace Section 11.7.4 and general force equations with a new proposed section and equations based on research published by Kehoe and Freeman.

Persuasive —

See Kehoe Comment 2, Item 105.

4. Clarify the application of vertical seismic forces in conjunction with horizontal seismic forces on nonstructural components. Require consideration of vertical effects for components supported on cantilevers. Revise the $2/3$ factor used to estimate vertical seismic forces.

Persuasive —

See revisions to coordinate between Sections 2.6.11, 3.2.7.2, 3.4.2, 11.7.3, 11.7.4 and acceptance criteria for nonstructural components specified in 11.9. Section 2.6.11 specifies consideration of vertical forces on cantilevers. Vertical forces on nonstructural components need only be considered where specifically required in 11.9 (currently this is just 11.9.6 canopies and marquees). The proposed revision to the $2/3$ factor for vertical seismic forces was not incorporated, but should be considered further as new GT 2-25.

5. Editorial comment on C11.7.6 accepted. See revisions.

McClure	105	11.7	negative
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Seconds Kehoe comments on this ballot item.

See response to Kehoe comments on this ballot item.

Hess	106	11.8	negative
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This section on rehabilitation methods was the core of ATC 33 and has been reduced to one sentence.

Non-persuasive —

It was the decision of the PT that the standard would not specify specific methods of rehabilitation.

This was intended to allow the design professional the flexibility to use creative methods, or new methods not known at the time of publication, to accomplish the rehabilitation objective.

Rehabilitation methods that were present in the original ATC 33 publication have been retained in the commentary for reference. This same concept was applied to rehabilitation methods for structural components in Chapters 4 through 8.

Kehoe	106	11.8	negative
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Specify the Code Official as the approving authority.

Editorial —

Not accepted. The Code Official is the implied approving authority on all issues and need not be specified.

McClure	106	11.8	negative
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Seconds Kehoe comments on this ballot item.

See response to Kehoe comments on this ballot item.

Hattis	107	11.9	negative
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1. Revise commentary C11.9.1.5.1 to be consistent with current industry terminology.

Persuasive —

See revisions.

2. Add phrase to commentary C11.9.1.5.2 to cover revisions to the acceptance criteria.

Persuasive —

See revisions.

3. Update the reference to AAMA test method in C11.9.1.5.3 and C11.9.1.5.4 to AAMA 501.4-2000.

Persuasive —

See revisions.

4. Revise the acceptance criteria in Section 11.9.1.5.3 to be consistent with the latest changes to the NEHRP Provisions (proposal 8-16(2000), which is accepted).

Persuasive —

See revisions.

5. Revise Commentary C11.9.1.5.3 to be consistent with revised Equation 11-9.

Persuasive —

See revisions.

6. Revise the evaluation requirements of 11.9.1.5.4 for consistency with the revised acceptance criteria.

Persuasive —

See revisions.

Kehoe	107	11.9	negative
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1. Revise the classification of adhered veneer to either acceleration sensitive or deformation sensitive and describe when each situation applies.

Non-persuasive —

Classification as deformation sensitive requires both a force-based analysis and deformation analysis. Calculation of forces will satisfy the concern over the attachment. Proper calculation of deformation imposed by the structure will require the engineer to consider the backing and interconnection of the backing with the structure. If the system will result in no deformations in the veneer, the criteria is satisfied.

2. Remove thickness limitations on anchored veneer in 11.9.1.2. Explicitly list terra cotta as anchored veneer.

Non-persuasive —

The specified thickness are intended to specify when the masonry is considered veneer, not when it needs to be anchored. Material in excess of those thicknesses does not qualify as veneer and is not covered by this section.

3. Acceptance criteria for LS and IO performance are the same. Use of 11.7.3 force equations for LS can be more severe than 11.7.4 equations used for IO because 11.7.3 equations are upper bound. IO requirements should be more stringent than LS requirements.

Non-persuasive —

The requirements for LS and IO are not identical. For deformation sensitive components, IO deformation limits are more stringent (see 11.9.1.3.3 for example). It is true, however, that use of 11.7.3 force equations can be more stringent than 11.7.4. This issue should be considered further as a new GT.

4. Prescriptive requirements should not be permitted for the IO Performance Level.

Non-persuasive —

The PT does not have technical justification for changing the criteria from that contained in original FEMA 273 at this time.

McClure	107	11.9	negative
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Seconds Kehoe comments on this ballot item.

See response to Kehoe comments on this ballot item.

Kehoe	108	11.10	negative
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1. See Kehoe Comment 3, Ballot Item 107.
2. See Kehoe Comment 4, Ballot Item 107.
3. NFPA 13 is for fire suppression piping and should not be used for other types of piping.

Persuasive —

See revisions.

4. Editorial comment on Section 11.10.5.3.1 is accepted. See revisions.
5. Specify a method for evaluating pipes at seismic joints in Section 11.10.5.4.

Non-persuasive —

Section 11.7.5 provides direction on how to consider relative movements at seismic joints.

McClure	108	11.10	negative
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Seconds Kehoe comments on this ballot item.

See response to Kehoe comments on this ballot item.

Kehoe	109	11.11	negative
--------------	------------	--------------	-----------------

1. See Kehoe Comment 3, Ballot Item 107.
2. See Kehoe Comment 4, Ballot Item 107.
3. Lateral forces on storage racks in Section 11.11.1.3 should be treated like non-building structures similar to the 1997 UBC.

Persuasive —

No change made. This issue should be considered further as a new GT.

4. Section 11.11.1.4 Evaluation Requirements should provide guidance on how to consider the items listed, or should be deleted.

Non-persuasive —

The verbiage satisfies the intent, which is to direct the engineer on what to consider. How the items are considered is left to the discretion of the engineer and the code official.

5. Hydraulic elevators are not as susceptible to damage as traction elevators. Less than 4-stories tall need not be considered for LS or IO performance.

Non-persuasive —

The PT lacks technical justification to relax the criteria at this time.

McClure	109	11.11	negative
Seconds Kehoe comments on this ballot item.			
See response to Kehoe comments on this ballot item.			

Hess	112	11.14	affirm w/comment
<i>Editorial</i> —			
Accepted. See revisions.			

APPENDIX A:

<i>Author</i>	<i>Item</i>	<i>Section</i>	<i>Vote</i>
Breiholz	114	A.2.1	affirm w/comment
<i>Editorial</i> —			
Accepted. See revisions.			

M. Minority Opinion Report

Minority Opinion Report

At the 3rd meeting of the ASCE Standards Committee on Seismic Rehabilitation held in San Francisco on August 23 and 24, 2000, the 3rd SC Draft of the FEMA 356 *Prestandard for the Seismic Rehabilitation of Buildings* was unanimously accepted for ballot by those in attendance. That acceptance was conditional upon incorporation of revisions discussed at the meeting and the completion of further study on selected portions of the document as directed by the committee. That work has been completed and incorporated into the Prestandard. The results of these further studies are reported in Appendices N through Q of this Global Topics Report.

In spite of this unanimous approval, certain issues remained important to a minority of committee members, even after committee deliberations. At that meeting, it was agreed that the ASCE/FEMA 273 Prestandard Project Team would receive and publish minority opinions from standards committee members in a Minority Opinion Report. This report was to be included as an appendix to the Global Topics Report.

The following opinions have been submitted by individual members of the ASCE Standards Committee on Seismic Rehabilitation. The opinions expressed are those of the individual, and do not necessarily reflect the opinions of the ASCE/FEMA 273 Prestandard Project Team, or the standards committee as a whole.

Minority Opinion**Submitted by Frank E. McClure****FEMA 356****Section 3.2.10, Overturning, Section 3.2.10.1, Linear Procedures**

FEMA 356, July 21, 2000, Section 3.2.10.1 does not provide clear and unambiguous guidance to address the BSSC Case Studies Report, FEMA 343, Section 6.2, Technical Adequacy, Issue T-1 concerning the treatment of overturning in 1997 FEMA 273, predecessor to FEMA 356.

FEMA 356, Section 3.2.10.1 has been revised to include a new Equation (3-6) to reduce the conservatism concerning the overturning checks in FEMA 356. However, this revision does not address the issue raised in FEMA 343, Section 6.2, Technical Adequacy, Issue T-1 which states:

"This modification should result in overturning demands that are consistent with current codes for new constructions, but it does not address the resulting inconsistency in demand forces above the foundation interface and those reduced forces below it."

Another issue with FEMA 356, Section 3.2.10.1 is the statement: "Alternatively, the load combination represented by Equation (3-6) shall be permitted for evaluating the adequacy of dead loads alone to resist the overturning."

Does this above wording mean that Equation (3-6) can be applied when calculating the overturning effects that result from the application of the "Pseudo Lateral Loads", Equation (3-10) to the structural components or elements above the foundation-soil interface, at the superstructure to top of foundation connection and/or to the elements or components anywhere in the superstructure? An example would be to check the adequacy of a partial-penetration butt weld in a splice in a structural steel column in the superstructure.

1997 FEMA 273, Section 3.3.1.3, states: "This load, the pseudo lateral load, when distributed over the height of the linear-elastic model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event."

If the overturning moment, M_{ot} , is reduced by the application of the factor, R_{ot} , anywhere in the superstructure and the superstructure elements or components are checked or designed for the resulting reduced displacements (forces), will there be a fully developed and adequate load path with all the elements and components being capable of developing the "Pseudo Lateral Load" calculated displacement (loads) during the design event?

FEMA 356, Section 3.2.10.1 wording should be revised to clarify that the use of Equation (3-6) should only apply at the foundation-soil interface.

End of Minority Opinion

Minority Opinion

Submitted by Frank E. McClure

FEMA 356

Section 3.3.1.3.1, Pseudo Lateral Load

FEMA 356, July 21, 2000, does not provide clear and unambiguous guidance on how to calculate the vertical and horizontal forces acting on the connections at the bottom of the superstructure to the top of the foundation.

FEMA 356, Section 3.3.1.3.1, Pseudo Lateral Load states: "The pseudo lateral load in a given horizontal direction of a building shall be determined using Equation (3-10). This load shall be used to design the vertical elements of the lateral force-resisting system."

Consider a one bay three story concentric structural steel braced frame using chevron diagonal bracing. FEMA 356 does not provide guidance on how to calculate the vertical and horizontal forces acting on the steel base plates or other anchorage systems at the first story intersections of the structural steel columns and diagonal chevron bracing members at the top of the foundation system.

FEMA 356, Section 3.2.10.1, Linear Procedures provides guidance on how to calculate the vertical forces acting on the connections at the bottom of the superstructure, but does not provide guidance on how to calculate the horizontal forces acting on the above described connections at the bottom of the superstructure.

FEMA 356, Equation (3-5) reduces the overturning moment, M_{ot} , by a factor, $C_1 * C_2 * C_3 * J$, when calculating the vertical tension and compression forces to check the adequacy of the stabilizing effects of dead loads. The resultant vertical forces acting on the base plates or other anchorage systems must be combined with the horizontal forces resulting from the Pseudo Lateral Load, calculated using Equation (3-10). Should these horizontal Pseudo Lateral Loads be reduced by the same factor, $C_1 * C_2 * C_3 * J$, which is used to reduce the overturning moment, M_{ot} , when combined with the vertical forces acting on the base plates or other anchorage systems?

I do not recommend that Equation (3-6) be used to calculate the forces acting at the base of the superstructure connection to the top of the foundation system, but only be used to calculate the vertical forces at the foundation-soil interface. To use Equation (3-6) in calculating the vertical forces in the components or elements above the foundation-soil interface in the superstructure would allow "weak links" in the complete and adequate load path to be accepted and/or designed because of the large reduction of forces due to the application of a large R_{ot} to the overturning moment, M_{ot} .

However, if the final FEMA 356 permits the use of Equation (3-6) in the superstructure and at the base of the superstructure connections, then a similar question could be asked. Should the Pseudo Lateral Load be reduced by the factor, $C_1 * C_2 * C_3 * R_{ot}$, to calculate the horizontal shearing force acting on the base plates or other anchorage systems?

FEMA 356 should be revised to answer the above questions to provide proper guidance on how to calculate the vertical and horizontal forces at the connections of the superstructure to the top of the foundation.

End of Minority Opinion

Minority Opinion
Submitted by R. McConnell
FEMA 356
Section 1.6, Seismic Hazard

As demonstrated in charts provided the Committee, there are serious problems regarding extreme increases in the seismic force values required by this document for some areas of the nation, particularly if we incorporate the use of the USGS MCE maps as now prescribed.

The MCE levels do not appear acceptable for practice in the areas of concern. One example is the area of Champion, MO where the “design level” (2/3 time the MCE value) is approximately six times the USGS probabilistic level of 10% exceedance in 50 years. That “design level” also happens to be 43% higher than the highest requirement in California. What “hard” justification is there for this?

This document, FEMA 356, makes matters worse by its requirement for the use of the full value of the site MCE for its “BSE-2”.

One simple alternative to limit compounding the extreme levels is to modify the present BSE-2 definition by requiring that the full MCE level be used only to the point where it is 50% higher than the 10% at 50 year level. Beyond that, the two-thirds value would be used. There are still “troubles ahead”, but this would help somewhat.

For those interested in pursuing this in more detail, they may obtain copies of two disks from BSSC: the USGS “Design Parameters” by E.V.Leyendecker (MCE, etc. levels at any U.S. coordinates); and a disk containing the MCE and 10%/50 year values for over 164,000 populated sites in the U.S. The latter also contains map ratios and charts for comparison study.

I urge adequate review of these items.

End of Minority Opinion

Minority Opinion
Submitted by R. McConnell
FEMA 356

Section: General, Preface

In my opinion, acceptance of FEMA 356 will be difficult, and lacks simply presented, but sufficiently detailed, justification. Also, some believe that there may not have been adequate concern at the outset for writing this document to get equivalent results requiring minimal effort for transition.

Several years ago, in the first of ASCE meetings on this project, I presented a similar method for multiple materials limit analysis for seismic resistance that used “R” values as presently used in the major codes. I still maintain that such transition consistency of various definitions and procedures could have been a simpler and adequate route.

Added to concerns for lack of simpler transition and detailed justification for changes, the case-studies report, FEMA 343, is insufficient for review and comparisons. I could not check various procedures and comparisons with prior codes using the limited information presented in 343 or available to me through BSSC. It is my understanding that all three “studies” that had two firms, doing independent efforts on a single structure, resulted in significant differences by each pair. No surprise. (Are we being “possessed by procedures”?) I looked into the Memphis case to the extent possible, and feel that it needs more study. (It is significantly important due to the concern for the MCE map levels in that area.) It would have helped considerably to have had the traditional “coupon”, or “schematic”, samples of types; and/or sample calculations for each case. Also, the MCE design values will compound any comparisons’ wide variations across the country.

Use of this document is going to be considerably difficult for all who have not been directly involved in its preparation. The goal is more proficiency and accuracy in analysis. This document is likely to be more vulnerable to error in its implementation.

End of Minority Opinion

N. Special Study 10— Issues Related to Chapter 7



Memo

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To: Jon Heintz
From: Mike Mehrain
Date: November 20, 2000
Subject: Issues Related to Chapter 7, FEMA 356

Dear Jon,

Here is a summary of my understanding of the various issues regarding Chapter 7 resulting from the Third SC Meeting. Following my suggested changes, review by Dan Shapiro and discussions with Dan Abrams, the issues were discussed in our project team meeting on November 17, 2000. The decision of the project team is indicated below. Section numbers refer to the third SC draft version of FEMA 356.

1. (Section C.7.1) FEMA 356 needs to replace LSP with the “Special Procedure” included in FEMA 310, not merely a reference to it as in this paragraph. The criticisms of LSP for URM are:
 - (a) The m values provided in Chapter 7 are too large, and even $m=1.5$ can only be acceptable if comparison studies show this to be correct.
 - (b) Secondary elements are not applicable to URM.

Suggested Change: None. This was already studied as a Global Topic. The project team believes that the FEMA 356 methodology is applicable to URM. Further case studies are necessary to identify the superiority of one method over another.

2. (Section 7.2) Historical information is based on “working stress” method and this paragraph could lead the engineer to use wrong numbers.

Suggested Change: Add commentary as follows: The engineer should be aware that values given in some existing documents are working stress value rather than “expected” or “lower bound” strength used in this document.

-
3. (Section C.7.3.2.1) Cracking based on past earthquake or settlement cannot be the only criteria for classifying masonry condition.

Suggested Change: Change the last two sentences of this section so that FEMA 306 categories provide an upper limit, i.e. walls with moderate damage may not be categorized as good; walls with heavy or extreme damage shall be categorized as poor condition.

4. (Section 7.3.2.1) References in the third SC draft are all old, and the latest version must be used.

Suggested Change: Dates have already been removed from the 90% draft.

5. (Section 7.3.2.4.ii) Delete this paragraph.

Suggested Change: Agree to delete.

6. (Section 7.3.2.6) One interprets this paragraph to say that $V_{te} = V_{t0}$ and that this should be replaced by either $V_{te} = 0.67 V_{t0}$ or that V_{te} is the second decile of V_{t0} values.

Suggested Change: Define v_{te} = average of bed-joint shear strength, v_{t0} , given in Equation 7-2.

Also change Section 7.3.2.4.iv to read: "Individual bed-joint shear strength test values, v_{t0} , shall be determined in accordance with Equation 7-2."

The project team does not agree that we need to define this strength differently compared to other materials. Also note that the effect of this requested change and that of item 18 tend to cancel out.

7. (Section 7.3.2.7) This paragraph should be changed to require that, for URM use gross stiffness and for reinforced masonry, use cracked stiffness equal to (say) 50% of gross (similar to concrete).

Suggested Change:

– *Delete the word "uncracked" from the first sentence.*

– *Delete the entire second sentence.*

– *Replace Section 7.4.4.1, item 1 with:*

"The shear stiffness of reinforced masonry walls shall be based on uncracked section properties".

– *Replace Section 7.4.4.1, item 2 with:*

"The flexural stiffness of reinforced masonry walls shall be based on cracked section properties. It shall be permissible to use an effective moment of inertia equal to 50 percent of gross section modulus."

Project team believes 50% gross property is applicable to reinforced masonry in flexure only. This is consistent with the concrete chapter.

8. (Section 7.3.2.9.1) A total of 3 masonry tests and 2 reinforcement tests is not adequate. Use the “comprehensive” testing of Section 7.3.2.9.2 as minimum requirements.

Suggested Change: No change. This is consistent with the rest of the document.

9. (Footnotes 2 and 3 to Table 7-1) The use of 1960 as a critical date for use of masonry is not appropriate, especially in the eastern part of the United States. Furthermore, mortars may be solid or air entrained with drastically different values. Footnote 2 and 3 (which are commentary statements) may be deleted.

Suggested Change: Agree to delete.

10. (Table 7-2) The factor of 1.6 in Table 7-2 is too high. Replace with 1.3. This recommended factor is based on Kariotis’ tests during the Techmar research.

Suggested Change: Change factor 1.6 to 1.3.

11. (Section C.7.3.3.2.1, 2, 3) These sections refer to use of methods that have not proved to be reliable in the past and should be deleted from this document.

Suggested Change: No change. These methods can be used in conjunction with traditional tests.

12. (Section 7.4.v) This section makes reference to documents that are old and use “working stress” design. The references should be changed to 1997 MEHRP or 2000 IBC. Also, the definition of the lower bound strength and expected values are not clear. They can be deleted and reference made to the particular section that clearly defines lower bound as mean minus one sigma and expected as the mean strength.

Suggested Change: Agree The 90% draft has already improved this section. No more changes are necessary.

13. (Section C.7.4.1.3.1) The sentence in item 1 does not have a solid reasoning behind it and should be deleted.

Suggested Change: Agree to delete.

14. (Section 7.4.2.1.iv) This paragraph appears to be using a Secant stiffness method, which is not the principle used in FEMA 273 and should be deleted.

Suggested Change: Delete this paragraph.

15. (Section F.7.4.3.2.ii) Add to the commentary after equation C7-2 to warn the engineer that completely fixed condition is often not the case in actual buildings.

Suggested Change: Not critical but this commentary can be added.

16. Some engineers believe that toe crushing and bed joint sliding are not realistic modes of failure in URM walls and piers. They believe that there are only two forms of failure:

(a) Masonry shear for which Equation 7.3 should be used. This failure should be considered force controlled.

(b) Rocking, for which equation 7.4 should be used. This form of failure should be considered deformation controlled.

They believe that equations 7.5 and 7.6 have no basis and if these equations are used, we should provide sufficient research to substantiate these equations.

Suggested Change: No change.

Project team believes that keeping the four failure patterns presents a more reliable approach. Further case studies, as indicated in response to item 1, would clarify this issue.

17. (Section 7.4.2.2.2) We have not specified the method to test for f_{dt} . Is this based on Brazilian test or do we always use a v_{me} for determination of f_{dt} as shown in section 7.4.2.2.B.iii?

Suggested Change: No change.

18. (Section 7.4.2.2.B.iii) This relationship is incorrect. f_{dt} is the maximum stress while v_{me} is the average stress, therefore, we should say $f_{dt} = 1.5 v_{me}$.

Suggested Change: No change. See item 6 for comments.

19. (Table 7-3) The tabulated values of m in Table 7.3 are very large. They should be cut down to about one-half of those indicated. Also, for ease of interpretation, the values for rocking should be spelled out (e.g. for I.O. to say need not be lower than 1). Also, delete the portion regarding Secondary Walls.

Suggested Change: No change, except for clarifying rocking values. Project team believes these values are justified.

20. (Table 7-4) These nonlinear acceptability criteria have no experimental backing and are quite high. They should be reduced. Better to be removed totally and not allow nonlinear analysis of URM buildings.

Suggested Change: No change. Project team does not agree with these comments.

21. (7.4.3.2.iii) Define effective void ratio in this paragraph. Does this apply to “out-of plane” only?

Suggested Change: Definition: Effective Void Ratio is the ratio of collar joint area without mortar to the total area of collar joint. A commentary should be added: this section applies to treatment of veneer for out-of-plane behavior of walls, only. For in-plane resistance, effective thickness is the sum of all wythes irrespective of condition of color joint.

22. (C.7.4.3.3) Correct the reference to TR-08, 1984.

Suggested Change: Agree.

23. (Sections 7.4.4.2.1 and 7.4.4.2.2) Equations should not be related to expected or lower bound strength. The equations are the same for both. If lower bound material properties are used, we obtain lower bound strength and if expected material properties are used we get expected strength.

Suggested Change: The 90% draft already includes some editorial clarifications. Additional verbiage has been added to the standard to explain that when shear is a deformation-controlled action, expected shear strength may be calculated with the same equations using expected material properties.

24. (Section 7.4.4.2.1) The Whitney Stress Block for masonry is .80 rather than .85. Also, the max. compressive strain in masonry is .0025 for concrete masonry and .0035 for brick masonry.

Suggested Change: No change.

25. (Section 7.4.4.3) Shear controlled reinforced masonry shear walls should be treated as deformation controlled with appropriately low m values similar to concrete shear walls.

Suggested change: Change the paragraph to read “Shear in shear controlled and flexure in flexure controlled reinforced masonry walls and piers shall be considered deformation controlled actions. Vertical...”

26. (Table 7-6) The m values in Table 7.6 are too numerous and relationship between m value and the L/h does not appear to be correct. This table should be changed to follow the general pattern of the concrete section. Furthermore, values should be added for shear controlled masonry walls. The FEMA 310 document is an acceptable substitute for this table.

Suggested Change: No change for now. We can change this later. However, add one row for “shear controlled walls” and use m values from Table 6-21, and the associated footnote.

27. (Table 7-7) Similar to item 26.

28. (Section 7.4.5.3.i) Delete all paragraphs in this section.

Suggested Change: Agree to delete. Also delete “For linear procedures” from Section 7.4.5.2.i.

29. (Section 7.5.1.2.ii) Reword to just say that actions in masonry infills are deformation controlled.

Suggested Change: Agree to this change. This may be moved to Section 7.5.2.3.3.

30. (Section 7.5.2.1.iii) This should have a reference to FEM 1 and CSMIP.

(Research by Kariotis et al 1994).

Suggested Change: Add a sentence in Section 7.5.2 as follows:

“The contribution of stiffness and strength due to infill is permitted to be based on non-linear finite element analysis of a composite frame substructure with infill panels that account for presence of openings and post yield cracking of masonry.

Commentary: This section refers to use of programs such as FEM 1.

Alternatively, a diagonal strut analogy per Section 7.5.2.1 and 7.5.2.2 may be used.”

31. (Section F.7.5.2.1.iv) This representation (Figure C7.3) should be deleted because it is primarily conjecture with no confirmation of its acceptability.

Suggested change: No change. This is helpful to conceptualize the behavior.

32. (Section 7.5.2.2.A.ii) The strength of masonry infill is not related to the shear strength of the masonry and can only be obtained by nonlinear finite element analysis. Dan Abrams has done further work and this section should be updated accordingly.

Suggested change: No change. New research is consistent with this section.

33. (Sections 7.5.2.2.B.iii and 7.5.2.2.C.ii) Delete this paragraph because the force could be substantial enough to cause failure of the column. Alternatively, reduce the 50psi to a much smaller value.

Suggested Change: Keep these sections but reduce 50 psi to 20 psi.

34. (Sections 7.5.2.3.A.ii and 7.5.2.3.B.ii) Do not disregard the frame if its strength is small. Also, define frame strength V_{fre} . Is it shear capacity of the column? The combined effect of frame and masonry infill is different from masonry alone, even for low strength of frame. Furthermore, the shear failure of column due to the presence of masonry infill may not be identified if masonry is treated alone

Suggested Change: (1) Define V_{fre} = Shear capacity of column. (2) In both paragraphs, delete the sentence “If the expected ... 7.4.4”. (3) Delete “0.3 □” from Tables 7-8 and 7-9.

35. (Table 7-10) The tabulated numbers are too low for masonry infill. Tests have shown that masonry infills have substantial resistance to lateral loads perpendicular to the plane of the infill. Either do not require a limit or increase these values to approximately 30.

Suggested Change: No change. Project team prefers to keep a set of conservative numbers for simplicity. The engineer can always use equation 7-20 to permit a thinner wall.

36. (Section 7-8) There is no minimum reinforcement specified in definition of Reinforced Masonry Wall.

Suggested Change: FEMA 310 has a definition that should be used here.

O. Special Study 10— Wood Issues

ASCE/FEMA 273 Prestandard Project

Special Study Report **Wood Issues**

prepared by

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October 17, 2000
(Revised November 22, 2000)

EXECUTIVE SUMMARY

This report covers the following five tasks related to a general update of Chapter 8 of the Prestandard draft. A sixth task involving edits and revisions to improve the consistency of the chapter was also included. A summary of these tasks is included below. The review of Chapter 8 of the Prestandard has revealed some areas, beyond the scope of this Special Study project, for which further study is recommended.

1. *Review applicability of recent research (UCI testing, CUREe research program).*

Preliminary data from the CoLA/UCI testing program has been reviewed. This data supports the numerical acceptance factors for linear and nonlinear procedures that appear in the current draft and forms the basis for the proposed strength criteria for wood structural panel shear walls. The proposed relationship between lower-bound and expected strengths is based on the CoLA/UCI test results.

Action Items:

- Revise Section 8.4.9.2 of the Prestandard based on the underline/strike-through revisions (to the 90% Draft) contained in Appendix A of this report.
- Review additional testing and research once it becomes available.

2. *Update wood reference documents and revise technical provisions if required.*

Since the original publication of FEMA 273, a consensus standard for Load and Resistance Factor Design (LRFD) for engineered wood construction (ASCE 16-95) has been published. This standard, which has been adopted by reference into the 2000 *NEHRP Recommended Provisions* for new buildings, provides material component strengths that are consistent with the expected strength approach of the Prestandard. Conversion from allowable stresses, which is the current approach in the Prestandard, has been moved to become an alternative described in the commentary. The *National Design Specification for Wood Construction* (NDS) was maintained as a commentary reference, updated to the 1997 edition, for default allowable stresses. However, this conversion methodology is revised as described in Task #4.

Action Items:

- Revise Chapter 8 of the Prestandard based on the underline/strike-through revisions (to the 90% Prestandard document) contained in Appendix A of this report.

3. *Review contradiction between Tables 8-1 and 8-2 regarding differences between stiffness of wood assemblies when classified as shear walls versus diaphragms.*

Inconsistencies have been identified based on comparisons of computed shear wall and diaphragm deflections. The shear wall stiffness values appear to be adequate, but the diaphragm stiffness values appear to be significantly too large and the equation for determining diaphragm deflection, in which the stiffness values are used, appears to incorrectly represent the effect of aspect ratio. The equation produces results that may be reasonable for an aspect ratio of 3:1, but grossly underestimates deflections at lower aspect ratios. The source of stiffness values and deflection equations for non-plywood sheathed shear walls and diaphragms has not been identified. Proposed revisions to the stiffness values and the equation are presented, but they are not based on rigorous study.

Action Items:

- Revise Chapter 8 of the Prestandard based on the underline/strike-through revisions (to the 90% Prestandard document) contained in Appendix A of this report. Add a global issue identifying the need for this issue to be revisited as additional research becomes available.

4. *Review applicability of factors used to convert allowable values to expected strength.*

Although the LRFD specification will form the basis for computing expected strengths in the Prestandard, it is our opinion that a methodology for converting allowable stress values into expected strengths is still useful. The Prestandard has been revised to permit an “approved” method for conversion, and one such method is included in the commentary.

The development of the LRFD standard for engineered wood construction is based on the ASTM D5457-93 *Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design*. This ASTM standard provides two methodologies for the development of LRFD reference resistance values: one uses test data and the other uses conversion from approved allowable stress values. The latter method is similar in approach, but numerically different from the FEMA 273 methodology. For consistency with the LRFD reference standard, the conversion method in the commentary to the Prestandard has been revised to match the ASTM D5457-93 format conversion methodology and will refer to the 1997 *NDS* for allowable stress default values.

Action Items:

- Revise Chapter 8 of the Prestandard based on the underline/strike-through revisions (to the 90% Prestandard document) contained in Appendix A of this report.

5. *Review and comment on applicability of ABK TR-03 regarding diaphragm shear strengths with roofing.*

The test results contained in ABK TR-03 suggest an increase can be permitted for the yield strength of straight-sheathed diaphragms when built-up roofing is present.

Action Items:

- Revise Table 8-1 of the Prestandard based on the marked-up table (from the 90% draft) contained in Appendix B of this report.

6. *Review of general consistency, clarity and usability of Chapter 8.*

Our review of Chapter 8 has resulted in several recommendations for improving the consistency, clarity and usability. When significant, the changes are noted in this report and contained in the Appendix A revisions; where minor or editorial, they are not noted in the report but are contained in the Appendix A revisions. Specific changes are listed as action items below.

Action Items:

- Revise Chapter 8 of the Prestandard based on the underline/strike-through revisions (to the 90% Prestandard document) contained in Appendix A of this report.
- We recommend adding a section following 8.3 to provide general requirements consistent with Chapters 5 and 6. This is included in the Appendix A revisions. Subsequent sections would need to be renumbered.

-
- We recommend adding a section following 8.6 that addresses wood elements and systems other than shear walls, diaphragms, and foundations (e.g. knee-braced frames, rod-braced frames, braced horizontal diaphragms, and components supporting discontinuous shear walls). Placing this information in one location would improve the clarity and usability of the Prestandard. This revisions is indicated in Appendix A, including notes regarding sections that would need to be renumbered.

INTRODUCTION

This report contains proposed modifications to the 90% Draft (9/29/00) of the FEMA 356 *Prestandard for the Seismic Rehabilitation of Buildings* (referred to herein as FEMA 356) based on five identified tasks listed below:

1. Review applicability of recent research (CoLA/UCI testing, CUREe research program).
2. Update wood reference documents and revise technical provisions if required.
3. Review contradiction between Tables 8-1 and 8-2 regarding differences between stiffness of wood assemblies when classified as shear walls versus diaphragms.
4. Review applicability of factors used to convert allowable values to expected strength.
5. Review and comment on applicability of ABK TR-03 regarding diaphragm shear strengths with roofing.

This study has also included a sixth task, which involves general edits and revisions to improve the consistency and usability of the chapter. This involves some reorganization of sections and many changes to section headings. Where these proposed revisions are significant, a discussion is included in this report; where they are editorial and minor, they are not included in this report, but are contained in the underline/strike-through in Appendix A.

OBJECTIVES

The tasks noted above were addressed with the following overall objectives in mind:

- Update and revise the prestandard to reflect recent research and code-development activities.
- Allow yield values to be based on 1) testing in accordance with Section 2.8, 2) principles of mechanics, 3) LFRD capacities (with $\phi = 1$) times an additional factor as needed (for shear walls only, based on recent testing), or 4) converted ASD values (as described in the commentary).
- Characterize the maximum force developed by 1) testing, or 2) multiplying yield values by an appropriate factor. Consideration of this maximum force is limited to nonlinear analysis procedures and limit-state analysis to compute force-controlled actions.
- Provide lower-bound values that are based on 1) mean minus one standard deviation test results, or 2) yield values multiplied by a factor. The default factor was revised based on available test results.
- Reorganize the main sections so that they are consistent with Chapters 5 and 6. Also, divide wood elements into four categories: shear walls, diaphragms, foundations, and “other wood elements and components.”
- Revise a number of other items for correctness, consistency, and clarity. These items are discussed in detail in the report.

FINDINGS

A majority of the revisions associated with Tasks #2 and #4 are based on a shift in reference documents for default material properties and expected strengths. Since the original publication of FEMA 273, a consensus standard for Load and Resistance Factor Design (LRFD) for engineered wood construction (ASCE 16-95) has been published. This standard, which has been adopted by reference into the 2000 *NEHRP Recommended Provisions* for new buildings, provides material component strengths that are consistent with the expected strength approach of FEMA 356. The development of the LRFD standard for engineered wood construction is based on the ASTM D5457-93 *Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design*. This ASTM standard provides two methodologies that may be used to establish LRFD reference resistance values: one uses test data and the other uses a soft conversion from approved allowable stress values. Published by the American Forest & Paper Association (AF&PA), the 1996 *LRFD Manual for Engineered Wood Construction* contains the ASCE 16-95 standard as well as commentary and design supplements. The AF&PA *Manual* and supplements contain reference resistance values for wood components and connections that have been developed using the ASTM D5457-93 standard. These reference resistance values will now form the basis for the default expected strengths in FEMA 356.

Conversion from allowable stresses (listed in an “approved code”), which is the current approach in the FEMA 356, will be kept as an alternative described in commentary. However, this conversion methodology has also been revised to be consistent with the format conversion methodology contained in ASTM D5457-93, which is similar in approach, but numerically different from the current FEMA 356 methodology. The commentary will still contain a reference to the *National Design Specification* for default allowable stress values.

The major revisions to FEMA 356 are summarized with background explanatory information in this report. Minor revisions, including updating current references and non-technical edits, are not explicitly noted here but are included in the accompanying underline/strike-through. Note that while we have proposed revisions to many of the section headings, this report refers to the section headings as contained in the 90% draft.

Chapter 8: Wood (Systematic Rehabilitation)

Section 8.3.2 Properties of In-Place Material and Components

Since this is the primary material property section referred to by the sections for specific components and assemblies (8.4.4, 8.5.2, etc), clarify the path to default materials by adding text to Section 8.3.2.1.1.

Section 8.3.2.1.2 Specified Material Properties

Nominal or specified material properties for wood construction are usually based on allowable stress values and therefore should not be taken as expected material properties without an appropriate conversion to strength values. Nominal or specified properties can serve as a basis for computing expected strengths. Section has been clarified.

Section 8.3.2.5 Default Properties

For wood components and connections, remove conversion factors for allowable stress values, and add reference to ASCE 16-95 for default expected strength values. Indicate that expected strengths shall include all applicable adjustment factors as specified in ASCE 16-95.

Indicate that ASTM D5457-93 or another “approved” method for computing expected strengths from code-recognized allowable stress values is permitted. A reference to the 1997 NDS for default allowable stress values and the specifics of the ASTM D5457-93 methodology for conversion to strength values are provided in the commentary.

The recent CoLA/UCI testing included shear walls sheathed with gypsum wallboard. The yield deflection and displacement ductility factors determined in the tests are in excellent agreement with the values shown in the present draft of FEMA 356 (unchanged from FEMA 273).

As identified above, Task #3 involves the apparent contradiction between the shear stiffness, G_d , values for shear wall assemblies in Table 8-1 and diaphragm assemblies in Table 8-2. For diaphragms, the values are 100 times greater than for shear walls of the same material. In reviewing these values, we have also uncovered an apparent inconsistency with the use of the equations for calculating yield deflections of shear walls (Equation 8-1) and diaphragms (Equation 8-5). Due to the differences between these equations, determining the appropriate relative values of G_d was very difficult, as discussed below.

Our initial review of the stiffness values involved comparing the yield deflections of the shear walls and diaphragms listed in Tables 8-1 and 8-2 using Equations 8-1 and 8-5, respectively. Intuitively, for equal widths (b), the yield deflection of a shear wall with a height (h) should be about half that of a diaphragm with length $L=2h$, since shear-related deflections (panel shear and nail slip) are expected to dominate. Calculations based on various aspect ratios (h/b and L/b) indicate that this is not the case.

A second review compared the yield deflections of the shear walls and diaphragms listed in Tables 8-1 and 8-2, using Equations 8-1 and 8-5, with the yield deflections of wood structural panel sheathed shear walls and diaphragms in accordance with Equations 8-2 and 8-6, respectively. The yield deflections for the various shear wall assemblies were in reasonable agreement with those for structural panel shear walls. That is, for various aspect ratios, the relationship between yield deflections of non-structural panel sheathing and structural panel sheathing appeared reasonable and intuitive. However, the yield deflections for diaphragms with non-structural panels (and unblocked structural panels and structural panel overlays for which there are G_d values) did not compare well to those for structural panel diaphragms. The non-structural panel diaphragms were much too stiff. In addition, the deflection of non-structural panel diaphragms seemed much too highly dependent on aspect ratio. Equation 8-5 for diaphragms considers the effects of aspect ratio as L^4/b^3 , which does not match the treatment of aspect ratio in Equation 8-6 (L^3/b for flexure and no consideration for the other terms), nor does it agree with Equation 8-1 for shear walls, which is independent of aspect ratio.

Based on sample calculations for various diaphragm configurations, it is clear that the values of G_d in Table 8-2 are too large. However, the proper values can not be accurately addressed without first dealing with the apparent flaws in Equation 8-5. Equation 8-5 generally compares well to the yield deflections for structural panel diaphragms where the aspect ratio is 3:1. This leads us to believe that the equation could have been developed to match the ABK diaphragm testing that was performed on 60' by 20' samples. However, many of the diaphragms listed in Table 8-2 are not currently permitted to have aspect ratios of 3:1 and calibration of an equation that is so highly dependent on aspect ratio to one aspect ratio would not be appropriate.

We cannot provide a simple and rigorous formula for the calculation of diaphragm deflections. However, we can take an approach that is consistent with that taken in the development of FEMA 273. The shear wall equation and Gd values appear to have been developed using the following two-step process: 1) select an equation form that is consistent with the predominant mode of behavior (panel shear and nail slip, both of which are shear-related), and 2) calibrate a stiffness factor to produce reasonable agreement with tests and more detailed calculations. Because the yield deflections calculated using Equation 8-5 and Table 8-2 are clearly incorrect, we have adopted a similar calibration approach, but with an additional constraint. For clarity and usability, we propose that the Gd values for diaphragms (in Table 8-2) be divided by 100 so that they match the values for similar shear wall assemblies (in Table 8-1). Therefore, the calibration to match more detailed calculations is by means of a factor applied to an equation of the same form as Equation 8-1. By comparing the relationship between shear wall and diaphragm displacements for plywood sheathed elements (using Equations 8-2 and 8-6) and other assemblies (using the Gd values along with Equation 8-1 and the proposed Equation 8-5), we determined that a factor of 2 should be applied in the denominator of the proposed equation which then becomes $\Delta y = (v_y L) / (2Gd)$. The calculated yield deflections are in good agreement. Therefore, we propose that this approach be taken until additional research supports further refinement. The calibration described above neglected chord slip for diaphragms and anchor deformation for shear walls. A chord slip term (consistent with the anchorage slip term in Equation 8-1) has not been added to the proposed Equation 8-5 since the effects of chords are presumably accounted for in the varying values of Gd for chorded and unchorded diaphragms.

Task #5, as indicated above, involves reviewing the applicability of ABK TR-03 (*Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing*, ABK Joint Venture, Topic Report 03, December 1981) regarding diaphragm shear strengths with roofing. This document contains the background and results of the diaphragm testing program and primarily includes raw data and force-deflection plots. A companion volume providing interpretation of the diaphragm testing (ABK TR-05) was never published. The yield strength values in Table 8-2 apply to bare sheathing without considering roofing. This is reasonably accurate for most assemblies since the roofing provides negligible strength. However, since the yield strength of single straight sheathing is very low, the presence of roofing may have a significant effect. The ABK testing program included tests of straight-sheathed diaphragms with built-up roofing. Tests without roofing were not performed. A review of the raw data for the test with roofing (without retrofit nailing of the roofing) gives a yield strength of about 200 plf and a maximum strength of about 240 plf. This is significantly greater than the yield strength value of 120 plf contained in Table 8-2. Assuming that 120 plf is appropriate for sheathing without roofing, we have added a footnote to the table permitting an increase of 50% for single straight sheathing in which built-up roofing is present. This results in a yield strength of 180 plf; the 1.5 factor is slightly conservative to reflect the paucity of data (there was only one ABK test of this assembly) and the significant strength degradation observed in the test.

The ABK testing included several of the lumber sheathed diaphragm assemblies listed in Table 8-2, and all of the tests were based on diaphragms with aspect ratios of 3:1 (60' by 20' specimens). All of these tests resulted in acceptable behavior and led to the development of design values for each assembly that were included in the ABK methodology. Therefore, it was decided by the Project Team, that the permitted aspect ratios (as specified in Tables 8-3 and 8-4) for all lumber sheathed diaphragms could be increased to 3:1. In addition, to provide a more smooth transition to the point where diaphragms are not considered effective lateral-load-resisting elements, the Project Team decided to allow for the acceptance criteria (m -factors or deformation ratios) to decrease linearly from the value at the maximum tabulated aspect ratio down to 1.0 for an aspect ratio of 4:1. Therefore, diaphragms (both lumber and structural panel sheathed) are permitted to have aspect ratios of 4:1 if they remain elastic.

By way of comparison, the 2000 NEHRP *Recommended Provisions*, permit maximum aspect ratios of 4:1 for blocked structural panel sheathing and 3:1 for unblocked structural panel sheathing and diagonal lumber sheathing (straight sheathing is not permitted at all). Although there is no rigorous basis for this revision, the Project Team agrees that it is reasonable to provide a transition in the acceptance criteria rather than a step function at the maximum tabulated aspect ratio beyond which diaphragms are considered ineffective.

There are two final issues regarding Tables 8-1 and 8-2. First, the shear stiffness for a shear wall consisting of wood siding over diagonal sheathing was incorrectly transferred from FEMA 273. The value should be 11,000 rather than 1,100. This value was changed on the marked-up table. Second, the shear stiffness for a chorded diaphragm of single diagonal sheathing (500,000) appears suspect, though it was correctly transferred from FEMA 273. For most conditions, the stiffness of a chorded diaphragm is twice that of an unchorded diaphragm, but for single diagonal sheathing it is only 25% higher (500,000 vs. 400,000). Also, assuming there is a correlation between the stiffness of shear wall and diaphragm assemblies, a proposed value of 800,000 for the diaphragm (before dividing by 100 as recommended above) is in perfect agreement with the value of 8,000 for the shear wall in Table 8-1. The 500,000 value was not changed, but a change to 800,000 (subsequently divided by 100) should be considered.

Markups of Tables 8-1 and 8-2 are included in Appendix B.

Section C8.3.2.5 Default Properties

Add commentary describing ASCE 16-95 and resistance values contained in the AF&PA *Manual* and supplements.

Provide the ASTM D5457-93 format conversion methodology for allowable stress values. This methodology involves multiplying the allowable stress value (based on a normal, 10-year duration) by a format conversion factor which is defined as $K_F = 2.16/\phi$, where ϕ varies based on component action. The various ϕ values are provided.

For format conversion, the ASTM D5457-93 standard clearly notes that “it shall not be claimed that reference resistance values generated in this manner achieve a stated reliability index.” However, this method appears to be most consistent with the LRFD reference for default material properties. A comparison between published values in the LRFD and NDS supplements for wood components and connections indicates that LRFD values (with $\phi = 1.0$) and format conversion of *NDS* values will result in equivalent expected strengths for seismic loading.

Indicate that the *LRFD Manual* contains a guideline for computing expected strengths from published allowable stress values (rather than average ultimate test values) for connection hardware.

Section 8.3.5 Rehabilitation Issues

For consistency with the steel and concrete chapters (5 & 6), move this section and associated commentary to a new Section 8.X.4 (see below). It is not appropriate for this section to be a subsection of “Material Properties and Condition Assessment.” The text of this section is unchanged.

Section C8.3.5 Rehabilitation Issues

See section 8.3.5 above.

Section 8.X General Assumptions and Requirements [*NEW SECTION*]

For consistency with other materials chapters (steel and concrete), we recommend adding a new section between Sections 8.3 and 8.4 and renumbering all subsequent sections. This section provides the appropriate location for introduction of stiffness requirements, design strengths and acceptance criteria, a specific subsection for the treatment of connections, and rehabilitation measures. This is especially useful as a place to reference from the sections for specific components and assemblies.

Section 8.X.1 Stiffness

New section (consistent with chapter 5) indicating that component stiffnesses shall be calculated in the sections concerning the specific components (shear walls, diaphragms, foundations, and “other wood elements and components”). Provide discussion on computing stiffness of wood material components and connections for linear and nonlinear procedures. This is also where the generalized force-deformation relation is introduced (Figure 8-1) with explanation of the parameters *c*, *d*, and *e*. We also propose that this figure be significantly revised for consistency with the rest of the document. Figure 6-1(b) could be used with minor revisions.

Section 8.X.2 Design Strengths and Acceptance Criteria

New section (consistent with chapter 5), no text following main heading.

Section 8.X.2.1 General

New section, indicating that actions shall either be deformation-controlled or force-controlled and that design strengths are as described in the following sections.

Section 8.X.2.2 Deformation-Controlled Actions

New section (consistent with chapter 5), describing the procedure for determining expected strengths, and referring to the sections for specific assemblies (shear walls, diaphragms, etc.). This section also contains guidelines for determining expected strengths and deformation capacities for wood components and connections that are not explicitly covered in the subsequent sections. Expected strengths are taken as the LRFD values, including all applicable adjustment factors, and ϕ is taken as 1.0.

Section 8.X.2.3 Force-Controlled Actions

New section (consistent with chapter 5), describing the procedure for determining lower-bound strengths. It indicates that, in lieu of more specific information, lower-bound strength values for wood components shall be taken as expected strength values, including all applicable adjustment factors, multiplied by 0.85.

FEMA 273 did not include a factor relating lower-bound and expected strengths for wood elements. Earlier drafts of FEMA 356 included a judgment-based factor of 0.75. The factor proposed in this study (0.85) is based on mean minus one standard deviation values for the recently completed CoLA/UCI testing of shear walls. FEMA 356 Section 8.3.2.4.2 also indirectly supports this level of certainty by requiring additional testing when the results of two tests differ by more than 20%.

The maximum forces developed in the CoLA/UCI shear wall tests were consistently 1.5 times the yield force. The maximum forces developed in the APA diaphragm tests were generally 2 times the yield force. Other wood components and connectors exhibit similar overstrength. This overstrength should be considered when calculating force-controlled actions.

Section 8.X.3 Connections

New section. This section is intended to provide a centralized location for providing requirements and guidelines for the treatment of connections, connectors, and connection hardware. Most of the text has been gathered from other parts of the chapter, and there are no technical changes.

Section C8.X.3 Connections

New Section. Commentary indicates that strength of entire connections, consisting of the connection hardware, connectors, and connected elements, must be considered. This should be clear based on the definitions in Section 8.7, but some guidance in this section would be helpful. For example, rather than simply taking the published average ultimate test values for a hold down device as the expected strength of the hold down assembly, the engineer also should consider the strength of the stud bolts, the strength of the anchor bolt, and the strength of the net section of the stud itself.

Section 8.X.4 Rehabilitation Issues

New section (consistent with chapter 5). This is the same as the previous Section 8.3.5 but we propose relocating the section for consistency with the steel and concrete chapters.

Section C8.X.4 Rehabilitation Issues

New section (consistent with chapter 5). This is the same as the previous Section C8.3.5 but we propose relocating the section for consistency with the steel and concrete chapters.

Section 8.4 Wood and Light Frame Shear Walls

Section 8.4.X General

New section with text from previous main Section 8.4. This section is intended to contain all the general information to clarify the references from the following subsections.

Add discussion regarding consideration of openings in shear walls. This was previously included in commentary by reference to the diaphragm section.

Remove text that yield strength is defined as 80% of ultimate as this is not always the case for wood components and assemblies.

Some connection information has been moved to Section 8.X.3, and there is a reference back to that section.

Section C8.4.X General

New section with text from previous main commentary Section C8.4.

Concerning shear wall aspect ratios, replace reference to 1994 UBC with the 2000 *NEHPR Recommended Provisions*. Indicate that the *Provisions* limit the aspect ratio for structural panel shear walls to 2:1 for full design shear capacity and permit reduced design shear capacities for walls with aspect ratios up to 3.5:1.

Add discussion and references for considering on the effects of openings in wood shear walls.

Section 8.4.1 Types of Light Frame Shear Walls

A few general changes are proposed for this section. Section headings are revised for consistency, and references are updated. We propose to remove the discussion concerning strength and stiffness degradation from commentary for various assemblies. Where applicable it will be added to analysis sections for specific assemblies (8.4.4, etc). Also remove references to the C_2 value as it will always be 1.0.

Section 8.4.3 Knee-Braced and Miscellaneous Timber Frames

Section 8.4 “Wood and Light Frame Shear Walls” is not the appropriate place for this subsection. Therefore we recommend moving it to a new section for “other wood elements and components” following Section 8.6, as shown in Appendix A.

Section 8.4.4 Single Layer Horizontal Lumber Sheathing or Siding Shear Walls

Section 8.4.4.1 Stiffness

In Equation 8-1 “G” is not the modulus of rigidity, but rather is the stiffness of the shear wall assembly as indicated in FEMA 273 and the Third SC draft. The notation “ G_d ” should be restored. In general, Equation 8-1 and the values for G_d for this and other shear wall assemblies appear reasonable as discussed in the comments on Section 8.3.2.5.

Section 8.4.4.3 Acceptance Criteria

Wording of section, but not content, has been revised for consistency throughout the chapter. Also applies to following sections (8.4.5.3, etc.)

For clarity, we propose a few changes to the linear and nonlinear acceptance criteria (Tables 8-3 and 8-4, respectively). These are included in Appendix B.

Section 8.4.9 Structural Panel or Plywood Panel Sheathing Shear Walls

Section 8.4.9.1 Stiffness

We propose to modify the values for e_n based on a comparison with the values of e_n for yield load as specified in the commentary of ASCE 16 (see also 1997 UBC Standard 23-2 and the commentary to the 1997 NEHRP Recommended Provisions). Using the equations for e_n and the maximum permitted load per nail (which is roughly equivalent to the load per nail at shear wall yield), the values for e_n are 0.13 for 6d nails and 0.08 for 8d and 10d nails. Also include in the text a requirement to increase e_n by 20% for panel grades other than Structural I as is specified in ASCE 16, etc.

Section 8.4.9.2 Strength

Consistent with the LFRD approach introduced in Section 8.3.2.5, the yield strength values for structural panel sheathed shear walls have been revised. FEMA 356 currently provides two methods for computing expected strength: 1) use of 80% of the values in Table 8-5 and 2) Equation 8-3 for nailing patterns not included in the table. Neither of these methods is consistent with the LFRD approach. The values in Table 8-3 are inconsistent with the values listed in the AF&PA LFRD *Manual*, the identical values in the 2000 *NEHRP Recommended Provisions*, and the results of the recent CoLA/UCI testing. Therefore, we recommend removing Table 8-3 and instead providing a reference for obtaining listed shear wall strengths. (This is similar to the current method for structural panel diaphragms, see Section 8.5.7). In lieu of changing Equation 8-3 to conform with LFRD values, we propose to delete it, and permit the calculation of shear strength using “principles of mechanics.” In commentary, refer to the method contained in the American Plywood Association (APA) Research Report 154 (*Wood Structural Panel Shear Walls*, Tissell, 1997), which has a more complete method for determining shear wall strength by calculation (that is still simple).

We also reviewed the appropriate conversion from ultimate strength to yield strength. Currently, FEMA 356 indicates that yield strength should be 80% of ultimate (or maximum) strength. We reviewed shear wall test data contained in APA Research Report 154 and APA Research Report 158 (*Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading*, Rose, 1998). In addition, we considered unpublished preliminary test data from the City of Los Angeles (CoLA) / University of California at Irvine (UCI) research program as indicated in Task #1. We considered raw data, force-deflection plots, and values for the yield limit state (YLS) and the strength limit state (SLS) as indicated in this research.

Our intent is to provide a factor to obtain yield capacities from the $\phi = 1.0$ values (in accordance with Section 8.X.2.2) from the referenced sources. From the CoLA/UCI data, we considered 17 representative test groups (13 plywood, 4 OSB) of 3 shear wall tests each. Based on these test results the yield strength (expected strength) should be 80% of the $\phi = 1.0$, LRFD value for plywood and 65% of the $\phi = 1.0$, LRFD value for OSB. These results are in general conformance with the APA testing which notes that OSB has a lower yield strength than plywood. Refer to Appendix C for supporting information.

Section C8.4.9.2 Strength

Provide references to the *AF&PA Manual* and the *NEHRP Recommended Provisions* for listed shear wall strengths. Provide reference to APA document for calculation of strength.

Section 8.5 Wood Diaphragms

Section 8.5.X General

New section with text from previous main Section 8.5. This section is intended to contain all the general information to clarify the references from the following subsections.

Move discussion regarding consideration of diaphragm openings from Section 8.5.11 to this section to simplify the referencing. Section 8.5.11 was previously only referred to in commentary.

Some connection information has been moved to Section 8.X.3, and there is a reference back to that section.

Section C8.5.X General

New section with text from previous main commentary Section C8.5.

Add discussion and references for considering the effects of openings in wood diaphragms.

Section 8.5.1 Types of Wood Diaphragms

A few general changes are proposed for this section. Section headings are revised for consistency, and references are updated.

Section 8.5.2 Single Straight-Sheathed Diaphragms

Section 8.5.2.1 Stiffness

In Equation 8-5 “G” is not the modulus of rigidity, but rather is the stiffness of the shear wall assembly as indicated in FEMA 273 and the Third SC draft. The notation “ G_d ” should be restored. As discussed in the comments on Section 8.3.2.5 we have identified some issues associated with Equation 8-5 and the values for G_d for this and other diaphragm assemblies.

Section 8.5.2.3 Acceptance Criteria

Wording of section, but not content, has been revised for consistency throughout the chapter. Also applies to following sections (8.5.3.3, etc.).

For clarity, we propose a few changes to the linear and nonlinear acceptance criteria (Tables 8-3 and 8-4, respectively). These are included in Appendix B.

Permitted aspect ratios for various diaphragms have been revised as discussed in the report Section 8.3.2.5.

Section 8.5.7 Wood Structural Panel Sheathed Diaphragms

Section 8.5.7.1 Stiffness

We propose to modify the values for e_n based on a comparison with the values of e_n for yield load as specified in the commentary of ASCE 16 (see also 1997 UBC Standard 23-2 and the commentary to the 1997 *NEHRP Recommended Provisions*). Using the equations for e_n and the maximum permitted load per nail (which is roughly equivalent to the load per nail at diaphragm yield), the values for e_n are 0.13 for 6d nails and 0.08 for 8d and 10d nails. Also include in the text a requirement to increase e_n by 20% for panel grades other than Structural I as is specified in ASCE 16, etc.

Section 8.5.7.2 Strength

Consistent with the LRFD approach introduced in Section 8.3.2.5, the yield strength values for structural panel sheathed diaphragms have been revised. FEMA 356 currently bases yield strength on test results (ultimate shear) or conversion from allowable values in the UBC. We propose to provide a reference for determining shear wall strengths. We also propose to permit calculation of yield strength based on “principles of mechanics.” In commentary, refer to the method contained in the American Plywood Association (APA) Research Report 138 (*Plywood Diaphragms*, Tissell and Elliott, 1993), which has a comprehensive methodology for determining diaphragm strength by calculation.

We also reviewed the appropriate conversion from ultimate strength to yield strength. Currently, FEMA 356 indicates that yield strength should be 80% of ultimate (or maximum) strength or 2.1 times allowable stress values. Although there is not as much cyclic testing available for diaphragms as there is for shear walls, we reviewed shear wall test data contained in APA Research Report 138.

Our intent is to provide a factor to obtain yield capacities from the $\phi = 1.0$ values (in accordance with Section 8.X.2.2) from the referenced sources. From the APA data, we considered 3 representative tests (all plywood). Based on these test results the yield strength (expected strength) should be 100% of the $\phi = 1.0$, LRFD value. (There is no data available to suggest different values for OSB). These results are in general conformance with the ABK TR-03 testing of plywood diaphragms reviewed as discussed in the comments in Section 8.3.2.5. Refer to Appendix C for background information.

Section C8.5.7.2 Strength

Provide references to the *AF&PA Manual* and the *NEHRP Recommended Provisions* for listed diaphragm strengths. Provide reference to APA document for calculation of strength.

Section 8.5.8 Wood Structural Panel Overlays on Straight or Diagonally Sheathed Diaphragms

Section 8.5.8.2 Strength

FEMA 356 currently bases yield strength on test results (ultimate shear) or conversion from allowable values for a comparable wood structural panel diaphragm; our proposal does not change the philosophy of this approach. This section will refer directly to Section 8.5.7.2 for yield strength of “the corresponding wood structural panel diaphragm.” Section 8.5.7.2, its commentary, and the sections to which it refers provide four methods to determine the strength. They are testing, principles of mechanics, LRFD reference resistances, and converted ASD capacities.

Section 8.5.9 Wood Structural Panel Overlays on Existing Wood Structural Panel Diaphragms

Section 8.5.9.1 Stiffness

APA Research Report 138 (*Plywood Diaphragms*, Tissell and Elliott, 1993) explicitly states that the diaphragm deflection equation (Equation 8-6) does not apply to double layer panel diaphragms. This is presumably due to the difficulty in dealing with the nail slip term. Therefore, we propose to include the panel over panel overlay in Table 8-3 and use the G_d values associated with panel over sheathing diaphragms for deflection calculation in accordance with Equation 8-5. Once the issues with the G_d values for diaphragms and Equation 8-5 are resolved, this is judged to be adequate for estimating deflections of panel over panel diaphragms. Note that the strength criteria for panel over panel diaphragms will remain as they are currently stated in FEMA 356.

Section 8.5.11 Chords and Openings in Wood Diaphragms

For ease of use, delete this section and move its contents into the general discussion for diaphragms (Section 8.5).

Section 8.5.12 Posts not Laterally Restrained or Part of a Knee-Braced Frame System

This section was added to the 90% draft in response to Global Issue 8-8. In its current form, it seems confusing and incomplete. We propose to change the heading to reflect what the section is rather than what it is not. Our recommended section, “Components Supporting Discontinuous Shear Walls”, also includes text for beams that support discontinuous walls, as this condition can occur.

In addition, the Section 8.5 “Wood Diaphragms” is not the appropriate place for this subsection. Therefore we propose moving it to a new section for “other wood elements and components” following Section 8.6, as shown in Appendix A.

Section 8.Y Other Wood Elements and Components

New section. This section contains general requirements for elements and components other than shear walls, diaphragms, and foundations. It is essentially an organizational change intended to improve to usability of the chapter. Refer to Appendix A.

**P. Special Study 12—
FEMA 310 and FEMA 356
Differences**

ASCE/FEMA 273 Prestandard Project

Special Study Report **FEMA 310 and FEMA 356 Differences**

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November 30, 2000

Introduction

The ASCE Standards Committee on Seismic Rehabilitation of Buildings is now responsible for producing both of the standards for seismic evaluation (FEMA 310) and seismic rehabilitation (FEMA 356). These two documents, while similar, were produced at different times in separate forums. FEMA 310 has already gone through standards committee ballot and has had numerous revisions. FEMA 356 has had many global topic studies performed, resulting in significant changes. The goal of these two documents is that they be used together. FEMA 310 would be used for the initial evaluation of buildings and FEMA 356 would be used either for advanced analysis or rehabilitation. Therefore, the two documents need to be checked for consistency against one another.

In examination of both documents, two major differences are apparent:

1. There is a difference in the seismic demands in evaluation versus design. The difference is philosophical and extends back to FEMA 178 when a 0.85 and 0.67 were applied to the static base shear. FEMA 310 was developed to maintain this consistency with FEMA 178. FEMA 356 is a rehabilitation document, so the forces remain at design level. After much discussion, it was decided that the difference would remain between the two documents since the documents are used for different purposes. However, FEMA 310 commentary would be revised to indicate that evaluation level demands would have a lower probability of achieving the desired performance level.
2. The FEMA 310 analysis methodology is less complex than FEMA 356. When FEMA 310 was developed, it was recognized that the requirements for evaluation should be less strenuous than for rehabilitation. Therefore, only the LSP was used and the terms and analysis requirements were simplified. Other requirements, such as material properties and materials testing were also relaxed. Since the FEMA 310 methodology is really a simplified subset of FEMA 356, it was decided that the difference would remain, once again acknowledging the difference between evaluation and design.

Once these two differences were recognized, the two documents were very consistent. There were minor differences in the methodology due to changes in FEMA 356 from the Global Topics Studies performed. There were minor differences in the definitions and cross-references due to changes in FEMA 310 during the standards committee ballot process.

Revisions to Documents

FEMA 356: Definitions and cross-references due to the FEMA 310 ballot process will be revised in FEMA 356 prior to standards committee ballot.

FEMA 310: Methodology revisions in FEMA 356, such as period formulation and foundations, due to Global Topics Studies will be revised in FEMA 310 during the public ballot process.

Discussion of Document Differences

The following Table summarizes the list of differences identified between FEMA 310 and FEMA 356, the affected sections in each document, and the action required. In the sections that follow, each item is discussed in greater detail including an explanation of the nature of the difference, a discussion about the difference, and changes recommended for each document. If an issue listed in Table 1 was examined, and no significant differences were found, no further discussion is provided. Although this study concluded that these issues were not significantly different in the two documents, they are listed here for future reference.

Document Differences	FEMA 310 Reference	FEMA 356 Reference
Issue Examined – Differences Found – Revisions To Be Made		
Level of Investigation, Site Visit Requirements	Sections 2.2-2.3	Section 2.2
Building Type Definitions	Table 2-2	Table 10-2
Site-Specific Requirements	Section 3.5.2.3.2	Section 1.6.2
Period Formulation	Section 3.5.2	Section 3.3.1
Foundation Analysis	Section 4.2.4.3.4	Chapter 4
m-Factors	Tables 4-3 to 4-6	Tables in Chapters 5-8
0.75 Factor for Evaluation	Section 5.2.1	Section 3.3.1
Reference Tables	None	Table C10-20
Issue Examined – Differences Found – No Revisions To Be Made		
Performance Level Definitions	Section 2.4	Section 1.5.1
Further Evaluation Requirements/Limitations	Table 3-3	Table 10-1
Ground Motion	Section 3.5.2	Section 1.6.1
Deformation vs. Force-Controlled Actions	Section 4.2.4	Section 2.4.4.3
URM Special Procedure	Section 4.2.6	Section 7.4.2, 7.4.3
Nonstructural Procedure	Section 4.2.7	Section 11.7
Issue Examined – No Differences or Minor Differences Found – No Revisions To Be Made (or already made)		
Definitions, References, Notation	Chapter 1	Throughout (Chapter 10 esp.)
Soil Factors	Section 3.5.2	Section 1.6.1.4
C-Factor	Section 3.5.2	Section 3.3.1
Statements in Checklist	Section 3.7	Tables C10-1 to C10-19
Analysis Procedure	Section 4.2.2	Section 3.3
Mathematical Modeling	Section 4.2.3	Section 2.4.4.2, 3.2.7, 3.2.2.2.2
Nominal vs. Expected Strengths	Section 4.2.4.4	Section 2.2.2, Chapters 5-8
Allowable vs. Ultimate Factors	Section 4.2.4.4	Chapters 5-8
Checklist Statement Language	Sections 4.3-4.8	Sections 10.3

Table 1 – Summary of Document Differences

Topic Name:	Level of Investigation, Site Visit Requirements
FEMA 310 Reference:	Section 2.2-2.3
FEMA 356 Reference:	Section 2.2
Difference:	FEMA 310 is always less detailed than FEMA 356 as it is judged that less investigation is required for evaluation as opposed to a retrofit. Requirements for testing have a big impact here.
Discussion:	Differences in level of investigation are consistent with the philosophy of differences between evaluation and rehabilitation.
Changes to FEMA 310:	Section 2.2 bullets for Tier 3 level of investigation to be revised to refer to source document selected for the Tier 3 Evaluation.
Changes to FEMA 356:	None

Topic Name:	Building Type Definitions
FEMA 310 Reference:	Table 2-2
FEMA 356 Reference:	Table 10-2
Difference:	Definitions of building types are not in sync. FEMA 310 has been revised through the ballot process.
Changes to FEMA 310:	None
Changes to FEMA 356:	Revise to match FEMA 310 definitions.

Topic Name:	Site Specific Requirements
FEMA 310 Reference:	Section 3.5.2.3.2
FEMA 356 Reference:	Chapter 1.6.2
Difference:	Requirements for site-specific ground motion criteria are different. FEMA 356 allows for use a mean spectra whereas FEMA 310 uses mean + one sigma.
Discussion:	Studied in Global Issue 2-11. FEMA 273 did not specify statistical basis. FEMA 356 has been revised to specify the use of mean probabilistic spectra and 150% of median deterministic spectra.
Changes to FEMA 310:	Revise FEMA 310 to match FEMA 356
Changes to FEMA 356:	None

Topic Name:	Period Formulation
FEMA 310 Reference:	Section 3.5.2.4
FEMA 356 Reference:	Section 3.3.1
Difference:	The formulas for period formulation in each document in different. FEMA 356 has a β factor in it.
Discussion:	Subject of a special study in FEMA 356 development to review recently published research and reduce conservatism in calculated periods.
Changes to FEMA 310:	Revise FEMA 310 to match FEMA 356
Changes to FEMA 356:	None

Topic Name:	Foundation Analysis
FEMA 310 Reference:	Section 4.2.4.3.4
FEMA 356 Reference:	Chapter 4
Difference:	FEMA 356 has gone to the R_{OT} approach for determining foundation forces. FEMA 310 has the 2/3 and 1/3 reductions in force. Both methods yield similar forces, as shown in the Fourth Ballot Response on FEMA 310.
Changes to FEMA 310:	Recommend changing procedure to R_{OT} method of evaluation.
Changes to FEMA 356:	None

Topic Name:	<i>m</i> -factors
FEMA 310 Reference:	Tables 4-3 to 4-6
FEMA 356 Reference:	Tables in Chapters 5-8
Difference:	The FEMA 310 Tables are more abbreviated than FEMA 356 and the <i>m</i> -factors in FEMA 310 on average are slightly higher than FEMA 356. The reason for this increase is to account (partially) for the 0.85 and 0.67 factors in FEMA 178 that account for forces used in design versus evaluation.
Discussion:	Differences are intentional as noted above.
Changes to FEMA 310:	Update C5.2.1 to refer to FEMA 356
Changes to FEMA 356:	<ol style="list-style-type: none">1. Reference FEMA 310 is C1.12. Reference 0.75 factor in FEMA 310, Tier 3 in C1.3

Topic Name:	0.75 Factor for Evaluation
FEMA 310 Reference:	Section 5.2.1
FEMA 356 Reference:	Section 3.3.1
Difference:	FEMA 310 states that if you use FEMA 356 (or any design document for that matter) for evaluation, you can apply a 0.75 factor on those forces. This is the argument on forces used in design versus evaluation.
Discussion:	Differences are intentional as noted above.
Changes to FEMA 310:	Update C5.2.1 to refer to FEMA 356
Changes to FEMA 356:	<ol style="list-style-type: none">1. Reference FEMA 310 is C1.12. Reference 0.75 factor in FEMA 310, Tier 3 in C1.3

Topic Name:	Reference Tables
FEMA 310 Reference:	Not Applicable
FEMA 356 Reference:	Table C10-20
Difference:	FEMA 356 still references FEMA 178. The reference numbers in FEMA 310 should be cross-checked against the latest ballot version of FEMA 310.
Discussion:	FEMA 178 is still used on Federal Projects and in SB 1953. Leave in as reference. Table should be updated to latest version of FEMA 310 section numbers and statements.
Changes to FEMA 310:	None
Changes to FEMA 356:	Table C10-20 updated to reflect latest FEMA 310 section numbers and statements.

Topic Name:	Performance Level Definitions
FEMA 310 Reference:	Section 2.4
FEMA 356 Reference:	Section 1.5.1
Difference:	The definition for Life-Safety and Immediate Occupancy are different in each document. The FEMA 310 definition has been refined by the ballot process and includes both the definition and commentary in Chapter 1. FEMA 310 does not have a Collapse Prevention Performance Level.
Discussion:	FEMA 310 has set the minimum Performance Level at Life Safety. A more rigorous evaluation per FEMA 356 would need to be performed to justify a lower performance level. The definitions of performance levels in each document are similar. FEMA 310's definitions are more direct while FEMA 356's definitions are more carefully worded. The definitions should be made consistent through the ballot process.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

Topic Name:	Further Evaluation Requirements/Limitations
FEMA 310 Reference:	Table 3-3
FEMA 356 Reference:	Table 10-1
Difference:	These tables are similar in form (but not values), but they do serve slightly different purposes.
Discussion:	The tables in each document were derived from the same source. However, each table has a different purpose. The FEMA 310 table is used to denote when the checklist methodology breaks down and a full analysis is required. The FEMA 356 table reflects the limitations of the Simplified Rehabilitation Method.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

Topic Name:	Ground Motion
FEMA 310 Reference:	Section 3.5.2
FEMA 356 Reference:	Section 1.6.1
Difference:	FEMA 310 follows NEHRP and the 2000 IBC by allowing only the use of the MCE maps. FEMA 356 allows the use of the MCE maps or the 10-in-50 maps.
Discussion:	The FEMA 310 check is a subset of a FEMA 356 check for a defined performance level and earthquake hazard. No changes recommended.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

Topic Name:	Deformation vs. Force-Controlled Actions
FEMA 310 Reference:	Section 4.2.4
FEMA 356 Reference:	Chapter 2.4.4.3
Difference:	Definitions of these terms are different in the documents. The definitions for FEMA 310 have been refined through the ballot process.
Discussion:	Definitions in FEMA 356, Section 2.4.4.3 are more rigorously defined in terms of component force-deformation behavior. FEMA 310 definitions are more direct statements consistent with FEMA 356 concepts for linear procedures.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

Topic Name:	URM Special Procedure
FEMA 310 Reference:	Section 4.2.6
FEMA 356 Reference:	Section 7.4.2, 7.4.3
Difference:	FEMA 356 has no special procedure for URM with flexible diaphragms. FEMA 310 has converted the FEMA 178 Methodology and is still going under refinement.
Discussion:	The issue has been considered under Global Issue 3-8 and Special Study. Portions of the procedure have been included in FEMA 356. Use of the Special Procedure in FEMA 310 is permitted as part of the “break” for evaluation, but rehabilitation requires the procedures of FEMA 356.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

Topic Name:	Nonstructural Procedure
FEMA 310 Reference:	Section 4.2.7
FEMA 356 Reference:	Chapter 11.7
Difference:	FEMA 356 has an analytical and prescriptive procedure whereas FEMA 310 only has the prescriptive procedure. The prescriptive procedures are almost identical except FEMA 310 does not account for vertical effects.
Discussion:	FEMA 310 is a subset of FEMA 356. No change required. Analysis could be done in Tier 3 using FEMA 356.
Changes to FEMA 310:	None
Changes to FEMA 356:	None

**Q. Special Study 13—
Study of Nonstructural Provisions**

FEMA 356
Prestandard for the Seismic Rehabilitation of Buildings
Study of Nonstructural Provisions

November 28, 2000

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Scope of Study

The provisions for evaluating and rehabilitating nonstructural components in FEMA 356, were thought by the author to contain a number of inconsistencies. One source of inconsistency is the differences in definition of the performance levels for nonstructural components as defined in Chapter 1 and the procedures for evaluating nonstructural components as set forth in Chapter 11.

The purpose of the study is to clarify the intent of the performance levels, to attempt to bring consistencies between the two portions of the document, and to establish rational procedures for evaluating nonstructural components at each performance level.

Background

In FEMA 356, there are four Performance Levels that are defined for nonstructural components: Operational, Immediate Occupancy, Life Safety, and Hazards Reduced. An additional Performance Level of Not Considered is also defined. The FEMA 356 prestandard specifically states that it does not include specific design procedures or acceptance criteria for the Operational Performance Level. Criteria have been developed for evaluation and rehabilitation of typical nonstructural components for other performance levels.

One of the fundamental issues in this study is the definition of Hazards Reduced Performance Level. At the recent Standards Committee meeting, it was stated that Hazards Reduced Performance was intended to address the situation in typical practice in which an engineer would rehabilitate the high hazard nonstructural components in the building. It is also the intention of Hazards Reduced Performance that the nonstructural components that are evaluated or rehabilitated to this performance level should meet the same criteria as for Life Safety Performance Level. The rationale for using the same criteria for Hazards Reduced and Life Safety is 1) if rehabilitation is required, the rehabilitation of the critical components should provide Life Safety protection for those components and 2) once bracing is provided, there would not be a significant difference in the design for Life Safety versus a lesser criteria. In other words, if a nominal bracing system is needed at all, that bracing could generally meet a stricter criteria.

This background definition provides the basis for this study. If the definition or intent of the Hazards Reduced Performance Level is changed, there will need to be other changes required to the document to maintain the consistency between the definition and the criteria.

Nonstructural Performance Levels

The issues of nonstructural performance are covered in two sections of FEMA 356; section 1.5.2 in chapter 1 and chapter 11. Ideally, the definitions of the performance levels in chapter 1 should have some correlation with the rehabilitation criteria for the nonstructural components as set forth in chapter 11. The following is a description of the considerations necessary for coordination.

Chapter 1

Section 1.5.2

Section 1.5.2.4 provides the definition of the Hazards Reduced Performance Level. As stated above, the intention of the Hazards Reduced Performance Level is to address the high risk nonstructural components in the building. These high risk components are likely those that, if they were to fail, would create the greatest falling hazard to the occupants of the building and the public that might be outside the building. These nonstructural components would primarily be those objects that are relatively heavy and in areas of public assembly or public access.

Due to the wide variety of conditions that may be encountered in a building, it would not be practical for FEMA 356 to provide a list of all items that should be addressed in order to meet this goal. The engineer should be allowed judgement to determine which nonstructural components would be considered high risk and should be addressed at this performance level. The engineer alone should not necessarily make the determination as to which nonstructural components should be considered critical. The owner may need to provide input as to areas and components of concern. The local jurisdiction may also have requirements for addressing certain nonstructural components as a minimum requirement, such as parapets and hollow clay tile walls in primary exit routes.

With these considerations, the definition of Hazards Reduced Performance Level has been revised as follows:

Nonstructural Performance Level N-D, Hazards Reduced, shall be defined as the post-earthquake damage state that includes damage to nonstructural components that could potentially create falling hazards. High risk nonstructural components shall be secured and shall not fall into areas of assembly or onto primary public thoroughfares. Exits, fire suppression systems, and similar life-safety issues are not addressed in this Performance Level.

In this revision, the strict definition of the items to be considered, such as falling debris over 500 pounds or having a dimension in excess of 6 feet, are removed. There may be situations in which heavy or large items could fall without endangering the public and may not be required to be rehabilitated. By eliminating this restriction and by including the modifier *high risk* in the requirements for securing and protection from falling, judgement is allowed to be used in selecting the nonstructural components that are to be considered in the evaluation and rehabilitation. The definition also explicitly states that exiting, fire suppression, and other life-safety issues are not considered in Hazards Reduced Performance. Table 11-1 has also been modified to include minimum recommendations for nonstructural components that need to be evaluated and rehabilitated, if necessary.

The commentary for this section has also been revised to account for the changes. The intent of the Hazards Reduced Performance Level is clearly stated as addressing a subset of nonstructural components. The commentary also suggests that those components evaluated in the Hazards Reduced should be evaluated and rehabilitated to Life Safety Performance Level.

Tables C1-5 through C1-7

The specific nonstructural components that are listed in tables C1-5 through C1-7 have been categorized as architectural; mechanical, electrical, and plumbing; and contents. The designation of which components fall into each category is different than that used to distinguish categories in Table 11-1. The grouping in Tables 11-1 is consistent with the designations used in Section 11.9 through 11.11. Light fixtures, for example, are listed as architectural components in Tables C1-5, but are listed as electrical equipment in Table 11-1. Some of the nonstructural components in Tables C1-5 through C1-7 were relocated to be consistent with Table 11-1.

In most instances in these tables, there was an expressed difference in performance between the Hazards Reduced Performance Level and the Life Safety Performance Level. As described above, the intent of Hazards Reduced is the consideration of selected nonstructural components and that these components would be evaluated using the same criteria as for Life Safety. As described below, the expected performance of some nonstructural components, as described in Tables C1-5 through C1-7 was not consistent with differences in the evaluation procedures in Chapter 11

The expected performance of some nonstructural components at the Hazards Reduced and Life Safety Performance Levels are revised to be consistent with similarities in the evaluation requirements between the Hazards Reduced and Life Safety Performance Levels. For some nonstructural components, there is no requirement for evaluation listed in Table 11-1 for Life Safety or Hazards Reduced Performance Levels. In these cases, the performance descriptions in Tables C1-5 through C1-7, have been revised so that the performance is consistent, where appropriate. The expected performance in Tables C1-5 through C1-7 have also been clarified for Hazards Reduced Performance to be the same as Life Safety Performance for those components that are not required to be evaluated or rehabilitated in Table 11-1. The expected Hazards Reduced Performance in Tables C1-5 through C1-7 for components that are considered high risk and which are required to be evaluated in Table 11-1 have been increased to account for required evaluation and rehabilitation.

The intent of Tables C1-5 through C1-7 is that nonstructural components evaluated or rehabilitated to the Hazards Reduced Performance Level should perform the same as if the nonstructural components had been evaluated or rehabilitated to the Life Safety Performance Level. For nonstructural components that are not evaluated or rehabilitated at the Hazards Reduced Performance Level, the expected performance should be less than the performance for Life Safety. Therefore, a footnote is added to Tables C1-5 through C1-7 to indicate that the performance of the nonstructural components would be the same as the Life Safety Performance Level if the component were considered critical.

Chapter 11

Section 11.1

In section 11.1, the scope of the chapter is clarified to identify the performance levels that are specifically addressed in the chapter. Since Operational Performance Level is not covered, a statement in the section on scope specifically notes that it was not covered. This is also explained in the commentary. The commentary is also revised to state that the core of the chapter deals with Hazards Reduced Performance Level, in addition to Life Safety and Immediate Occupancy. As intended, the commentary explains that the requirements for Hazards Reduced Performance will generally be based on the requirements for Life Safety.

Section 11.2

Section 11.2 describes the general procedure for rehabilitating nonstructural components, including a list of steps. The section references Table 11-1, which contains the requirements for various types of nonstructural components.

Item 1 of the procedure, states that a rehabilitation objective is established including a performance level and a zone of seismicity with reference to Section 11.4. Section 11.4 references Section 1.4 for the selection of the rehabilitation objective. The terminology used in section 1.4 states that the rehabilitation objective is a goal consisting of a target building performance level and an earthquake hazard. The term performance level in this item is clarified to be target building performance level to be consistent with Section 1.4. The term zone of seismicity is not used in section 1.4 and therefore has been changed to be earthquake hazard level for consistency. A sentence has also been added to clarify that the provisions of chapter 11 are not applicable in the case where the nonstructural performance level of the building is Nonstructural Performance Not Considered.

In item 2, two sentences are added to state that there needs to be an assessment to determine which nonstructural components will be considered when the Hazards Reduced Performance Level is used. Since Hazards Reduced considers a portion of the nonstructural components, then it is necessary to designate which components are the high risk ones and will therefore be evaluated and rehabilitated. This selection, as mentioned above, should be approved by the owner, and possibly by the building official. Table 11-1 has been revised to include recommendations for minimum nonstructural components to be considered. Commentary has been added to section C11.2 to discuss considerations of nonstructural components for Hazards Reduced Performance Level.

In item 5, the term *acceptability* has been changed to *classification* to be consistent with the terminology in section 11.6. Acceptability is covered in section 11.3.2.

Item 6 is one of the key steps in the procedure that is listed in section 11.2 since it describes the evaluation procedure for the nonstructural components. The evaluation procedure is clarified to be based on the acceptance criteria in Section 11.3.2. However, there appeared to be a significant oversight in the development of the acceptance criteria. In Section 11.9 through 11.11, the acceptance criteria referred to Section 11.3.2 and provided some guidance on acceptable drift levels, but no guidance on acceptable structural capacities. Section 11.3.2 refers to section 11.9 through 11.11 for the acceptance criteria. It appears that an explicit acceptance criteria for forces on the bracing for the nonstructural components was currently missing from the document and needs to be included. Because of this omission, section 11.3.2 is revised as described below to discuss the acceptance criteria.

Item 7 discusses the rehabilitation of nonstructural components and the acceptance criteria. Similar to the comment above, the reference for the acceptance criteria is changed from Section 11.9 through 11.1 to be section 11.3.2. Also included in this item is a statement that the connection between the nonstructural component and the structure should be based on Chapters 5 through 8. Often the bracing of nonstructural components involves more than providing a bolt to connect the nonstructural component to the structure. In these cases, there are structural elements, such as braces, that transfer the lateral forces from the nonstructural component to the connection to the structure. These bracing elements also need to be evaluated, and therefore the second sentence of this item is revised to include bracing elements as well as the connections as needing to be checked to determine their acceptability. The use of Chapters 5 through 8 to determine the acceptability of the nonstructural component bracing has been revised to refer to section 11.3.2 since the force levels used for nonstructural components, using section 11.7, are at strength design levels. The criteria in Chapters 5 through 8 however, are for expected strength or lower bound strength, and therefore are not consistent.

Table 11-1 provides a description of the requirements for determining the acceptance for various types of nonstructural components. For a given type of nonstructural component, the requirements may vary depending on the performance level and the region of seismicity. The requirements in the table do not vary much by zone of seismicity. In fact, the requirements for Immediate Occupancy were the same for all zones of seismicity and there are only a few minor differences between the requirements for Life Safety for zones of high seismicity and moderate seismicity. Immediate Occupancy requirements have been combined for all performance levels in Table 11-1. The requirements for Life Safety and Hazards Reduced Performance have been combined for High and Moderate Seismicity. Where necessary, a footnote has been provided to distinguish between requirements for High and Moderate Seismicity to keep consistency with the previous table.

Hazards Reduced Performance Level is now included in the Table 11-1. The nonstructural components for which evaluation and rehabilitation are recommended have been included in the table. There are some nonstructural components that need not be evaluated at the Life Safety Performance Level in Table 11-1. No evaluation would be needed for these nonstructural components for Hazards Reduced Performance Level. The architectural items included to be considered for Hazards Reduced Performance are heavy architectural items such as the exterior cladding, heavy ceilings, and parapets and appendages. The only mechanical systems included are piping containing hazardous materials. Integrated ceiling light fixtures are also included in the table. Storage racks are included in the table, with a footnote that this requirement applies where the storage racks are in areas of public occupancy, such as stores. Although this table is explicit in the requirements for components to be included at Hazards Reduced Performance Level, this should not prevent the engineer from adding or subtracting from these items, if appropriate.

Section 11.3

As mentioned previously, the acceptance criteria for nonstructural components is not well defined. Section 11.3.2 references acceptance criteria in section 11.9 through 11.11, whereas sections 11.9 through 11.11 reference acceptance criteria in section 11.3.2. Sections 11.9 through 11.11, provide guidance for deflection limits for deformation-sensitive components, but no guidance for determining the structural capacity of the braces and anchorage for the nonstructural components. In FEMA 273, the acceptance criteria for bracing of nonstructural components was not well defined in terms of how to check the components of the bracing for the applied forces.

The current draft prestandard (in section 11.2, item 7) indicates that the intent of the document is to apply the criteria in Chapters 5 through 8 to check the bracing of nonstructural components. As described above, the force level applied to nonstructural components in Section 11.7 is at a strength design level, and therefore it would not be appropriate to use the criteria in Chapters 5 through 8 that are not at strength design levels. Section 11.3.2 is therefore revised to indicate that the acceptance criteria for the designated forces on the nonstructural components should be based on strength design basis. Deformation limits specified in Section 11.9 through 11.11 have not been changed.

The intended definition of Hazards Reduced Performance is again added to the commentary as being the same as for Life Safety Performance, except that it applies to designated high risk components. The standards text in section 11.3.2 indicates that Life Safety acceptance criteria should be used for those components checked using Hazards Reduced Performance. The commentary has been expanded to provide an explanation for this intention. The commentary includes a statement that it may be permissible to use a lower acceptance criteria for Hazards Reduced Performance.

Some types of bracing for nonstructural components are based on proprietary components, and strength design values may not be available. Commentary is added to direct the engineer to allow for a conversion of the allowable stress design capacities to strength design for the nonstructural bracing.

Conclusions

The procedures for evaluation and rehabilitation of nonstructural components using the current draft of FEMA 356 has been reviewed. A number of inconsistencies in the definitions and within the procedures are identified and corrected.

The Hazards Reduced Performance Level was not well defined or explained. The intent of Hazards Reduced Performance Level has been stated to be the application of Life Safety Performance to a subset of nonstructural components. This would allow for rehabilitation of nonstructural components designated as critical falling hazards without requiring all nonstructural components to be evaluated and rehabilitated. This is thought to represent common practice in which only the heavy falling hazards in public areas are rehabilitated. Revisions to the text and tables in Chapter 1 and Chapter 11 have been recommended to provide consistency in the provisions in Chapter 11 and the definitions and expected performance in Chapter 1.

In the process of the review of the nonstructural provisions, other related issues were identified. The most important of these issues is the acceptance criteria for forces on the bracing for nonstructural components. There are two approaches that could be taken to provide for acceptance of the nonstructural bracing components. One approach would be to use existing allowable stress or strength design values for determining whether the braces are adequate for the applied forces. A second approach would be to develop acceptance values for bracing components that are consistent with the values used for the structural components of the lateral force resisting system.

It appears that the latter approach would be extremely difficult and was not intended to be used. The acceptance criteria has clarified as being on a strength design basis.

The use of strength design capacities to determine the acceptability of bracing and anchorage of nonstructural components represents a simple, straightforward method of checking these items. The primary advantage of this approach is to easily allow the use and evaluation of systems that are traditionally used for nonstructural bracing, such as anchor bolts and small steel framing members. The disadvantage is that there is a fundamental difference in philosophy between the evaluation of structural components and nonstructural components. This difference requires further study to develop nonstructural evaluation criteria that are consistent with the approach used for structural components.

Attached are appendices that contain the recommended changes to the text based on this study. The additions have been underlined and the deletions are shown with strikethrough text. Only Chapters 1 and 11 are included.