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About This Compilation

The SEAOC Blue Book - Seismic Design Recommendations is the premier publication of the SEAOC Seismology Committee. The name “Blue Book” is renowned worldwide amongst engineers, researchers, and building officials. Since 1959, the SEAOC Blue Book has been a prescient publication of earthquake engineering. This new edition of the Blue Book builds upon the tremendous effort of those who have forged earthquake engineering practice via the previous half-century of Blue Book editions. This Blue Book edition was originally unveiled at the 2008 SEAOC Convention, held September 23-27, 2008 at the Fairmont Orchid, Kona, Hawaii and has since been expanded with the addition of three more articles and the update of others for this publication. The previous (seventh) edition of Blue Book - Recommended Lateral Force Requirements and Commentary was published in 1999, and it is available as an online reference.

Much in Seismic Design Recommendations is new relative to the previous edition of the Blue Book, from the title, layout and format of the articles to the general commentary and philosophical discussion of earthquake engineering issues. Seismic Design Recommendations provides insight and discussion of earthquake engineering concepts; interpretations of sometimes ambiguous or conflicting provisions of various codes, standards, and guidelines; and practical guidance on design implementation. This has all developed through countless hours of effort contributed by many SEAOC members, with considerable oversight and contribution from more than 30 SEAOC Seismology Committee members over more than a six-year period. This format was adopted in recognition of the current national code development process and available publications where emphasis is placed on code provision commentary, such as NEHRP Recommended Provisions For New Buildings and Other Structures - Part 2: Commentary (FEMA 450). The Seismology Committee hopes that this publication will invigorate the reader to seek further insight and foresight into earthquake engineering issues and guide their promulgation in building codes and material standards.

The SEAOC Seismology Committee intends to continually develop new articles and update the ones included here based on changes in codes, practice, and research. This is done at the request of our membership and to influence development and refinement of building codes and standards and to indicate needed areas of further research. This June 2009 Compilation should be considered the ephemeral version of these articles, and readers are encouraged to check for updates and later versions of these by contacting SEAOC and accessing the SEAOC Website. An electronic version of the articles is available to SEAOC members as part of their membership privileges.

Most importantly, the online Blue Book will continue to be the vehicle for formalizing and expanding upon positions and opinions of the Association through the work of the SEAOC Seismology Committee.

SEAOC Seismology Committee
June, 2009
Acknowledgments

The 2009 SEAOC Blue Book, *Seismic Design Recommendations*, reflects the work of the 2002 through 2009 SEAOC Seismology Committees, the SEAOC Board, and other SEAOC members who contributed time and effort in one or more capacities as authors, editors, and reviewers. It has built upon the work of all the SEAOC Seismology Committees from 1959 to the present. A list of past and present members of Seismology Committees as well as contributors to the articles published below.

John Diebold, past Chair of the SEAOC Seismology Committee, was the lead individual for the management of much of the process of producing Blue Book articles. Kevin Moore, the immediate past SEAOC Seismology Committee Chair, was instrumental in managing the effort and patiently provided the liaison with the SEAOC Board of Directors. Mehran Pourzanjani the current chair managed the expansion of the document with the addition of more articles and the transition of it from online format to book form. In addition to the work of the contributors to articles and the SEAOC Seismology Committee, the effort to create the original online form of the Blue Book involved the work of two technical editors, David Bonowitz and Robert Reitherman, and the behind-the-scenes work of Tracy Brown, the SEAOC web facilitator.

Dave Bonowitz, the Blue Book technical editor from 2002 through 2005, worked with the SEAOC Board of Directors and SEAOC Seismology Committee to set the direction for the new Blue Book. He worked with the Seismology Committee to determine article topics, contact authors and reviewers, collect the initial drafts of articles, and set up a review process, all of which formed the basis for the online publication.

Tracy Brown, SEAOC’s web facilitator, has worked with the Seismology Committee, technical editors, SEAOC Board, and others to enable the original version of the Blue Book to be placed on-line for access for SEAOC members only. She worked with consultants who set up the security and payment system to enable SEAOC members free access, via downloading pdf files. Tracy has also tracked down an electronic copy of the 1999 Blue Book, which is planned to be placed on-line as a reference document.

Bob Reitherman, the Blue Book technical editor from 2006 to 2008, worked with the SEAOC Seismology Committee to edit and format each article, checked citations of sections of codes or standards, tied up loose ends such as missing references or needed graphic refinement of figures, and identified any ambiguous wording that required further input from the Seismology Committee to ensure that final editorial changes were done accurately. To achieve the Committee’s goal of more widely publicizing the new online series of articles, he has arranged for the ongoing publication of abbreviated versions of select Blue Book articles in *STRUCTURE* magazine.

### ARTICLE CONTRIBUTORS

Altoontash, Arash  
Aschheim, Mark  
Bligh, Ray  
Bonneville, David  
Bonowitz, David  
Cobeen, Kelly  
Cochran, Michael  
Comartin, Craig  
Diebold, John  
Ekwueme, Chukwuma  
Fennel, Andy  
Forslin, Brent  
Gallagher, Ron  
Gates, William R.  
Ghosh, S.K.  
Gillengerten, John  
Goel, Subhash C.  
Hale, Tom H.  
Hamburger, Ronald  
Hanson, James  
Hilmy, Saeed I.  
Hohbach, Doug  
Huang, Y. Henry  
Hudson, Martin  
Islam, Safiel  
Itani, Ahmad  
Johnson, Martin W.  
Kamp, Christopher  
Kariotis, John C.  
Kersting, Ryan  
Khadivi, John H.  
Lai, James S.  
Lawson, John  
Lew, Marshall  
Lopez, Walterio  
Lyons, Robert T.  
Maffei, Joe  
Magee, Doug  
Malhotra, Praveen  
Malley, James  
Maraniani, Peter J.  
Miranda, Eduardo  
Mochizuki, Gary  
Moore, Kevin  
Moore, Mark  
Nelson, Rawn  
Prasad, Badri  
Porush, Alan  
Pourzanjani, Mehran  
Power, Henry C.  
Rodrigues, Nicolas  
Sabelli, Rafael  
Saunders, Mark C.  
Scheel, Norman  
Schmid, Ben  
Staelin, Bill  
Thompson, Douglas  
Thompson, James H.  
Tokas, Chris  
Uriz, Patxi  
Uang, Chia-Ming  
Van Dorpe, Tom  
Zsutty, Theodore  
Zacher, Edwin G.
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1958 – 2003 SEAOC Seismology Committee

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John A. Blume 1957-1959
David R. Bonneville 1993-1997
David Bonowitz 1999-2002
Dominic E. Campi 1995-1997
Raymond W. Clough 1966-1967
R. Gordon Dean 1962
Henry J. Degenkolb 1957-1959
Neville Donovan 1975,1976
Eric Elsesser 1963,1974-1976
Sigmund A. Freeman 1975-1987
Mark Gilligan 1985-1987
Leslie W. Graham 1968
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William T. Holmes 1977-1979
William T. Wheeler 1957-1960
H. Bolton Seed 1975,1976
Roland L. Sharpe 1969,1972-1974
Constantine Shuhaibar 2001-2002
Hudson W. Smith 1976-1979
Sanford Tandowsky 1970-1974
Stanley E. Teixeira 1965-1966
Charles C. Thiel 1987
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**SEAOSC**

Robert Bachman 1989-1995,1994*
Stephenson B. Barnes 1957-1959
Reuben W. Binder 1957-1963
Gregg Brandow 1985-1987
Robert Byrum 1968
Ted Christensen 1982-1984
Leroy L. Crandall 1957-1959
Schafer J. Dixon 1975,1976
John J. Driskell 1967-1968
Murray Erick 1975,1976
Ben Gordon 1968,1969
Norman B. Green 1957-1959
Norman R. Greve 1975-1978
Gary C. Hart 1988
Henry Y. Huang 1998-2005
Saif Hussain 1996-2000
Saiful Islam 1996-2005
Albin W. Johnson 1964*,1965
James Johnson 1977-1979
Roy G. Johnston 1957-1959,
Norman Jones 1967
Tom T. Kamei 1971-1973
James J. Kessler 1966-1968
James S. Lai 1986-1990,1989*
Gerald D. Lehmer 1978-1985
Ed Lindskog 1957-1959
Bob Lyons 1999-2005
Ernst Maag 1957-1959
Herald Omsted 1957-1959
John W. Robb 1974-1980
Thomas A. Sabol 1994-1996
Hassan Sassi 1998-1999
MJ Sidner 1957-1959
John M. Seinbrugge 1957-1959
Hans G. Steinman 1971,1972
Donald R. Strand 1971-1975*
James H. Thompson 1970
Earl H. Vossenkemper 1969,1970
William T. Wheeler 1957-1960
Robert Whitelaw 1985-1987
Robert Wilder 1957-1959

**SEAOC**

R.W. Bradley 1957-1959
Allen H. Brownfield 1957-1959
Walter D. Buehler 1968
David B. Campbell 1985
Joyce Copeland 1997-1998
David A. Crane 1969-1971,1977
Bruce Doig 1987,1988
Herman F. Finch 1957-1963
Charles Greenlaw 1985
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Hoi W. Wong 1978-1979

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Edward A. Hanlon, Jr. 1988,1993
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Kevin P. Lyons 1988
Vilas S. Mujumdar 1991-1993
Alfred L. Parme 1984
Carl Schulze 1994-1998

*Chair
Seismic Design Recommendations is the formal title of the latest SEAOC Blue Book, a publication of the Seismology Committee of the Structural Engineers Association of California known to engineers, researchers, and building officials around the world for over forty years.

This online edition, launched in January 2006, responds to the needs of SEAOC members and long-time Blue Book users. In addition to the new title and the obvious new medium, this edition takes on a new role within the code development process, focused less on the drafting of code provisions and more on their interpretation and implementation. This Blue Book presents the Seismology Committee’s recommendations for the practice of earthquake design. It supplements building codes and standards and, in places, suggests alternative code approaches. It is not intended, however, to replace the building code or to represent a legal standard of care.

The Blue Book’s Role
Since 1959, SEAOC’s Blue Book, formerly titled Recommended Lateral Force Requirements and Commentary, has been at the vanguard of earthquake engineering in California and around the world. This new edition, the eighth overall and first online, looks to extend that tradition.

While much is new, starting with the transition to an online format, the Blue Book will still offer background and commentary on the building code, it will still be the companion document most useful to practicing structural engineers, and it will still be the vehicle for formalizing and explaining positions of the SEAOC Seismology Committee.

Each Blue Book article covers a specific topic, sometimes a particular code provision and sometimes a more general area of practice. Each article is intended to answer the following questions:

• What is the historic importance of the topic, in performance terms? What are the implications of substandard design?

• What is the building code’s approach to the topic? How were specific provisions derived? How is the provision or practice typically implemented? What is the Seismology Committee’s recommended practice?

• How should the provision or standard practice evolve? What short-term code changes does Seismology propose? What long-term studies will be needed?

Not every article can answer every question. Indeed, it is useful to identify where answers are missing. One function of the Blue Book in its new role is to spot unresolved issues and thereby motivate new research.

Historic contribution. From its first edition in 1959 through the 1970s, the Blue Book was the de facto precursor of UBC earthquake provisions used throughout the western United States and in many places overseas (Porush and Zacher, 1987). There was, in essence, a direct pipeline from the SEAOC Seismology Committee to the building code. The early history of the Blue Book and its predecessors is covered briefly in article 01.02.010 and in greater depth by Strand (1984) and Holmes (1998).

Even with the publication of ATC 3-06 (1978), SEAOC retained its control of actual code provisions. The landmark ATC document built on the Blue Book to thoroughly revise and rationalize earthquake-resistant design. In turn, Seismology took ATC 3-06 and with an eight-year effort produced the provisions and commentary that would become both the 1988 Blue Book and the 1988 UBC (Porush and Zacher, 1987).

The 1988 UBC marked a new generation of earthquake design codes, and the Blue Book was its indispensable companion, explaining the existing code provisions even as it proposed new ones. Picking up where ATC 3-06 left off, the 1990, 1996, and 1999 editions linked the code’s provisions to past judgment and to contemporary research. (A close review of Blue Books, NEHRP Provisions, and ATC documents through the 1980s and 1990s finds nearly identical text in several key passages. The documents’ drafting committees, perhaps due to overlapping memberships, were clearly happy to borrow from and share with each other in the service of common goals.)
Now, a new generation of building codes again demands a commentary that keeps pace. This time, however, there are plenty of candidates. Consider the available resources, almost none of which existed when the Blue Book underwent its last major overhaul in the 1980s:

- The NEHRP Provisions and Commentary (BSSC, 2001a and 2001b) are the unofficial resource documents for ASCE 7-02 (which itself has a small but useful commentary).

- Material standards such as ACI 318 and AISC 341 now provide their own commentaries.

- Organizations such as ATC, CUREE, EERI, and FEMA (not to mention SEAOC and its local sections) regularly hold workshops and conferences devoted to earthquake engineering. For example, CUREE (1998) produced a four-volume Proceedings of papers on just the 1994 Northridge earthquake.

- Engineering journals carry more papers about earthquake-resistant design than ever. *Earthquake Spectra* (2000) even produced a theme issue devoted to recent codes and standards.

- National and regional research centers such as EERC and NCEER regularly produce bound reports, as do leading university labs.

- At least three massive earthquake-related structural design “handbooks” have recently been completed (Naeim, 2001; Chen and Scawthorn, 2002; Bozorgnia and Bertero, 2004). All were written and edited by prominent SEAOC members. In addition, there are now numerous “code companion” books for sale, not to mention textbooks devoted to seismology and geotechnical earthquake engineering.

- Even the notes and seminars produced by ICC (formerly ICBO), including SEAOC’s Design Manuals, compete in some way with the Blue Book’s historic role as explainer and interpreter of the code.

**A new role.** Does this mean the Blue Book is no longer necessary? On the contrary, the volume of material now available to engineers makes a practical, independent perspective more valuable and useful than ever. The Blue Book merely needs to embrace a new role, concerned less with writing code provisions and more with improving actual engineering practice. The Seismology Committee remains involved with code development, of course, principally by placing representatives as members or observers of official code-writing committees. But in large part, the Committee and the Blue Book now function as reviewers and contributors to the code development process, not as principal authors. (See article 01.02.010 for a discussion of the current code development process.)

In its new role, the Blue Book no longer needs to bridge the gap between research and code; NEHRP and the new material standards do that. Rather, the Blue Book’s job is to bridge between code and practice. This emphasis will show itself in subtle ways. For one thing, much of the hardcore technical background can now be left to others. The Blue Book’s role is no longer to justify a code provision but to assess its impact on practice and to confirm whether it is supported by relevant analysis, testing, experience, or judgment.

The article format also responds to this new emphasis. It is no longer necessary for the Blue Book to restate and comment on code provisions section by section. The new role calls for a broader view—broader historically and broader technically—that examines the code in the context of performance and explains its requirements in ways that foster good design. Thus, the new Blue Book will have, for example, an article about the concept of redundancy, not just a commentary on the related code section; one article about out-of-plane wall anchors, not multiple commentaries to match the code’s scattered provisions; and an article about parking structures as a class of buildings that gets no attention as such from the code. While the Blue Book will still propose new code language where appropriate, its primary purpose will be to guide earthquake design practice by illuminating the code with history and context.

A third contribution by the new Blue Book will be to speed the adoption of new ideas by local engineers and code officials. The Seismology committee can approve a position and present it online faster than the new standards process can approve it for the code. When the Blue Book was the main resource document for the UBC, it was at
most three years between Seismology Committee endorsement and, assuming ICBO approval, publication in the code. Now, the lag between a new idea and an enforceable code provision (barring emergencies) is considerably longer. As discussed further in article 01.02.010, California code provisions that will be used in 2010 will have been based on proposals made eight years earlier and already balloted by early 2004. If the code is eight years away, endorsement of a new finding or interpretation in the Blue Book as a Seismology Committee position can be valuable to SEAOC members.

**Seismology Committee Positions**

Despite its reduced role in the code development process, the Blue Book will continue as the publication that formalizes positions of the SEAOC Seismology Committee and thereby represents an independent and authoritative statewide consensus.

Seismology positions can be code change proposals, code interpretations, recommended practices, accepted alternatives, or voting positions on proposals put forward by other organizations. Positions are taken regularly by the Committee and are documented in Committee position statements, FAQ responses, correspondence (usually to code officials), or simply in meeting minutes. (See also the Committee’s web page: http://www.seaoc.org/Pages/committees/seismology_post.html.) As in the past, Committee positions will be incorporated into the Blue Book. Now, however, a specific article can be revised and posted independent of other articles.

Seismology Committee positions form the basis of SEAOC’s proposals and public comments on emerging codes and standards. These positions have been influential in the past because the Blue Book was acknowledged as a reliable predictor of the next code. Even with the shift to national codes and standards, however, the Seismology Committee is still routinely asked to resolve questions of code interpretation. Indeed, with the new national standards process (and the attendant national markets), a Seismology Committee position might carry new significance for engineers and code officials in California and the western United States.

A Seismology Committee position is:

- The consensus of a 12-person SEAOC committee, as opposed to the view of an individual engineer or researcher as might be presented in a journal, handbook, seminar, or trade publication
- The recommendation of practitioners, as opposed to the proposal of a vendor who might be perceived as having a conflict of interest
- Based on “California practice,” as opposed to national multi-disciplinary consensus.

The idea of a California practice in excess of minimal code requirements predates the first Blue Book. What former Seismology Committee member Henry Degenkolb meant by the term was an understanding of earthquake effects that code equations of the time could not convey, for example, running bottom bars through the joints of a concrete frame and generally tying the structure together (EERI, 1994, p. 33). But with ATC 3-06 and the coming of national codes, Degenkolb and others recognized a watering down of California practice for purposes of achieving national consensus (p. 147 ff.).

Today, the national standards resolve regional concerns by exempting low seismic zones from most detailing requirements. But there is still room for the Blue Book to improve on the code for local conditions. California practice can still mean prudent measures beyond the letter of the code, like bracketed analysis, site-specific geotechnical study, or tighter quality control. It can also mean attention to those principles that the code still has trouble quantifying, like regularity, reliability, and capacity design.

**The New Blue Book**

The principal changes relative to the 1999 seventh edition (Seismology, 1999) are:

- New title and role. As proposed building code provisions, past Blue Books bridged the gap between research and regulation, between engineering theory and its codification. The new *Seismic Design Recommendations*, no
longer focused on code development, will begin to emphasize the proper implementation of code provisions in practice.

• New base code. The Blue Book commentary will be based primarily on ASCE 7, the loading standard that will form the basis of future earthquake design codes in California and, eventually, much of the world. Users of the IBC or the NEHRP Provisions (BSSC, 2001a) will recognize the ASCE 7 requirements. Users of the UBC or the 2001 CBC might not. To help ease the transition, and because the 2001 CBC will still be in effect into 2007, Blue Book articles will also refer to UBC/CBC provisions and will in many places compare them with ASCE 7.

• New material standards. The days of the single-volume building code are past. Now, model codes for earthquake design (including the 2003 IBC and NFPA 5000) refer to outside standards for loads (ASCE 7) and for each material (see article 00.02.020). Blue Book articles will introduce and comment on the new standards.

• New content and references. Since the 1999 Blue Book, substantial research programs have been finalized and their findings packaged for use by code-writers and designers. Blue Book articles will look at how this new work (by the SAC Joint Venture, the CUREE/Caltech Woodframe Project, and others) has been, or should be, incorporated into new codes.

• New format. The Blue Book will no longer offer a full set of recommended code provisions. This is a necessary concession to the new national code-writing process (see article 01.02.010). With no formal set of provisions, new Blue Book articles are free from the rigidly numbered and often confusingly ordered format of a code. Instead, material will be arranged in self-contained articles on circumscribed topics.

• New medium. In early 2004, the Seismology Committee and the SEAOC Board of Directors decided to convert the traditionally bound Blue Book into an online journal of earthquake-related structural engineering. Articles will be formatted so that they may still be collected into a bound volume (or a compact disc), but the primary “delivery mode” will be as individual articles in portable document format (pdf) posted on a public website. As always, the Blue Book will be free to SEAOC members and available for sale to non-members. Internet-based search functions will allow users to look for articles on specific topics or code sections.

The decision to go online, as well as the switch to an article format, responds to the needs of SEAOC members and Blue Book users. It offers more flexibility in the publication schedule, allows quicker updates to specific sections, and facilitates user feedback. Perhaps most important, the online format helps the Blue Book keep pace with overlaps and near-constant revisions to codes and material standards developed on different schedules.

Production. Work on the Blue Book’s eighth edition began in early 2003 with an outline of article topics that mirrored the code’s organization but broke from its rigid format. With about 80 distinct articles identified, the Committee selected individual experts to serve as lead writers and reviewers, as well as a technical editor to ensure consistency of tone and scope. Since the Blue Book is ultimately the position of the Committee and not of any individual, approval of each article still requires a vote of the full Seismology Committee.

The Committee determined that new Blue Book articles would be based on the 2002 version of ASCE 7 and would compare that standard with the 2001 CBC (and 1997 UBC). Meanwhile, the 2003 NEHRP Provisions were in balloting, ASCE 7-05 was well into development, and California was preparing to decide on the model code that would form the basis of the 2004 CBC. In this environment, the Committee recognized that an online distribution would provide essential flexibility and would better serve SEAOC’s members than the traditional bound format, parts of which were certain to be obsolete soon after publication. Conversion to the online format thus began in early 2004. Initial postings of articles reference ASCE 7-02, with later postings of articles referencing ASCE 7-05.

Each new article began as a compilation of past Blue Book commentary related to a single topic. That commentary has now been enhanced by adding newer references and by clarifying past interpretations in light of new code provisions. Most of the past material has also been rewritten so that each article can now stand on its own, apart from a section-by-section commentary on the code.
Because of the reformat, not every bit of information from the 1999 Blue Book will be restated in the new articles. Also, in order to provide a regular publication schedule, some topics will not be covered in the online edition for some time. On the other hand, the flexibility of the journal model allows publication of timely articles without waiting for completion of an entire edition.

**Organization.** While Blue Book articles will be numbered, their organization is not intended to match the sequence of any particular code. Further, the flexibility afforded by online distribution makes it unnecessary to publish articles in sequence. Nevertheless, the outline is intended to move broadly from the general to the specific, and the numbering scheme is expected to support future revision and expansion.

Each article will have a seven-digit number. The first two digits will signify one of 15 broad subjects – what would be the chapters of a bound publication. The other five digits will identify subtopics. The main sections:

- 01. Philosophies of earthquake design
- 02. Quality assurance
- 03. Ground motion characterization
- 04. Seismic force resisting systems
  - 04.01 Characterization
  - 04.02 Basic seismic force-resisting systems
  - 04.03 Structure configuration and response
  - 04.04 Structural modeling
  - 04.05 Structural analysis procedures
- 05. Element checks and design
- 06. Nonstructural components
- 07. Foundations
- 08. Steel
- 09. Concrete
- 10. Composite structures
- 11. Masonry
- 12. Wood & light-frame
- 13. Seismic isolation
- 14. Energy dissipation devices
- 15. Non-building structures

Approved for publication by the Seismology Committee, February 2005

**References**


**Keywords**

preface  
online  
blue book

**How To Cite This Publication**

Articles in the SEAOC Blue Book series should be cited as follows:

In the writer’s text, the article should be cited as:

(SeaOC Seismology Committee 2006)

In the writer’s reference list, the reference should be listed as:

**Glossary of Acronyms**

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANSI</td>
<td>American National Standards Institute.</td>
</tr>
<tr>
<td>BOCA</td>
<td>Building Officials and Code Administrators International, Inc. One of three statutory members of the ICC. Producer of the National Building Code (NBC).</td>
</tr>
<tr>
<td>BSSC</td>
<td>Building Seismic Safety Council. Producer, for FEMA, of the NEHRP Provisions and Commentary. For more information, see BSSC (2001b, p.431-444).</td>
</tr>
<tr>
<td>IBC</td>
<td>International Building Code. Published as a model code by ICC since 2000.</td>
</tr>
<tr>
<td>ICBO</td>
<td>International Conference of Building Officials. One of three statutory members of the ICC. Producer of the UBC.</td>
</tr>
<tr>
<td>ICC</td>
<td>International Code Council. Formed from three statutory members: BOCA, ICBO, and SBCCI. Producer of the IBC.</td>
</tr>
<tr>
<td>NCSEA</td>
<td>National Council of Structural Engineers Associations, of which SEAOC is one.</td>
</tr>
<tr>
<td>NEHRP</td>
<td>National Earthquake Hazards Reduction Program, a federally-funded program under which the NEHRP Provisions and Commentary (BSSC, 2001a; 2001b) are produced. The NEHRP Provisions and Commentary have been produced by BSSC for FEMA since 1985 as recommended design provisions and as a resource for building code development. The 2000 provisions form the basis of the earthquake design provisions in ASCE 7-02. For more information about NEHRP, see BSSC (2001b, p.431-444).</td>
</tr>
<tr>
<td>STC</td>
<td>Seismic Task Committee. A technical task committee of the ASCE 7 standards committee and developer of earthquake design provisions for ASCE 7.</td>
</tr>
<tr>
<td>UBC</td>
<td>Uniform Building Code. Published as a model code through 1997 by ICBO. Since 1961, its earthquake design provisions were adapted from the SEAOC Seismology Committee’s Blue Book.</td>
</tr>
</tbody>
</table>
**Keywords**
glossary
acronyms

**How To Cite This Publication**
Articles in the SEAOC Blue Book series should be cited as follows:

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2006)

In the writer’s reference list, the reference should be listed as:

Recent and current codes and standards


ASCE (2004). Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-05 Pre-publication copy for final review and editing), Structural Engineering Institute of the American Society of Civil Engineers.


Keywords
ASCE 7, ATC 3-06, Blue Book, BSSC, Building code, CBC, IBC, ICBO, NEHRP Provisions, Seismology Committee, Standards, UBC

Introduction
Earthquake design provisions have changed rapidly and substantially since 1980. So has the process by which they are developed and codified. What was once the franchise of SEAOC and, in particular, the Seismology Committee is now a national effort with rigorous procedures for consensus building and balloting. In this new environment, industry associations and academics play more significant roles than in the past. While individual practicing structural engineers are still essential participants in the code development process, the roles of code officials and professional associations like SEAOC have been reduced. The result is a building code that might become more stable over time and more uniform between jurisdictions, but also one that might lose its adaptability to local conditions and custom. Some also anticipate that new design codes will become even more complex and difficult to apply. This article describes the old code development process and the new one, as well as the organizations and publications that motivated the transition.

California Building Code status.
The California Building Code sets minimum requirements for all California jurisdictions. The 2001 edition, which is still in use as of mid-2005, continues to use the 1997 UBC as its model code. The Seismology Committee supports the selection of a new model code because the 1997 UBC is out of date. In 2003, the California Building Standards Commission selected NFPA 5000 as its model code. In March of 2005, the CBSC rescinded that decision (CBSC, 2005), opening the door for adoption of the IBC as the model code for the CBC. The Seismology Committee has taken a position in support of the most current IBC as the model code for the CBC.

The UBC and the Blue Book
Since at least the middle of the eighteenth century, communities around the world have anticipated and responded to damaging earthquakes by adopting special requirements for building design and construction. Equivalent lateral force procedures already in use in many countries by the early 1920s are examples of modern design provisions, which can be said to be marked by rational approximations of earthquake effects as structural loads. More on early code approaches can be found in Geschwind (2001), Holmes (1998), Olson (2003), Strand (1984), and Tobriner (1984).

In 1928, the Pacific Coast Building Officials (an organization of code officials) published the first edition of the Uniform Building Code (PCBO, 1928). In a non-mandatory appendix, following an approach adopted in Japan and incorporating lessons from the 1906 San Francisco earthquake, the 1927 UBC recommended that structures designed for seismic resistance should have the strength to resist a total lateral force proportional to the building weight. The design lateral forces were to be applied at specific floor levels in each of two orthogonal directions. In addition, the appendix recommended that each structure be firmly bonded and tied together, to assure that it acts as a unit.

For more than sixty years following the publication of the 1927 UBC, California structural engineers, working principally as volunteers through the SEAOC Seismology Committee, led international efforts to develop seismic provisions for building codes. These efforts began formally in the 1940s with separate efforts in northern and southern California (EERI, 1994, p.134-5). In 1959, SEAOC published its Recommended Lateral Force Requirements – the first Blue Book – as a joint statewide effort with design procedures that mirrored those in the contemporary UBC; in 1960 came a commentary on preferred seismic design practice (Seismology, 1959; 1960).

Other than prohibitions against the use of unreinforced masonry and requirements for anchoring wood frame construction to foundations, early editions of the Uniform Building Code had few detailing provisions. For the 1967 UBC, however, the Seismology Committee introduced ductile detailing requirements for reinforced concrete frames based on pioneering work by John Blume, a prominent SEAOC member (Blume et al., 1961). Over the next thirty years, the Committee championed a succession of similar enhancements. Table 1 lists some of the criteria and detailing requirements introduced into building codes during this period in response to observed earthquake
Development of Earthquake Design Provisions for Building Codes

performance. Many other important changes were based on research by universities, the U.S. Geological Survey, and individual practicing engineers. (Additional background on some of these changes is given in individual Blue Book articles and in references such as Degenkolb, 1986.)

By the mid-1980s, the Blue Book and the UBC provisions based on it were recognized around the world as leading references for the design of earthquake-resistant buildings. In the United States, seismic design requirements in the ANSI A58.1 standard (National Bureau of Standards, 1982), the forerunner of ASCE 7, as well as those in codes published by BOCA and SBCCI (see the glossary below) were based on Blue Book recommendations.

Table 1. Recent west coast earthquakes and building code provisions they motivated

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>UBC Edition</th>
<th>Enhancement</th>
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<tbody>
<tr>
<td>1971 San Fernando</td>
<td>1973</td>
<td>Direct positive anchorage of masonry and concrete walls to diaphragms</td>
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<tr>
<td></td>
<td>1976</td>
<td>Seismic Zone 4, with increased base shear requirements</td>
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<td></td>
<td></td>
<td>Base shear dependence on site conditions through coefficient $S$</td>
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<tr>
<td></td>
<td></td>
<td>Occupancy Importance Factor $I$ for certain buildings</td>
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<tr>
<td></td>
<td></td>
<td>Interconnection of individual column foundations</td>
</tr>
<tr>
<td>1979 Imperial Valley</td>
<td>1985</td>
<td>Diaphragm continuity ties</td>
</tr>
<tr>
<td>1985 Mexico City</td>
<td>1988</td>
<td>Requirements for columns supporting discontinuous walls</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Separation of buildings to avoid pounding</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Design of steel columns for maximum axial forces</td>
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<tr>
<td></td>
<td></td>
<td>Restrictions for irregular structures</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ductile detailing of perimeter frames</td>
</tr>
<tr>
<td>1987 Whittier Narrows</td>
<td>1991</td>
<td>Revisions to site coefficients</td>
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<td></td>
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<td>Revision to spectral shape</td>
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<td></td>
<td></td>
<td>Increased wall anchorage forces for flexible diaphragm buildings</td>
</tr>
<tr>
<td>1989 Loma Prieta</td>
<td>1991</td>
<td>Increased restrictions on chevron-braced frames</td>
</tr>
<tr>
<td></td>
<td>1994</td>
<td>Limitations on $b/t$ ratios for braced frames</td>
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<tr>
<td>1994 Northridge</td>
<td>1997</td>
<td>Ductile detailing of piles</td>
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<tr>
<td></td>
<td></td>
<td>Restrictions on use of battered piles</td>
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<tr>
<td></td>
<td></td>
<td>Requirements to consider liquefaction</td>
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<tr>
<td></td>
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<td>Near-fault zones and corresponding base shear requirements</td>
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<td></td>
<td></td>
<td>Revised base shear equations using $1/T$ spectral shape</td>
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<td>Redundancy requirements</td>
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<td></td>
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<td>Design of collectors for overstrength</td>
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<td></td>
<td></td>
<td>Increase in wall anchorage requirements</td>
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<tr>
<td></td>
<td></td>
<td>More realistic evaluation of design drift</td>
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<td></td>
<td></td>
<td>Steel moment connection verification by test</td>
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</tbody>
</table>

The UBC development process. The UBC, published triennially (with interim supplements) by ICBO, was a model code adopted by local jurisdictions throughout the western U.S., including California and its major cities. For each code cycle, ICBO received proposals for changes and published them in a monograph for public review. Any individual or organization was eligible to submit proposals. ICBO committees, composed of ICBO-member building officials, then held public hearings before voting to reject or accept each proposal.

Building on its historic role, the Seismology Committee was an active participant in this process. One or more Committee representatives typically attended the ICBO hearings to speak in support of SEAOC proposals and to provide a Seismology opinion on changes proposed by others. Often, the Seismology Committee would meet with proponents of competing proposals to negotiate a compromise position prior to the code hearings.
SEAOC’s recommended code change proposals, especially those regarding earthquake design provisions, were usually accepted by ICBO. In some code cycles, notably 1988 and 1997, ICBO accepted SEAOC proposals that amounted to essentially complete rewrites of existing code chapters. Thus, while SEAOC was not the only organization contributing to the UBC seismic provisions, it did have a dominant influence, and the Blue Book, which formalized Seismology Committee positions and interpretations, was widely viewed as a preliminary version of the coming seismic code. This unofficial (though reliable) pipeline from the Seismology Committee to the UBC gave SEAOC members unusually direct access to the code development process. It also gave the Seismology Committee the responsibility for justifying, explaining, and interpreting the code’s eventual provisions – even those with which the Committee disagreed. Generally, the Blue Book was the vehicle by which the Committee discharged its important obligation, and its Lateral Force Requirements and Commentary would come to be read essentially as a commentary on the UBC.

Despite their wide acceptance, the Blue Book’s recommended provisions were developed in a relatively closed process, with relatively few participants. As now, the Seismology Committee was composed of voting delegates from SEAOC’s four local associations (Central California, Northern California, San Diego, and Southern California). Ideas for code changes usually came to the Committee through one of the local associations and were sometimes assigned to subcommittees for development prior to Committee deliberation. Regardless of the origin of a particular proposal, it was the longstanding practice of the Committee to propose only those code changes that had the consensus support of the statewide Committee.

In addition to the voting delegates, Seismology Committee meetings were (and still are) frequently attended by observers from other SEAs (especially Oregon and Washington), by ICBO members and other code officials, by representatives of the construction industry and trade organizations, by academics and researchers, and by individual SEAOC members with interests in the Committee’s positions. These observers, whose contributions are always welcome, often introduced persuasive information and arguments and were instrumental in crafting proposed code language. Nevertheless, there was no formal process for open review, comment, and response with respect to Seismology Committee positions.

Nationalization: NEHRP and the IBC

Although the UBC included seismic design parameters for the entire United States and had been used outside California and even overseas, some viewed the UBC as a west coast document not suitable for national adoption. Increased interest in interstate and national design and construction argued for a single set of provisions that could be enforced nationwide. New national codes, standards, and recommended provisions would eventually lead to new roles for both SEAOC and the Seismology Committee.

NEHRP Recommended Provisions. Today, the NEHRP Provisions and Commentary (BSSC, 2001a; 2001b) are the primary resources for seismic provisions in national codes and standards. They trace their development directly to the landmark document known as ATC 3-06 (ATC, 1978), which, like the Blue Book, came about largely through the efforts of SEAOC and its members.

The 1971 San Fernando earthquake convinced leading engineers that the UBC provisions, which dated back to the 1959 Blue Book and its precursors, were due for substantial updating, and that such a comprehensive overhaul would need to be done outside the regular code cycle. SEAOC thus created the Applied Technology Council as an independent non-profit organization to seek funding for and to carry out structural engineering research aimed at improving design practice and codes. ATC 3-06, published in 1978 with funding by the National Science Foundation and the National Bureau of Standards (now NIST), recommended extensive changes and improvements relative to the UBC and the Blue Book. The Seismology Committee participated in the four-year ATC effort as an independent reviewer; many ATC participants were also SEAOC and past Seismology Committee members. The substantial changes recommended by ATC 3-06 would eventually be incorporated into the 1988 UBC and the 1988-90 (Fifth Edition) Blue Book (Porush and Zacher, 1987).

The Building Seismic Safety Council was established under the auspices of the National Institute of Building Sciences to establish national consensus with respect to ATC 3-06 and to modify it as needed for incorporation into
model building codes nationwide. The modified document became the first NEHRP Provisions in 1985. Since then, the NEHRP Provisions have been revised and updated on a triennial cycle, with changes approved by a series of rigorous consensus balloting processes, first within technical subcommittees, then at the level of the Provisions Update Committee (a coordinating oversight body), and finally through the BSSC member organizations. (The BSSC committees that maintain the NEHRP Provisions and Commentary are composed of about 100 volunteer members with voting privileges and are reconstituted for each revision cycle. BSSC itself comprises about 65 member organizations, including SEAOC, each of the local SEAOC sections, and several other state SEAs. Additional background on BSSC is found in the 2000 Commentary (BSSC, 2001b, p. 431 ff.).)

The NEHRP Provisions were incorporated into model building codes for the first time in 1993, when both BOCA and SBCCI adopted seismic provisions (for the NBC and SBC, respectively) based on the 1991 NEHRP Provisions. This adoption was motivated in part by a 1990 Executive Order (Bush, 1990) requiring new buildings housing federal agencies to comply with the NEHRP Provisions. Also in 1993, ASCE 7 was revised to include seismic provisions matching those of NEHRP.

ICBO also considered adopting NEHRP-based seismic provisions for the 1994 UBC but stayed with its own provisions after the Interagency Committee for Seismic Safety Coordination (a joint committee of federal agencies) found that those provisions (which had been recommended by the Seismology Committee and based on ATC 3-06), provided safety equivalent to NEHRP (Melvyn Green & Associates, 1995).

As discussed below, model building codes no longer print most of their own technical provisions; instead, they adopt their provisions by reference to approved consensus standards developed by other organizations. As a result, future NEHRP Provisions are not likely to have a direct influence on building codes. Rather, they are expected to be a means by which new technologies and procedures can be introduced to the structural engineering community. In addition, the previously cited Executive Order empowers the NEHRP Provisions as a check on the technical content of standards developed by industry groups. This gatekeeper function will be exercised in part through the adoption or modification of national standards in the Provisions and in part through the advocacy efforts of the Code Resource Support Committee (CRSC), a federally supported subcommittee of BSSC’s Provisions Update Committee charged with coordinating and advising the model code development organizations on seismic policy (Mahoney and Sheckler, 2001).

The International Building Code. In 1995, BOCA, ICBO, and SBCCI agreed to coalesce into a single model building code development organization to be known as the International Code Council with the intent of publishing a single nationally applicable building code. Until then, their three model codes were each dominant in certain regions, the UBC being the model code in California and most of the western U.S. Each organization agreed to publish its last edition in the late 1990s and to focus future efforts on the joint publication, in 2000, of the first International Building Code.

In preparation for the 2000 IBC, SEAOC, BSSC, and ICC each took steps to align, or “converge,” the UBC and the NEHRP Provisions in order to provide a uniform resource for the new code. BSSC would introduce into the 1997 NEHRP those provisions (some borrowed from the Blue Book) that west coast engineers felt were imperative in the seismically active western U.S. as well as lessons from the 1994 Northridge and 1995 Kobe earthquakes. At the same time, SEAOC’s Seismology and Code Committees would revise the Blue Book and propose code changes to provide UBC users with a transition to NEHRP-based codes and strength design standards (Bachman, 1995; Cobeen, 1995). This effort resulted in the major revisions to earthquake design provisions in the 1997 UBC, its final edition. (For more on IBC precursors, see Ghosh and Chittenden, 2001, and Hamburger and Kircher, 2000.)

Current Model Codes: IBC and NFPA

Most U.S. jurisdictions establish building codes by adopting (and sometimes modifying) a model code. Large cities, including Los Angeles and San Francisco, are historical exceptions, but even they today rely on model codes for their basic provisions. Prior to the IBC, the most commonly used model codes were the UBC, NBC, and SBC, produced, respectively, by ICBO, BOCA, and SBCCI (see the glossary below).
After the efforts of the 1990s to nationalize building codes, the leading model codes are the IBC and NFPA 5000. The IBC is published on a three-year cycle with interim amendments. As with the UBC before it, the revision process is open, with decisions made initially by the vote of ICC committee members at public hearings and ultimately ratified by the full ICC membership. (For more on ICC procedures and its development process, see the ICC website: www.iccsafe.org.) While any individual or organization may propose a change to the IBC, and while acceptance of a proposal is a function primarily of its technical merits, it often benefits a proposal to have the support of organizations with national stature, such as ASCE or the CRSC. Thus, organizations active in code development, including SEAOC, frequently build coalitions and develop compromise proposals prior to the code hearings. The Seismology Committee submits its code change proposals primarily with or through NCSEA in order to gain wider support.

NFPA (see the glossary) announced its intent to publish an alternative to the IBC as the 2000 IBC was nearing completion; its 2003 edition of NFPA 5000 was published in 2002. NFPA 5000 cites many of the same reference documents as the 2000 IBC, but its technical provisions do differ in a few significant aspects. Examples include specific provisions for testing and inspection, foundation design, and conventional light-frame construction, as well as general procedures for adoption and modification of referenced material standards (OSHPD, 2003).

The NFPA 5000 development process is also somewhat different from the IBC’s, as it relies on ANSI-compliant balloting to achieve national consensus. Some engineers feel that such a development process, while valuable for certain materials or design issues, might not be best for earthquake design provisions, which vary in importance between different parts of the country. Several reasons are given:

- In NFPA’s ANSI process, code provisions are developed by technical committees appointed by NFPA, each of which must include specified numbers of representatives from consumers’ groups, suppliers, and regulators. By contrast, the ICC committee responsible for updating the IBC consists mostly of code officials. Therefore, in the view of the Seismology Committee, the ICC committees are more accountable to the public and better represent public interests.

- In the ANSI process, any committee member voting against a change proposal must provide a substantive explanation of his or her vote. Also, ANSI-compliant committees must meet geographic and disciplinary balance requirements, which lead to committees with members who are not experts in earthquake design. In the view of the Seismology Committee, the combination of non-experts and strict procedures for negative votes might allow a proposal to be approved without appropriate scrutiny.

- In the ANSI process, change proposals originate within the committee. After the committee reaches consensus, the proposals are made available for public review and comments, which the committee must resolve. In the ICC process, by contrast, change proposals initiate from outside the committee, and public input, including that of professional organizations like SEAOC, is solicited before the committee votes. While NFPA committees do include representatives from NCSEA, the Seismology Committee believes the ICC process to be more conducive to input (and rebuttal) from SEAs, from individual engineers, and from the public.

The next CBC is expected to be based on the IBC. With either new model code, however, engineers used to the 2001 CBC and its UBC basis will face many new code provisions when the update occurs. These will involve new concepts such as seismic design categories and completely revised earthquake ground motion parameters based on national hazard maps, not seismic zones. These new concepts are presented in separate Blue Book articles.

**Standards**

For purposes of describing building code development, a standard is a document produced in accordance with ANSI rules for committee representation, balloting, consensus building, etc. Past model codes and guidelines, including the UBC, the NEHRP Provisions, and even the 2003 IBC, are not themselves standards in this sense. All the model codes do, however, adopt standards developed by other organizations. For example, both the 2003 IBC and the 2003 NFPA 5000 adopt their structural loading criteria, including earthquake design provisions, by reference to the 2002 edition of the standard known as ASCE 7. Similarly, material design provisions are adopted by reference to
standards produced by such organizations as ACI and AISC. The model code thus becomes more like a shell document, with each key section referring in turn to a separate standard.

This contrasts with past model codes, which were self-contained documents. Even when past model codes would adopt provisions authored by others (such as those of ACI or even those of the Blue Book), they would reproduce those provisions in their entirety within the model code volume. The new approach of adopting by reference has little effect on the actual design criteria; it does, however, require users to consult multiple documents instead of a single code volume.

There is no law that requires the use of standards in model building codes. Indeed, as discussed above, a national standards process might be inappropriate for some design provisions. In general, however, standards bring a necessary division of labor to the code development process. They also provide a measure of uniformity and comparability between various model codes. Material standards are further expected to ensure the quality of building products and to create a “level playing field” in competitive markets (Henry and Johnson, 2003). On the other hand, to the extent that the adoption of standards represents deference to the expertise of standard-writing committees, model code organizations will prefer adoption of standards without modifications. This affects the access of individuals and organizations to the code development process. That is, instead of proposing changes to the model code, interested parties now need to propose changes to the particular standard at issue and therefore need to be familiar with the development schedules and procedures of several standard-writing groups.

Implications of the New Process
Development of earthquake design provisions for building codes has evolved substantially since the first UBC and even since its final edition in 1997. Nationalization and standardization, which were expected to minimize differences between model codes and across jurisdictions, have led to a complex process of linked and sometimes competing documents developed through near-constant and sometimes overlapping revision cycles. Once, the Blue Book could reasonably be viewed as a preview of the next UBC’s earthquake provisions. Now, a new code development process means new roles for engineers’ organizations such as the Seismology Committee, SEAOC, NCSEA, and even the NEHRP Technical Subcommittees.

Figure 1 illustrates the new process and the Seismology Committee’s new role. In the figure, a solid line indicates direct or near-direct adoption of one document by another, and a dashed line indicates input into the development process. The figure is necessarily incomplete with respect to the parties involved; for example, it does not show the many other groups that contribute to and influence the development process, including industry groups, other professional associations, and NCSEA (through which Seismology makes many of its proposals). The figure indicates the way in which a local building code (for example, the California Building Code) is based on a model code, which itself is largely a collection of references to separate standards for loading (ASCE 7) and material and system design (AISC 341, etc.).

Figure 1 also shows how the Blue Book is no longer the source document for any code or standard. Rather, it is the repository of Seismology Committee positions and the source of the Committee’s proposals to NEHRP, ICC, NFPA, and the standard-writing groups.

Figure 1 represents in a rough way the development of current model codes such as the 2003 and 2006 editions of the IBC. This process, however, is still relatively new. A meaningful measure of its success will require several cycles. Nevertheless, the Seismology Committee anticipates that the new process could impact structural engineering practice in several ways:

- Adoption of national design standards by reference might eventually simplify practice by minimizing differences between competing model codes. Through 2004, however, that promise has not been fulfilled, as the IBC and NFPA 5000 make different adoptions with different modifications (OSHPD, 2003).

- Adoption of national standards by reference might eventually simplify practice for engineers working in multiple jurisdictions, reducing the likelihood of oversight or omission due to unfamiliar local codes. Through
2004, however, that promise, too, remains unfulfilled. The 2001 CBC adopts a number of material standards and itself represents a baseline for California jurisdictions, yet California cities and counties continue to adopt local modifications. When the state eventually adopts one of the national model codes, it is also expected to make modifications to reflect established practices (especially for special occupancies), assuring that earthquake design in California will still differ from design in other states using the same model code.

- Adoption of standards by reference will complicate practice for engineers used to working with a self-contained code such as the UBC. Engineers will perhaps need to consult and coordinate the provisions of several documents for even a relatively straightforward design.

- Despite its emphasis on national standards, the new code development process has not yet developed criteria by which to quantify new seismic force resisting systems. In their absence, this task is sometimes performed by fee-based evaluation services that allow proponents to establish their own acceptability criteria. In the view of the Seismology Committee, these practices negate the benefits of the standards process. It is therefore the position of the Seismology Committee that systems not yet codified, regardless of the findings of evaluation services, should only be used if they comply with code provisions for undefined systems.

- Nationalization of the code might reduce the number of large changes from one edition to the next. Since the 1971 San Fernando earthquake, new earthquake design provisions have been introduced in each three-year cycle, and the rapid evolution has complicated engineering practice. Some expect the ANSI process for developing national reference standards to slow the rate of change. However, that was not the case with the development of the 2003 NEHRP Provisions or with ASCE 7-05, as both documents considered long lists of change proposals.

- Emergency revisions and amendments may still be enforced by local California jurisdictions for cause, but it is unclear how such measures will be handled by national organizations.
The trend toward building codes as placeholders for other reference standards will mean a longer lag between a code change proposal and an eventual code provision. The implications of such a lag on design quality are not yet known. As Figure 1 illustrates schematically, if the development schedules of the standards, the model codes, and their resource documents (such as the NEHRP Provisions) are not carefully coordinated, the lag between a good new idea and its enforcement can be six or even eight years. For example, ASCE 7-05 (the 2005 edition) will be based on the 2003 NEHRP Provisions. Proposals to both of these documents were due in 2002. If the 2007 CBC adopts a 2006 model code, which in turn adopts ASCE 7-05, that means that the code in use in California in 2010 (the 2007 CBC) will have been based on proposals made eight years earlier and will already have been balloted as of early 2004. The lag is complicated further by the long balloting schedule required for consensus standards. Because the ASCE 7 balloting takes so much longer than the NEHRP balloting, reviewers and voters, including the Seismology Committee, found themselves voting on ASCE 7-05 before its source document, the 2003 NEHRP, was finalized. (ASCE 7 is expected to go to a five-year revision cycle. This may alleviate the voting overlap, but it will only increase the lag between proposals and provisions.)

The lag between new thinking and an enforceable code also has potential implications for engineer liability. Under the new process, it will often be the case that a revised loading or material standard exists long before a local code adopts it. Should an engineer double her design effort by consulting the new standard as well as the obsolescent code, or should she ignore the not-yet-adopted standard even if it might provide better performance? The Seismology Committee supports the consideration of new consensus thinking during design, but it recognizes the burden this places on the practicing engineer (and potentially on jurisdictions) under the new code development process.

Compared with the old process, which was dominated by professional associations like SEAOC, ANSI rules for developing standards require more participation of and give more control to industry groups – and less to public interests. While standards committees include design professionals, researchers, and others with non-commercial interests, it is the opinion of the Seismology Committee that committees developing design provisions for a specific material (concrete, masonry, steel, wood, dampers and isolators, etc.) tend to support criteria that would make the use of that material more economical. While these committees usually act responsibly, they are sometimes reluctant to adopt more restrictive provisions even when evidence of failures or damage indicates that it is appropriate to do so, giving some observers the perception of a conflict of interest. With less influence on standard-writing committees, it will be more difficult for outside parties, including Seismology and NCSEA, to play their traditional watchdog role. With separate standards for each structural material, it will also be more difficult for Seismology and others to ensure consistency of design philosophy among different types of construction. Finally, if standards committees are slow to recognize inadequate provisions for their own material, it might also become more difficult to implement code changes quickly when an earthquake or other disaster reveals flaws in current practice.

Approved for publication by the Seismology Committee, May 2005

Glossary

The following acronyms are used in the text above. They represent organizations and documents instrumental in the building code development process.

ANSI American National Standards Institute.


BOCA Building Officials and Code Administrators International, Inc. One of three statutory members of the ICC. Producer of the National Building Code (NBC).
Development of Earthquake Design Provisions for Building Codes

BSSC  Building Seismic Safety Council. Producer, for FEMA, of the NEHRP Provisions and Commentary. For more information, see BSSC (2001b, p.431-444).


IBC  International Building Code. Published as a model code by ICC since 2000.

ICBO  International Conference of Building Officials. One of three statutory members of the ICC. Producer of the UBC.

ICC  International Code Council. Formed from three statutory members: BOCA, ICBO, and SBCCI. Producer of the IBC.

NCSEA  National Council of Structural Engineers Associations, of which SEAOC is one.

NEHRP  National Earthquake Hazards Reduction Program, a federally-funded program under which the NEHRP Provisions and Commentary (BSSC, 2001a; 2001b) are produced. The NEHRP Provisions and Commentary have been produced by BSSC for FEMA since 1985 as recommended design provisions and as a resource for building code development. The 2000 provisions form the basis of the earthquake design provisions in ASCE 7-02. For more information about NEHRP, see BSSC (2001b, p.431-444).


STC  Seismic Task Committee. A technical task committee of the ASCE 7 standards committee and producer of earthquake design provisions for ASCE 7. (See NEHRP.)

UBC  Uniform Building Code. Published as a model code through 1997 by ICBO. Starting in 1961, its earthquake design provisions were adapted from the SEAOC Seismology Committee’s Blue Book.

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Development of Earthquake Design Provisions for Building Codes


For many years model code provisions for earthquake design have acknowledged three fundamental assumptions:

1. Nonlinear, inelastic behavior is expected, but proposed designs may be analyzed with linear procedures.
2. Cyclic loading is expected, but structures may be designed for monotonic loads.
3. Real deformations will greatly exceed those predicted by linear analysis with reduced forces, but the real values may be approximated as multiples of the linear analysis results.

Taken together, these simplifying assumptions have made it possible to understand, predict, and regulate structural response in terms suitable for adoption in building codes. Over time, additional code provisions have developed to account for cases in which one or more of these assumptions does not apply.

Whether these assumptions are sufficient depends on one’s performance objectives. Nevertheless, they remain the basis of most earthquake design in 2008, though they are largely the consequence of traditional engineering practice and now obsolete technology. First a technical necessity, and still an economic one, this set of assumptions is seen as the basis of a rational approach to earthquake design that balances safety with economy.

As modern building codes developed, linear elastic design for pseudo-static loads was standard practice. While methods of plastic analysis were developed in the 1960s, actual building designs were still based on elastic analyses, if not approximate methods or even rules of thumb. Some researchers and practitioners as early as the 1950s and early 1960s began to recognize the significance of inelastic response, though the Blue Book and codes of that era only implicitly addressed the subject. According to Bozorgnia and Bertero (2004),

> To consider the inherent ductility and energy dissipation capacities of different structures, a coefficient $K$ was introduced in the base shear equation $V = KCW$, where $K$ values were specified for four types of building construction. According to Blume et al. (1961): "The introduction of $K$ was a major step forward in code writing to provide in some degree for the real substance of the problem -- energy absorption -- and for the first time to recognize that equivalent acceleration or base shear coefficient $C$ alone is not necessarily a direct index of earthquake resistance and public safety."

The very first Blue Book (SEAOC Seismology Committee 1959) introduced an explicit requirement for ductility, which a seismically knowledgeable structural engineer would have interpreted as the flip side of the statement that elastic-based design forces were reduced from expected force levels in large earthquakes. That ductility requirement
was for structures exceeding 13 stories or 160 feet in height, and "ductility" was defined as equivalent to that of a steel moment-resisting frame. Design of Multistory Reinforced Concrete Buildings for Earthquake Motions by Blume, Newmark, and Corning (1961) was one of the most influential early works incorporating this thinking that was aimed at practitioners and building codes, and it had the effect of developing reinforced concrete seismic design provisions that could qualify for the equivalent ductility stated in the Blue Book. The 1966 edition of the Blue Book (SEAOC Seismology Committee 1966) was the first to include ductile reinforced concrete requirements. Several papers on inelastic seismic response at the Second World Conference on Earthquake Engineering in 1960, such as by Newmark and Veletsos (1960), Penzien (1960), and Blume (1960), were presented, and that year may be taken as a benchmark for when inelastic response became a major research topic. However, practical design methods based on inelastic analysis had not yet permeated ordinary design practice. Guidance to engineers to build "toughness" into their structures was based to some extent on a realization that inelastic response in a large earthquake would be a reality, but it is not known how many structural engineers conducting seismic designs at the time fully understood the inelastic thinking that was more often implicit rather than explicit in codes and guidelines.

Nonlinear analysis software was available by the early 1970s, but it was rarely used for design (Martini et al. 1990). Even with the introduction of affordable and powerful desktop computers in the 1980s, the computation time and data storage requirements made nonlinear analysis impractical for the design of everyday buildings. State of practice reviews in the 1980s by Brooks (1987) and Habibullah (1987) do not even mention nonlinear analysis. While nonlinear analyses are performed with greater frequency today, ASCE 7-05 still does not require nonlinear analysis for any structure type.

During the 1970s and 1980s, widely distributed seismic design guidance for engineers, such as ATC-3 (ATC 1978), Housner and Jennings (1982), and Berg (1983) increasingly emphasized that so-called “reduced” design forces were known to be well below those that an elastic structure would actually endure in a code-level earthquake. By this time, an increasing library of strong motion records had accumulated. While the 1/3 g peak ground acceleration from the 1940 El Centro record had long been the most often used accelerogram for analysis and testing, the 1971 San Fernando Earthquake, with its 241 strong motion records (Maley and Cloud 1973), including the Pacoima Dam record with a peak over 1 g, broadened engineers’ minds as to the level of shaking in earthquakes. Building codes could have kept the structure elastic by increasing the minimum design forces to match the expected level of shaking. Instead, the codes took a different approach. As noted in the Commentary of the 1974 Blue Book (SEAOC Seismology Committee 1975 p. 7-C),

The actual motions generated by an earthquake may be expected to be significantly greater than the motions used to generate the prescribed minimum design forces. The justification for permitting lower values for design are many-fold and include: increased strength beyond working stress levels, damping contributed by all the building elements, an increase in ductility by the ability of members to yield beyond elastic limits, and other redundant contributions.

That Blue Book Commentary discusses ground motion levels and induced inelastic response in structures in detail, which indicates how much had changed in the 15 years since the first Blue Book was produced. On the other hand, inelastic response was still usually taken into account in design practice by ductile detailing provisions, while analyses were still elastic. Code-based design (i.e., the elastic procedures allowed in building codes) retained the previous framework: reduced design forces, presumed significant inelastic behavior, and provision for inelastic response via appropriate detailing provisions. Indeed, when the format of the code’s base shear equation was modified in 1988 and the R factor was introduced to explicitly reduce elastically computed force levels, which followed the lead of ATC 3-06 (ATC 1978), values of the new design parameters were selected to assure that customary design forces would not change (Porush and Zacher 1987).

The notion of fully elastic behavior was ruled out as well by construction economics and architectural convention. Had codes been changed to require elastic response in large earthquakes, the necessary structural configurations and member sizes would have severely impacted the building industry. Fairly standard building types would have suddenly become unwieldy and architecturally inefficient, if not prohibitively expensive. In all likelihood, such substantial changes would have been resisted by political forces, just as the adoption of early earthquake codes had been (Olson, 2003) and even as many code changes still are.
Is Code-Based Earthquake Design Sufficient?

The real test of these design assumptions is whether they lead to buildings that work. The 2000 NEHRP Commentary (section 5.2) says they do: “This approach can be taken because historical precedent … demonstrates that … it is possible to perform an elastic design of structures for reduced forces and still achieve acceptable performance” (BSSC, 2001b, p. 69). But similar statements were made before the 1971 San Fernando Earthquake and again before the 1994 Northridge Earthquake, only to be questioned after those events. The assertion that code-based design is supported by past performance is thus optimistic.

The NEHRP Commentary’s assessment relies in part on the theory of “ductility reduction” (BSSC, 2001b, p. 71) but is based largely on a general impression from post-earthquake reconnaissance that buildings designed and built to the latest codes rarely collapse and tend to perform better than the older structures around them. Whether this impression is reliable or statistically valid is debatable. Certainly, engineered structures of modern materials can be expected to outperform older vernacular architecture still common in seismically active areas around the world, but this does not prove the adequacy of the newer design. Several structure types once considered code-compliant are now seen as deficient in some respect. Non-ductile concrete frames, wood-framed apartment buildings with “tuck-under” parking, and pre-Northridge welded steel moment frames are examples.

Furthermore, some of the oft-cited reports are simply out of date. For example, a 1968 report described contemporary building codes as having produced “designs that have successfully withstood severe earthquakes in the past with little or no damage at all” (Goel, 1968). But the basis for that statement was a 1955 report on the 1952 Kern County earthquake, which affected mostly non-engineered structures. (The engineered structures shaken by that event were built in the 1920s and 1930s, and most were in Los Angeles, over 100 km away.) A statement about code sufficiency made after one earthquake must expire eventually.

Finally, there are important counterexamples. Earthquakes in Japan (1995) and Taiwan (1999) collapsed dozens of buildings erected within the previous ten years and designed with codes similar to the UBC (Nakashima et al. 1998, Uzarski and Arnold 2001). In both cases, engineers were able to distinguish in hindsight critical factors including irregular configuration, poor detailing, and outdated provisions in design codes that had later been revised. Nevertheless, the failed buildings were compliant when they were built. A review of post-earthquake reconnaissance reports spanning several decades has shown that engineers initially, routinely, attribute poor performance of high-end structures to substandard construction quality; only later is a flawed design concept recognized (Reis and Bonowitz, 2000).

In the United States, the two earthquakes that most influenced modern (post-World War II) building codes were San Fernando in 1971 and Northridge in 1994. Each was followed by volumes of reconnaissance literature and precedent-setting reviews of building code provisions. Despite the 1968 report cited above, the San Fernando Earthquake just a few years later caused “serious damage to many presumably ‘code-designed’ buildings” and revealed that “codes of that day contained serious gaps that had to be closed” (SEAOC Seismology Committee, 1990, p. xiii).

SEAOC’s review of Northridge Earthquake implications was equally critical (Seismology, 1996, Appendix A). First, certain structure types—wood-framed buildings with gypsum board or stucco shear walls, concrete parking garages, and welded steel moment frames—were singled out for their unexpected poor performance. Second, while code-based designs generally met their objective of avoiding collapse and preserving life safety, it was unclear whether lightly shaken buildings provided enough resistance to damage to be considered successful. With respect to the fundamental assumptions of code-based design, the SEAOC Northridge Commentary Committee was forced to ask “whether the current elastic equivalent lateral force seismic code format can be used to reliably predict performance over the full range of ground motions indicated, except in the simplest situations.”

The question, then, is whether code-based design has a one-point or a multi-point performance objective. Model codes use a one-point objective, stated with respect to the default design earthquake, as in 1997 UBC section 1626.1 (2003 IBC is similar): “to safeguard against major structural failures and loss of life.” The first Blue Book with
Commentary (SEAOC Seismology Committee 1960) had a similar goal, which was to "provide minimum standards to assure public safety." Later, Blue Books included an early variety of performance-based engineering, tying different performance to different earthquake levels, seeking to resist a "minor" earthquake with no damage, a "moderate" earthquake with no structural damage, and a "major" earthquake with no collapse.

For a one-point life safety objective, the findings from the 1994 Northridge Earthquake can be said to have vindicated code-based design. But for a three-point objective, such as the one espoused by past Blue Books, the SEAOC Northridge Committee found that linear elastic analysis with reduced forces might not be adequate to achieve reliable, acceptable performance: “[T]he nonlinear behavior of the structure and the deformation demand of the expected or design ground motion should at least be considered” (SEAOC Seismology Committee, 1996, Appendix A).

In the end, to demonstrate the sufficiency of a given code one would need comprehensive performance data from real structures designed to that code and shaken by the design earthquake—data that does not exist. But real performance data does not exist to prove the sufficiency of alternative design assumptions either, let alone their superiority. Therefore, as a matter of judgment, considering the narrow purpose of code-based design, linear static methods with reduced forces are still considered appropriate for most structures. (Indeed, linear static analysis remains the only reasonable approach for typical wood structures, whose nonlinear and dynamic properties are notoriously complex.)

Finally, if code-based design is sufficient for its stated objective, it is sufficient only for a “well-planned and constructed structure” (SEAOC Seismology Committee, 1990, p. 2-C), that is, one that is “regularly configured, well proportioned, well detailed [and meets] not only the letter but the intent of code provisions” (Porush and Zacher, 1987). Three major aspects of building performance need to be adequately considered for the code-based approach to be reliable: inelasticity, cyclic loading, and realistically estimated deformations.

**Inelasticity**

Building codes prescribe earthquake design forces much lower than those that would be felt by a fully elastic structure. These design forces are based on “response spectra that are representative of, but substantially reduced from, the anticipated design ground motions” (BSSC, 2001b, p. 69; SEAOC Seismology 1999, Fig. C105-2, p. 167.) With these reduced forces, deformations calculated by linear elastic analysis are also less than what the structure must sustain. The actual design-basis ground motion is expected to load a code-designed structure beyond its elastic range. The code’s detailing provisions account for the necessary inelastic behavior. Element force levels and deformations in the fully responding inelastic structure are derived as multiples of the values predicted by the linear elastic analysis.

Because of this presumed inelasticity, structures subject to earthquake effects must be detailed appropriately even if the reduced earthquake forces do not appear to govern the design. As required by ASCE 7-02 section 9.1.1 (and similarly in 1997 UBC section 1626.3), “The specified earthquake loads are based upon post-elastic energy dissipation in the structure, and because of this fact, the provisions for design, detailing, and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.” In particular, wind forces may appear to govern the design of some buildings, but wind and seismic forces are fundamentally different. Wind pressures and the resulting forces tend to have somewhat predictable upper limits; the upper limits of seismic inertial forces are less predictable.

Inelastic behavior is allowed to accommodate earthquake effects only if that inelasticity does not impair the structure’s resistance to gravity loads. Recognizing this limit, code provisions have been added over time to restrict yielding or buckling in certain critical elements. For example, the 1988 UBC followed the lead of the SEAOC Seismology Committee and added special design equations for steel columns, requiring them to resist axial loads that include unreduced earthquake effects. Other special conditions for which inelastic response should be avoided are now addressed by the Ω0 or overstrength factor.
The actual amount of inelasticity presumed by code provisions is only roughly estimated by parameters such as $R$ and $\Omega_0$. For innovative systems and for performance objectives differing from the code’s, such a rough estimate might be inappropriate. Instead, engineers should identify the specific sources of inelasticity and quantify the expected inelastic response. This will likely require system-specific, and perhaps even project-specific, testing. Systems for which such testing is appropriate, even though nonlinear analysis is not strictly required for code-based design, include seismic isolation, viscous damped frames, buckling-restrained braced frames, steel moment-resisting frames with innovative beam-column connections, and post-tensioned precast concrete frames. As engineers continue to refine designs and learn lessons from earthquakes, even more familiar systems might require new testing.

**Cyclic Loading**

Earthquake ground shaking is expected to cause several reversing cycles of inelastic deformation. This is another reason why seismic detailing is essential even when wind “controls” the member sizing. The code’s detailing requirements are intended to provide the energy absorption and toughness needed to resist actual earthquake forces and deformations, allowing the structure to respond inelastically without catastrophic failure.

Code parameters do not, however, allow an accurate estimate of the number of inelastic cycles a structure or its yielding components are likely to see. Again, physical testing might be warranted for systems and details that lack a sufficient track record, even if the design is ultimately based on linear analysis with reduced forces. Of course, how to simulate the inelastic cycling of a real earthquake in a series of controlled tests is not trivial.

**Deformation**

Linear elastic analysis with reduced design forces naturally results in predicted deflections (and deformations, and interstory drifts) that are well below those expected in the actual design earthquake. To make up for this, ASCE 7-02 amplifies the calculated deflection by the factor $C_d$. The corresponding factor in the 1997 UBC is $0.7R$. The amplified deflections are then compared to specified drift limits. Still, real deformations are not well predicted, as noted in the 2000 NEHRP Commentary: “major earthquake ground motions can cause deformations much larger than the specified drift limits” (BSSC, 2001b, p. 2).

This is a potential concern because deformations are probably the best indicators of earthquake damage and of performance in general. “Considering damage levels, it is becoming well recognized that, because of expected nonlinear structural behavior, deformations should replace forces as the key parameter. In order to improve damage predictability, target damage limitations must be linked to deformations and actual deformations must be better estimated” (SEAOC Seismology Committee, 1996, Appendix A). In general, then, performance prediction probably requires a better estimate of true deformation than amplified elastic drifts. Especially where the performance objective involves limited damage, as opposed to just the avoidance of collapse, the simplifying assumptions of code-based earthquake design are likely to be inadequate.

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**Keywords**
equivalent static lateral force analysis
history of seismic codes
inelastic response
linear procedures
nonlinear modeling
How To Cite This Publication
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

**Background**

Inadequate design documentation can result in longer times required for plan review and permitting, additional time and expense for field coordination, additional requests for information, lack of flexibility during construction, an incomplete quality assurance plan, and construction delays.

The engineer of record knows the load paths, structural systems, and material capabilities needed to resist the required design loads. It is the engineer’s responsibility to communicate this information to others involved in the design, permitting, construction, and inspection of the building. Good documentation is also beneficial for planning later alterations.

Failure to consider such fundamental earthquake design elements as load paths and connections is believed to have contributed to poor performance in past earthquakes. Studies of damage from the 1994 Northridge earthquake, for example, indicated that deformation compatibility and load path requirements were overlooked by designers in several failed parking structures and soft-story wood framed apartment buildings (SEAOC Seismology Committee 1996, Appendix A).

Design documentation may be even more important for performance-based designs in which engineers seek to control damage and maintain function, as well as preserve life safety. Detailing and construction quality are crucial to enhanced structural performance, and both rely on thorough design documentation. Post-earthquake function of nonstructural components is also associated with enhanced performance objectives, so thorough documentation of those details is important as well.

**Types of Documentation**

Structural design documentation includes calculations, drawings, specifications, soils/geotechnical reports, local rainfall and snow data, building component data, equipment weight, size, and anchorage requirements, and other supporting data used to define the structural design. Different documents may be produced for different projects and for different phases of design. Each type of document, at each phase, might have a specific purpose. Collectively, the design documentation must allow plan check of the design, allow appropriate peer review, define the work for bidding purposes, facilitate construction in accord with the design intent, and provide a basis for necessary inspections.

The SEAOC Professional Practice Committee has produced a set of guidelines for the practice of structural engineering (SEAOC Professional Practice Committee 1999). With respect to structural design documentation, the guidelines identify three principal document types to be produced by the engineer of record (refer to the SEAOC guidelines for details):  

- Structural drawings: contract documents necessary for construction of the building.
- Structural specifications: contract documents that supplement the drawings and are also necessary for construction.
- Structural calculations: although not contract documents, and strictly speaking, they are not necessary for construction, these are often necessary to validate the design and to enable both plan check and design peer review.
review (SEAOC Seismology Committee 1999, section IB-7.2). Calculations are also required for most buildings by permitting authorities and by building codes.

Where various codes and standards refer to “construction documents,” that term may be understood to mean only the drawings and specifications, not the calculations. The SEAOC Professional Practice guidelines address the general content of design documents. This article addresses design documentation specifically related to building code provisions for earthquake design.

**Maintaining Design Documents**
The Seismology Committee recommends that the building owner maintain a set of structural drawings and calculations for use following an earthquake. Maintenance of original design documents also facilitates repairs, alterations, and additions through the life of the building. While it is not the engineer’s responsibility to maintain documents for these purposes or to cause the owner to maintain documents, the Seismology Committee encourages engineers to convey to their clients the value of maintaining structural design documentation. (On-site maintenance of documents is generally more convenient for immediate post-earthquake use. Off-site maintenance might be preferable in the event of severe structural damage.)

**Structural Drawings**
ASCE 7-02 section 9.2.1 and ASCE 7-05 section 11.2 defines construction documents as “the written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with this standard.” Similarly, 1997 UBC section 106.3.3 and 2003 IBC section 106.1.1 both require the construction documents to “indicate the location, nature and extent of the work proposed and show in detail that it will conform to the provisions of this code.” The dual requirement to describe the project and to demonstrate conformance suggests two types of information to be shown: building information and reference information.

**Building information.** With respect to earthquake design, the drawings must describe the seismic force-resisting system with enough detail to allow its proper construction and inspection. It is the engineer’s responsibility to assure that critical load path elements are adequately detailed. Model codes make a few specific requirements in this regard:

- All connections resisting design seismic forces must be detailed on the structural drawings (1997 UBC section 1633.2.3)
- Drawings or specifications must be prepared for structural and nonstructural components requiring special inspection for seismic resistance (2003 IBC section 1603.1.8, ASCE 7-02 section 9.6.1.7, ASCE 7-05 section 13.2.7).

In addition, the engineer should identify on the drawings or in the specifications those elements of the seismic force-resisting system that either provide or require bracing during construction. For example, AISC (2000, section 7.10) requires the engineer to identify the steel lateral load-resisting system so that the erector can brace it until supporting diaphragms are in place. It also requires identification of non-steel elements, to be built by others, that will support the steel framing in the completed structure.

**Reference information.** With respect to earthquake design, the 2003 IBC and 2006 IBC section 1603.1.5 lists 10 items (with 13 total design parameters) that must be shown on construction documents. The list in 2003 NFPA 5000 section 35.4.2.5 is similar. There is no corresponding list in the 1997 UBC.

Most of the items on the IBC list are parameters needed to derive the seismic base shear for the equivalent lateral force procedure in ASCE 7-02 section 9.5.5 and ASCE 7-05 section 12.8. Also on the list are such non-numeric items as the seismic use group, the seismic design category, the basic seismic force-resisting system(s), and the analysis procedure used. While the seismic base shear and response coefficient (listed by the IBC but not by NFPA 5000) are of little importance as predictors of earthquake performance, they can be useful reference values.
Therefore, the Seismology Committee supports the IBC documentation requirements in principle. However, it is the opinion of the Seismology Committee that only the parameters used to derive the seismic response coefficient(s) $C_s$ need to be listed; the calculated value $V$ and the building’s estimated weight $W$ should not.

For designs based on the UBC (or CBC), it is the position of the Seismology Committee that similar information should be shown on drawings unless it does not apply to the design, for example, where simplified design or conventional light-frame construction is used. For UBC-based designs, seismic zone factor $Z$, seismic source type, and the distance to the seismic source should replace the corresponding items in the IBC list.

Regardless of the model code used, it is the position of the Seismology Committee that, in general, the basis for the earthquake design should be stated and that the following information related to earthquake design should be shown on structural drawings:

- The date and edition of the governing building code
- Material properties used, including soil properties
- Parameters needed to establish the seismic response coefficient (2003 IBC)
- The basic seismic force-resisting system(s), as tabulated in 1997 UBC Table 16-N or ASCE 7-02 Table 9.5.2.2 or ASCE 7-05 Table 12.2.1 (2003 IBC, 2006 IBC, 2003 NFPA 5000)
- Data needed for design by others (for example, seismic relative displacements per ASCE 7-02 section 9.6.1.4, or ASCE 7-05 section 13.3.2 needed for the design of nonstructural components and their attachments).

Designs that go beyond the minimum requirements of the governing code, such as those based on optional analysis procedures, should supplement this list as appropriate.

**Structural Calculations**

While the words “engineering calculations” are absent from the definition of construction documents in ASCE 7-02, ASCE 7-05, and in model building codes, their production is implied and necessary for validation of code compliance. Structural calculations for earthquake design are mandated indirectly by ASCE 7-02 section 9.1.2.4: “When required by the authority having jurisdiction, design documents shall be submitted to determine compliance with these provisions.” Model code provisions that call for structural calculations include 1997 UBC section 106.3.2, 2003 IBC section 106.1 (“other data”), and 2003 NFPA 5000 section 1.7.6.2.1 (“other information”). These model code provisions presume that design documents will include structural calculations that demonstrate the adequacy of all vertical and lateral load path elements. However, the codes do not specify how calculations should be presented or what they should contain.

In general, the content and format of structural calculations should be left to the engineer of record. Some general guidelines are appropriate, however. Structural calculations should be sufficiently annotated so as to be understandable by most engineers and plan checkers. Specific building code sections should be cited to demonstrate compliance. Calculations should be sufficient to allow minor field revisions to the original design without reanalysis. The methodology used in the design calculations should be clearly presented so that results are reproducible. Design decisions, which may involve judgment, regarding member sizing and the acceptability of trial designs should be clearly stated. The final product should constitute a traceable link between design assumptions and construction drawings.

**Computer-generated calculations.** In general, documentation generated by computer should meet the same standards as hand-calculations. That is, they should be well annotated and results should be reproducible. The keys to good design documentation with computer-generated calculations are a clear graphical description of the computer model and tabular summaries of key results.

For traditional computer analyses that take a single input file and generate self-contained output files, the Seismology Committee recommends that the following items be included in the calculation package:
• Sketches as needed to describe the complete geometry of the computerized structural model
• A description of the computer program used, including the version number. The description should state what the program does within the context of the larger design process, distinguishing, for example, between analysis and design, or between member sizing and member checking. A user’s guide should be made available to plan checkers and peer reviewers upon request. It is always the responsibility of the engineer of record to confirm that the software works.
• Input data, clearly annotated. Values assumed by the program should be distinguished from those estimated or assumed by the engineer
• Output data, including project- and structure-specific identifiers, restatement of input data, and results with units clearly noted. Additional information, such as the date of the analysis, is frequently useful for later reference. On some projects, the volume of calculations is enough to hamper an efficient review. In these cases, at the discretion of the building official, it is acceptable to submit some or all of the calculations in electronic format.
• At least the first sheet of each computer run should be signed by (or should bear the initials of) the engineer responsible for the computer modeling and data processing. This recommendation is intended to provide accountability for the appropriateness of input data and for the sufficiency of output confirmation.

Where short output files are interspersed with hand calculations, these recommendations may be modified to fit the circumstances.

Where computer-generated calculations emulate hand calculations (that is, where the output format reads not as a data file but as a logical sequence of annotated statements) additional software documentation and hand verification are not necessary, providing the calculations are transparent, with assumptions and equations readily visible and clearly labeled. Any hidden tables, equations, or functions on which the calculations rely should be provided.

In contrast to “traditional” computer analyses, new programs may be more interactive, relying on graphical data generation and output, and offering the user a number of options for working with the same set of input data. These programs may require more effort on the part of the engineer in terms of documenting the final design. While the variety of software and engineering tools is too great to address with a set of prescriptive recommendations, the basic principles of design documentation stand: the structural calculations need to link the design assumptions with the construction drawings in such a way that the logic of the design process may be followed by peer reviewers and plan checkers.

**Emerging Practices**

New technologies are changing the nature of design documentation (STRUCTURE 2004). While most of these developments focus on communications and construction, earthquake design is also affected by improvements in software and computing. For example, nonlinear and response-history analyses can generate more data than is reasonable to submit or useful to review. Even the most basic structural designs now commonly rely on electronic media of some sort.

The Seismology Committee does not yet hold a position on any specific technology or emerging practice but in general encourages the use of new technology for design documentation. Regardless of format, however, design documentation must continue, as described above, to facilitate plan check, peer review, bidding, construction, and inspection. The Committee also encourages consideration (and further study) of potential problems related to new documentation practices, especially as they may affect earthquake design. These include:

• Learning curve issues: not all members of the design and construction team are equally facile or comfortable with new technology
• Coordination: while many new technologies make coordination easier and better, the ease with which electronic documents are updated and reproduced might lead some users to make frequent and late revisions
Security: the exchange of critical or sensitive documents by email or over the internet remains an evolving issue in engineering as in other fields.

Obsolescence: engineering software changes quickly, as do operating systems, file formats, and storage devices. While we can still read a set of paper plans from a century ago, it seems unlikely that a digital file will be effective as a “permanent” record.

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How To Cite This Publication

In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2006)

In the writer’s reference list, the reference should be listed as:

Keywords

- calculations
- computer-generated calculations
- design documentation
- drawings
- engineer of record
- specifications
The structural systems most commonly used to resist earthquakes are listed in ASCE 7-05 Table 12.2-1 and 1997 UBC Table 16-N. While the ASCE 7 table refers to them as “seismic force-resisting systems” (SFRSs) it is important to note that these are distinct elements within a possible combination of systems to form the overall lateral force-resisting system of a building, which would include diaphragms, foundations, and other load path components. The text of ASCE 7-05, however, is not careful about maintaining these distinctions, so the more common term “seismic force-resisting system,” or SFRS, is used here. That is, the SFRS is the “basic” system contemplated by ASCE 7-05. (Previously, in the title of ASCE 7-02 Table 9.5.2.2 listing SFRSs, the word “basic” was included.) While SFRS is difficult to define, it is commonly understood to mean the set of vertically oriented structural elements above the foundation that are expected to act together to resist interstory drifts and to carry design earthquake forces between levels of the structure.

Diaphragms, collectors, and other load path components, though essential to acceptable performance, are commonly analyzed and designed separately from the SFRS. This is a useful distinction, as it simplifies analysis and design, but it relies on the assumption that these other components are not needed as primary sources of ductility or interstory strength or stiffness. In other words, if the diaphragm, collectors, or other elements apart from the systems tabulated in ASCE 7-05 Table 12-2-1 must provide reliable inelasticity in order to develop the presumed SFRS, then these attributes must be considered integral to the SFRS design. Most codes avoid this complexity with provisions that attempt to limit inelasticity to the “basic” SFRS.

Consistent with typical code-based design practice, diaphragms, collectors, and other load path components are not considered part of the SFRS for purposes of this discussion and are discussed in separate articles.

The code tables give design parameters and height limits for each listed SFRS. AISC 7–05 Table 12.2-1 also includes an additional column that references applicable detailing provisions.

Several of the specific SFRSs are discussed in depth in separate Blue Book articles.

**System Selection**

Selection of the SFRS for a specific building is as much art as science. It is clearly a design decision of fundamental importance, yet there is no system that is best for all buildings. Factors to consider when selecting a seismic force-resisting system include:

- Performance. All of the code-approved systems are expected to meet the performance objective of the code. For enhanced performance objectives, however, some systems might be better than others. For example, some systems are better able to meet tight drift limits or assure speedy repair.

- Architectural and nonstructural coordination. A moment-resisting frame system can accommodate open spaces and unrestricted bays between columns. Braced frame and shear wall systems generally offer less flexibility in space planning and fenestration. The spacing of gravity columns, fire-rated partitions, and utility cores can also affect the relative efficiency of certain systems.

- Construction cost. The project budget might dictate systems of certain materials or cost-effective fabrication and erection procedures.
• Design budget. Some systems can be analyzed and designed effectively with simple hand calculations, while others require more expensive and time-consuming procedures.

Acceptable earthquake performance is a function of more than the selected structural system. Configuration and integration of the SFRS within the building are fundamental to good design, concerning such issues as irregularities, torsion, redundancy, and the combination of systems.

**Design Parameters**

The three design parameters tabulated by ASCE 7-05 in Table 12.2-1 are:

1. A response modification coefficient, $R$
2. An overstrength factor, $\Omega$
3. A deflection amplification factor, $C_d$

In addition, limitations on allowable applications of systems to different Seismic Design Categories and for height of structure are listed.

The theoretical basis of these three parameters and the relationships between them are discussed in depth in separate articles. This article discusses only the relative values assigned to the various seismic force-resisting systems. Parameters assigned to a given system might be subject to code limits based on how the system is configured or combined with other systems within a building.

In general, the current design parameters are intended to provide the same high confidence that each listed SFRS will meet the implied performance objectives of the code. The current parameters began as broad distinctions among four basic structural system types discussed below. Parameters for specific systems were assigned considering the characteristic performance expected of each main type, and parameters for newer systems have generally been added by judgmental comparison with older, established systems.

**Response modification coefficient.** $R$ is the parameter that most succinctly represents the potential capacity of a system for ductile response and energy dissipation. The tabulated $R$ value is effectively “adjusted” by the adjunct parameters $\rho$ and $I$ that account respectively for structural redundancy and the occupancy-defined importance.

For the systems permitted in the highest seismic areas (represented by seismic design categories D, E, and F), the $R$ values in ASCE 7-05 are essentially identical to those assigned by judgment in the late 1970s (ATC 1978, Table 3-B). However, while the $R$ values have remained the same, design and detailing provisions have changed to reflect newer research and more recent earthquake performance. For example, special steel moment frames were assigned the highest $R$ value, 8, in 1978. They still have an $R$ value of 8 in ASCE 7-05, though design requirements have changed substantially based on an updated record of performance and testing.

$R$ values for dozens of systems added since 1978 have been based in part on test data but primarily on judgmental comparison with the older systems. For example, ATC 3-06 assigned an $R$ of 3.5 to reinforced masonry shear walls in a bearing wall system. Since then, details with more ductile behavior have been developed. ASCE 7-05 retains the 3.5 value but classifies the traditional detailing as an “intermediate” system and limits its use. In seismic design categories D through F, special reinforced masonry shear walls are permitted instead, with an assigned $R$ value of 5.

Prior to ATC 3-06 (ATC 1978), the Blue Book and the UBC had identified four basic structure types (described below). Each was assigned a value of the coefficient $K$ based originally on the judgment of the SEAOC Seismology Committee and “influenced by its collective experience and observations in earthquakes of destructive intensity” (Seismology, 1960 p. 30). The Blue Book commentary related $K$ to a structure’s “inherent resistance to earthquakes” and referred to “careful analyses and evaluations of the performance of structures in major and moderate earthquakes” (Seismology 1967, p. 44). The relative $K$ values, which remained unchanged through the 1985 UBC,
were intended “to give all types of structural systems an equal probability of [acceptable] performance under a designated earthquake” (Seismology 1975, p. 15-C).

Still, the Committee acknowledged that $K$ was largely a “judgment factor” (Seismology 1967, p. 44) and recognized that the record of earthquake performance for each system type would certainly grow, necessitating modifications. The first major modifications came with ATC 3-06. While the new $R$ values were based on the then-existing $K$ coefficients of the Blue Book, ATC-3-06 also expanded the system table from four basic types to 21 material-specific systems, making distinctions among them to reflect expected differences in “toughness” and “damping” (ATC 1978, C3.3.1). ATC-3-06 notes plainly that its $R$ values were “based on [the] best judgement and data available … and need to be reviewed periodically” (Table 3-B, note 1). Efforts to rationalize and assign consistent $R$ factors are described further, separately from this article.

The 1997 UBC $R$ values differ somewhat from ASCE 7-05, though the two sets share a common ancestor in the Blue Book parameters that predated ATC 3-06. The $R$-values of the UBC were converted from earlier $K$ values with the intent of leaving design base shear levels unchanged (Porush and Zacher 1987). Roughly, $R = 8/1.4K$, where the 1.4 factor accounts for a shift from allowable stress design to strength design. With this conversion, the ductile moment frames, which had been assigned a $K$ factor of 0.67, would have qualified for $R$ of about 8.5. Bearing wall systems, called “box systems” in earlier UBC editions, had been assigned a $K$ of 1.33, which converted to an $R$ factor of about 4.3. Thus, some of the differences between 1997 UBC and ASCE 7-05 (for example, 8.5 vs. 8 for special steel moment frames and 4.5 vs. 5 for special concrete shear walls in bearing wall systems) do not represent technical disagreements so much as different genealogies of code development. In both cases, the code values continue to reflect the judgment of previous generations.

**Deflection amplification factor.** The $C_d$ values in ASCE 7-05 also trace back directly to ATC 3-06. Low $C_d$ values indicate relatively brittle systems (BSSC 2004b, p. 44).

The 1997 UBC is similar in its drift provisions to ASCE 7-05, but it does not give system-specific $C_d$ values. Instead, the UBC amplifies deflections by the value $0.7R$. The $0.7R$ amplifier effectively requires design for 70% of the drift of the theoretically elastic structure. The corresponding value in ASCE 7-05 would be represented by the ratio $C_d/R$. To the extent that this ratio differs from 0.7, the two codes will lead to different calculated drifts and could result in different designs.

The issue is most relevant for structures likely to be governed by drift rather than strength, that is, tall flexible structures with high $R$ values, such as special moment-resisting frames. ASCE 7-05 Table 12.2-1 includes three moment frame systems permitted for use in buildings taller than 65 feet in seismic design categories D-F. For each of these, $C_d/R = 5.5/8 = 0.69$, so the design requirement is consistent with the UBC’s 0.7 value.

However, for the seven other similarly permitted systems in ASCE 7-05 with $R$ of 8, the ratio of $C_d/R$ ranges from 0.50 to 0.81. (If systems with $R$ as low as 5 are included, the ratio ranges from 0.5 to 1.0.) There are also some clear inconsistencies. For example, dual systems with special concentrically braced frames are to be designed for 81% of the elastic drift, but dual systems with eccentrically braced frames need design for only 50%. It is not clear why the $C_d/R$ ratios for these two systems should be so different, and on opposite sides of the 70% value for special moment frames.

For some systems, then, designers using the 1997 UBC or 2001 CBC face the question of whether to amplify drifts by $0.7R$ or by a larger $C_d$ value required by a more recent, and presumably more advanced, design standard like ASCE 7-05 or the 2003 NEHRP Provisions (BSSC 2004a). (For example, for special reinforced concrete shear walls in bearing wall systems, ASCE 7 and BSSC set $R = C_d = 5$, so the design drift is 100% of the theoretical elastic drift, not 70% as it is in the UBC.) One would think that the more conservative provision might be the safer design choice. Given the inconsistencies described above, however, it is the position of the Seismology Committee that for purposes of determining the design drift in buildings governed by the 1997 UBC or the 2001 CBC, there is no need to go beyond the requirement of the code in an attempt to live up to more recent documents.
**Height Limits**

The height limits imposed by ASCE 7-05 Table 12.2-1 and 1997 UBC Table 16-N, particularly in high seismic areas, are intended as checks on the judgmentally assigned design parameters. That is, since the design parameters were based largely on judgment and supported by limited post-earthquake observations (as opposed to exhaustive analysis or testing), it was important to the code developers to restrict certain systems to the range of application with which they were familiar. For example, ATC 3-06 noted that “the lack of reliable data on the behavior of highrise buildings whose structural systems involve shear walls and/or braced frames makes it convenient at present to establish some limits” (ATC 1978, section C3.3.4). The current 65-foot limits on light-framed and timber systems also reflect the limits of practical experience.

The precise numerical values, however, are based on judgment and convenient benchmarks, not on testing, analysis, or post-earthquake observation. The basic limit for non-moment frame systems in both the UBC and ASCE 7-05 is 160 feet, a value established by the first Blue Book to supplement an earlier Los Angeles code requirement for buildings taller than 13 stories (Seismology 1959, section 2313(j)). A height limit of 13 stories, approximately 150 or 160 feet, was imposed by Los Angeles zoning regulations since approximately the turn of the nineteenth-twentieth centuries. A variance was granted for the Los Angeles City Hall built in the 1920s, at 454 feet the tallest building in California until 1964. Several buildings in downtown Los Angeles were built up to that 13-story zoning cap in the first half of the twentieth century and called “limit-height” buildings. In 1943, “the City of Los Angeles introduced an innovative seismic code that related the seismic coefficient to the flexibility of the building—the first such code in the United States and among the first anywhere.” (Berg 1983, p. 26) This “dynamic” equivalent static lateral force formula was written in such a way that the calculation only worked for a building up to 13 stories in height—because none taller was then allowed in that jurisdiction. When the zoning-based height limit of Los Angeles was abolished in the 1950s, the problem California structural engineers faced was the development of seismic requirements for taller highrises. Some engineers favored requiring moment-frame structures in buildings less than 160 feet tall, while others felt that older buildings that would not conform to the revised seismic regulations for buildings of a given height could be automatically considered deficient and obsolete.

Thus the 160-foot limit has its origins in this Los Angeles city planning experience rather than an explicit seismic design rationale. The first Blue Book commentary noted clearly that “the limitations of 13 stories and 160 ft have been established arbitrarily and are subject to further study” (Seismology 1960, p. 43). When ATC 3-06 revisited the system table, it retained the 160-foot limit for high seismic areas as well as the note that its limiting values were “arbitrary, and considerable disagreement exists … regarding [their] adequacy” (ATC 1978, section C3.3.4).

The provision introduced into the UBC after Los Angeles eliminated its height limit required any building taller than 160 feet to have a “ductile” moment resisting frame, ostensibly because shear wall and braced frame systems were perceived to lack “multiple lines of defense” or “a second line of resistance” (Seismology 1967, p. 43, 46). While that limit has largely been retained for high-seismic areas, codes now permit other specific systems for heights up to 240 feet. As noted above, this is also an arbitrary value, but it does recognize other systems that, if not equivalent to special moment-resisting frames, are at least expected to provide ductility and reliability sufficient for acceptable performance. 1997 UBC Table 16-N identifies four such systems, setting a 240-foot limit in zones 3 and 4 for:

- Steel eccentrically braced frames in building frame systems
- Concrete shear walls in building frame systems
- Steel special concentrically braced frames in building frame systems
- Steel special truss moment frames.

ASCE 7-05 Table 12.2-1 is similar, though somewhat less clear. Note d refers to section 12.2.5.4, which permits systems with “steel braced frames” and “special reinforced concrete cast-in-place shear walls” up to 240 feet in seismic design categories D and E if they are arranged and configured for good resistance to torsion. The text of the provision is inconsistently worded, however, and therefore potentially unclear. Specific clarifications to section...
12.2.5.4 and to Table 12.2-1 notes d and e are recommended below. It is the position of the Seismology Committee that the increased height limits in ASCE 7-05 section 12.2.5.4 should apply only to:

- Cast-in-place (but not precast) special reinforced concrete shear walls in bearing wall systems, building frame systems, or dual systems with intermediate moment frames
- Steel eccentrically braced frames with either type of connection at columns away from links
- Special (not ordinary) steel concentrically braced frames in building frame systems, but not in dual systems with intermediate moment frames.

ASCE 7-05 does not explicitly allow special steel truss moment frames (STMF) up to 240 ft. The 1997 UBC does allow STMF up to 240 ft. This discrepancy may be the result of an oversight during the development of ASCE 7-05. Without a technical basis for specifically limiting the height of the STMF, the Seismology Committee supports the height limit as posed in the 1997 UBC and 1999 Blue Book.

In ASCE 7-05 Table 12.2.1, the height limits for some dual systems with intermediate moment frames are less than for a building having the same bearing wall or building frame system without the intermediate moment frame. This is inconsistent: The dual system maximum height should be equal to or greater than the limit tabulated for the case where the bearing wall or building frame system is used alone.

**System Attributes**

Both ASCE 7-05 and the UBC group the seismic force-resisting systems into broad categories. Generic definitions for each category are given in ASCE 7-05 section 11.2 and 1997 UBC sections 1627 and 1629.6. The four main categories—bearing wall systems, building frame systems, moment-resisting frames, and dual systems—as well as their generic definitions, trace back directly to ATC 3-06 (ATC 1978, Table 3-B) and, with slight modification, to the first Blue Book (Seismology 1959, Table 23-C).

In terms of resistance to lateral loads, the original categories distinguished primarily between moment-resisting frames and stiffer wall or braced frame systems. The stiffer systems were further divided according to whether gravity loads were carried by bearing walls or by the columns of a “complete” space frame. Indeed, the default system—the three-dimensional space frame—was defined in terms of how it carried gravity, not lateral, loads. Earthquake loads for other systems were prescribed relative to the default, either one third higher (for suspect bearing walls) or one third lower (for the first modern SFRS, the ductile moment-resisting frame). Dual systems, combining moment-resisting frames with stiffer elements, comprised the fourth main category. Overall, these early design provisions expressed a strong preference for moment-resisting frames as the only system expected to provide ample “energy absorption capacity over the elastic and plastic range” (Seismology 1960, p. 33).

Historically, each of the main system categories has been expected to provide a certain characteristic performance under earthquake loads. All modern systems, however, are premised on some measure of overstrength, inelastic capacity, and load redistribution (BSSC 2001, p. 70). These expectations, implicit in building code design parameters and detailing provisions, have been based on (usually judgmental) assessments of:

- Past performance when subject to strong ground motion
- Demonstrated inelastic deformation capacity
- Relative vulnerability of gravity load carrying systems
- Capacity for system overstrength and force redistribution after initial yielding
- Multiple modes of resistance, including redundant frame lines and backup systems.

ATC 3-06 subdivided the four main categories, principally by material, into 18 specific lateral force-resisting systems. Three inverted pendulum systems completed the list. ATC 3-06 also distinguished between “special” and “ordinary” moment frame systems, with the special systems required to incorporate the latest ductile detailing in order to qualify for the traditionally low design forces for moment frames. (ATC 1978, Table 3-B)

Since ATC 3-06, the list of SFRS types in the UBC, and ASCE 7 has grown by:
• Addition of new systems, such as eccentrically braced frames and special truss moment frames
• Addition of new materials, such as composite braced frames and steel sheet panels on light framing
• Recognition of traditional systems, such as plain masonry, typically for use only in low seismic areas
• Addition of new dual system combinations, including some with intermediate moment frames
• Further distinction of traditional systems by special, intermediate, and ordinary detailing.

Historically, it has been the position of the Seismology Committee that “exaggerated” forms of the defined systems should not necessarily qualify for tabulated $R$ values (Seismology 1990, p. 20-C). Special moment frames with isolated one-bay bents, shear walls with large openings, and strong beam/weak column frames were given as examples. More recently, however, provisions for redundancy, modeling, and detailing have tried to address some of those concerns. Still, it is the Committee’s position that because the tabulated design parameters are largely based on judgmental notions of “typical” structures, any precedent-setting applications should be held to the requirements for undefined systems.

**Bearing wall systems and building frame systems.** These two basic system types both use relatively stiff shear wall or braced frame elements to carry lateral earthquake loads. The principal difference is in how the SFRS interacts with the gravity load-carrying system of the building.

Historically, neither of these categories was expected to provide the highest level of inelastic deformation capacity. With the introduction of “special” reinforced walls and braced frames, however, the characteristic performance within these categories has come to vary widely, depending on the material, detailing, and configuration of the system. Still, all of these systems are relatively stiff, and their design tends to be governed by strength requirements more than by drift limits.

A building frame system is said to have “an essentially complete space frame” to carry gravity loads. In particular, the gravity loads were to be carried by columns, not by bearing walls (ATC 1978, p. 337). Originally, this meant a three-dimensional grid of beams and columns independent of a discrete SFRS. By contrast, bearing wall systems (called box systems in the UBC and the Blue Book through 1985) had gravity load-carrying walls and partitions that interrupted or replaced columns in the overall building grid. The vertical load-bearing walls were allowed, but not required, to double as lateral load-carrying shear walls.

From the beginning, the presence of “a minor portion of bearing walls” was not intended to trigger the bearing wall provisions; stairwell and basement walls, for example, were not expected to affect “the action of a multi-storied building” (Seismology 1960, p. 22). ASCE 7-05 section 11.2 defines a bearing wall quantitatively in terms of the vertical load it resists but does not say how many bearing walls create a bearing wall system. Whether “minor” should be understood to mean “small” or “few” is not clear. The 2000 NEHRP Commentary (BSSC 2001, p. 74) suggests only that building frame systems should not have bearing walls that carry gravity load from more than “a few percent of the building area.”

Given more recent design provisions for deformation compatibility and ductile detailing of gravity load-carrying elements, the question of whether significant gravity loads are carried by walls or frame columns is no longer meaningful. Still, for purposes of code compliance, the designer must make a selection and in doing so should judge whether the presence of bearing walls will influence the post-yield capacity of the gravity system. (The larger question is whether laterally stiff bearing walls might influence the response of the intended SFRS, but that is unrelated to the distinction between bearing wall and building frame systems.)

The original distinction between bearing wall and building frame systems was based on a perceived need for a “second line of resistance” where shear walls carried the bulk of earthquake loads. One way to provide this backup was with the ductile moment frame of a dual system. Another way was to provide a distinct and complete frame to carry the gravity loads, as described in the 1967 Blue Book commentary:
The presence of a load carrying frame is desirable in a shear wall building because it may provide vertical stability to the building and prevent total collapse after damage is sustained by the shear walls. The frame also acts to tie the building together and redistribute the lateral force to undamaged elements of the bracing system (Seismology 1967, p. 46).

Walls and braced frames were understood to lack ductility (at least when compared with moment frames). If they were also counted on to carry significant gravity load, they were seen as potential collapse hazards. Code writers addressed this concern with a 33% increase in earthquake design loads for bearing wall systems ($K$ of 1.33, as opposed to the default value of 1.00). The intent was to protect against collapse of the gravity system by encouraging robust, or “complete,” gravity framing, or in its absence, by reducing the ductility demand on suspect bearing wall elements.

More recently, the distinction between bearing wall and building frame systems was somewhat reinterpreted. Although the code definitions of these two basic system types have scarcely changed since the earliest Blue Books, the distinction has been thought of as less about the completeness of the gravity system on its own than about the degree to which principal SFRS components carry both earthquake and gravity forces. Concentrically braced frames offer the most common example: If the diagonal braces carry gravity load in compression, the system has been deemed a “bearing wall” system (Seismology 1990, p. 12-C). Past Blue Books listed such systems specifically as “Braced Frames Where Bracing Carries Gravity Load” (Seismology 1990, Table 1-G), a distinction that persists in the 1997 UBC and the 1999 Blue Book (Seismology 1999, C104.6.2). Forthcoming codes, however, will list steel braced frames only as building frame systems, acknowledging that only one set of design parameters is needed for these systems (BSSC 2004a, Table 4.3-1), a position with which the Seismology Committee concurs. To clarify, the new position of the Seismology Committee is that braced frame systems need not be distinguished as bearing wall or building frame systems, and that the distinction made by past Blue Books may be discarded.

Indeed, the penalty in the code for bearing walls is no longer so great, nor is it the same for all systems. As Table 1 indicates in its comparison of bearing wall $R$ values (labeled here $R_{BW}$) with building frame $R$ values ($R_{BF}$), the benefit of going to a building frame system is an increase in $R$ and a subsequent decrease in the design base shear. Depending on the system, the decrease is between 7% and 17% in ASCE 7-05, 15-21% in the 1997 UBC.

In practical terms, the original distinction between bearing wall and building frame systems has faded. Since good seismic performance at expected force levels is known to be a function of detailing and load path, the real effect of a small difference in the design base shear is negligible. Indeed, this difference in $R$ is less than other potential code “penalties” for certain irregularities or low redundancy. Furthermore, current provisions for overstrength, deformation compatibility, capacity design of connections, and other factors account more directly for the likely ill effects of non-ductile failure in SFRS components that carry both earthquake and gravity forces. (See New Thinking below.)

### Table 1. Comparison of $R$ values in selected bearing wall and building frame systems

<table>
<thead>
<tr>
<th>SFRS type</th>
<th>ASCE 7-05</th>
<th>1997 UBC1</th>
<th>1997 UBC1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Wall</td>
<td>Building Frame</td>
<td>$R_{BW} / R_{BF}$</td>
</tr>
<tr>
<td>Ordinary steel concentrically braced frame</td>
<td>NA</td>
<td>3.25</td>
<td>NA</td>
</tr>
<tr>
<td>Special reinforced concrete shear walls</td>
<td>5.0</td>
<td>6.0</td>
<td>0.83</td>
</tr>
<tr>
<td>Special reinforced masonry shear walls</td>
<td>5.0</td>
<td>5.5</td>
<td>0.91</td>
</tr>
<tr>
<td>Light-framed walls with rated wood structural panels</td>
<td>6.5</td>
<td>7</td>
<td>0.93</td>
</tr>
</tbody>
</table>

1 SFRS types per ASCE 7-05 Table 12.2-1. 1997 UBC SFRS type descriptions vary slightly from the ASCE 7-05 descriptions.
Moment-resisting frame systems. Moment-resisting frames were the first structural systems expressly designed for inelastic response under expected seismic loads. Since the first Blue Book editions, they have been exempted from height limits (indeed, they have been required for tall buildings), assigned the most optimistic design parameters, and prescribed as essential backup systems for less ductile walls and braced frames: “The ductility provided by this type of framing may well prove to be the difference between sustaining tolerable and, in many cases, repairable damage, instead of catastrophic failure” (Seismology 1967, p. 45). Since then, some of the early high expectations have been shown to be premature, as poor performance led to substantial research and improved design provisions for concrete frames after the 1971 San Fernando earthquake and for steel frames after the Northridge earthquake in 1994, as is discussed in separate articles. The latest of these provisions, while representing state of the art research, have not yet been tested in large numbers by real earthquakes.

Nevertheless, current requirements for special moment-resisting frames are expected to provide as much or more ductility and energy dissipation capacity as any codified SFRS. Moment-resisting frame systems are also generally more flexible than shear wall and braced frame systems, and their design is frequently governed by code drift limits.

As shown in ASCE 7-05 Table 12.2-1, the moment-resisting frames are further classified as special, intermediate, or ordinary. Each class, and each system, has specific height limits and detailing requirements. As discussed above, the quantitative height limits are largely arbitrary, and the prohibitions are judgmental, reflecting the limits of past experience. The ductile proportioning and detailing requirements for special moment-resisting frames are appropriate for their relatively high $R$ values.

1997 UBC section 1629.6.4 defines moment-resisting frame systems as having “essentially complete” space frames to carry gravity loads. This UBC language, which echoes the longstanding description of a building frame system, goes back to early Blue Book editions (Seismology 1967, p. 46). While neither ASCE 7-05 nor the 2000 NEHRP Provisions makes a similar stipulation, their predecessor, ATC 3-06, and the NEHRP Commentary both do (ATC 1978, Table 3-B; BSSC 2001, p. 74). As discussed above, a complete space frame means gravity loads are carried by a grid of columns, not by bearing walls. The UBC thus recognized that bearing walls do not pair well with moment-resisting frames; bearing walls tend to be laterally stiff and can interfere with the intended flexural action of the moment frame. While this stipulation remains in code definitions, it is now largely unnecessary if newer code provisions for structural modeling and deformation compatibility are followed.

Since the first Blue Book, it has been accepted practice that not all bays of the space frame need to be moment-resisting (Seismology 1960, p. 22). The engineer may designate selected portions of the space frame as the actual SFRS, as long as these portions satisfy the design requirements and provide the intended behavior. The purpose is to allow the engineer to select the most effective configuration. Still, the current design parameters were assigned at a time when the typical practice involved rather complete framing, usually around the full building perimeter and sometimes through the interior as well. Over time, architectural styles, construction economics, and optimization techniques gave rise to buildings with a minimal number of discrete frames, each only one or two bays wide (Reis and Bonowitz 2000). These optimized designs, unanticipated by early code provisions, are likely to require special attention to issues involving foundation uplift, load path elements (collectors), diaphragm connections, and the behavior of large or deep structural sections. The same potential concerns apply to narrow shear walls and braced frames.

Dual systems. The use of a moment-resisting frame as backup to a shear wall or braced frame system is discussed briefly above and in other Blue Book articles.

Inverted pendulum and cantilevered column systems. “Cantilevered column” and “inverted pendulum” systems are discussed in separate Blue Book articles.

Undefined systems. ASCE 7-05 section 12.2.1 permits use of systems that are not in Table 12.2-1 “if analytical and test data are submitted that establish the dynamic characteristics and demonstrate [acceptable] lateral force
resistance and energy dissipation capacity.” 1997 UBC makes a similar allowance and enumerates, in section 1629.9.2, seven specific characteristics that must be addressed.

In general, acceptance of a proposed system is left to the discretion of the code official. Between the ASCE 7-05 and the 1997 UBC provisions, however, the Seismology Committee supports and prefers the more specific UBC requirements. Further, it is the position of the Seismology Committee that the design of undefined or not-yet-codified systems should be peer reviewed with reference to the five broad criteria listed above under System Attributes.

As noted above, it is the position of the Seismology Committee that even a “defined” system should be held to the requirements for undefined systems when the application is precedent-setting in terms of building height, SFRS aspect ratio, member span or dimension, structural material, or other parameters.

New Thinking

The four structure types in the 1960 Blue Book became 41 system types in the 1997 UBC, with 15 different values of $R$. ASCE 7-05 Table 12.2-1 lists 83 different systems with 17 different values of $R$.

As discussed above, many current code provisions and design parameters are holdovers from early Blue Book editions or from ATC-3-06. Most are based on judgment, somewhat outdated if not obsolete, and many of the early provisions are now irrelevant, contradictory, or wrong. Considering the degree to which that judgment still influences system classification and design parameters, the number of codified structural systems has grown beyond reason.

Proposals for simplifying the SFRS tables and rationalizing the assigned design parameters were considered for the 2003 NEHRP Provisions but not adopted by BSSC. It is the opinion of the Seismology Committee that the guiding principles for a new SFRS table should include:

- Fewer specific system types and fewer distinct values and combinations of numerical design parameters. Too much specificity gives an undue impression of precision.

- Usefulness as a table of basic qualification criteria. Generic design parameters would clarify the expected performance of a proposed system. Instead of parameters being assigned to each new system, a proposed new system would need to qualify (by testing, for example) for the pre-defined generic parameters.

- Distinction between basic system types based primarily on the mode of seismic resistance, not the structural material or the gravity system, in particular, elimination of the distinction between “bearing wall” and “building frame” structures. Where additional conservatism is deemed appropriate for SFRS walls that also carry substantial gravity load, any modification of design load or wall capacity should be applied to the individual wall element, not to the whole structure.

- Distinction between specific systems based on expected reliability and ductility—special, intermediate, and ordinary, in current terms—not structural material or configuration. The generic design parameters would be used to align the various system groups so as to ensure roughly equivalent performance and reliability. Modifications to the generic design parameters (presented in footnotes, perhaps) could then be used to accommodate known attributes of specific systems. Material standards would give the necessary system-specific detailing provisions.

- Consistent classification of potential dual systems. Ideally, dual systems would be removed from the SFRS table. Almost any tabulated system that is permitted in the seismic design category (SDC) of interest could be combined with a special moment frame, and the dual system design parameters could be taken as the average of the parameters of the two component systems.
Table 2 shows what such a simplified SFRS table might look like in principle for seismic design categories D and E, which account for most of California. Some systems listed as NP could be accepted in SDC B and C, and some additional prohibitions or height limits might be appropriate for SDC F. Table 2 is conceptual only. For consistency with ASCE 7-05 Table 12.2-1, additional notes will be required to for specific materials, systems, and configurations.

**Table 2.** Conceptual SFRS table (see text for explanation)

<table>
<thead>
<tr>
<th>System</th>
<th>$R$</th>
<th>$\Omega_r$</th>
<th>$C_d$</th>
<th>Height limits [ft] SDC D and E$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Light frame walls</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With shear panels</td>
<td>7</td>
<td>3</td>
<td>4</td>
<td>65</td>
</tr>
<tr>
<td>With diagonal braces</td>
<td>4</td>
<td>2</td>
<td>3</td>
<td>65</td>
</tr>
<tr>
<td><strong>Shear walls</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special</td>
<td>6</td>
<td>2</td>
<td>5</td>
<td>160$^d$</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5</td>
<td>2</td>
<td>4</td>
<td>NP</td>
</tr>
<tr>
<td>Ordinary</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>NP</td>
</tr>
<tr>
<td><strong>Braced frames</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special</td>
<td>6</td>
<td>2</td>
<td>5$^c$</td>
<td>160$^d$</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5</td>
<td>2</td>
<td>4</td>
<td>35</td>
</tr>
<tr>
<td>Ordinary</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>NP</td>
</tr>
<tr>
<td><strong>Moment-resisting frames</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special</td>
<td>8</td>
<td>3</td>
<td>5</td>
<td>NL</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>NP</td>
</tr>
<tr>
<td>Ordinary</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>NP</td>
</tr>
<tr>
<td><strong>Cantilevered columns</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>NL</td>
</tr>
<tr>
<td>Ordinary</td>
<td>1.5</td>
<td>2</td>
<td>1.5</td>
<td>NP</td>
</tr>
</tbody>
</table>

$^a$ NP = Not permitted. NL = No limit.

$^b$ Load increases (or reduced $R$ values) might be appropriate for wall elements carrying significant gravity load.

$^c$ Increased values might be appropriate for steel eccentrically braced frames.

$^d$ Height limits may be increased for some systems, similar to ASCE 7-05 section 12.2.5.4.

**Recommended Code Changes and Corrections**

**Steel concentrically braced frames.** The Seismology Committee recommends changes to 1997 UBC (and 2001 CBC) Table 16-N with respect to the design parameters and height limits for ordinary steel concentrically braced frames in both bearing wall and building frame systems.

**Steel moment-resisting frames.** The Seismology Committee recommends changes and clarifications to ASCE 7-05 Table 12.2-1 and to 1997 UBC (and 2001 CBC) Table 16-N with respect to the nomenclature, design parameters, and height limits for steel moment-resisting frames.

**Dual systems.** The Seismology Committee recommends changes to 1997 UBC (and 2001 CBC) Table 16-N as needed to prohibit the use of dual systems with ordinary moment-resisting frames in Seismic Zones 3 and 4. Also, in ASCE 7-05 Table 12.2-1, the $R$ values and height limits for some dual systems with intermediate moment frames are internally inconsistent, sometimes stipulating lower values for a dual system than applies to the same SFRS without the moment frame, as discussed above under Height Limits.
Height limits in seismic design categories D, E, and F. As discussed above under Height Limits, ASCE 7-05 relaxes the height limits for certain structures as noted in section 12.2.5.4, triggered by Table 12.2-1 notes d and e. The Seismology Committee recommends the following clarifications to ASCE 7-05 Table 12.2-1 notes d and e (with the proposed additions shown with underlining and deletions with strikethrough):

\[d\] Height limits may be increased to 240 ft (73.2 m) for some systems. See Section 12.2.5.4. for a description of building systems limited to buildings with a height of 240 ft (75 m) or less.

\[e\] Height limits may be increased to 160 ft (48.8 m) for some systems. See Section 12.2.5.4. for building systems limited to buildings with a height of 160 ft (48.8 m) or less.

In addition, the Seismology Committee recommends the following clarifications to ASCE 7-05 section 12.2.5.4.

12.2.5.4 Increased Building Height Limit for Steel Braced Frames, Special Steel Truss Moment Frames, and Special Reinforced Concrete Walls. The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (5 m) for structures assigned to Seismic Design Category F that have steel eccentrically braced frames, special steel concentrically braced frames, special steel truss moment frames, or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements in each story:

1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).
2. The braced frames, moment frames, or shear walls in any one plane shall resist no more than 60 percent of the total design seismic forces story shear in each direction, neglecting accidental torsional effects.
3. The design seismic shear in any braced frame, moment frame, or shear wall in any one plane resulting from torsional effects shall not exceed 20% of the total design seismic shear in that element.

References


**Keywords**

- bearing wall systems
- building frame systems
- combined systems
- deflection amplification factor, $C_d$
- dual systems
- ductility
- height limits
- overstrength factor, $\Omega_0$
- response modification factor, $R$ factor
- Seismic Design Categories
- seismic force-resisting system, SFRS
- special steel truss frames
- steel concentrically-braced frames
- steel moment-resisting frames
- torsion
- vertical load-carrying system
Intent of System Factors

It has been shown from past experience that a structure can be economically designed for a fraction of the elastic seismic design forces while still able to serve its intended purpose to meet the basic life safety performance objective. This design philosophy implies that inelastic behavior and damage in the structure are expected. However, a design process that incorporates inelastic time-history analysis is too complex for routine design. The intent of the Response Modification Factor, \( R \), is to simplify the design process such that only linearly elastic static analysis (i.e., the equivalent lateral force procedure) is needed for the majority of building design. The design seismic force is, thus, reduced from the elastic seismic force by a reduction factor \( R \).

The design seismic forces and the associated elastic deformations are low and do not represent the response quantities that are expected during severe earthquake shaking. While some deformation-controlled members detailed to provide ductility are expected to deform inelastically, other force-controlled members that are designed to remain elastic would experience a significantly higher seismic force level than that predicted based on the design seismic forces. To account for this effect, the code uses a Seismic Force Amplification Factor, \( \Omega_o \), such that the realistic seismic force in these force-controlled members can be conveniently calculated from the design seismic forces. (\( \Omega_o \) is termed the Structural Overstrength Factor in ASCE 7-02/05.) To control drift or to check deformation capacity in some deformation-controlled members, a similar approach is also adopted in the code that uses a Deflection Amplification Factor to predict the maximum deformations from that produced by the design seismic forces. (This factor is defined as \( C_d \) in ASCE 7-02/05.)

Implementation of System Factors

Figure 1, which shows a typical response envelope of a structure, can be used to explain the \( R \)-factor seismic design procedure. This response envelope can be established from either testing or a pushover analysis. The structure first responds elastically, which is then followed by an inelastic response as the lateral forces are increased. The redundancy that is built into the system allows a series of plastic hinges to form in the structure, leading to a yielding mechanism at the strength level \( V_Y \).

Based on the fundamental period of the structure, a designer first calculates the elastic design base shear, \( V_e \). \( V_e \) (see point E in Figure 1) is then reduced by a factor \( R \) to a design seismic force level \( V_s \) (point S), beyond which elastic analysis is not valid:

\[
V_s = \frac{V_e}{R} \quad \text{(Eq. 1)}
\]

To estimate internal forces that develop in force-controlled members for capacity design, the corresponding forces at the design seismic force \( (V_s) \) level are then amplified by a system overstrength factor, \( \Omega_o \). From an elastic analysis, the drift at the \( V_s \) level is also amplified by a deflection amplification factor, \( C_d \) to estimate the maximum (inelastic) drift; the calculated drift cannot exceed the limit allowed by the code.

Historical Development of System Factors

In order to understand the process of the evaluation of the system factors, it is helpful to review their historical development as used in the previous editions of the SEAOC Recommended Lateral Force Requirements and Commentary (Blue Book) and other seismic provisions in the US.
Response modification factor: The $R$ factor can be traced back to the $K$ factor, which appeared in the first edition of the Blue Book, (SEAOC Seismology Committee 1959). The specification of the response modification factor for the equivalent lateral force procedure has evolved from an implicit to an explicit form.

In an implicit form, the general base shear formula of $V_w = [a \text{ constant factor}] \times W$ for working stress design has been in use for many years. The form of $V_w = (ZKC)W$ was used in the 1959 Blue Book recommendations. This expression was later modified to include Importance and Site factors to become

$$V_w = (ZIKCS)W$$

(Eq. 2)

These expressions are implicit in the sense that neither the design ground motion nor the system factor is expressed in an explicit form like Eq. (1). Although the $K$ factor and the remaining terms ($ZICS$) individually do not have any physical meaning, the product, which was calibrated from experience and observed building performance in past earthquakes, does provide a reasonable estimate of the design seismic forces.

It was not until the 1985 Blue Book (SEAOC Seismology Committee 1985) that an explicit form that considered the physical principles was favored:

$$V_w = \frac{V_e}{R_w}$$

(Eq. 3)

where $V_e = (ZIC)W$. Note that the subscript $w$ in $R_w$ was provided to indicate that the $R$ factor was reducing elastic demand force levels to working stress levels. Switching from working stress design to strength design, the 1997 Blue Book (SEAOC Seismology Committee 1997) uses the form in Eq. (1) to prescribe the design seismic forces, where $V_e = (C, I / T)W$. In ASCE 7-02, $V_e$ is expressed as $S_D I / T \leq S_{DS} I$.

Although the format of design base shear has been changed over the past four decades, it can be shown that the design base shear, after some adjustment to account for the difference between working stress design and strength design, does not vary too much. The purpose of the progressive changes was not primarily to change resistance requirements, but rather to provide a rational meaning in terms of response spectrum demand and the inelastic response reduction capabilities of a given structural system.
The system factor $K$ is the forerunner of $R$. Originally, the basis for the $K$ factor was the definition of four basic lateral force-resisting systems ranging from the special moment-resisting frame with $K=0.67$ to the bearing wall system with $K=1.33$. The corresponding values for the $R_w$ and $R$ approximately follow the relation of (Rojahn and Hart 1989)

$$R_w \approx \frac{8}{K} \quad R \approx \frac{R_w}{1.4}$$

(Eq. 4)

where the 1.4 reflects the conversion from working stress design to strength design. For a given level and type of the specified site ground motion, there was no compelling reason to change the relative design load levels for the four basic systems. In general, the structural system characteristics that were considered in the evaluation of the previous and present factors include:

1. Observed and/or predicted system performance under strong ground motion.
2. Level of inelastic deformation capability.
3. Vulnerability of the vertical load-bearing system.
4. Degree of redundancy in the lateral force-resisting system.
5. Multiplicity of lines of resistance, such as back-up frames.

The structural systems were then divided into categories within which some of these characteristics were common. These categories and their abilities to meet the listed characteristics are briefly reviewed below.

**Moment-resisting frame system**
- Good performance where energy dissipative special details are used
- High level of inelastic response capacity
- System can perform inelastically without jeopardizing vertical capacity and stability
- Systems in buildings that have performed well in past earthquakes have been highly redundant

**Dual System.**
- Good elastic response control for moderate shaking; good energy dissipation for strong shaking
- Damage to primary lateral force-resisting system does not affect stability of the vertical system
- Backup system provided in the form of a moment frame

**Building Frame System**
- Performance varies depending on material and configuration
- Level of inelastic response is dependent on type of lateral system employed
- Damage to the lateral system should not lead to failure of vertical system
- Nominal backup system only in the form of a space frame

**Bearing Wall and Bearing Braced Frame System**
- Performance varies depending on material and configuration
- Lower level of inelastic response capability
- Lateral system failure could lead to vertical system failure.

The expected performance of other systems was then evaluated relative to these reference systems in order to determine the intermediate $R$ values. The same characteristics discussed above were used, and considerations focused on the following issues:

- The degree to which the system can be allowed to go beyond the elastic range, its degree of energy dissipation in so doing, and the stability of the vertical load-carrying system during inelastic response due to maximum expected ground motion. The detailing necessary to achieve this inelastic performance is covered in the design requirements and material chapters. Also, regardless of the $R$ value and its corresponding system, all elements of the structure must be adequately tied together to transfer loads to the lateral force-resisting elements.
The consequence of failure or partial failure of vertical elements of the seismic force resisting system on the vertical load-carrying capacity and stability of the total building system

The inherent redundancy of the system that would allow some progressive inelastic excursions without overall failure. One localized failure of a part must not lead to failure of the system.

When dual systems of building frame systems are employed, important performance characteristics include the deformation compatibility of the systems and the ability of the secondary (backup) system to maintain vertical support when the primary system suffers significant damage at the maximum deformation response. The backup system, when compatible with the primary system, can serve to redistribute lateral loads when the primary system undergoes degradation and should stabilize the building in the event that the primary system is badly damaged. If they are incompatible, then the total system performance may be very unreliable and therefore unacceptable.

The relationship between the load side of the design equation that includes the \( R \) reduction and the capacity or material side should not be confused. Qualities related to the seismic load modification characteristics of the structural system should be represented by the \( R \) value. Qualities related to the performance and reliability of individual structural elements should be represented by either the assigned strength or the strength reduction factor on the resistance side.

**Deflection amplification factor:** A deformation or story drift check in the force-based design procedure has been performed in either of two formats in the US: serviceability and ultimate limit state check. Prior to the 1997 SEAOC Blue Book (or UBC), the serviceability drift check was intended to minimize nonstructural damage caused by more frequent minor or moderate earthquakes; for this purpose an interstory drift limit of 0.005 of the story height is generally accepted (Freeman 1977). In the 1985 UBC, the story drift limit for the design seismic forces \( V_w \) in Eq. (2) is 0.005\( K \); note that it is dependent on the system factor \( K \). To understand the implication of including \( K \) in the drift limit, consider the minimum required structural stiffness, which is represented by the initial slope of the response curve shown in Figure 2. As both the design base shear and the drift limit contain the \( K \) factor, the minimum stiffness, which is represented by the slope of segment OW, required to minimize nonstructural damage is independent of the ductility-related system factor \( K \) because this factor is cancelled out in the design process. This practice of including the \( K \) factor in the drift limit (0.005) is justified because the threshold for nonstructural damage is the same, which is irrelevant to the structure’s ductility capacity (Uang and Bertero 1991). The serviceability drift check is performed in the elastic range because it is not expected that structural damage would occur in a minor or moderate earthquake.

After the \( K \) factor was replaced by \( R_w \) in the 1987 SEAOC Blue Book (or 1988 UBC), the story drift limit for serviceability check was converted by Eq. (4) as follows:

\[
0.005K = 0.005 \left( \frac{8}{R_w} \right) = \frac{0.04}{R_w} \quad \text{(Eq. 5)}
\]

An upper bound (0.005) was placed on the above drift limit. For longer period structures (taller than 65 ft or \( T \geq 0.7 \) sec), the above drift was conservatively reduced to 0.03/\( R_w \) with an upper bound of 0.004.

The above practice was, however, abandoned in the 1997 SEAOC Blue Book (or UBC), which was in favor of checking story drift as an ultimate limit state for severe design earthquake ground motions. Originally developed by ATC 3-06 (and NEHRP Provisions later), this second format checks inelastic story drift expected from the design ground motion for a drift limit several times larger than 0.005 of the story height. The expected inelastic drift, \( D_{ue} \), is computed by amplifying the story drift, \( D_s \), by the deflection amplification factor, \( C_d \) (see Figure 1). The associated drift limit is in the range of 0.015 to 0.025 of the story height. The 1997 SEAOC Blue Book (or UBC) follows the same approach, except that \( C_d \) is replaced by 0.7\( R \).
System overstrength factor: For capacity design, the seismic force amplification factor for force-controlled elements was first introduced in the 1987 Blue Book. Although no specific symbol was designated for this factor, the term $3R_w/8$ was used to estimate maximum seismic forces that can be developed in these elements. In the 1997 Blue Book (or UBC), the system overstrength factor, $\Omega_o$, was used for the same purpose. The same factor was later adopted in the 2000 NEHRP (or ASCE 7-02/05).

Theoretical Relationship Between System Factors
When a structural system has been designed for the design seismic load level and details established by its $R$ value, there are two behavior properties that allow the structure to perform adequately under the design earthquake ground motion. Referring to Figure 1, by idealizing the actual response envelope by an elastic-perfectly-plastic response, the system ductility factor, $\mu_s$, can be defined as $D_u/D_s$. With this ductility capacity, the elastic seismic force $V_e$ can be reduced to the yield mechanism strength $V_y$, by a ductility reduction factor, $R_d$. That is, $V_y = V_e/R_d$. Since the prediction of $V_y$ requires an inelastic analysis, for design purposes this strength level can be further reduced to the $V_s$ level beyond which first significant yield of the member, e.g., formation of a plastic hinge. The reserve of strength beyond $V_y$ is defined as structural overstrength and is measured by the system overstrength factor $\Omega_o$ ($=V_y/V_s$). Therefore (Uang 1991a),

$$R = \frac{V_y}{V_s} = \frac{V_e}{V_y} = \frac{V_e}{V_s} = R_d \Omega_o \quad \text{(Eq. 6)}$$

The $C_d$ factor is

$$C_d = \frac{D_u}{D_s} = \frac{D_u}{D_s} \frac{D_u}{D_s} = \mu_s \Omega_o \quad \text{(Eq. 7)}$$

An alternate interpretation of Eq. (6) can be made with the aid of Figure 3. Base shear $V_s$ is the design seismic force design level. The path of S to M represents the key points on the structure pushover curve. Assuming for the time being that $V_e$ is the actual linear elastic threshold capacity of the structure, the resistance path from the yield at point S to point M represents how the total resistance of the entire system increases as the understressed and redundant members become fully developed with the increasing pushover deformation. The total increase from $V_y$ to the maximum strength $V_F$ at point M is represented by the system overstrength factor $\Omega_o$. In addition, the 5 percent damped elastic base shear demand ($V_e$) for the design earthquake is along the demand path E to M. If the structure...
were to remain fully linear elastic, without yield at point S, then the resulting base shear demand would be $V_e$. However, since the structure has inelastic behavior from S to M, there are changes in the equivalent dynamic characteristics, which effectively modify the demand response spectrum from path EC to EM. Along the resistance path SM, the period increases from $T$ to $T'$ as nonlinear softening takes place, and there is a concurrent increase in the equivalent damping. This increase in effective period and damping results in a decrease in demand from $V_e$ to $V_y$ (Iwan 1980). This reduction in the dynamic response is represented by the $R_d$ factor, which is the other contributing factor in Eq. (6).

![Graph showing Seismic resistance versus demand](image)

**Figure 3.** Seismic resistance versus demand

Other forms for the $R$ factor have also been proposed. For example, Whittaker et al. (1999) suggested that a redundancy factor, $R_R$, be included as the third contributing factor in Eq. (6). Note that in this Seismic Design Recommendations article, redundancy, as well as some issues like vertical irregularity, have been treated separately and not as part of the $R$ factor.

**Practice in Other Countries**

**Canada.** The 1995 National Building Code of Canada (NBCC 1995) uses $V_t = U (V_e / R)$ to calculate the design base shear. The $R$ factor, ranging from 1 for brittle systems to 4 for the most ductile systems, accounts for ductility reduction only. A “calibration” factor $U (= 0.6)$ is used in order to achieve a design comparable to that based on previous editions. This approach effectively assumes that the system overstrength factor is 1.67 (=1/U). In the proposed 2005 edition (Heidebrecht 2003), the $R$ factor is replaced by the system ductility reduction factor, $R_d$, with a value ranging from 1 to 5, and 1/U is replaced by the system overstrength factor, $R_o$, with a value ranging from 1 to 1.7.

Note that $V_e$ in the 1995 edition represents the effect of a 475-year mean recurrence interval seismic event, while $V_t$ in the 2005 edition is based on a 2500-year mean recurrence interval seismic event without a 1/3 reduction as is used in ASCE 7-02. The deflection amplification factor is $R$ and $(R_o R_d)$ for the 1995 and 2005 editions, respectively. For typical buildings other than schools or essential facilities, the limit of the story drift ratio is 0.02 in the 1995 edition; following the SEAOC Vision 2000 recommendation (SEAOC 1995), this value is increased to 0.025 as a limit state for collapse prevention. Although the drift limit appears more relaxed, the stiffness requirement is more stringent in the proposed 2005 edition because the value of $V_e$ and its amplified displacements are much higher.

**Eurocode.** Eurocode 8 (CEN 1998) explicitly requires that both the serviceability and ultimate limit states be considered for seismic design. For the ultimate limit state, the behavior factor, $q$, is used to characterize both the
system ductility and overstrength. For reinforced concrete buildings, a basic value ($q_o$) is assigned to each type of lateral load-resisting system; the value ranges from 2 to 5. The basic value is then modified (i.e., reduced) as follows:

$$ q = q_o k_D k_R k_W \geq 1.5 $$

(Eq. 8)

where $k_D$ is used to account for the ductility capacity (1.0, 0.75, and 0.5 for high, medium, and low ductility class, respectively), $k_R$ is to account for irregularity in elevation (1.0 for regular structures, and 0.8 otherwise), and $k_W$ reflects the prevailing failure mode (less than 1.0 for walls with aspect ratio less than 3). A different format is used for steel building design. In this case, the code allows the designer to calculate explicitly the system overstrength factor ($=1/\beta_u$, which is the same as $C_s / C_s$ using the terminology in Figure 1.) In addition, a constant value for the system ductility factor is assigned to each framing system:

$$ q = C \left( \frac{\alpha_u}{\alpha_1} \right) \leq 8 $$

(Eq. 9)

Taking the ductile steel frame system as an example, the value of $C$ is 5. The value of $q$ needs to be reduced by 20% if the building has a vertical irregularity.

Once the value of $q$ is determined, the design response spectrum is reduced from $V_e$ by a reduction factor that is period dependent. The reduction factor is equal to $q$, except for the very short period range. In this range, which represents the ascending branch of the elastic spectrum, the force reduction varies from 1 (i.e., no reduction) at $T = 0$ sec to $q$ at a period where the constant acceleration region starts. A similar approach of using a period-dependent seismic force reduction factor is also used by the Mexican code.

The deflection amplification factor, termed as the displacement behavior factor $q_d$, is assumed equal to $q$. The amplified displacements are then used for, among other things, checking the $P$-$\Delta$ effect and building separation. Eurocode 8 does not specify a limit for the amplified story drift as a check for the ultimate limit state requirement. Instead, a drift limit of 0.004 to 0.006 of the story height is specified as a serviceability limit state requirement for a seismic event with a larger probability of occurrence than the design earthquake; the intensity of the serviceability design earthquake is 0.4 to 0.5 that of the design earthquake.

**New Zealand.** Like Eurocode 8, both the serviceability and ultimate limit states are considered explicitly for seismic design. No ductility is considered for the serviceability limit state, and significant member yielding is not permitted. The intensity of the design spectrum is equal to one-sixth that of the 475-year mean recurrence interval elastic design spectrum; this intensity level is significantly less than that (2/5 to 1/2) used by the Eurocode 8, but it is comparable to that implied by the 1988 UBC (1/6 to 1/8, see Uang and Bertero 1991). Drift limits intended to protect nonstructural components are also recommended.

For the ultimate limit state, the “structural ductility factor”, $\mu$, is specified. Taking either the reinforced concrete or steel moment frames as an example, the value of $\mu$ is 1.25 and 3 for “elastically responding structures” and “limited ductility” structures, respectively. For “ductile structures”, the value of $\mu$ is constant (= 10) for $T \leq 0.5$ sec, reducing linearly to 6 at $T = 0.7$ sec, and remains constant thereafter. (Based on the high values of $\mu$, it appears that the “structural ductility factor” includes both the contribution from system ductility and overstrength.) The equal displacement rule (i.e., $R = \mu$) is then applied to the period range $T \geq 0.7$ sec. Between 0.7 and 0.45 (or 0.6) sec, the seismic force reduction transitions from that based on the equal displacement rule at $T = 0.7$ sec to that based on the equal energy rule (i.e., $R = \sqrt{2\mu - 1}$) at $T = 0.45$ sec. Below which the reduction remains constant, not reducing to 1.0 at $T = 0$. sec.

As an appendix in the Commentary, a procedure is suggested that can be used to establish the $\mu$ factor for systems not defined in the code.
Japan. The Building Standard Law of Japan (IAEE 1992) adopts a two-level seismic design procedure. For the serviceability limit state (or level 1) design, the intensity of the design earthquake is one fifth that for the ultimate limit state (level 2). Member yielding is not permitted and the story drift limit is 0.005 of the story height. The level 2 design requires that the ultimate story shear strength be at least equal to $D_y V_e$. The value of the system characteristics factor $D_y$, which is period independent, varies from 0.25 to 0.5 for steel buildings. Taking ductile steel frame as an example, $D_y = 0.25$, which implies that the system ductility factor is equal to $1 / D_y = 4$. Therefore, the Japanese code specifies a minimum value for the ultimate story shear strength (i.e., $C_y$ force level in Figure 1). Although this requirement checks the ultimate story shear strength, it requires some form of nonlinear analysis. To simplify the design, the level 2 design can be waived if a more conservative seismic design base shear is used in the level 1 serviceability design (Uang 1991b).

Rational Derivation of System Factors For New Systems and Calibration for Existing Systems

One observation that can be made from a comparison of seismic codes in different parts of the world is that a two-level seismic design procedure that considers both the serviceability limit state for the more frequent, less intense design earthquake shaking and the ultimate limit state for a rare but much more intense design earthquake is used by several codes (Japan, New Zealand, and Eurocode). Although an indirect check of story drift for serviceability limit state was used in the Blue Book prior to 1997 (Osteraas and Krawinkler 1990, Uang and Bertero 1991b), such practice was abandoned in favor of checking deformations at the ultimate limit state. Since the latest trend in seismic code development has been to control both nonstructural and structural damage at different levels of earthquake intensity, it is worthwhile to evaluate which of the one- or two-level design procedures is more appropriate to achieve the stated objectives of this Seismic Design Recommendations article. It is known that the spectral shape is also dependent on the return period and seismic zone. Whether it is appropriate to use the same spectral shape for both the serviceability and ultimate limit states also needs to be examined.

The system factors ($R$, $C_o$, and $\Omega_o$) are intended for the ultimate limit state requirements. The following issues need to be considered in improving the reliability of these system factors.

Format of $R$ factor. A review of several seismic codes mentioned above shows that it is generally recognized that the $R$ factor is the product of two components: the system ductility reduction factor, $R_d$, and the system overstrength factor, $R_o$. Some codes specify constant values for these components or the product of them; lower bound values are generally specified to be conservative. But some codes (Eurocode 8 for steel design, and the Japanese code) allow the designer to quantify the component of system overstrength. At present the US seismic provisions do not allow the calculation of the $R_o$ factor. In the future it is anticipated that, as an incentive, the designer may be allowed to perform a pushover analysis (Krawinkler and Seneviratna 1998) so that a higher $R_o$ value than that assumed by the code can be used.

Period-dependent $R$ factor. The US practice has been to use a period-independent $R$ factor. Miranda and Bertero (1994), summarizing the work of a number of researchers, indicate that the system response reduction factors such as $R$ may need to be period dependent, particularly for systems with very short periods. This is because the inelastic displacements tend to be larger than those of a linear elastic system in this short period range. Note that these studies were based on a single-degree-of-freedom (SDOF) system, which does not have system overstrength because the SDOF model is determinate. The justification of this Blue Book Recommendations article and several other codes for the use of a period-independent $R$ factor is based on the following. First, the plateau (i.e., constant acceleration range) of the design response spectrum has been extended to $T = 0$ sec to replace the ascending branch of the spectrum in the very short period range; this process introduces a significant amount of system overstrength for short-period structures, which can compensate for the reduced ductility factor. Second, refining the ductility reduction may not be justified considering that the prediction of period for very stiffness building structures, where the effect of soil compliance becomes more significant, is questionable using the code period formulae. Further research on the behavior of representative building models and review of observed performance of actual structures in this period range is necessary to establish the need for and extent of a more refined period dependent-modification of the $R$ factor.
**System overstrength.** As a component of the $R$ factor, a lower bound value of the system overstrength should be used for $R_o$. But a higher bound value should be used to establish $\Omega_s$ for capacity design. The 1997 Blue Book assumes $\Omega_s = 1.1R_o$.

Some limited research has been conducted to evaluate the amount of system overstrength for certain types of lateral load-resisting systems, but a systematic study that includes all building types of different heights, width, seismic zones, non-seismic loadings (gravity and wind), etc. is yet to be conducted. One sample study (Jain and Navin 1995) on reinforced concrete frames shows that the system overstrength not only varies with the number of stories but also, much more significantly, is affected by the seismic zones; the system overstrength in low seismic zones can be several times larger than that in high seismic zones because the effect of gravity loads is more significant in the former case. Before results from a systematic study become available for establishing a more rational system overstrength factor, specifying an $R$ factor with a format similar to Eq. 9 that allows the designer to establish the system overstrength factor from nonlinear analysis is an option.

To assist in the evaluation of the system overstrength factor, the NEHRP Recommended Provisions (BSSC 2001a, 2001b) in its Commentary suggest that the factor be subdivided into three categories such that $\Omega_s = \Omega_D\Omega_M\Omega_S$ (see Fig. 4). $\Omega_D$ represents the ratio in lateral strength between Points 2 and 1 in the figure, where Point 1 is the prescribed minimum design seismic force level, and Point 2 represents the point of “nominal” first significant yield (e.g., the formation of a plastic hinge in a moment frame) based on nominal material strengths. This portion of the overstrength varies considerably from one system to another, yet it is the one that can be quantified easily by elastic structural analysis tools. First, it is system dependent. For systems like braced frames and shear wall structures, $\Omega_D$ can be very low and close to unity; for other systems like steel special moment-resisting frames whose design is usually dictated by drift limitations, it is common that the $\Omega_D$ value varies between 2 and 3. Second, $\Omega_D$ is highly dependent on the seismic zone.

$\Omega_M$ represents material overstrength. This portion of the system overstrength, i.e., the ratio in lateral strength between Points 2 and 3 in the figure reflects the difference between the nominal and actual material strengths, including strain rate effects. Reinforced masonry, concrete, and steel provisions have historically used a factor of 1.25 to account for the ratio of mean to specified strengths. A survey of wide-flange shapes indicated that the ratios of mean to specified yield strengths were 1.37 and 1.15 for A36 and A572 Gr. 50 steels, respectively. $\Omega_S$ represents the system overstrength beyond the first significant yield point (Point 2 in the figure). It is dependent on the level of

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**Figure 4.** Components of System Overstrength

$\Omega_M$ represents material overstrength. This portion of the system overstrength, i.e., the ratio in lateral strength between Points 2 and 3 in the figure reflects the difference between the nominal and actual material strengths, including strain rate effects. Reinforced masonry, concrete, and steel provisions have historically used a factor of 1.25 to account for the ratio of mean to specified strengths. A survey of wide-flange shapes indicated that the ratios of mean to specified yield strengths were 1.37 and 1.15 for A36 and A572 Gr. 50 steels, respectively. $\Omega_S$ represents the system overstrength beyond the first significant yield point (Point 2 in the figure). It is dependent on the level of
redundancy contained in the structure as well as the extent to which the designer has optimized the various elements that participate in lateral force resistance. See the NEHRP Provisions for further discussion on these components.

For residential construction, recent tests on a woodframe model house shows that the contribution of stucco on the stiffness and strength of the structure is very significant (Filiatrault et al. 2002). Testing on isolated cold-formed walls with cold-formed studs and plasterboard also shows that boundary conditions like end return walls can increase the lateral resistance by a factor of 3 (Gad et al. 1999). Ignoring components like return walls and other out-of-plane walls can significantly underestimate the system overstrength.

**System ductility factor.** Compared to the system overstrength factor, the determination of system ductility reduction factor is more complicated; the latter requires that the system ductility factor be determined first. The ductility factor for the simplest case of an SDOF system is

$$\mu_s = \frac{D_u}{D_y}$$  \hspace{1cm} (Eq. 10)

where for an idealized elastic-perfectly plastic system shown in Figure 5 (a) the yielding displacement, $D_y$, is well defined, and $D_u$ represents the maximum displacement. To establish $R$, $\mu_s$ represents the ductility capacity that the system can provide. Ductility capacity is generally established from experimented testing, most of which is conducted at the member or subassemblage level. Difficulties arise when applying the idealization to define the ductility capacity even at the member level. Park (1998) provided four possible definitions for each of $D_y$ and $D_u$ (see Figure 6). While these definitions may be appropriate for some types of structural components (e.g., reinforced concrete or steel members) with clearly defined significant yielding, it is not obvious how to apply the definitions to other types of components where the “first significant yield” is difficult to define. As an example, Figure 5(b)-b shows the cyclic response of an isolated woodframe shear wall. The hysteretic behavior is highly nonlinear and does not show a distinct yielding point due to the distributed yielding and fracture of the nailed connections. Depending on the definition in Figure 5 (a), the value of $D_u$ can vary considerably. Furthermore, the determination of $D_u$, which represents the usable deformation capacity at “failure,” is also troublesome because structural components usually exhibit strength degradation in addition to stiffness degradation at large deformation levels. To define $D_u$, Park (1998) suggested that it is reasonable to recognize at least part of the post-peak deformation capacity, see Figure 6(b). But no standard rules exist that define the allowable strength degradation. According to Appendix C4.A of the New Zealand code, the strength cannot degrade below 80% of the peak strength. In the US, the AISC Seismic Provisions in its Appendix S uses 80% of the nominal beam strength as the acceptance criteria for determining rotation capacity of welded steel beam-column moment connections. A consensus-based definition of $D_y$ and $D_u$ for the determination of ductility capacity, even at the component level, is needed.

![Deformation Load](a) Elastic-perfectly plastic idealization

![Wall Force (kips) Drift (in)](b) Hysteresis response of woodframe shear wall (Gatto and Uang 2001)

**Figure 5.** Definition of Ductility Capacity
To simplify the discussion when it is extended to the member ductility capacity at the story level, consider a one-story frame with a yielding beam element supported by two elastic columns pinned at the base. The story ductility capacity (or factor) is the same as the member (rotational) ductility capacity only when the columns are rigid. Otherwise, the story ductility capacity is less than the member ductility capacity, because the story ductility capacity, which is a function of the relative flexural stiffness between the beam and column, reduces as the columns become more flexible.

The problem is further complicated when one extends the concept of story ductility to system ductility in a multistory building. Since the concept of modal response breaks down for an inelastic system, opinions are divided as to what scalar quantities should be used as the force and deformation in defining system ductility. Base shear or overturning moment has been used by some as the force term; roof lateral displacement or the story drift in a critical story have been used as the deformation term. The “first significant yield” for defining $D_y$ can be either determined from a pushover analysis, or an inelastic time-history analysis in which the first yield can be strongly influenced by the higher mode effect. Depending on the definitions used, it should not be surprising that the values of system ductility can vary considerably (Reyes-Salazar 2002). Further study is needed to establish a rational definition of the system ductility factor.

The system ductility capacity is also strongly influenced by the type of yielding mechanism that may form. Capacity design provisions, which are required for moderate and high seismic regions, appear effective in mitigating weak-story mechanisms. But studies show that a global yielding mechanism may not always be achieved. The system ductility capacity can be reduced significant if the damage is concentrated in a limited number of stories.

**System ductility factor.** Assuming that a system ductility factor for the deformation capacity of the structure can be defined, several modern codes assume that the ductility reduction rule like that proposed by Newmark and Hall (1982) for SDOF systems can apply. Studies that support this postulation do not exist.

**Deflection amplification factor.** A comparison of the deflection amplification factor in relation to the response modification factor of several seismic provisions is summarized in the following table:
### Table 1 Comparison of Response Modification and Deflection Amplification factors

<table>
<thead>
<tr>
<th>Seismic Provisions</th>
<th>Response Modification factor</th>
<th>Deflection Amplification factor</th>
<th>Deflection Amplification Factor</th>
<th>Response Modification factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>UBC</td>
<td></td>
<td>(3/8) (R_w)</td>
<td>0.375</td>
<td>0.7</td>
</tr>
<tr>
<td>1994</td>
<td></td>
<td>(0.7R)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1997</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASCE 7-02</td>
<td>(R)</td>
<td>(C_d)</td>
<td>0.5–1.0</td>
<td></td>
</tr>
<tr>
<td>Eurocode 8</td>
<td>(q)</td>
<td>(q)</td>
<td>1.0(^c)</td>
<td></td>
</tr>
<tr>
<td>Mexico</td>
<td>(Q)</td>
<td>(Q)</td>
<td>1.0(^c)</td>
<td></td>
</tr>
<tr>
<td>New Zealand</td>
<td>(\mu)</td>
<td>(\mu)</td>
<td>1.0(^c)</td>
<td></td>
</tr>
<tr>
<td>NBC of Canada</td>
<td></td>
<td>(R)</td>
<td>(U = 0.7)</td>
<td></td>
</tr>
<tr>
<td>1995</td>
<td></td>
<td>(R/R_o)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2005 (proposed)</td>
<td></td>
<td>(R/R_o)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^{a}\)period-dependent in the short period range; reduces to 1.0 at \(T = 0\) sec.

\(^{b}\)period-dependent in the 0.45 (or 0.6)–0.7 sec range; does not reduce to 1.0 at \(T = 0\) sec.

\(^{c}\)greater than 1.0 in the short period range.

Note that the \(C_d/R\) ratios as specified in ASCE 7-02, range from 0.5 to 1.0. Two observation can be made. First, the theoretical relationship between this ratio can be derived from Eqs. (6) and (7):

\[
\frac{C_d}{R} - \frac{\mu \Omega_o}{R_o \Omega_o} = \frac{\mu \Omega_o}{R_o}\]

(11)

For an SDOF system, Newmark and Hall (1982) suggested that the \(C_d/R\) ratio should be equal to 1.0 in the equal displacement range, and larger than 1.0 in the shorter period range. Table 1 shows that, except for the US seismic provisions, codes of all other countries follow this rule.

Study on the deflection amplification factor is limited. Based on a study of four instrumented multistory buildings, one study showed that \(C_d\) is less than \(R\) (about 0.7 to 0.9\(R\)) if it is used to estimate roof lateral displacement. But \(C_d\) can be larger than \(R\) for story drift estimation, especially when \(T/T_g\) is larger than 0.3, where \(T_g\) is the predominant period of the ground motion (Uang and Maarouf 1994). Further study is needed to justify the US practice of specifying a \(C_o\) which is less than \(R\). If it is demonstrated that the values of \(C_o\) should be increased to \(R\), such a revision would result in much stiffer structures. Unless the consensus based on the observed building performance from past earthquakes indicates a need for stiffer structures, the impact of increasing the \(C_o\) factor can be mitigated by increasing the drift limits, which may reflect the more realistic drift limit at failure.

### Remaining Challenges

Current efforts (Applied Technology Council 2007) are being made to quantify components that contribute to the system factors. Considering the complexity of a building structure with components such as gravity load-carrying systems and “nonstructural” components that are not accounted for, and the limitations of the tools for analyzing idealized models, it is expected that in the foreseeable future engineering judgment and lessons learned from observed building performance after earthquakes will also continue to play a vital role for adjusting the values of these system parameters.

One example that current codes still cannot address is the issue of permanent drift, which can be large especially for near-fault ground motions. A system (e.g., buckling-restrained braced frame) can have excellent energy dissipation capacity, but large permanent drift may not be acceptable by the building owner. Traditionally, the better estimated performance of the dual systems has resulted in assigning high \(R\) values and more relaxed height limitations. While the strength requirement for the backup steel moment-resisting frames is relatively low, the large deformability of the moment frames which remain elastic long after the structural walls or braced frames have experienced damage provides a restoring mechanism to minimize permanent drift of the building. While no provisions on permanent drift currently exist, as an incentive for better performance of the building after a major seismic event the values of \(R\)
factor for those that fall in the Building Frame Systems may be adjusted downward slightly and higher values of $R$ are assigned to those designed as a Dual System.

The $R$ factor formulation in Eq. (6) shows that two systems, one with a high ductility capacity and low overstrength and the other with a low ductility capacity and high overstrength, can have the same value of $R$. Nevertheless, numerous studies have shown that the largest source of uncertainty in predicting seismic response is from the earthquake ground motion input. It has been the approach of this Blue Book Recommendations article to prefer ductility capacity through proper ductility requirements to strength to deal with seismic action higher than assumed. On this basis, although two systems just mentioned should conceptually have the same value of $R$, it may be appropriate to assign a higher weighting factor to ductility capacity than overstrength.

The $R$-factor design approach was developed as a compromise to achieve an economical design by accepting inelastic action in the structure, yet a greatly simplified elastic analysis is preferred for routine design. Although a close examination of this design procedure and the associated system factors shows that the components that contribute to these factors can be complicated, it is worthwhile to keep this basic intent in mind. Future efforts to improve this force-based method and to rationalize the system factors that greatly complicate the design process may defeat the original intent.

**Keywords**
deflection amplification factor, $C_d$
equivalent lateral force procedure
overstrength factor, $\Omega_0$
response modification factor, $R$ factor

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SEAOC Seismology Committee (1959). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.


**How To Cite This Publication**

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2007)

In the writer’s reference list, the reference should be listed as:

**History**

The first Blue Book commentaries published by the Structural Engineers Association of California saw potential benefits in a ductile moment-resisting frame acting as a “secondary line of defense” (SEAOC Seismology Committee 1960, p. 22), limiting the response and assuring the safety of a structure after stiffer wall elements had cracked (SEAOC Seismology Committee 1967, p. 43). San Francisco structural engineer and Seismology Committee member Henry Degenkolb was an early proponent of what would become the modern dual system, preferring earthquake-resistant structures with a “primary system for stiffness and a backup system for toughness” (Degenkolb 1994). This could be restated in current terms as “a primary system for serviceability and a secondary or backup system for ductility.” The concept of the dual system is believed to have grown out of observations of buildings that survived the 1906 San Francisco earthquake. Many surviving multistory buildings studied after that event had steel frames with infill walls, effectively providing a sort of dual system.

Modern dual systems have been built in California at least since the 1950s, but their use has not been prevalent. The most common usage has been in mid- and high-rise buildings taller than 160 ft for which past editions of the UBC required a special moment-resisting frame and in which a shear wall or braced frame core was used to reduce drift. Dual systems have also occasionally been used in shorter buildings to justify higher \( R \) factors than those assigned to shear wall or braced frame systems, or when the additional cost of potentially improved performance was justified by the importance of the building.

Dual systems were first codified in the 1959 Blue Book (Table 23-C) and the 1961 UBC (Table 23-F):

> Buildings with a complete horizontal bracing system capable of resisting all lateral forces, which system includes a moment resisting space frame which, when assumed to act independently, is capable of resisting a minimum of 25% of the total required lateral force.

These systems were assigned \( K \) factors of 0.8, equivalent to a current \( R \) of about 7. The shear wall or braced frame system was designed for 100% of the lateral seismic design forces, and the moment resisting frame was designed independently for 25% of the same forces. A dual system structure was not considered as good as one with a “ductile moment resisting space frame” taking 100% of the seismic force, which was allowed to use a \( K \) of 0.67 (\( R \) of about 8.5).

The 25% value was and still is “judgmentally selected” (BSSC 2001, section 5.2.2.1).

By the 1985 UBC (Table 23-I) the definition had expanded:

> Buildings with a dual bracing system consisting of a ductile moment-resisting space frame and shear walls or braced frames using the following design criteria:

a. The frames and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames.

b. The shear walls or braced frames acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces.

c. The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.

In 1988 UBC section 2312(d)5, the dual system requirements became somewhat more lenient, as the primary walls or braced frames were no longer required to be designed for 100% of the design forces:

> Dual system. A structural system with the following features:
(i) An essentially complete space frame which provides support for gravity loads.
(ii) Resistance to lateral load is provided by a specially detailed moment-resisting space frame (concrete or steel) which is capable of resisting at least 25 percent of the base shear and shear walls or braced frames.
(iii) The two systems shall be designed to resist the total lateral force in proportion to their relative rigidities.

Though codified in 1988, this change in requirements for the primary system design forces from resisting 100% of the design seismic force to only a portion based on relative rigidity was also made in ATC 3-06 (1978, Table 3-B).

Also in 1988, the $R_w$ value for dual systems with special moment-resisting frames (SMRFs) rose to match that for SMRFs alone. The values of $R_w$ for dual systems with concrete shear walls or steel EBFs was set at 12, while the value for dual systems with steel concentric braced frames was set at 11.

Thus, $R_w$ was increased in 1988 while the total required strength of the dual system was also reduced. The explanation for this apparent contradiction involved perceived conservatism in traditional dual system design practice and a recognition of new analytical tools: “[The 100% rule] permitted design without the necessity of evaluating interaction effects with the moment-resisting frame. With the availability of computer programs for the analysis of the complete structure model, the walls or braced frames can be designed for more realistic interaction forces, and the 100 percent rule is no longer needed” (SEAOC Seismology Committee, 1990, commentary section 1D.6.d.2). The “more realistic” interaction forces were still based on elastic analyses, however. Elastic force distributions are still not “realistic” in the inelastic range in which structures actually perform.

The 2000 NEHRP Commentary states that a dual system has “a redundant lateral-force-resisting system that is a moment frame …” (BSSC, 2001, section 5.2.2.1). Similarly, ATC 3-06 (1978, section C3.3.4) described the moment frame as a “secondary defense system with higher degrees of redundancy and ductility.” These might have been true statements in the days when the shear walls or braced frames were required to resist 100% of the design shear, but now that the forces are distributed according to stiffness, the moment frame can no longer be considered “redundant” to the primary system. Although the Seismology Committee is not recommending an immediate return to the 100% approach, it is important to recognize that as the codes have evolved, the required strength of dual systems has been significantly reduced. (The term “redundant,” as used in this context by the NEHRP and ATC 3-06 commentaries, has nothing to do with the code’s redundancy coefficient.)

The 1988 UBC approach provides the basic background for the current provisions in both ASCE 7-05 and the 2001 CBC (or 1997 UBC). Since 1988, the code has permitted many more types of dual systems. It is clear from 1988 and earlier provisions that the moment frame in a dual system was intended to be highly ductile, similar to what is now known as a Special Moment-Resisting Frame. In attempts to be more specific and comprehensive, and to accommodate conditions in areas of lower seismicity, national model codes and standards now prescribe seismic design parameters for many untried or untested systems. Some of these might not perform in the manner traditionally expected for dual systems.

**Current Practice**

Dual systems are defined in ASCE 7-05 section 12.2.5.1 and 2001 CBC section 1629.6.5. The two definitions are essentially the same, with different wording. The ASCE 7-05 version reads:

For a dual system, the moment frames shall be capable of resisting at least 25 percent of the design seismic forces. The total seismic force resistance is to be provided by the combination of the moment frames and the shear walls or braced frames in proportion to their rigidities.

The CBC/UBC is somewhat more definitive. A dual system must satisfy the following:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

The CBC definition is clearer in the requirement for “an essentially complete space frame,” although it is believed that this is also the intent of ASCE 7-05. The position of the Seismology Committee is that the same requirement should be met when ASCE 7-05 is used for design.

The term “essentially complete space frame” lacks definition, although the implied intent is that the frame should be capable of providing complete support of gravity loads while also being able to resist at least 25 percent of the design seismic forces if the primary system were completely removed from the building. This is suggestive of the taller San Francisco buildings that survived the 1906 earthquake that were cited for the original adoption of dual system provisions into the building code.

However, it is not always necessary nor even beneficial to create column-like or beam-like elements within walls, since these elements change the reinforcement patterns and the resulting ductility of the walls (i.e., column reinforcing provided within a shear wall will act as boundary reinforcing, not as a column). Likewise, when braced frames are present the typical bracing connections will conflict with the normal behavior of any beam-column moment connections. Therefore, it is better within the context of the design rules currently defined for dual systems to physically separate at least the beams of the secondary moment frame from the elements of the primary lateral load resisting system. Current understanding of the inelastic response of a superimposed system such as a moment frame embedded within a shear wall would suggest that the design would need to consider the bracing effects of cracked wall segments acting as compression diagonals within the infilled frame bays, which have not been defined in standard building codes (ASCE 2000, 2003). However, structures where the secondary moment frame is laterally offset from an adjacent wall system are not infill structures and are considered valid dual systems.

**Moment frame design.** The design of the secondary moment frame requires using an envelope of the maximum forces from two separate analysis - first, an analysis of the combined primary and secondary system using the full design seismic force to determine forces in all primary and secondary elements, and second, an analysis of the moment frame and gravity system, but without the elements of the primary system, using both gravity loads and lateral forces meeting the 25% design requirement.

Determination of the 25% design requirement for the moment frame calls for some code interpretation. Using the UBC or CBC, the 25% forces are traditionally based on the distributed base shear of the equivalent lateral force procedure (sections 1630.2 and 1630.5), using tabulated $R$ values for the dual system and $T$ values for the total dual structure ($C_t = 0.02$ by default). The 25% design forces for the moment frame using ASCE 7-05 should be based on a vertical force distribution with the exponent $k$ derived for the total structure (section 12.8.3).

With either the CBC or ASCE 7, the requirement to “resist” 25% of the design forces or design base shear may be thought of as a minimum strength requirement. Moment frame drifts under the 25% forces condition need not be checked even if they exceed the drifts of the dual system under the 100% load condition.

Code moment frame design and detailing requirements that the seismology committee considers applicable to the design of the secondary moment frame include the use of special load combinations, consideration of torsional effects on the moment frame as a stand-alone system, and ductile detailing provisions for frame connections. The seismology committee does not consider it necessary to include any special redundancy factor calculations for the stand-alone moment frame.

In addition, if moment frame columns are used as shear wall boundary elements, they must be designed for the required overturning actions in the wall.
If the design of a dual system design uses a dynamic analysis procedure to determine design forces for the combined system, the requirement to determine design forces for 25 percent of the design base shear in the moment frame does not mean that an additional dynamic analysis is also required for the secondary moment frame. The 25 percent requirement may be met by a separate static analysis of the secondary moment frame, using 25 percent of the base shear required for the frame as a whole vertically distributed with the exponent k derived for the total structure.

**Dual system types.** ASCE 7-05 Table 12.2-1 lists 21 types of dual systems, 13 with SMRFs and 8 with IMRFs. It is based on the 2003 NEHRP Provisions (BSSC 2004), which list 13 types with SMRFs and 8 with IMRFs. 2001 CBC Table 16-N has 15 types: 6 with SMRFs, 3 with IMRFs, 5 with OMRFs, and one with Masonry Moment Resisting Wall Frames (MMRWF). In ASCE 7-05, dual system R factors range from 8 to 3. In the 2001 CBC, R factors range from 8.5 to 4.2.

Table 1 summarizes the different types. A dash in the table cell indicates that the system is not codified. “NP” indicates those dual systems that are not permitted for building structures by the CBC/UBC for Seismic Zones 3 and 4 or by ASCE 7-05 for Seismic Design Category D. (Some are permitted for SDC D but not for SDC E or F.)

Table 1 reveals significant differences between the reference documents. Several of the systems not permitted by ASCE 7 for high seismicity are simply not included in the CBC. Similarly, composite systems have not been included in the CBC, although some may have sufficient ductility for use in California. The CBC does not distinguish “ordinary” and “special” shear walls; shear walls in the CBC are similar to those defined as special in ASCE 7-05. Further, the definitions and provisions for steel IMRF and OMRF are different in the two documents.

Table 1 shows that many of the dual systems using IMRFs or OMRFs have R factors that are about the same or lower than R for the shear wall or braced frame system alone. These dual systems therefore do not seem to offer any significant design advantage. For these reasons, and as discussed further below, it is the position of the Seismology Committee that these dual systems should not be used for earthquake design. Any restriction on design freedom caused by this position is mitigated by the fact that most of the systems in question are already Not Permitted or severely limited for the high Seismic Design Categories that apply throughout most of California.

Some new systems were added to ASCE 7-05 in 2005 that resemble dual systems although they are not categorized as such. Separate and greater R-values are tabulated for both eccentric braced frame and buckling-restrained braced frames, if moment-resisting beam-column connections are provided in combination with the braced frames. No specific design rules have been included to define any other strength requirements for the beams or columns than what is given for the combined system, and connection detailing requirements (SMF, IMF or OMF) have not been defined. The Seismology Committee recommends that the beam-column connection comply with special moment frame detailing provisions, and that potential interactions between braced frame connection plates and the connections be specifically considered in the connection design or the condition avoided.

Another structural system added in ASCE 7-05 that has some resemblance to a dual system is the “Shear wall-frame interaction system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls.” This system is poorly defined and has poor ductility, but is only permitted in SDC B.

Both ASCE 7-05 and the 2001 CBC set height limits on certain dual systems. In general, the limits on dual systems simply match those on the component systems. As noted in the NEHRP Commentary, the limits themselves (typically 100 ft or 160 ft) are essentially arbitrary (BSSC, 2001, section 5.2.2.4). The 160 ft limit derives originally from a Los Angeles Building Code provision related to height limits in that city (Layne et al., 1963). Table 2 presents a summary of height limit restrictions defined in ASCE 7-05. For many non-moment frame primary systems such as shear wall or braced frame systems, providing a secondary moment frame provides the only code-defined way to use these systems in buildings greater than 160 feet in height. Although these existing code height limitations may be arbitrary, this table represents the most common reason for when dual systems are used today.
### Table 1. Comparison of dual systems in ASCE 7-05 and 2001 CBC

<table>
<thead>
<tr>
<th>Shear wall or braced frame type</th>
<th>ASCE 7-05 ( R ) for non-dual building frame system</th>
<th>ASCE/CBC ( R ) value for dual system</th>
<th>2001 CBC ( R ) value for dual system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SMRF</td>
<td>IMRF</td>
<td>OMRF</td>
</tr>
<tr>
<td>Steel EBF, moment-resisting connections away from links</td>
<td>8 / NA</td>
<td>8</td>
<td>—</td>
</tr>
<tr>
<td>Steel EBF, non-moment-resisting connections</td>
<td>7 / 7</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Steel SCBF</td>
<td>6 / 6.4</td>
<td>7</td>
<td>6, NP</td>
</tr>
<tr>
<td>Special reinforced concrete shear wall</td>
<td>6 / 5.5</td>
<td>7</td>
<td>6.5</td>
</tr>
<tr>
<td>Ordinary reinforced concrete shear wall</td>
<td>5 / NA</td>
<td>NP</td>
<td>NP</td>
</tr>
<tr>
<td>Composite EBF</td>
<td>8 / NA</td>
<td>8</td>
<td>—</td>
</tr>
<tr>
<td>Composite CBF</td>
<td>5 / NA</td>
<td>6</td>
<td>5.5</td>
</tr>
<tr>
<td>Composite steel plate shear wall</td>
<td>6.5 / NA</td>
<td>7.5</td>
<td>—</td>
</tr>
<tr>
<td>Special composite reinforced concrete shear wall with steel elements</td>
<td>6 / NA</td>
<td>7</td>
<td>—</td>
</tr>
<tr>
<td>Ordinary composite reinforced concrete shear wall with steel elements</td>
<td>5 / NA</td>
<td>NP</td>
<td>—</td>
</tr>
<tr>
<td>Special reinforced masonry shear wall</td>
<td>5.5 / 5.5</td>
<td>5.5</td>
<td>—</td>
</tr>
<tr>
<td>Intermediate reinforced masonry shear wall</td>
<td>4 / NA</td>
<td>NP</td>
<td>NP</td>
</tr>
<tr>
<td>Bucking-restrained braced frame, moment resisting beam-column connections</td>
<td>8</td>
<td>8</td>
<td>—</td>
</tr>
<tr>
<td>Special steel plate shear walls</td>
<td>7</td>
<td>8</td>
<td>—</td>
</tr>
<tr>
<td>Ordinary steel CBF</td>
<td>3.25 / 5.6</td>
<td>NP</td>
<td>NP</td>
</tr>
<tr>
<td>Ordinary reinforced masonry shear wall</td>
<td>2 / NA</td>
<td>—</td>
<td>NP</td>
</tr>
<tr>
<td>Ordinary composite braced frame</td>
<td>3 / NA</td>
<td>—</td>
<td>5.5, NP</td>
</tr>
<tr>
<td>Ordinary composite reinforced concrete shear wall with steel elements</td>
<td>5 / NA</td>
<td>NP</td>
<td>NP</td>
</tr>
<tr>
<td>Masonry shear wall</td>
<td>NA / 5.5</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

**Performance Issues**

The nature and construction of dual-system structures have evolved significantly from the time when the system was first identified to today. The original buildings cited for comparison had highly perforated but nearly continuous wall systems around the entire perimeter, with complete space frames resembling moment frames located either directly adjacent to or embedded within the wall systems. Today’s systems generally have one or more “core” elements as primary systems with separated secondary moment frames located around the building perimeter, providing both the desired redundancy and supplemental torsional stability to the structure. These systems become interactive as the building height increases. In low-rise structures there is very little interaction between the two...
systems and the primary system tends to resist the majority of seismic forces throughout the structure height. In mid-rise and tall structures, the differences in lateral drift characteristics between the primary system and the secondary moment frame tends to dominate design, so that the secondary moment frame in the upper stories tends to resist nearly 100% of the design seismic forces, while in the lower stories the primary system resists the majority of forces. Therefore in mid-rise and tall structures the expected inelastic behavior transitions from moment-frame behavior in the upper portion to dual-system behavior near the base.

Table 2. Comparison of Height Limits for Dual Systems in ASCE 7-05

<table>
<thead>
<tr>
<th>Primary System</th>
<th>w/o Dual System</th>
<th>with Intermediate Moment Frames capable of resisting at least 25% of prescribed seismic forces</th>
<th>with Special Moment Frames capable of resisting at least 25% of prescribed seismic forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special Reinforced Concrete Shear Walls</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>B, C = NL, D = 160 ft, E, F = 100 ft</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Composite Steel &amp; Concrete Concentric Braced Frames</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>B, C = NL, D = 160 ft, E = 100 ft, F = NP</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Special Steel Concentric Braced Frames (SCBF)</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>B, C = NL, D = 35 ft, E, F = NP</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Composite Steel &amp; Concrete Eccentric Braced Frames</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Steel Eccentric Braced Frames (EBF)</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Composite Steel Plate Shear Walls</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Special composite reinforced concrete shear walls with steel elements</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Special Reinforced Masonry Shear Walls</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Buckling-Restrained Braced Frame (BRBF)</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Special steel plate shear walls</td>
<td>B, C = NL, D, E = 160 ft, F = 100 ft</td>
<td>—</td>
<td>B, C, D, E, F = NL</td>
</tr>
<tr>
<td>Ordinary Reinforced Concrete Shear Walls</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
</tr>
<tr>
<td>Ordinary composite reinforced concrete shear walls with steel elements</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
</tr>
<tr>
<td>Intermediate Reinforced Masonry Shear Walls</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
</tr>
<tr>
<td>Ordinary composite braced frames</td>
<td>B, C = NL, D, E, F = NP</td>
<td>B, C = NL, D, E, F = NP</td>
<td>—</td>
</tr>
</tbody>
</table>
Important performance characteristics of dual system behavior include the deformation compatibility of the combination of systems and the ability of the secondary (backup) system to maintain vertical support when the primary system suffers significant damage at the maximum deformation response. The backup system, when compatible with the primary system, can serve to redistribute lateral loads when the primary system undergoes degradation and should stabilize the building in the event that the primary system is badly damaged. If the deformation and ductility characteristics of the two systems are incompatible, then the total system performance may be very unreliable and therefore unacceptable. For example, a highly ductile buckling-restrained braced frame system would not be compatible with a secondary intermediate moment frame system.

In mid-rise or tall buildings, rotation demands on the moment frame joints in the upper parts of the building may be large, and the intended dual system performance might be reversed; that is, the moment frames might yield first, with the stiff core acting as the secondary or backup system. Given these scenarios, the moment frames in mid-rise or tall buildings should be required to be SMRFs, while those in the low-rise buildings might not need the same level of ductility capacity, or would derive little benefit from a having dual system.

These issues are not yet adequately considered by current building codes. For example ASCE 7-05 Table 12.2-1 allows several different dual systems with IMRFs for SDC D, which covers standard occupancies in much of California. For a 12-story building (about 160 ft tall), the IMRF, as described above, could be the unintended primary system in the upper floors. In the same table, however, steel IMRFs are not permitted as stand-alone systems in multi-story SDC D buildings taller than 35 ft, and concrete and composite IMRFs are not permitted at all. If the shear walls and braced frames were designed for 100% of the design shear, this inconsistency would be less of a concern. But with design forces distributed according to relative rigidities with elastic analysis, the ductility of the moment frame system is a concern, and SMRF systems should be mandatory. It should be recognized that the height limits for a dual system with IMRF might be more restrictive than that of the primary system itself, since the design of the primary system by itself in a mid-rise or tall building would generally need to be more substantial in order to meet drift limits.

Although there is criticism of the generally arbitrary nature of the existing height limits in building codes (SEAOC Seismology Committee, 1960, p. 43; ATC, 1978, section C3.3.4.), dual systems are an example where the nature and limitations of the systems themselves appear to warrant height limits. A more rational approach to height limits for dual systems would be based solely on the potential characteristics of each system type and the performance intentions of the structure Occupancy Category, and not on geographic location. However, the political nature of building codes may continue to require some definition of height limits for dual systems based on Seismic Design Category.

The Seismology Committee is not aware of any performance assessments of dual system buildings in past earthquakes. Dual system buildings were excited by the 1989 Loma Prieta earthquake but have not been targeted for comparative study, and dual systems were relatively rare in the areas strongly affected by the 1994 Northridge earthquake.

**Recommendations**

As noted above, the Seismology Committee position is that dual systems with IMRFs or OMRFs should not be used in SDC D, E, or F California. Also, when the ductility of the moment frame is as low, or lower, than that of the shear wall or braced frame system, the concept seems to be at odds with the original intent of the dual system.

In ASCE 7-05 Table 12.2-1, the \( R \) values and height limits for some dual systems with intermediate moment frames are internally inconsistent.

Other potential directions for the development of dual system code provisions include:

- Allow any combination of lateral systems, as long as a properly substantiated nonlinear analysis is performed, and all elements of the system are rationalized for the displacements to which they are subjected. For dual systems, the nonlinear analysis would need to check the compatibility of the moment frame with the shear wall or braced frame and confirm that the expected moment frame joint rotations are acceptable.
• A rational approach to definition of the 25% strength basis for the secondary system needs to be established. For some combinations of systems a greater or lesser strength basis may be beneficial or warranted, or a nonlinear analysis may be needed to justify use of the system.

• New structural systems that have been added for EBF and BRBF systems that provide separate $R$ values when moment connections are provided at beam-column connections lack adequate definition of the moment frame system and are contributing to code complexity. These types of variant combinations should be folded into the dual system provisions, eliminated or avoided where possible.

As further developments in design procedures progress toward more explicit recognition of actual expected forces and deformations, it is likely that design of one system within a complex dual system for a portion of a somewhat artificial base shear will be de-emphasized.

Dual systems with SMRFs as defined in the 2001 CBC and ASCE 7-02 are expected to meet code-level performance objectives. Still, the Seismology Committee recommends additional study to identify optimal system combinations and to define the optimal strength and stiffness requirements for both parts of the dual system. Although the 25% strength requirement for the moment frame has been codified since the 1961 UBC, its appropriateness has not been systematically verified by nonlinear analysis. Modern analytical techniques could be used to determine ideal strength and stiffness for the moment frame. Such studies would likely indicate different design solutions for different building heights and system aspect ratios.

Recent nonlinear studies of various lateral-force resisting systems have demonstrated that collapse prevention requires adequate stiffness to resist $P$-$\Delta$ effects at actual drift levels (FEMA 2005, Section 4.4). Further study to determine $P$-$\Delta$ effects on moment frames in dual systems might improve their efficacy.

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Keywords
dual systems
height limits
moment frames
overstrength factor, $\Omega_0$
response modification factor, $R$ factor,
seismic force-resisting system, SFRS
special moment-resisting frame, SMRF

How To Cite This Publication

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2007)

In the writer’s reference list, the reference should be listed as:

Cantilever column systems are seismic force-resisting systems in which the lateral forces are resisted entirely by columns that act as vertical cantilevers. Cantilever columns provide a simple alternative to a moment frame, braced frame, or shear wall for a variety of low-rise structures. In particular, they are useful in low-rise light-frame construction of two stories or less in combination with other structural systems to provide resistance along an independent line of resistance, as may be needed for an addition to an existing building. Cantilever column elements currently compete with proprietary prefabricated shear panels and proprietary light-gauge braced elements in light-frame shear wall systems.

System Factors and Height Limits
ASCE 7-05 (ASCE 2006) references detailing requirements that pertain to other systems in its tabulation of system factors for cantilever column systems. That tabulation is reproduced here as Table 1. However, when it comes to guidance on applying the other system requirements, little or no further guidance is provided, other than pointing to the respective materials reference standards. It is the SEAOC Seismology Committee’s opinion that this means the column is to be designed as a frame column with flexure to meet the requirements of the respective materials standards.

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section where Detailing Requirements are Specified</th>
<th>Modification Coefficient, R note a</th>
<th>System Overstrength Factor, Ω note g</th>
<th>Deflection Amplification Factor, Cd note b</th>
<th>Structural System Limitations and Building Height (ft) Limit note c</th>
</tr>
</thead>
<tbody>
<tr>
<td>G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:</td>
<td>12.2.5.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Special steel moment frames</td>
<td>12.2.5.5 and 14.1</td>
<td>2/2</td>
<td>1/4</td>
<td>2/2</td>
<td>35 35 35 35 35</td>
</tr>
<tr>
<td>2. Intermediate steel moment frames</td>
<td>14.1</td>
<td>1/2</td>
<td>1/4</td>
<td>1/2</td>
<td>35 35 35b NPb,i NPb,i</td>
</tr>
<tr>
<td>3. Ordinary steel moment frames</td>
<td>14.1</td>
<td>1/4</td>
<td>1/4</td>
<td>1/4</td>
<td>35 35 NP NP NP</td>
</tr>
<tr>
<td>4. Special reinforced concrete moment frames</td>
<td>12.2.5.5 and 14.2</td>
<td>2/2</td>
<td>1/4</td>
<td>2/2</td>
<td>35 35 35 35 35</td>
</tr>
<tr>
<td>5. Intermediate concrete moment frames</td>
<td>14.2</td>
<td>1/2</td>
<td>1/4</td>
<td>1/2</td>
<td>35 35 NP NP NP</td>
</tr>
<tr>
<td>6. Ordinary concrete moment frames</td>
<td>14.2</td>
<td>1</td>
<td>1/4</td>
<td>1</td>
<td>35 NP NP NP</td>
</tr>
<tr>
<td>7. Timber frames</td>
<td>14.5</td>
<td>1/2</td>
<td>1/2</td>
<td>1/2</td>
<td>35 35 35 35</td>
</tr>
</tbody>
</table>

notes

a Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.

b Reflection amplification factor, Cd, for use in Sections 12.8.6, 12.8.7, and 12.9.2

c NL = Not Limited and NP = Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft. Heights are measured from the base of the structure as defined in Section 11.2.

d See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.

e See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.

f Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.

g The tabulated value of the overstrength factor, Ω0, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

h See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.

i See Sections 12.2.5.8 and 12.2.5.9 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category F.
Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m²) and in penthouse structures. Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.

The nature of the cantilever is that it does not provide much redundancy. That is the reason for the low \( R \) values and severe building height restrictions for these systems. The \( R \) value for a cantilever column system ranges from 1 for Ordinary Concrete Moment Frames to 2.5 for Special moment frames of steel or concrete. Intermediate moment frames of steel or concrete and timber frame columns have an \( R \) of 1.25. The Deflection Amplification Factors (\( C_d \)) have the same values as the \( R \) factors for these systems. Structural Overstrength Factors (\( \Omega_s \)) are 1.25 for all systems, except that timber frame columns are given a 1.5 overstrength factor. Cantilever column systems are limited to a maximum building height of 35 ft measured from the base of the structure. Only Special moment frame systems are permitted in all of the Seismic Design Categories included in the NEHRP Provisions (BSSC 2004). Intermediate steel moment frame and timber frame systems are allowed up through Seismic Design Category D. Ordinary steel moment frames and Intermediate concrete moment frames are allowed up through Seismic Design Category C. Ordinary concrete moment frames are limited to Seismic Design Category B.

By referring to the material standards, there could be a wide variety of interpretations concerning a cantilever column, especially with regard to the various steel standards. The cantilever could be structural steel designed per AISC 341 (AISC 2002) or could be light gauge cold formed steel material designed per AISI Lateral Design Standard (AISI 2004), AISI NAS (AISI 2005), or ASCE 8-02. One might even consider that an element similar to the proprietary light gauge framing elements that are substituted for woodframe shear walls could be designed and detailed as a cantilever column element.

**Design Requirements**

In addition to meeting the material standards requirements for the column element, ASCE 7-05 Section 12.2.5.2 limits the axial load on individual cantilever column elements. For load combinations of Section 2.3 (LRFD), the axial load shall not exceed 15 percent of the design strength of the column to resist axial loads alone. For load combinations of Section 2.4 (ASD), the axial load stress on individual cantilever column elements shall not exceed 15 % of the permissible axial stress. This axial load limit assures sufficient column ductility or lateral stability when the column hinges at the base, because the resulting column sway mechanism with one column hinge at the base is the only means of lateral force resistance for this system.

As the base connection is key to the performance of cantilever systems, the foundation and other elements providing overturning resistance need to have sufficient strength to resist the load combinations with overstrength factors as per Section 12.4.3.2. As the overstrength factors are reasonably small, 1.25 for steel and concrete moment frames or 1.5 for timber frames, this requirement should be easy to meet.

These two requirements are fairly straightforward and imply that other requirements for diaphragm design, deformation compatibility, and drift compliance shall also be met. However, ASCE 7-05 does not directly address the stiffness requirements of the base connection.

It is the opinion of the SEAOC Seismology Committee that unconstrained flag pole footings and isolated spread footings should generally not be used for the base connection, given the large contribution of foundation rotation and soil deformation expected for those foundation types. The exception would be where the soil deformation and foundation rotation can be adequately accounted for. The emphasis is on deformation compatibility, including validation of the base fixity of the column-to-foundation connection. Where a grade beam connects to an adjacent vertical element, adequate stiffness for the cantilever base is easy to ascertain. The adjacent vertical structural element(s) may be an adjacent cantilever column, a gravity load post, or a building wall, which can provide some restraint for the grade beam foundation.

**Combinations with Other Lateral Force-resisting Systems**
ASCE 7-05, Section 12.2.3.2 provides some guidance regarding combinations of lateral force-resisting systems. In general, it is required that the lowest $R$ value of any of the combined systems in a particular direction be used for the design in that direction. It is also required to use the largest overstrength factor and deflection amplification factor related to the combined $R$ values that are used in the same direction of the systems. This is consistent with the understanding under earlier codes. ASCE Section 12.2.4 requires that detailing provisions for structural components common to different framing systems that are used to resist seismic motions in any direction shall be designed using the detailing requirements of Chapter 12 that are required by the highest response modification coefficient, $R$, of the connected framing systems.

The use of the least value of $R$ along an independent line of resistance is also addressed in ASCE 7-05 Section 12.2.3.2 as follows: “Resisting elements are permitted to be designed using the least value of $R$ for the different structural systems found in each independent line of resistance if the following three conditions are met: (1) Occupancy Category I or II building, (2) two stories or less in height, and (3) use of light-frame construction or flexible diaphragms. The value of $R$ used for design of diaphragms in such structures shall not be greater than the least value for any of the systems utilized in that same direction.”

This essentially provides for nearly the same provisions as are recommended by a SEAOC Seismology Committee position paper, “Cantilever Column Elements in Light Frame Shear Wall Systems LRFD,” which is applicable to ASD and LRFD (SEAOC Seismology Committee 2004). The position paper provided necessary guidance for the existing practice of using cantilever column elements within a predominantly light-frame wood shear wall lateral force-resisting system. The position paper has four requirements, and presumes use of the applicable building code load factors:

1. The maximum inelastic response displacement of the cantilever column (at the higher $R$ value of the mixed system), with consideration of the base plate and anchor bolt deflection, shall be limited to the lesser of 0.01H or the approximate deflection of the adjacent shear walls in the same direction.
2. The design of the column, its connection to the diaphragm, its connection to the foundation, and the foundation shall be based on loads factored up by the ratio of the larger $R$/lower $R$ (e.g. 6.5/2.5=2.6).
3. The column axial design force ratio shall be based on a $K = 2.1$ and shall not exceed the force ratio of $Pu/\phi Pn \leq 0.15$
4. A reinforced concrete grade beam shall join the cantilever column to the adjacent vertical structural element(s) with sufficient stiffness to satisfy the deflection limit along each line of resistance. Other reinforced concrete foundation systems may be used, providing the foundation rotation and stiffness is included in the deflection calculation.

The emphasis is on deformation compatibility, including validation of the base fixity of the column-to-foundation connection. The limitation on the shear wall deflection versus that of the cantilever column in a deformation compatibility check is a part of the SEAOC Seismology Committee position. When mixing different systems, it is important to ensure that the relative deformations of systems along adjacent lines of resistance are within a reasonable tolerance to avoid tearing apart the system. These combinations can also include other flexible diaphragm structures.

**Cantilever Column vs. Inverted Pendulum Structures**

By definition, an inverted pendulum structure designation can apply to buildings in any of the structural systems listed in ASCE 7-05 Table 12.2-1 and is not a lateral force-resisting system designation. Instead, it is a class of structures where more than 50% of the structure’s mass is concentrated at the top of a slender, vertically cantilevered structure and in which the stability of the mass at the top of the structure relies on the rotational restraint to the top of the cantilevered portion of the structure.

ASCE 7-05 Section 12.2.5.3 states that “regardless of the structural system selected, inverted pendulums as defined in Section 11.2, shall comply with this section. Supporting columns or piers of inverted pendulum-type structures...
shall be designed for the bending moment calculated at the base determined using the procedures given in Section 12.8 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base." This is simply an additional requirement to be used for design of these structures.

While some cantilever column systems can be classified as inverted pendulum-type structures, that designation should not be assumed to apply. Most cantilever column systems do not fall into this category, as there are usually several cantilever columns connected together at their tops and the stability of the mass at the top generally does not depend on the rotational restraint at the top of the cantilever.

**Reduction of the Redundancy Factor**

ASCE 7-05, Section 12.2.4.2 Item a and Table 12.3-3 allow for the redundancy factor $\rho$ for cantilever columns to be reduced to 1.0 in Seismic Design Categories D, E, or F where each story resisting more than 35% of the base shear can meet the following requirement: "Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b)."

**Further Considerations**

Cantilevered column systems have potential uses beyond the limited scope allowed by ASCE 7-05. The SEAOC Seismology Committee would like to see some clarification of the use of limited-height cantilever column elements for a part of a building such as a rooftop penthouse or decorative element. These could be designed and detailed similarly, with some height restriction applicable to that cantilever system, as distinct from the height restrictions that pertain to the overall building.

Proprietary cold-formed light-gauge that are being used to substitute for woodframe shear panels may look like and have similar aspect ratios to cantilever columns. This raises the question of whether the testing of systems or elements of systems is adequate to justify the application of a system beyond the system factors and height limits prescribed by the applicable building code. Should these code requirements be revisited and adjusted based on the available test data?

The SEAOC Seismology Committee still eagerly awaits the results of in-progress studies (ATC 2008, NEHRP Consultants 2008) on system factors and implications for systems such as the cantilever column systems, and it recommends that further study and testing be done to compare cantilever column designs to some of the proprietary narrow-panel walls that have been approved by the International Code Council (ICC). Such a study could provide further insight into adjusting system factors for these systems in the future.

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**Keywords**
cantilever
column
combinations
inverted pendulum
mixed system

**How To Cite This Publication**
In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Cantilever column systems are defined in ASCE 7-05 as “a seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base.” The code does not explicitly acknowledge the fact that many applications of cantilever columns elements are within mixed systems, especially in light-frame construction. A strict reading of the code indicates that a structural system where lateral forces are only partially resisted by columns acting as cantilevers from the base is not technically a Cantilever Column System. However, in general the provisions for Cantilever Column Systems can be applied to cantilever columns elements in mixed systems, since it will lead to conservative results regarding system strength. One caution however is that the system strength provided by the Cantilever Column System provisions may lead to stiffness incompatibilities with surrounding elements of the lateral force-resisting system. Therefore, in mixed systems, designers should consider introducing a ductile grade beam to the base of the column and perform a stiffness analysis to ensure deformation compatibility with the other surrounding lateral force-resisting elements.

Historically, cantilever columns have been used by designers to provide an alternative to a moment frame or shear wall for a variety of light-frame structures including single and multi-family dwellings. In particular, they are useful in providing lateral force-resistance around garage openings, whether for two-story houses or for tuck-under garages found in multi-family construction. In addition, cantilever columns are useful in retrofit applications, such as an existing single-family dwelling where no qualifying lateral force-resisting elements were previously provided. Cantilever column elements currently compete with proprietary prefabricated shear wall panels and proprietary light-gauge bracing elements in light-frame shear wall systems.

**Historical Development of Cantilever Column Provisions**
The code classification of cantilever columns was first introduced into the 1997 UBC and was defined as an element that “cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces applied essentially at the top.” A relatively low $R$ factor of 2.2 was chosen. The introduction of this system occurred amid much discussion of the problems observed in the 1994 Northridge Earthquake, which included a weak-story collapse of the Northridge Meadows Apartment Complex. This failed structure had a tuck-under open parking garage with pinned end gravity columns. The 3-in diameter gravity columns buckled due to accidental fixity at the base and due to high drifts experienced at the open front of the structure. In light of these concerns, the 1999 SEAOC Bluebook cautioned that the cantilever column system should “only be used when the use of more desirable systems in not feasible” and also stipulated that “deformation compatibility checks are particularly important if the cantilever column system is used as part of a mixed system...” These cautions were based upon the belief that cantilever columns have “very limited redundancy,” “resist all lateral forces,” and have “no independent vertical load-carrying system.” As the design practice reacted to these changes ten years ago, engineers began converting cantilever column elements to moment frames, or to proprietary narrow shear wall systems that were listed as having an $R$ factor equivalent to wood-sheathed shear walls. The reason for avoiding the use of cantilever columns was that its $R$ factor of 2.2 would have been applied to the entire structure along that axis. At the time, many engineers complained that the cantilever column, as used in this typical class of structure, was unduly penalized in comparison to the alternatives. Accordingly the SEAOSC Light Frame Committee (June 2000), and thereafter the SEAOC Seismology committee published a Position Paper (May 2004) that stated it was appropriate to apply a less restrictive $R$ factor to the rest of the structure in certain cases. This relief from the restrictive $R$ factor included the requirements of a deflection check and a limit on axial load of 15 per cent of capacity, as well as other qualifications. The paper was titled “Cantilever Column Elements in Light Frame Shear Wall Systems” and was published on the SEAOC web page in both ASD and LRFD format. The statement provided necessary guidance for the existing practice of using cantilever column elements within a predominantly light-frame wood shear wall lateral force-resisting system.
further discussion see the section on Horizontal Combinations with Other Systems” in the Blue Book article, “Cantilever Column Systems” (SEAOC Seismology Committee 2008).

Application System Factors and Height Limits

As pointed out in that article, ASCE 7-05 has identified system factors for Cantilevered Column Systems (See Table 1). The table does not clarify if the material detailing requirements are dependent upon the column or the foundation.

Table 1. Cantilevered Column Systems, from ASCE 7-05, Table 12.2-1

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section where Detailing Requirements are Specified</th>
<th>Modification Coefficient, ( R_e )</th>
<th>System Overstrength Factor, ( \Omega_0 )</th>
<th>Deflection Amplification Factor, ( C_d )</th>
<th>Structural System Limitations and Building Height (ft) Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:</td>
<td>12.2.5.2</td>
<td>2/2</td>
<td>1/4</td>
<td>2/2</td>
<td>35</td>
</tr>
<tr>
<td>1. Special steel moment frames</td>
<td>12.2.5.5 and 14.1</td>
<td>2/2</td>
<td>1/4</td>
<td>2/2</td>
<td>35</td>
</tr>
<tr>
<td>2. Intermediate steel moment frames</td>
<td>14.1</td>
<td>1/2</td>
<td>1/4</td>
<td>1/2</td>
<td>35</td>
</tr>
<tr>
<td>3. Ordinary steel moment frames</td>
<td>14.1</td>
<td>1/2</td>
<td>1/4</td>
<td>1/2</td>
<td>35</td>
</tr>
<tr>
<td>4. Special reinforced concrete moment frames</td>
<td>12.2.5.5 and 14.2</td>
<td>2/2</td>
<td>1/4</td>
<td>2/2</td>
<td>35</td>
</tr>
<tr>
<td>5. Intermediate concrete moment frames</td>
<td>14.2</td>
<td>1/2</td>
<td>1/4</td>
<td>1/2</td>
<td>35</td>
</tr>
<tr>
<td>6. Ordinary concrete moment frames</td>
<td>14.2</td>
<td>1</td>
<td>1/4</td>
<td>1</td>
<td>35</td>
</tr>
<tr>
<td>7. Timber frames</td>
<td>14.5</td>
<td>1/2</td>
<td>1/2</td>
<td>1/2</td>
<td>35</td>
</tr>
</tbody>
</table>

a Response modification coefficient, \( R \), for use throughout the standard. Note \( R \) reduces forces to a strength level, not an allowable stress level.

b Reflection amplification factor, \( C_d \), for use in Sections 12.8.6, 12.8.7, and 12.9.2

\[ \text{NL} = \text{Not Limited and NP} = \text{Not Permitted. For metric units use 30.5 m for 100 ft and use 48.8 m for 160 ft. Heights are measured from the base of the structure as defined in Section 11.2.} \]

\[ \text{See Section 12.2.5.4 for a description of building systems limited to buildings with a height of 240 ft (73.2 m) or less.} \]

\[ \text{See Section 12.2.5.4 for building systems limited to buildings with a height of 160 ft (48.8 m) or less.} \]

\[ \text{Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.} \]

\[ \text{The tabulated value of the overstrength factor, \( \Omega_0 \), is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.} \]

\[ \text{See Sections 12.2.5.6 and 12.2.5.7 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category D or E.} \]

\[ \text{See Sections 12.2.5.8 and 12.2.5.9 for limitations for steel OMFs and IMFs in structures assigned to Seismic Design Category F.} \]

\[ \text{Steel ordinary concentrically braced frames are permitted in single-story buildings up to a height of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 psf (0.96 kN/m²) and in penthouse structures.} \]

\[ \text{Increase in height to 45 ft (13.7 m) is permitted for single story storage warehouse facilities.} \]

Given that a pure Cantilever Column System is assumed to be of low redundancy, a low \( R \) value and low allowable building height for these systems is appropriate. Conversely, in the case of a mixed system with sufficient redundancy, a higher \( R \) may be appropriate. As stated in the related Blue Book article (SEAOC 2008), where there is little guidance on applying the other system requirements, it is the SEAOC Seismology Committee’s opinion that this means the column is to be designed as a frame column with flexure to meet the requirements of the respective materials standards. Whether or not this approach is used, it is emphasized that the cantilever column elements must be checked for deformation compatibility with other parts of the mixed system.

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**Keywords**
cantilever column
combined systems
inverted pendulum
light-frame
mixed system
$R$ factor

**How To Cite This Publication**
In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Background
Many buildings use a combination of seismic force-resisting systems (SFRSs). As explained in a separate article, a SFRS is defined to consist of the vertically-oriented lateral force-resisting components of a structure, whereas the overall seismic resistance of the building includes the role of other components such as diaphragms and foundations. For combinations of SFRSs, building codes traditionally limit or modify design parameters such as $R$ (response modification factor), $\Omega_0$ (overstrength factor), and $C_d$ (deflection amplification factor).

Code provisions for SFRS combinations are intended to result in structure geometries that assure a generally uniform distribution of deformations to all SFRS elements at both elastic and inelastic response levels. While simple in concept, this is often difficult to achieve, since systems whose rigidities are well balanced while elastic can exhibit quite different behavior when they are inelastic. Values of $R$, $\Omega_0$, and $C_d$ should reflect both the properties of the selected structural systems and the relative ability of the total structure to match the idealized deformation pattern.

Building code provisions for combined systems from the 1991 through the 1997 UBC editions did not change significantly and were based on the recommendations of the 1990 Blue Book (SEAOC Seismology Committee 1990). Changes that occurred since the 1997 UBC are described in the following sections.

Vertical Combinations
When used to resist loads acting on the same horizontal axis, vertical combinations should avoid geometries in which inelastic behavior is concentrated in the lower system. For example, shear walls or braced frames (except in small penthouse structures) should not be used above a seismic moment-resisting (SMRF) system. The near collapse of the main building of Olive View Hospital in the 1971 San Fernando earthquake is an example of unacceptable performance with this combination.

ASCE 7-02 section 9.5.2.2.2.1, ASCE 7-05 section 12.2.3.1, and 1997 UBC section 1630.4.2 address this situation by limiting the $R$ value of the lower system so that it does not exceed the $R$ of the upper system. The intent is to delay the onset of yielding in the lower system until the point at which the entire structure will yield together. An exception to this rule applies to seismic isolation, where essentially all seismic deformations are concentrated into a base level designed to accommodate them.

Even where the upper system is more flexible, deformations should not be concentrated in one part of a combined system. In general, an uneven deformation distribution violates the premise of the equivalent lateral force method used for most code-based design. To address this issue, the 1990 Blue Book introduced a two-stage analysis procedure that allows the linear static equivalent lateral force procedure where the lower system is stiff enough to act essentially as a fixed base for the upper system (SEAOC Seismology Committee 1990, section 1E.3.a). The 1997 UBC presents this procedure in section 1630.4.2, with reference to section 1629.8.3 item 4 for qualifying conditions.

ASCE 7-02 omitted the two-stage procedure. For Seismic Design Categories D, E, and F, section 9.5.2.2.4.4 specified that special moment-resisting frames, which are relatively flexible, must be “continuous to the foundation” if such moment frames are required for the seismic force-resisting system. A review of Table 9.5.2.2 indicates that a SMRF is required in SDC D-F (either by itself or as part of a dual system) for any building taller than 160 ft; this is presumed to be the trigger for section 9.5.2.2.4.4. Where SMRFs are used but not required, they may be supported by more rigid lower systems if vertical irregularity requirements are met. This provision also appears in the 2000 NEHRP Provisions, but without commentary (BSSC 2001, section 5.2.2.4.4).
The 2003 IBC section 1617.6.1.2 corrected the ASCE 7-02 omission by duplicating the same basic language that appears in the 1997 UBC. ASCE 7-05 section 12.2.3.1 has now corrected the omission and added the two-stage procedure.

**Effect of redundancy coefficient.** The UBC and ASCE 7-05 two-stage procedure for rigid podium structures properly accounts for the influence of redundancy coefficients on effective $R$ values. These effects must also be accounted for in other vertical combinations.

**Horizontal Combinations**

Systems used in horizontal combinations are of potential concern if the individual systems deform in significantly different elastic or inelastic patterns. Differential inelastic deformations lead to substantial redistribution of forces between the combined systems, resulting in high collector forces between the systems or high torsion when the systems are offset from each other. ASCE 7-02 section 9.5.2.2.2.1 and 1997 UBC section 1630.4.4 limited the combined system $R$ value on a given axis to the lowest value of the individual systems used on that axis. The intent was to delay yielding in any one system until all the systems could yield together.

When a different system is used for each principal direction, the concern was that a flexible, high-$R$ system in one direction would cause large transverse displacements to a brittle, low-$R$ bearing wall system framing in the other direction. Excessive out-of-plane distortions of the low-$R$ system can lead to either failure of the bearing wall or separation of the wall from the roof or floor framing it supports. For these conditions, both ASCE 7-02 section 9.5.2.2.2.1 and 1997 UBC section 1630.4.3 identified conditions in which the lowest value of any system was to be used for all systems in any direction. ASCE 7-02 triggered this requirement for all Seismic Design Categories if any system had an $R$ value less than 5. In comparison, the UBC was triggered only when any system was a bearing wall system, and then only in Seismic Zones 3 and 4.

The ASCE 7-02 provision in some situations was overly restrictive. The threshold $R$ value of 5 properly considered plain concrete and masonry shear walls, which are vulnerable, and properly exempts light-framed walls with wood structural panels, which are not. But it also included less vulnerable systems such as steel ordinary concentrically braced frames in bearing wall systems ($R = 4$) and even steel intermediate moment-resisting frames ($R = 4.5$).

In response to these issues, ASCE 7-05 section 12.2.2 has taken an alternate route, by completely removing any orthogonal systems requirement. The justification made for the removal was that other code provisions for displacement compatibility that have been added since the original provisions were adopted have essentially replaced the need for the former provision. By implication, then, the replacement section in ASCE 7-05 is section 12.12.4 (for Seismic Design Category D, E, or F).

**Exception for separate lines of resistance.** ASCE 7-05 section 12.2.3.2 includes a separate procedure for structures having different structural systems on independent lines of resistance, where the following three conditions are all met: (1) Occupancy type I or II building, (2) two stories or less in height, and (3) use of light-frame or flexible diaphragms. This procedure permits each line of resistance to be designed using the value of $R$ that is greatest for any systems found along that line of resistance, and independent of the $R$ value required for any other line of resistance in the building. The value of $\Omega_0$ and $C_d$ used for each line of resistance at any story is the largest value of this factor for the $R$ value used in the same direction being considered. The $R$ value used for design of diaphragms is the lesser of $R$ for any systems utilized within all lines of resistance in that direction of application of seismic forces.

The Seismology Committee interprets this exception to apply to a range of situations in which light framing is combined with cantilever columns, moment frames around large wall openings, and diagonally braced wall framing. In particular, the exception is taken to apply to elements placed on either side of garage openings in typical houses. Likewise, it also may apply to interior lines of resistance such as braced frames that are located within concrete or masonry wall buildings, provided that the diaphragm type is flexible.
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Keywords
combined systems
deflection amplification factor, $C_d$
lines of resistance
overstrength factor, $\Omega_0$
response modification factor, $R$ factor,
seismic force-resisting system, SFRS

How To Cite This Publication
Articles in the SEAOC Blue Book series should be cited as follows, with this example based on an article published in September, 2006. The publication month and year are shown in the footer of Blue Book articles; the article title is shown in the header.

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2006)

In the writer’s reference list, the reference should be listed as:

Approximate Period Determination

ASCE 7-05 (ASCE 2006) addresses determination of building periods in section 12.8.2. The approximate period, $T_a$, is intended to provide a conservative estimate of the elastic fundamental period of a building. This period depends on the mass and the stiffness of the building. Until the building is designed, its period cannot be determined using the principles of structural mechanics. However, a period is needed to commence the seismic design of the building, because the design force is dependent on the period. Thus seismic codes and related guidance documents provide approximate methods that can be used to estimate a building’s period with minimal information available on the building design. Such methods use simple formulas that involve only a general description of the structural system (such as steel moment frame, concrete moment frame, eccentrically braced frame) and overall dimensions (such as height) to estimate the period of vibration. It is important that $T_a$ be chosen to be lower than the expected value of the period, so that if the period estimate is later not refined, the base shear and the resulting design will remain on the conservative side. This assumes the typical site characteristics and seismic environment where the response spectrum will plot decreasing response with increasing period.

Moment-resisting Frame Systems. As pointed out in the 2003 (and earlier) NEHRP Commentary (BSSC, 2004, section 5.2.2), “taking the seismic base shear to vary as $1/T$ and assuming that the lateral forces are distributed linearly over the height and the deflections are controlled by drift limitations, a simple analysis of the vibration period by Rayleigh’s method leads to the conclusion that the vibration period of moment resisting frame structures varies roughly as $h_n^{3/4}$ where $h_n$ equals the total height of the building.” Based on this, ATC 3-06 (ATC 1978), the NEHRP Provisions through the 1997 edition, and all codes and standards based on these documents, including the 1997 UBC and the 2001 CBC, contained an approximate period formula: $T_a = C th_n^{3/4}$.

ATC 3-06 originally gave values of $C_t = 0.035$ and 0.025 for steel and concrete frames, respectively. These coefficients represented judgmental lower-bound fits to fundamental periods of vibration as computed from accelerograms recorded in the upper stories of 17 steel frame buildings and 14 concrete frame buildings during the 1971 San Fernando earthquake, as explained in the ATC-3-06 Commentary. The data upon which the ATC 3-06 values were based were reexamined for concrete frames (Bertero et al. 1988) and the $C_t$ coefficient adjusted to 0.030 in the 1988 edition of the NEHRP Provisions, which value was also adopted into the 1988 edition of the UBC. The 0.030 coefficient for concrete moment frames was reexamined using an expanded database (Bendimerad et al. 1991), and an increase to 0.035 was recommended. This recommendation, however, has not been implemented in any code or standard or in the NEHRP Provisions. The $C_t$ values for steel and concrete moment frames remained 0.035 and 0.030, respectively, in the 1997 NEHRP Provisions, the 1997 UBC and the 2000 IBC.

The values for $C_t$ given are intended to be reasonable lower bound--not mean--values for structures designed according to the 2000 IBC. Surveys and studies of the particular buildings that provided the period data for the ATC 3-06 equations (Bertero et al. 1988) have shown that these original equations, even with the modified $C_t = 0.030$ for concrete frames, provide predictions that are about 80 to 90 percent of the lower bound values of measured periods at deformation values near first yield of the structural elements.

While this might indicate a large, perhaps excessive degree of conservatism, the buildings involved were designed for lateral force requirements prior to those of the 1976 UBC, which were significantly lower than those in the 1976 UBC as well as those given in more recent seismic codes. Furthermore, the controls on interstory drift for all
elements, and on irregularity and member detailing provisions, as given in recent seismic codes, are generally more restrictive than those used for the buildings in the original period evaluation study. Therefore, given reasonably similar nonstructural elements, the population of structures that conform to the drift provisions of IBC 2000 will have increased stiffness and correspondingly lower period values than the structures designed according to previous codes.

In recent years, large strong motion instrumentation programs operated by agencies such as the California Division of Mines and Geology (now California Geological Survey) and the United States Geological Survey have collected a substantial inventory of recordings of building response to ground motion. Evaluation of the data from this expanded data base indicated that adjustments to the approximate period formulas were warranted. The basis for the adjusted formulas was developed in a study by Goel and Chopra (1997). The data used to develop these adjustments is presented in the 2000 NEHRP Commentary, section 5.4.2 (BSSC 2000). The values of the coefficient $C_t$ and the exponent $x$ given in ASCE 7-05 Table 12.8-2 for moment-resisting frame structures represent the lower bound (mean minus one standard deviation) fits to the measured data. Use of the lower bound helps assure that the estimated period is conservatively low.

Figure 1 compares the approximate periods obtained from the new formulas used by ASCE 7-05 to those obtained from the older formulations used by the 1997 UBC for steel and concrete special moment-resisting frame (SMRF) structures of different heights. As can be seen, differences are negligible for both steel and concrete mid-rise moment resisting frame structures. However, as building height exceeds approximately 100 ft, there is a significant increase in the predicted period for moment-resisting frame structures of concrete, as compared to previous predictive equations, with the difference in period between taller steel and concrete structures reduced significantly.
Other Lateral Force-Resisting Systems. For buildings with other than moment-resisting frames, the approximate period was determined in editions of the UBC through 1985 from the following equation:

$$T_a = 0.05 \frac{h_n}{\sqrt{D}}$$

where $D$ is the plan dimension of the building in the direction of analysis.

There was a proposal made during the development of ATC 3-06 to substitute $L_v$, the length of the longest element of the vertically-oriented resisting system, for $D$ in the above formula. This did not account for stiffness contribution of all of the other vertical resisting elements in the system, and was not accepted. The above formula was retained in ATC 3-06, and in all the NEHRP provisions through the 1991 edition. Periods for nine reinforced concrete shear wall buildings computed from accelerograph records during the 1971 San Fernando earthquake were compared against predictions by the above formula. See Figure 2, which is reproduced from the Commentary to the 1991 NEHRP Provisions. It can be seen that the formula provides a reasonable lower bound to the measured data.

Figure 2. Comparison of Periods Calculated from Formulas with Periods Calculated from Strong Motion Records
The above formula was changed to $Ta = 0.02h_\text{n}^{3/4}$ in the 1988 UBC and subsequently in the 1994 NEHRP Provisions. Goel and Chopra (1997) have provided a comparison of predictions by this formula against the periods of 16 reinforced concrete shear wall buildings (27 data points) identified from their motions recorded during earthquakes (see Figure 3). It was found that, for a majority of buildings, the code formula gave periods longer than the corresponding measured values.

![Figure 3. Comparison of Periods Calculated by Formulas with Periods of Reinforced Concrete Shear Wall Building Strong Motion Records](image)

The 1988 UBC introduced the following alternative to the 0.02 coefficient for structures with concrete or masonry shear walls;

$$C_t = 0.1/\sqrt{A_c}$$

$$A_c = A_e \left(0.2 + (D_e/h_\text{n})^2\right)$$

where $A_c =$ the minimum cross-sectional shear area in any horizontal plane in the first story, in square feet, of a shear wall

and $D_e =$ the length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.

The above formula remained in the UBC through its 1997 edition. The development of the UBC formulas was described in detail in previous Blue Books (Seismology 1999, Section C105.2.2).
Goel and Chopra (1998) compared the above formula against some of the measured periods of Fig. 3. They concluded that the alternate formula almost always gives a value of the period that is much shorter than the corresponding measured period. In their opinion, “… the degree of conservatism seems excessive for most buildings considered in this investigation.”

The period of shear wall buildings is highly dependent not only on the height of the building but also on the number, lengths, and thicknesses of shear walls present in the building. Analytical evaluations performed by Goel and Chopra (1998) indicate that equations of the form of ASCE 7-02 Eq. 9.5.5.3.2-2 and 9.5.5.3.2-3 provide a reasonably good fit to the data. However, the form of these equations being rather complex, the simpler equation of the form of Eq. 9.5.5.3.2.1 has been retained from the 1997 and earlier editions of the NEHRP Provisions, with the newer, more accurate information presented as an alternative (BSSC, 2001, Section 5.4.2).

Figure 4 provides a comparison of the alternate approximate period formula of the 1997 UBC for buildings with concrete and masonry shear walls with that of ASCE 7-02/7-05. In the ASCE formula, \( T_a \) is a function of \( A_B \), the base area of the structure in square feet, while in the UBC formula it is not. Thus, comparison can only be provided for specific base areas, and is provided for \( A_B = 10,000 \text{ ft}^2 \) as well as for \( A_B = 20,000 \text{ ft}^2 \). The comparisons are made for \( D_i/h_i = D_i/h_n \) (shear wall height \( h_i \) = building height \( h_n \) is assumed) = 0.1 and 0.2 (length of an individual shear wall divided by building height), and for different ratios of \( \Sigma A_e/A_B = \Sigma A_i/A_B \), the total cross-sectional area of shear walls running in one direction divided by the base area.

While there are some impractical combinations of parameters included in Figure 2, the overall comparisons vis-à-vis the formula \( T_a = 0.020 h_n^{3/4} \) are still relevant. For \( D_i/h_i = D_i/h_n = 0.1 \), the ASCE 7 formula yields impractically high values of \( T_a \). The formula appears to require some adjustments for low values of \( D_i/h_i \). The UBC formula, on the other hand, appears to yield period values that compare in an expected way with \( T_a = 0.020 h_n^{3/4} \).

Updated data for classes of construction other than those with moment-resisting frames and shear walls are not available to date. As a result, \( C_t \) and \( x \) values for other types of construction shown in ASCE 7-05 Table 12.8-2 are values largely based on limited data obtained from the 1971 San Fernando Earthquake that have traditionally been used in the 1997 and earlier editions of the NEHRP Provisions. The 2003 NEHRP provisions classified values for \( C_t \) for buckling-restrained braced frames (BRBFs) and eccentrically braced frames (EBFs) with a \( C_t \) value of 0.03. However, BRBFs were inadvertently left out of this same table in ASCE7-05, which relegates the system to the “Other” classification with a value of 0.02. It is the position of the SEAOC Seismology Committee that the use of \( C_t = 0.03 \) (per 2003 NEHRP) is acceptable for BRBF systems analyzed and designed under ASCE 7-05.

The optional use of \( T_a = 0.1N \) (ASCE 7-05 Eq. 12.8-8) is an approximation for frame buildings of low to moderate height, which has long been in use. That formula was part of the Uniform Building Code through its 1985 edition. It has now been revived by ASCE 7-05, with limitations on the story height (lower-bound) and the number of stories (upper-bound).

**General Observations.** All the equations in ASCE 7-05 section 12.8.2.1 provide period estimates that are lower than most measured period values in the elastic range, and definitely much lower than nearly all measured values in the cracked section state for concrete buildings and the partially yielded state for steel buildings. However, these estimated period values, when used in the design base shear equations (section 12.8.1.1), provide design values that are judged to be appropriate and consistent with past design practice. For the usual case of a descending spectrum, the decrease in demand due to the increase in period as the structure deforms into the inelastic range is already taken into account by the \( R \) value of a given structural system. Therefore, period formulas should provide the period of a structure in its elastic state.
Figure 4. Comparison of 1997 UBC and ASCE 7-02 Period Calculations for Buildings with Concrete or Masonry Shear Walls
Strictly speaking, the formulas presented for approximate period are appropriate only for structures with rigid diaphragms. Structures, especially low-rise structures with flexible diaphragms, will generally have periods related to the stiffness of the diaphragm, not just the stiffness of the vertical resisting elements. There are at present no proposals for considering this phenomenon in determining the approximate periods of such structures. ASCE 7, the IBC, and the NEHRP provisions do not prohibit the use of the approximate formulas in determining the periods of structures with flexible diaphragms.

**Rational Period Determination**

ASCE 7-02 section 9.5.5.3 / ASCE 7-05 section 12.8.2 permits the evaluation of period $T$ by any properly substantiated analysis. If a mathematical model has been formulated for dynamic analysis, then the period of the first mode of vibration in a given principal direction may be used for $T$.

A rationally computed period is a function of modeling assumptions and is dependent in particular on stiffness assumptions. The smaller the assumed stiffness, the longer the rationally computed period, which in many cases translates directly into a lower design base shear. This incentive exposes the rational computation of period to possible abuse. Although it would have been preferable to prevent such possible abuse by specifying fairly rigid modeling rules, it has so far not been possible to forge consensus behind a complete set of rules to meet all possible variations and conditions encountered in design. Thus past practice has been, and practice continues to be, to impose direct control on rationally computed period. For design purposes, it may not be taken any larger than a coefficient, $C_u$, times the approximate period $T_a$. Values of $C_u$ are given in ASCE 7-05 Table 12.8-1.

Larger values of $C_u$ (and thus higher allowed values of $T$) are permitted where lower soil-modified seismic ground motion is expected. This is because buildings in areas with lower lateral force requirements are thought likely to be more flexible. As pointed out in the 2000 NEHRP Commentary (section 5.4.2), the equations for $T_a$ are “tailored” to the types of construction commonly used to resist high lateral forces. NEHRP continues, “It is unlikely that buildings in lower seismic risk areas would be designed to produce as high a drift level as allowed [by the 2000 NEHRP Provisions] due to stability problems ($P$-$delta$) and wind requirements. In any case, the use of a large $T$ will not really result in lower design force in cases, quite common in lower seismic areas, where the gravity and wind, rather than the gravity and seismic, load combinations govern design. The use of a low $C_u$ value in lower seismic risk areas might change practice by making design seismic forces larger in comparison to the design wind forces.

In ASCE 7-05 Table 12.8-1, values of $C_u$ for high design accelerations are based on data plotted in the 2000 NEHRP Commentary (BSSC, 2001b, section 5.4.2). Analysis of those data indicates that an upper bound on period may be estimated as 1.4 times the lower bound. Since the approximate period $T_a$ is already based on the lower bound, the code provision sets the upper limit on $T$ at 1.4 times $T_a$. For low design accelerations, NEHRP and ASCE 7-05 set the value of $C_u$ at 1.7 based on “subjective” judgment (BSSC, 2001, section 5.4.2). Intermediate values are derived by interpolation.

1997 UBC Section 1630.2.2 restricts rationally computed periods to be no more than 1.3 times the approximate periods in Seismic Zone 4, and 1.4 times the approximate period in Seismic Zones 1, 2, and 3. Thus, the 1997 UBC is more restrictive (conservative) than ASCE 7-05 in areas of high seismicity; it is more liberal than ASCE 7-05 in regions of low seismicity. When $T$ is restricted to 1.2 $T_a$, $V$ based on $T$ is restricted to no more than 83.3% of $V$ based on $T_a$. When $T$ is limited to 1.4 $T_a$, $V$ based on $T$ is limited to no more than 71.4% of $V$ based on $T_a$. The difference is significant.

Reasonable mathematical modeling rules should be followed such that the increase in period allowed by the $C_u$ coefficient is not obtained when the actual structure does not merit it.

The Commentary to the 2003 (and earlier) NEHRP Provisions (BSSC, 2004, section 5.2.2) contains the following warning: “For exceptionally stiff or light buildings, the calculated $T$ for the seismic-force resisting system may be significantly shorter than $T_a$ … for such buildings, it is recommended that the period value $T$ be used in lieu of $T_a$ for calculating the seismic response coefficient, $C_r$.”
For historical reference, the 2002 Revisions to the 2001 *California Building Code* made the following stipulation:

The value of $T$ computed by Method A (approximate period formulas of the 1997 UBC) shall not be taken as larger than the value of $\tilde{T}$ given by Method B (rational period determination of 1997 UBC). If Method B is not used to compute $T$, then the value of $T$ shall be taken as $T_a/IN_v$.

The justification for the above amendment was given as follows:

Method A … is an empirical method, developed based on period measurements made in structures over a period of years. The vast majority of these measurements were made in commercial-type structures, designed to earlier versions of the building code. However, … [Method A] fails to recognize that period is directly [in fact, inversely] proportional to the base shear coefficient (unreduced by the $R$-factor) that is used for design and that this coefficient is significantly larger for hospital buildings than for structures on which the calibration was based. As a result, the Method A period often exceeds the computed Method B period and can result in an unconservative estimate of required strength (if the Method A period is used in lieu of the Method B computation, as permitted by the CBC). This amendment would eliminate this error by making the Method A period [inversely] proportional to the importance factor and near field factors used in determination of the base shear.

The SEAOC Seismology Committee concurs with the NEHRP Commentary regarding this point. As the past amendment to the CBC is now outdated and no longer contained within the 2007 CBC, the Committee recommends consideration of the effect of the importance factor on the calculated Method A period in the absence of a Method B calculation.

**Drift Calculation**

ASCE 7-02 Section 9.5.5.7.1 waives the upper limit on calculated period for purposes of drift analysis. The 1997 UBC makes a similar allowance in Section 1630.10.3. This provision is intended to bring consistency between computed drift and the lateral forces under which drift is computed. As has been mentioned earlier, the reason for restricting the value of $T$ to be no larger than $C_vT_a$ in the force design of a structure is to prevent using too low a design force, based on an unrealistically long period. However, unrealistically long periods can be calculated based only on stiffnesses that are too low. Such low stiffnesses, in combination with lateral forces that are artificially required to be larger than the forces yielded by periods based on those stiffnesses, would result in drifts that are larger than realistic. Waiver of the requirement $T \leq C_vT_a$ permits computation of drift based on forces and stiffnesses that are consistent.

According to ASCE 7-02 section 9.5.5.7.1, for the purposes of drift computation, the value of the base shear $V$ need not be limited by the lower-bound value given by Eq. 9.5.5.2.1-5 ($V \geq 0.044ISd_0$), although the other lower bound limit ($V \geq 0.5Sd/I$ in SDC E, F) does apply. This provision is a very important one for tall buildings, which are typically drift-governed (rather than strength-governed). The drifts of tall buildings are quite likely to be controlled by the lower-bound limitations on the design base shear. The first limitation originated in the regulations implementing seismic code legislation enacted in California following the Long Beach Earthquake of 1933, and does not represent a physical reality (the constant-displacement branch of the design spectrum becomes applicable only at very long periods). That is why it is exempted. The second lower-bound accounts for large velocity and displacement pulses in near-fault ground motion, and therefore remains applicable. The 1997 UBC did not contain the same exemption. However, it was added as an erratum long after that code was published.

**Modal (Response Spectrum) Analysis**

ASCE 7-02 section 9.5.6.4 requires the determination of periods, mode shapes and participation factors for all relevant vibration modes when linear response spectrum analysis is used. Commentary on this section is given by NEHRP (BSSC, 2001, Section 5.5.3).
Section 9.5.6.4 requires the above determination to be made “… by established methods of structural analysis for the fixed-base condition….” While the “established methods” proviso is easy to understand, the “fixed-base condition” raises a question. A reasonable interpretation is that the “fixed-base condition” does not mean that the actual foundation flexibility may not be modeled. In fact, soil-structure interaction effects are specifically permitted to be taken into account by Section 9.5.9.3. The language in question originated in ATC 3-06 and has been in every edition of the NEHRP Provisions. It is likely that a fixed base was specified for two reasons. First, it would yield a shorter, more conservative period estimate. Second, soil-structure interaction effects have been explicitly addressed from the beginning. Also, modeling of foundation flexibility - particularly for dynamic analysis - was a major challenge in those early days, and indeed continues to be a challenge today. Finally, it is imperative, that one does not compute modal periods assuming a flexible foundation, and then also take advantage of soil-structure interaction per Section 9.5.9.3.

It may be noted that there is no upper bound or modal periods (including the fundamental period), as there is on rationally computed period by the equivalent lateral force procedure. Any such upper bound is unnecessary because the resultant base shear from dynamic analysis is scaled up to 85 or 100 percent of the design base shear of the equivalent lateral force procedure in cases where the former comes out to be smaller (ASCE 7-02 Section 9.5.6.)

**Soil-Structure Interaction**

ASCE 7-02 section 9.5.9.2.1.1 allows modification of the building period in recognition of beneficial soil structure interaction effects. Background to this section is given by NEHRP commentary (BSSC, 2001, section 5.8.2.1.1).

**Keywords**

- period of vibration

**References**

How To Cite This Publication
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Introduction

The purpose of structural analysis for new construction is to provide information necessary for producing an adequate design. Codified design provisions are assumed to result in structures that have acceptable performance. Seismic performance is a function of many variables including ground motion, estimation of design loads, modeling, detailing, material quality, and workmanship. Available analysis procedures differ in the level of effort required and in the accuracy and specificity of the results that can be obtained. The most accurate procedures can be expected to produce a reliable design—one that achieves its performance objective with the smallest uncertainty. Greater conservatism may be required when using less accurate procedures. If properly used, unacceptable performance, if it were to happen, could not be attributed to the incorrect calculation of force and deformation quantities.

Computed analytical results will differ depending on the analysis procedure employed. Accurate prediction of actual response remains a challenge with even the best analytical procedures (e.g. Lew and Kunnath, 2000). For new designs, uncertainties affecting analysis results can be accounted for with ductile detailing and a reasonably conservative evaluation of the hazard. In fact, for most earthquake-resistant design, it is expected that ductility and detailing will accommodate gross uncertainties in ground motion estimates and in analytical idealizations used to model the structure. Therefore, it is usually not necessary to analyze a proposed design to predict actual inelastic forces and deformations in individual members. Rather, the analysis, with appropriate loads, needs only to provide enough basic information, allowing the detailing to accommodate uncertainties in the actual force and deformation demands.

In this respect, analysis that supports the design of new structures has a practical advantage over analysis of existing structures. Existing structures with obsolete details, degraded materials, or irregular seismic resisting systems often require more accurate and careful analysis to gauge expected performance.

Structural analysis goes hand in hand with structural modeling. Whether analysis provides reliable information to the designer depends on whether the analytical model is able to capture key modes of behavior.

The basic choices to make in determining which analysis procedure to use have to do with whether to do a static or dynamic analysis and whether to use a linear or nonlinear model of the structure. Linear procedures are not applicable to structures in which the ductility demands are highly localized, such as when weak story mechanisms form in moment frame structures. In some cases where linear analysis is permitted, such as the evaluation of gravity framing for deformation compatibility or determination of the demands in coupling beams, it may lead to conservative (and inefficient) estimates of demands. Dynamic analyses are needed where the effects of higher modes may be appreciable. Higher modes can cause interstory drifts and story shears to be substantially greater than estimates made using static procedures for buildings with more than approximately 3 stories.

Structures designed using smaller R factors can be expected to have smaller inelastic demands relative to those occurring in structures designed with larger R factors. Linear procedures may be more appropriate for structures designed with smaller R factors since the accuracy of linear procedures improves as the degree of inelasticity in the structure reduces. Similarly, structures designed to more stringent performance objectives can be expected to have smaller inelastic demands. Thus, linear procedures may be acceptable for the more critical (important) occupancies, and for these occupancies, the greater accuracy afforded by dynamic analysis procedures may be indicated. In a similar vein, linear static procedures may be used for agricultural (non-human) uses, where reduced accuracy may be tolerated.
Light-framed wood structures typically have substantial contributions to lateral resistance from elements that are not part of the designated lateral force-resisting system. Static analysis of a linear model of the structure is ordinarily considered adequate for design; the difficulty of accurately modeling the structural components makes the presumed accuracy of more complex models doubtful.

Design procedures, analytical capabilities, and performance expectations have evolved considerably since the 1930s. Seismic design originally was performed using a single-valued seismic coefficient. Different seismic coefficients were used in the 1927 Uniform Building Code to account for different soil conditions. The 1935 Uniform Building Code identified three seismic zones, having varied seismic coefficients in relation to the level of seismic risk in each zone. The 1943 Los Angeles Building Code recognized that the distribution of lateral force varies over the height of the structure; this was also recognized by the 1949 Uniform Building Code. The dependence of the base shear coefficient on period was recognized in a model code developed by a joint committees of the San Francisco section of the American Society of Civil Engineers and SEAONC. The SEAOC model code of 1957 determined the design base shear as a function of the type of structural system, weight of the building, and a coefficient that varied inversely with period. With time, the base shear coefficient was made to vary with the period raised to the $1/3$ power. Subsequent changes accommodated the use of $R$ factors for different structural systems and the change from allowable stress design to strength design.

The 1957 SEAOC model code stated that intentions of the code were to safeguard against structural failures and the loss of life, recognizing that nonstructural damage may occur. A more comprehensive statement of performance objectives was put forth only in 1995, with the publication of Vision 2000 by SEAOC.

Analysis capabilities have been improved considerably since the development of the digital computer in the 1960s, the advent of the finite element method for the solution of large numbers of simultaneous equations that describe the response of multiple-degree-of-freedom (MDOF) systems, and the development of analysis software for nonlinear analysis. Typical commercial software now features nonlinear analysis capabilities.

The desire for improved accuracy has led to the acceptance of more sophisticated analytical procedures as they have become available over the years, leading to the present multiplicity of methods in codes and guidelines. Generally, the design of new buildings can be based on the results of relatively simple analysis procedures, since inelastic mechanisms can be detailed to provide for relatively good behavior. Existing buildings may have potential vulnerabilities that require analytical results of greater accuracy to assess. Performance expectations have begun to be specified with greater precision just as our analysis capabilities have improved.

Accurate performance predictions are known to be difficult, as models are likely to be inaccurate in critical aspects, such as strength, stiffness, mass, and damping (e.g. Kunnath et al. 2000). Further complications are uncertainties in actual capacities (e.g. member and material strengths and deformation limits) as well as demands (ground motions). Interactions between demands and capacities (e.g. load history influences on deformation capacities, and the influence of member strengths on deformation demands) are not well addressed at present. Furthermore, current models are capable of addressing only limited types of inelasticity (e.g. plastic flexural hinges, brace buckling and yielding) rather than addressing inelasticity associated with general combinations of axial force, moment, shear, and torsion. Materials that exhibit low cycle fatigue present further difficulties to model.

Differences in the results of code permitted analysis procedures indicate that some clarification of the roles and applicability of these procedures is needed. The move towards performance-based engineering implies or perhaps even requires improvements in the accuracy and precision of the analysis procedures used in practice. While major portion of design process is fundamentally deterministic in nature at present, the ultimate achievement of a known and perhaps uniform reliability in structural performance will require a more comprehensive framework for the treatment of uncertainties in demands and capacities. Substantial progress in this direction has already occurred in the SAC steel project (e.g. FEMA-350), where a method is provided to address the influence of quantified uncertainties in parameters influencing demands and capacities on the confidence level with which a desired performance objective is achieved.
**Code approaches**

ASCE 7-05 section 12.6 requires the proper structural analysis method be selected from five different procedures in Table 12.6-1. These analysis procedures may be classified according to whether a linear or nonlinear model of the structure is subjected to a static or dynamic analysis, as follows.

<table>
<thead>
<tr>
<th>Analysis Method</th>
<th>Structural Model</th>
<th>Nonlinear</th>
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<tbody>
<tr>
<td>Static</td>
<td>Equivalent lateral force analysis (section 12.8)</td>
<td>Nonlinear response history analysis (section 16.2)</td>
</tr>
<tr>
<td>Dynamic</td>
<td>Modal response spectrum analysis (section 12.9)</td>
<td>Nonlinear response history analysis (section 16.2)</td>
</tr>
<tr>
<td></td>
<td>Linear response history analysis (section 16.1)</td>
<td>Nonlinear response history analysis (section 16.2)</td>
</tr>
</tbody>
</table>

Not listed in the above table are the nonlinear static procedures of ASCE/SEI-41, which is a standard derived from FEMA-356 and FEMA 440. Other analysis procedures do exist and are allowed by ASCE 7-05 section 12.1.1 provided the analysis uses “a model consistent with the procedure adopted. Some alternative analysis procedures are discussed in the sections below.

ASCE 7-05 allows each of its five procedures for certain combinations of seismic design category (SDC), seismic use group (SUG), structural material, and structural configuration (height, period, and irregularity).

**Simplified analysis.** Simplified analysis is a conservative version of the more general Equivalent Lateral Force analysis. It does not require a drift check.

ASCE 7-05 allows Simplified analysis for buildings not exceeding three stories in height in Occupancy Categories I and II, in Site Classes A-D, with further limitations as specified in section 12.14.1.1. Simplified analysis is intended to be a simplified version of the Equivalent Lateral Force procedure for use in relatively simple buildings.

**Equivalent lateral force analysis.** The Equivalent Lateral Force method is the most commonly used design procedure, and has been codified in various forms since perhaps the 1960s. In this method, design seismic forces are determined by a linear elastic static analysis of the structure. In effect, time-varying inertial forces are replaced by equivalent static forces that are applied to each floor level. The design base shear is determined as a function of the elastic period of the building, subject to certain limitations for very flexible structures. The forces applied to each floor equilibrate the base shear and are distributed over the height of the structure in proportion to the weight of the floor and its height above the base, raised to a power $k$. Inherent and accidental eccentricities are considered in the static analyses. The effects of seismic actions are combined with the effects of gravity and other loads according to the load combinations of the governing code to determine the member design forces. Story drifts due to the lateral forces are computed and must not exceed limits that are tabulated as a function of the structural system and seismic use group.

The design of structures using an $R$ factor larger than approximately 1.5 establishes the intent of the designer to allow inelastic behavior to occur at defined locations in the structure. Under such conditions, the results of a linear elastic analysis cannot correspond to the actual force and deformation demands that can be expected during inelastic response. Elastic analysis results can be used for proportioning structural members where the locations of inelastic response are selected to assure that a desirable mechanism develops in the structure. Several code provisions (e.g.
the requirement that the column flexural strengths exceed the beam flexural strengths) are intended to avoid undesirable mechanisms. The implicit approaches used by ASCE 7-05 and the IBC to ensure the development of desirable mechanisms might not be effective for some geometric configurations, where undesirable member proportioning occurs and for high ground motions. Nonlinear analyses may be necessary to establish the expected mechanism(s) of such structures.

The Equivalent Lateral Force (ELF) method effectively assumes all mass participates in the first mode. The ELF lateral force distribution is altered in an attempt to address higher mode effects on story shears. Even so, shears determined in nonlinear dynamic analyses (presumed to be more accurate) can significantly exceed the values computed using the ELF method (e.g. Eberhard and Sozen, 1993). Higher modes can influence story drifts and other response quantities.

The ELF method is recognized to be inadequate for:

• structures with irregular distributions of mass and stiffness
• structures with irregular distributions of story strengths
• structures in which the translational response in the two orthogonal directions and the torsional response (about a vertical axis) are strongly coupled
• structures with flexible diaphragms on multiple floor levels.

ASCE 7-05 allows the ELF method to be used for all buildings in SDC B and C. In SDC D, E, and F, the ELF method may be used for all light-framed structures, all Occupancy Category 1 and 2 structures not exceeding two stories in height, and all other structures except those having $T > 3.5T_s$ and those having a horizontal/torsional irregularity or a vertical soft story, mass, or geometric irregularity.

ASCE 7-05 to some extent permits structures having weak stories be constructed in SDC D, E, and F (see section 12.3.3.1), they are allowed to be analyzed using the ELF method, despite the poor performance expected for such structures.

For example, light-framed 2- and 3-story buildings with severe soft or weak story and/or torsional irregularities are allowed to be designed using the ELF, despite experience indicating the potential for collapse of such structures. It is recommended that the such structures be re-designed to avoid severe soft and weak story and/or torsional irregularities, or that dynamic analyses be made to demonstrate the adequacy of the design. Structures with flexible diaphragms currently are allowed to be designed according to the ELF method, although this approach can be expected to be accurate only for single-story structures. For multiple stories, the diaphragms can respond at similar frequencies, developing forces that are different than what would be estimated by response spectrum analysis.

ASCE-7 allows the ELF method to be used to design any structure in SDC A, B, and C. Given the expectation of significant ductility demands based on the use of an R-factor greater than 1.0 in design, prudence suggests that the ELF be used only for regular structures having no more than approximately 3 stories. These recommendations as well as previous code limits are based primarily on engineering judgment, experience, and limited research studies.

For low rise structures, it is not clear that idealizing a structure with mass lumped just at the floor levels is reasonable, as mass in the walls will participate to some extent in out-of-plane modes that do not involve significant displacements of the floors (Scott et al. 1999). As the number of stories increases, lumped mass models become more realistic, but multiple-degree-of-freedom effects associated with higher mode response become significant as the number of stories increases, having significant effects above approximately 4 to 6 stories. The ELF procedure, which assumes that all the mass is in the first mode, therefore is conservative for low rise structures, while it may become unconservative as the number of stories increases.

ASCE 7-05 Table 12.6-1 indicates that the linear static procedures should not be used for buildings with a fundamental period greater than 3.5 times the site period, $T_s$. The ATC-55 project (published as FEMA-440) observed that higher modes contributed substantially to response quantities for buildings at shorter periods.
However, the point at which inaccuracies become excessive could not be clearly identified. The 2009 NEHRP Seismic Provisions does not recommend that nonlinear static procedures be used for the detailed design of buildings exceeding 40 feet in height. Where higher modes contributions are significant and accuracy is important, dynamic analysis should be conducted.

Linear dynamic analysis results should, theoretically, be divided by \( R \) for design comparable to the use of the ELF method for structures dominated by first mode response. However, to prevent misuse in the case of a very low design spectrum, the ELF base shear is used as a benchmark for scaling of the base shear and other response quantities determined using a dynamic analysis to ensure that similar buildings result regardless of the analysis procedure used in their design. Dynamic analyses result in a more accurate distribution of demands over the height of the structure. As an incentive, and to reflect the enhanced accuracy, dynamic analysis results are allowed to be scaled to obtain a base shear equal to 85% of the ELF base shear.

### New thinking.

Since higher modes naturally contribute little to floor displacements, both linear and nonlinear static procedures remain applicable for determining the required base shear strength for use in the preliminary design of the lateral force resisting system. Design approaches are being developed that enable the base shear strength and period to be used following a conventional design procedure, except that the base shear can be established based on the desired displacement and ductility response of the nonlinear system. After the preliminary design of the structural system is complete, dynamic analyses may then be done to determine quantities that are strongly influenced by higher modes, such as story shears and interstory drifts.

### Linear Dynamic Procedures

As computer algorithms for eigen solutions for highly indeterminate structural systems became feasible, explicit calculations of the dynamic response of actual structures took the place of theoretical solutions based on continuum mechanics. Beginning in the 1970s response spectrum analysis procedures were recognized by the UBC and the Blue Book.

These methods inherently account for higher modes, but still use a linear elastic model in conjunction with scaling to ELF base shear to determine design forces. Like the ELF method, the design quantities computed using Linear Dynamic Procedures (LDP) are useful for design if the intended locations of inelastic behavior have been defined to ensure a desirable ductile mechanism. The story shears and story drifts computed with an LDP reflect the presence of higher modes, although these are based on the response of a linear elastic model.

The base shear calculated by an LDP analysis once reduced by an \( R \) factor can be significantly smaller than the ELF base shear, so codes typically require the LDP results to be scaled to a value comparable to the ELF base shear. For modal response spectrum analysis, response quantities are scaled so that the base shear is at least 85% of the ELF value (ASCE 7-05 section 12.9.4). For linear dynamic response analysis, elastic response quantities that reflect the presence of higher modes are divided by \( R/I \). Thus, the main practical difference between the LSP and LDP methods is the distribution of lateral forces over the height of the building. The dynamic results inherently reflect the effects of irregularities on the response of linear models.

The lateral force distribution of the LDP reflects the presence of higher modes, based on linear elastic response. However, nonlinear dynamic analyses have shown that shear demands can exceed the shears obtained from a linear dynamic analysis in which the results scaled to match the base shear of the ELF procedure. Thus, the results of a linear dynamic analysis may be inaccurate for structures in which significant nonlinearity is expected. Even in these circumstances, the LDP may be sufficient for design purposes, but care may be needed to ensure that the intended inelastic mechanism develops in cases where irregularities are present and for existing buildings, in which the distribution of strength may not be sufficient to ensure that a desirable mechanism forms. Similarly, since the analyses are based on a linear elastic model, the LDPs do not adequately address weak story mechanisms (ASCE 7-05 vertical irregularity type 5), for which ductility demands are concentrated in just one or several stories. Unless
such structures are designed with a significantly lower $R$ factor, they should be analyzed using nonlinear procedures. The accuracy of the LDP improves as the expected degree of nonlinearity reduces.

**Modal response spectrum analysis.** The orthogonality of the undamped modes of vibration allows the linear elastic response to be represented by superposing the responses of the individual modes in time. Any number of modes may be used as long as at least 90% of the participating mass is represented. In modal response spectrum analysis, The peak modal responses are combined using modal combination rules such as the SRSS or CQC rules (see Clough and Penzien, 1993) as an approximation to the actual time-varying interaction of the modes. Adequate approximate representations of the response often may be obtained using just several of the lowest modes (see Aschheim, et al., 2002). However, some structures may have components that are excited by modes that have participation by only a small fraction of the total mass, and thus these modes may be neglected in the response spectrum analysis even though they contribute significantly to the demands on these components.

Smoothed elastic design spectra (mainly, uniform hazard spectra) such as those specified by codes (ASCE 7-05 Figure 11.4-1) are typically used in a response spectrum analysis. Practitioners should be aware that any single response spectrum analysis is an inherently unrealistic representation of demands, because (1) different seismic sources may have different contributions to the spectral hazard at different period ranges, and (2) even where a single source exists, where a large set of potential ground motions at a site are considered, the spectral values at a given probability of exceedance over a range of periods are obtained for different seismic events; that is, no single event has spectral ordinates that match the uniform hazard spectrum over a large range of periods. Thus, the uniform hazard design spectrum overestimates the contribution of all modes for an event at the stated exceedance probability. Nevertheless, the use of uniform hazard spectra in response spectrum analysis is broadly accepted.

**Linear response history analysis.** The response history is determined as a step-by-step solution in the time or frequency domains for the response of a structure to a signal that represents the motion of the ground. The solution may be determined using modal superposition, which involves a sufficient number of modes to represent at least 90% of the participating mass, or by direct integration, which inherently represents 100% of the mass associated with the model of the structure. As with response spectrum analysis, the use of only several modes is often adequate, but for there may be some components of the structure that experience relatively large demands by modes in which a small proportion of the total mass of the structure participates. Because the response is computed for a particular ground motion record, computed responses are functions of the specific excitations that are used.

The direct integration method of response history analysis should be used when the parameter being considered is excited by a mode of vibration having a low mass participation factor. One example is the analysis of a building having a 2-story column supporting heavy precast elements. Since the mode associated with column vibration would have a relatively low mass participation factor, this mode might be omitted from a response spectrum or response history analysis even when including a sufficient number of modes to represent at least 90% of the mass. As a result, the column forces would not be calculated correctly by a modal analysis approach. Another example is a structure that has two significant modes of vibration in the same direction with almost identical periods. Structures with flexible diaphragms sometimes fall into this category. In this last example, time history modal superposition can be used: but the modal response spectrum analysis can not.

**Nonlinear Static Procedures**

Nonlinear static procedures comprise a variety of analysis methods that are based on static pushover analyses of a nonlinear model of the structure and the evaluation criteria by which member force and deformation demands can be judged. Included in these procedures are the Capacity Spectrum Method and the Displacement Coefficient Method, as well as various adaptive and multiple mode pushover techniques.

In general, nonlinear static and dynamic analyses are performed in the presence of a service level live load, and the total (gravity plus seismic) deformation determined in the analysis at locations of concentrated inelastic action are
compared with deformation capacities. In contrast, linear analysis procedures generally determine reduced forces (elastic quantities divided by an R factor) for combination with factored loads. As a result, the nonlinear methods lack some of the conservatism that is inherently present in the linear approaches for design; whether such differences manifested in the actual design depend on the calibration of acceptability criteria, the degree to which elements or cross sections contribute to both gravity and lateral load resistance, and the relative intensity of gravity load.


In the various nonlinear static procedures, a nonlinear model of the structure is subjected to gradually increasing lateral forces. Because the nonlinear behavior of the structural components is modeled, softening of the structure is observed as inelastic behavior develops. The global behavior of the structure is characterized by a “capacity curve,” which is a plot of the shear force developed at the base of the structure versus the displacement at a “control point,” usually located at the roof. Member forces, story drifts, and plastic hinge rotations are determined when the control point displacement matches a “target displacement.” The capacity spectrum and displacement coefficient methods represent different approaches to estimating the target displacement, although they are both fundamentally based on the inelastic response of single-degree-of-freedom oscillators to earthquake ground motions. Although these methods can produce different estimates of the target displacement, these estimates are generally consistent with the wide range in results that can be determined in the response to a suite of ground motion records; the inherent variability in the response amplitudes of inelastic systems is a fact that must be acknowledged by the engineering community.

The Capacity Spectrum method is based on the work by Freeman (1978), who recognized that spectral demands could be plotted as a function of spectral displacement rather than period. The method relies on the concept of “equivalent linearization,” in which the peak displacement response of an inelastic system is estimated as the peak displacement response of an elastic system having increased damping and reduced stiffness. The Capacity Spectrum method usually is presented as a graphical method, in which the capacity curve and various curves corresponding to spectral demands at different levels of damping are portrayed graphically, and the target displacement is estimated according to the effective damping that corresponds to the ductility demand of the system. The damping reduction factors used in the capacity spectrum method have been changed over the years; those used in ATC-40 were made to better fit the results of the Capacity Spectrum method to the SDOF nonlinear analysis results.

The Displacement Coefficient Method is described in FEMA-273, FEMA-356, and ASCE/SEI-41. This method is generally known as a “displacement modification” approach, using the peak displacement of an elastic system as a basis for estimating the peak displacement of a system undergoing inelastic response. An equation usually is used to estimate the displacement; however, the displacement estimates can also be expressed graphically on a plot of acceleration versus displacement. The target displacement estimates produced by the Capacity Spectrum and Displacement Coefficient Methods are known to differ. However, if both approaches were calibrated perfectly, they would result in the same estimate. Differences in the estimates made by using these procedures and recommended improvements to these equivalent linearization and displacement modification approaches are described in FEMA-440 (2005).

Appendix A of Chapter 5 of the 2003 NEHRP Provisions and Part 3 of the 2009 NEHRP Provisions provide an implementation of the displacement coefficient method that allows a structure to be designed using the NSP without having to require detailed checks of member force and deformation demands if the structure has an effective yield strength in excess of an estimated strength as if the structure was designed according to the ELF method. This allows the NSP to be used for the design of drift-controlled structures (e.g. steel moment frames), as well as for ductility-controlled structures where sufficient strength is provided, without requiring detailed member checks. Detailed member checks are required for structures with lesser strengths.
Peer review is generally required for the nonlinear static procedures, to be sure that the methods are being applied correctly in circumstances where the specific features of the structure or ground motions could not have been anticipated by guidelines’ writers.

Pushover analysis is useful for estimating deformation demands and can expose weaknesses in a structure that may remain hidden in an elastic analysis (Krawinkler and Seneviratna, 1998). Recent work for ATC-63 suggests that the nature of the inelastic mechanism expected under dynamic excitation may or may not conform to expectations based on pushover analysis. This is of great significance in the evaluation of existing structures, for which various interventions may be chosen based on the nature of the mechanism that develops and the need to stiffen (or strengthen) the building to limit deformation demands. For new design, the NSP may be useful for determining the strength and stiffness required to limit deformation and ductility demands to prescribed values; such an application is described in Part 3 of the 2009 NEHRP Provisions. Of concern in both new design and evaluation, however, is the lack of representation of higher mode demands in the analysis. While higher modes typically contribute very little to displacement demands, their contribution to interstory drifts and story shears is not negligible. Thus, the responses determined in pushover analysis are conditioned on the members of the structural system having sufficient strength to withstand the forces resulting from all modes.

Because higher modes contribute little to the displacements, the NSP is useful for estimating peak displacements over the height of the structure. The NSP may also be used to estimate interstory drifts, story shears, and other response quantities of short period structures having up to approximately 4 to 6 stories in height, depending on the degree of accuracy required.

FEMA-356 has elaborate requirements for multiple independent analyses using different load vectors in an attempt to establish bounds on the values of the response quantities of interest. While such analyses may be of use for short period structures, they are inadequate for determining those response quantities in which higher modes have a significant contribution. A simple first mode pushover analysis is sufficient for estimating peak displacements, while dynamic analyses appear to be necessary for estimating quantities that are significantly affected by higher modes.

**New thinking.** Chopra and Goel (1999) and Fajfar (1999) have proposed to improve the Capacity Spectrum Method by using displacement modification approaches in place of the equivalent linearization approach recommended in ATC-40, while retaining the graphical representation of the Capacity Spectrum Method. This approach is illustrated for a ductility of 3 by the “AD” curve of Figure 1. With this traditional graphical representation, iteration is required to identify the peak displacement for which the ductility of the system (having a given yield point) matches the spectral demand. Iteration is not required when an alternate representation is used, in which the yield displacement is plotted rather than the peak displacement. With the yield point spectra representation (Aschheim and Black, 2000), the peak displacement is determined based on the ductility demand of the curve passing through the yield point (the “YPS” curve of Figure 1). Using this format, it is possible to identify families of yield points that all satisfy a specified limit on peak displacement and/or system ductility. This can be done for one or multiple performance objectives. The yield strength required to satisfy these performance objectives may be determined readily. The required strength and associated period can then be implemented in a quasi-ELF design procedure, if desired.

The treatment of higher modes in pushover analysis was suggested in ATC-40, and has been formalized in a method described by Chopra and Goel (2002). In the so-called “multimodal pushover analysis method,” response quantities determined by independent pushover analyses in the first and higher modes are combined using SRSS combinations. However, inelastic responses cannot strictly be combined, and the method was found to be of inconsistent reliability in the ATC-55 studies. An energy-based pushover approach was suggested by (Hernandez Montes et al.) to overcome some of the difficulties associated with the higher mode pushover curves, but SRSS combinations of the potentially inelastic response quantities still are of inconsistent reliability. Aydinoğlu (2003) has suggested a procedure in which higher mode contributions are accounted for in the pushover analysis in a step-by-step procedure, which avoids the difficulties associated with independent nonlinear analyses, although assumptions must be made about the relative signs of each modal action.
ASCE-SEI 41 (2007) includes the results of efforts to improve the use of inelastic procedures for design and evaluation that were begun under the ATC-55 project. The project report, published as FEMA-440 (2005), includes recommended changes to improve the equivalent linearization and displacement modification approaches for estimating the peak displacement, the load vectors to be used in pushover analyses, and the influence of soil-structure interaction on the response estimates. A maximum $R$ value was recommended for systems with strength degradation. One outcome of the project is a method for scaling ground motions for use in nonlinear dynamic analysis based on the peak roof displacement or the peak displacement of the equivalent SDOF oscillator, known as the “Scaled NDP.” This procedure is discussed in the “new thinking” section that follows.

![Figure 1. AD and YPS curves for the case $\mu = 3$, determined by applying a recommended displacement modification approach to a smooth code design spectrum.](image)

**Nonlinear Dynamic Procedures**

Nonlinear dynamic analysis became possible with the development of academic software beginning in the 1980s and the distribution of commercial software beginning in the late 1990s. Code provisions for nonlinear dynamic analysis were developed subsequently. Nonlinear dynamic analysis is considered to be the most accurate of the analysis procedures because it most closely represents the inelastic response, and has become the benchmark by which the accuracy of other procedures is evaluated. Nevertheless, the results obtained are sensitive to the particular ground motion used and to the modeling of the structure. Comparisons of “blind” predictions with the actual recorded responses of instrumented buildings often indicate the potential for significant differences in predicted and actual behavior (e.g. Kunnath et al. 2000). Relatively little guidance on modeling is available in the literature. Because nonlinear dynamic analysis inherently reflects the contribution of higher modes and the influence of nonlinear component behavior on response, this method is considered applicable to all structures that are amenable...
to being modeled. Simpler methods are usually used for light-framed construction because of difficulties in representing the load-resisting characteristics of the materials used in this form of construction. Most structural analysis software are not capable of modeling large deformations, gap opening and rocking behavior, the development of membrane effects or catenary actions in structural members nearing collapse, and inelasticity associated with general combinations of axial force, shear, moment, and torsion. For these reasons, judgment is needed in the selection of the analysis tool, recognizing its capabilities and limitations for modeling the anticipated behavior of the structural system.

**New thinking.** Recognition of the significance of higher modes on response has led to the development of analytical techniques to supplement or replace pushover analysis. Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2002) consists of a succession of independent nonlinear dynamic analyses using one or more ground motions scaled to different amplitudes. Peak responses such as peak roof displacement or maximum interstory drift are plotted as a function of the intensity of the excitation, which may be expressed by the scaled peak ground acceleration, the spectral acceleration at the first mode period, or by other demand parameters. The plot in essence establishes a capacity curve by dynamic analysis; the capacity curves determined in this way are not as smooth and monotonic as those determined in static pushover analysis, and are a function of both the excitation and the characteristics of the structure.

The presence of higher modes can cause peak story drifts, story shears, and overturning moments determined according to the NDP to be substantially higher than the values determined by the NSP. The Scaled NDP (Aschheim et al. 2007) can be used for obtaining estimates of these and other quantities affected by higher modes. This approach relies on the NSP for determining the target displacement of the structure, which generally has small or negligible contributions from higher modes. Nonlinear dynamic analyses are then done using ground motions scaled such that the peak displacement of the control point matches the target displacement estimated according to the NSP. Mean values of the response quantities are determined for different scaled ground motions. The mean values may then be modified by statistically-derived factors that account for the number of analyses performed and the variability of each response quantity—the precise values to be used depend on the desired level of confidence and estimated or assumed values of the coefficient of variation of each response quantity. Advantages of the method include the relatively low dispersion in computed response quantities and its compatibility with existing spectral descriptions of hazard, which are used to estimate the peak displacement. As an alternative, ground motions can be scaled based on the inelastic response of the equivalent SDOF system and then applied to the structure.

**Potential Approaches for Using Analysis Procedures in Design**

Conventional design approaches have focused on strength and stiffness (or period) and have relied upon traditional spectral descriptions of hazard. Periods can be estimated using code formulae to determine preliminary values of base shear for design. Conservative force-based designs could be obtained if the period was underestimated, even though such an approach produces unconservative estimates of displacement. Iteration is often required, because the actual period may differ significantly from the initial estimate.

Engineers usually can establish approximate member sizes relatively early in the design process. Member strengths are typically determined by changing the weight of the steel in the cross section, whether this be for steel members or reinforced concrete members. Based on simple kinematics, the yield curvature may be estimated by the yield strain divided by the depth to the neutral axis. Curvatures over the length of a member can thus be established in an approximate fashion. Changes in the amount of material typically have little or no affect on the member curvatures at yield. Thus, the yield displacement is nearly invariant with changes in member strength. Such thinking applies to both members and structural systems, provided that the relative distribution of stiffness and strength is maintained for the latter, even as the strengths of the individual members are changed. This is the reason why the yield displacements of steel moment frames, observed in first mode pushover analyses, typically are between 1 and 1.2% of the height of the structure.
Aschheim (2002) has shown yield displacements (as a percentage of the height of the structure) to be stable even as the number of stories and the strength required to achieve the desired performance varies substantially. If taken advantage of, the stability of the yield displacement can be used to reduce or even eliminate the need for iteration in performance-based design. Such an approach would be effective for the initial proportioning of a structure using Yield Point Spectra, for example, and is described in Part 3 of the 2009 NEHRP Provisions. The scaled NDP (discussed in the previous section) could then be used to establish the required member strengths and deformation demands.

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How To Cite This Publication
In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2009)

In the writer’s reference list, the reference should be listed as:

Introduction and Background

Collector elements (also called drag struts or drag elements) are elements of floor or roof structures that serve to transmit lateral forces from their location of origin to the seismic force-resisting-system (SFRS) of the building. Typically, collectors transfer earthquake forces in axial tension or compression. When a collector is a part of the gravity force-resisting system, it is designed for seismic axial forces along with the bending moment and shear force from the applicable gravity loads acting simultaneously with seismic forces.

When subjected to lateral forces corresponding to a design earthquake, most buildings are intended to undergo inelastic, nonlinear behavior. Typically, the structural elements of a building that are intended to perform in the nonlinear range are the vertical elements of the SFRS, such as structural walls or moment frames. For the intended seismic response to occur, other parts of the seismic-force path, particularly floor and roof diaphragm collectors and their connections to the SFRS, should have the strength to remain essentially elastic during an earthquake. This is the intent of most building codes and for this reason collectors should be designed for larger seismic forces than those for which walls, braced frames, or moment frames are designed.

Traditional seismic design practices prior to the 1997 UBC were generally based on providing discrete collector elements having specific reinforcement to transfer the entire required seismic load to the receiving end of the seismic force-resisting vertical element. This practice was partly due to, and based on, the low collector force requirements in the older seismic design codes.

In the 1994 Northridge Earthquake, failure of collector elements, which contained insufficient reinforcement, was observed in more than one pre-cast parking structure. Collector elements were observed to have yielded early on, rendering the collector elements unable to transmit the lateral force to the shear walls.

Overstrength

ASCE 7-05 section 12.10.2.1 requires collectors, their splices, and their connections to the SFRS to be designed for the special load combinations with the overstrength factor, $\Omega_0$. The overstrength factor represents an upper bound lateral strength and is appropriate for use in estimating the maximum forces developed in non-yielding elements of the lateral system during design basis ground motion.

The intention of the overstrength factor, when applied to collector elements, is to minimize the probability of diaphragm-to-SFRS connection failure and instead force all the yielding into the building's properly detailed SFRS elements. This requirement perhaps had its genesis in ATC 3-06 (ATC 1978), which recommends embedment of chord reinforcing “sufficient to take the reactions without overstressing the material in any respect.”

Applying the overstrength factor effectively increases the collector force demand by approximately 200% to 300%. It often becomes impractical to provide a collector element that is concentric with the shear wall/moment frame that has adequate strength to resist the full seismic force and transfer it to the ends of the vertical seismic force-resisting element. Additionally, by concentrating all the collector reinforcement in a small region in line with the wall, the elastic stiffness of the adjacent floor slab may be underestimated. This “traditional” methodology can result in a condition where the floor slab having larger area and being stiffer than the collector element would initially resist the collector seismic tension. If the floor slabs are not adequately reinforced for the seismic tension force, significant cracking may occur, until the reinforced collector element starts yielding and reaches its full strength.
Design Forces

ASCE 7-05 (ASCE 2006) Section 12.10.1.1 reads that collectors shall be designed to resist design seismic forces from the structural analysis, $F_x$ (12.8-11), but shall not be less than that determined in accordance with $F_{px}$ (12.10-1.). Both $F_x$ and $F_{px}$ are amplified by the overstrength factor. When collector design forces are designed for only inertial forces the redundancy factor can be set equal to 1.0 in accordance with section 12.10.1.1.

One method used by many practicing engineers to obtain collector design forces is to assume that the diaphragm acts as a simple beam with uniform distribution of shear in the direction normal to the lateral span and with increasing axial forces in collectors aligned with SFRS elements, as shown in Figure 1. This method neglects any distributed tension or compression in the direction of lateral forces. Figure 2 shows an alternative mechanism of force delivery to the SFRS based on diaphragm shear capacity. Another approach to collector design would be to use the strut and tie model as covered in detail in Appendix A of ACI 318-08 code (ACI 2008). It is the opinion of the SEAOC Seismology Committee that the Seismic Load Effect Including Overstrength Factor of ASCE 7-05 Section 12.4.3 shall apply to the strut and tie model. Any mechanism of force delivery can be assumed in analysis provided the complete load path has adequate strength.

**Figure 1.** Uniform distribution of shear normal to lateral span

**Figure 2.** Forces delivered to SFRS based on diaphragm shear capacity

Section 12.3.1 of ASCE 7-05 requires that the structural analysis explicitly include consideration of the stiffness of the diaphragm, unless the diaphragm can be idealized as flexible or rigid. By using a semi-rigid assumption for diaphragms in the analysis model, collector forces, $F_x$, can be readily found. Analysis software that can model semi-rigid diaphragms has techniques for cutting sections through diaphragms modeled with horizontal finite elements.

**Interpretation of Minimum Collector Design Forces.** ASCE 7-05 Section 12.10.1.1, Diaphragm Design Forces, provides the equation for vertical distribution of diaphragm forces, with the maximum and minimum diaphragm forces of $0.4S_{DSI\bar{w}_{px}}$ and $0.2S_{DSI\bar{w}_{px}}$, respectively. When the minimum diaphragm force from Section 12.10.1.1 is used for collector design, it may not be rational to amplify the collector force by $\Omega_0$ as required by Section 12.10.2.1, since the $\Omega_0$ applies to the forces established by $R$ and the seismic force resisting system.
The SEAOC Seismology Committee therefore recommends $E_{mh}$ (in Section 12.4.3.1) for collectors be set equal to the greater of:

1) $\Omega_0 Q_E$, where $Q_E$ is the greater of:

   - $F_{px} (12.10-1)$ ignoring the $0.2S_{Dg}\text{Iw}_{px}$ minimum
   - $F_x (12.8-11)$

   or

2) $0.2S_{Dg}\text{Iw}_{px}$

**Transfer Slabs.** In addition to its inertial loads, the diaphragm must also be designed to transfer forces between vertical elements of the SFRS above and below the floor level in question. This requirement is triggered when the SFRS is discontinuous at the floor level and the load path clearly runs through the diaphragm. But it also applies where the stiffness of the SFRS changes significantly from one story to the next, such as with the introduction of basement walls at the ground level. In both cases the diaphragm forms part of the load path between SFRS elements.

For collectors of diaphragms that transfer forces between SFRS elements, the Seismology Committee position is that additional design considerations are necessary for two reasons. First, when the diaphragm forms part of the load path between SFRS elements, the forces transferred through it (that is, any design forces in addition to the prescribed $F_{px}$) are essentially those of the SFRS and are therefore subject to increases for redundancy. Second, if the diaphragm is acting essentially as an element of the SFRS, it should either be as ductile as the SFRS or remain essentially elastic.

Redundancy and overstrength provisions were not originally intended to be imposed simultaneously on any component. But if the SFRS is proportioned for $\rho$ greater than 1.0, then the maximum force it can deliver might be underestimated by $\Omega_0$ alone. Thus, reasonable arguments may be made that transfer diaphragm collectors should be designed for forces increased by both $\rho$ and $\Omega_0$. As noted in ASCE 7-05 section 12.4.3.1, however, the maximum expected force “need not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear response analysis utilizing realistic expected values of material properties,” and this statement forms the basis of the following Seismology Committee position.

Any portion of a diaphragm that transfers force between vertical elements of the seismic force-resisting system should be designed for the largest transfer force that can be delivered to it by adjacent load path elements, unless the diaphragm’s section proportions and details are shown to provide ductility equivalent to the adjacent SFRS elements. In other words, the diaphragm should not inadvertently become a non-ductile weak link in an otherwise ductile system. Thus, consistent with this philosophy as noted in ASCE 12.10.1.1 for transfer forces in diaphragms, a $\rho$ factor equal to the building redundancy factor shall be applied, but only to the portion of the load created by the change in SFRS stiffness.

**Direct Connection of Diaphragm to SFRS.** ASCE 7-05 Figure 12.10-1 clearly indicates that no “collector” is required when the diaphragm is directly connected to a full-length shearwall. The overstrength factor does not apply to this direct connection.

**Illustration of Design Methodology**

The building industry, specifically the structural engineering profession, has struggled with a reasonable method to construct concrete collector elements in reinforced concrete floor and roof slabs that can accommodate high force demands associated with $\Omega_0$ and other code provisions. Of particular relevance is the ability of the slab sections, rather than the beams, to act as collectors. The methods described below make the design and construction of reinforced concrete collector elements more tenable for certain types of structures.
This method focuses on the issue of the assumed seismic force distribution used in collector design, and it presents methods of design that the SEAOC Seismology Committee judges to be acceptable. The key aspect of any design method for collectors is that each segment of the seismic force path must be evaluated for adequate strength by checking the free body diagram of forces at all potential critical sections.

The purpose of the following discussion is to illustrate an alternative collector design approach where part of the seismic load is resisted by the reinforcement directly in line with the shear wall, which transfers the force directly to the end of the shear wall. The balance of seismic force is resisted by reinforcing bars placed along the sides of the wall and uses the slab shear-friction capacity at the wall-to-slab interface to transfer seismic forces to the wall. See Figure 3. (In this example, "shear wall" represents the vertical seismic force-resisting element; the condition for moment frames or other systems are similar).

![Figure 3. Perspective View of Wall and Collector](image_url)

Where the slab is adjacent to a shear wall and is used to resist seismic “collector forces,” there is an eccentricity between the resultant of collector force in the slab and shear wall reaction. This eccentricity can create secondary stresses in the slab transfer region (or “diaphragm segment”) adjacent to the wall. For a complete and consistent load path design, the effect of seismic force eccentricity in this “diaphragm segment” must be checked to determine that adequate reinforcement is provided to resist the induced stresses.

The design examples use a rational load path for collector forces and outline a design process that satisfies code requirements. The buildings used as a basis for the design examples were deliberately selected to be simple and structurally regular. Two design examples are provided. The first example illustrates the collector design for a post-tensioned roof slab in a building having concrete bearing shear wall seismic force-resisting system. The second example considers the same structure without post-tensioning.

**Slab Effective Width**

A key design issue in this approach is to determine the effective width of slab adjacent to the shear wall that is used to resist collector forces. Where a narrow effective width is assumed, eccentric force effects become small, but more reinforcement may be required to drag the collector forces in-line with the wall. On the other hand, if a wide slab
width is used as collector, more force can be transferred through the slab, reducing reinforcing bar congestion at the end of the wall; however, secondary stresses caused by force eccentricity would be larger.

The procedure outlined in these examples proposes to treat the choice of the effective slab width as a design parameter to be selected by the designer. The first example uses an assumed 45-degree influence line to determine the effective slab width; in the second example the effective slab width is arbitrarily selected to be equal to the wall length. For both cases, the resulting force eccentricity should be checked and, if required, additional reinforcement should be provided in the slab transfer region.

**Collector Design Procedure**

The following is a suggested outline for collector design.

- **Determine Collector Design Forces.** Determine the seismic force distribution to the vertical seismic force-resisting members by conventional analysis and draw the collector force diagram along the line of seismic force-resisting members (Figure 4).

Note that a linear variation of collector force along the line of a vertical seismic force-resisting member assumes that the tributary width of the slab is constant and the collector (which, in this case refers to both the element in line with the wall and its adjacent slab section) is stiffer than the other connecting members.

**Determine the Steel Area Directly in Line with Shear Wall.** It is proposed that the section of the collector that is directly in line with the wall be designed for all the applicable gravity load demand plus a reasonable portion of the total collector force that the designer can select considering the required number of reinforcing bars and practical limitations of reinforcing bar congestion at the end of the wall. Then, the balance of the collector force will need to be resisted by the adjacent slab section in accordance with the following design procedure.

**Select Effective Slab Width to Resist Collector Forces.** Example No. 1 presents a method to assign the effective slab width to resist collector forces based on an assumed 45-degree influence line, which originates from the "point of zero force" along the collector force diagram (see Figure 5).
Example No. 2 arbitrarily uses the following equation to assign an assumed effective slab width.

\[ B_{\text{EFFECTIVE}} = t_{\text{WALL}} + n \times \left( \frac{L_{\text{WALL}}}{2} \right) \]

where, \( n \) is the number of sides that slab exists adjacent to the collector.

Also, it is proposed that the designer may choose any other slab width that satisfies the check for secondary eccentric stresses.

**Determine Required Steel Area to Resist Collector Tension.** Determine the net tension force, \( T_{\text{NET}} \), and the required steel area, \( A_s \), at each section along the collector. (In these examples the required steel area is calculated only at the maximum force location). For most reinforced concrete slabs the net tension force is equal to the calculated collector tension, \( F_T \). For pre-stressed concrete sections, ACI 318-08 section 21.11.7.2 allows the use of the slab pre-compression force from unbonded tendons, \( F_{PT} \), when calculating \( T_{\text{NET}} \) as it is illustrated in Example No. 1. Hence:

\[ T_{\text{NET}} = F_T \Omega_0 \cdot F_{PT} \]
\[ A_s = \frac{T_{\text{NET}}}{\phi \cdot F_y} \]

where, \( F_T \) is the calculated collector tension force, \( \Omega_0 \) is system overstrength factor, \( \phi \) is capacity reduction factor as shown in ACI 318-08 Section 9.3.2.1, and \( F_y \) is yield strength of reinforcing steel.

The reinforcing area, \( A_s \), represents the total area of the required reinforcing steel. Part of this steel may be placed in the slab element directly in- line with the wall, and the balance may be distributed throughout the effective slab width adjacent to the wall. The collector reinforcement shall be placed, as much as practicable, symmetrically about the...
centroid of the concrete section in order to prevent additional out-of-plane slab bending stresses. Additional calculation shall be performed to determine the effect of collector force eccentricity relative to the shear wall reaction and required reinforcement.

For pre-stressed concrete sections, the magnitude of pre-compression force, $F_{PT}$, depends on the assumed effective slab width, which should be selected based on engineering judgment and verified by calculation to determine the required added reinforcement to resist effect of eccentric forces and secondary stresses.

**Check Collector Compression Stress.** Determine the total compression force, $C_{NET}$, and check concrete compressive stress at each section along the collector. (In these examples concrete compressive stress is checked only at the maximum force location). For most reinforced concrete slabs the total compression force is equal to the calculated collector compression, $F_C$. For pre-stressed concrete sections, since pre-compression force, $F_{PT}$, was used to reduce the net tension, $T_{NET}$, it must be accounted for in calculating to the total compression force, $C_{NET}$. Hence:

$$C_{NET} = F_C \Omega_O + F_{PT}$$

The ACI 318-08 Section 21.11.7.5 design concept for collectors in compression is that transverse reinforcement must be provided where large collector compressive forces exist. Since the collector forces have been increased by the overstrength factor, ACI 318-08 gives:

$$\frac{C_{NET}}{A_C} \leq 0.5 * f'_c$$

ACI 318-08 21.11.7.5

where $A_C$ is the gross cross-sectional area of the effective concrete section in compression. The magnitude of $A_C$ depends on the assumed effective slab width. Since the resultant of concrete compression forces would be eccentric relative to the shear wall, additional calculation shall be performed to determine the effect of collector force eccentricity relative to the shear wall reaction and required reinforcement.

**Check Diaphragm Segments for Eccentricity.** For conditions where all or part of collector reinforcement is placed at the sides of the shear wall, the transfer region (or the diaphragm segment adjacent to the wall) should be designed to resist the seismic shear and in-plane bending moment resulting from the eccentricity of the portion of collector force that is not transferred directly into the end of the shear wall. In keeping with the code intent to design collectors and their connections for the “maximum expected seismic force,” the stresses due to collector eccentricity in that diaphragm segment adjacent to the wall shall be determined using an overstrength amplification factor.

The specific diaphragm configurations, such as slab thickness variations, location of framing members, opening patterns, and other local conditions, could produce a complex stress state in the transfer region of eccentric collectors and affect the required slab reinforcement. Due to the vast variation of diaphragm configurations in actual design situations, a single all-encompassing design procedure could not be presented to be applicable to all possible cases. Hence, for the discussions in this section only an example of a simple diaphragm segment is provided to illustrate the general design requirements and a simplified rational procedure to satisfy these requirements.

Figure 6 shows an idealized partial plan at the edge of a diaphragm with the seismic resisting wall “a-d” and the seismic collector located eccentrically at a distance “e” relative to the wall. The figure also shows the diaphragm segment adjacent to the wall with internal forces acting on the free body “abcd,” (except the components of tension/compression forces perpendicular to the free body diagram are ignored in this example for sake of simplicity.) The collector design force is designated as $F_c$. In a general sense it consist of a compression and tension collector portions and the portion of diaphragm shear force along Line bc, respectively, designated as $(F_c \text{ comp})$, $(F_c \text{ tens})$, and $V_d$. Considering the seismic amplification factor and collector eccentricity, the maximum eccentric moment acting on the free body abcd is calculated as:
The applied eccentric moment should be resisted by the combined action of all the diaphragm internal forces, thus:

$$M_e = V_e h + M_1 + M_2 + M_3$$

where, the magnitude of internal forces, $V_e$, $M_1$, $M_2$, $M_3$, could be calculated in a rigorous analysis in accordance with their relative stiffness. However, for practical design purposes, the calculation can be greatly simplified by using capacity design concepts. The following discussion presents a possible procedure for determining the diaphragm segment design capacities. For example, the moment capacity of the slab region under direct tension from collector force, i.e. moment $M_3$ in the Figure 6, may be conservatively neglected. Furthermore, the shear capacity $V_e$ shall be calculated using only the capacity of shear reinforcing bars and neglecting the contribution of concrete section under tension. Hence, the strength limits for the shear force $V_e$ and bending moment $M_2$, are determined as:

$$M_2 = \varphi F_y A_{S2} (j.e)$$
$$V_e = \varphi A_{sv} F_y$$

where, $A_{S2}$ is the reinforcement areas perpendicular to section ab, $(j.e)$ is the effective moment arm, and $A_{sv}$ is the smaller of the reinforcing bar area parallel to sections ab and dc. Then, the required flexural strength, $M_1$, can be calculated as:

$$M_1 = M_e - (V_e h + M_2)$$

(Alternatively, it may be assumed that the bending moments $M_1$, $M_2$, and $M_3$ would reach their allowable strength limit and calculate the required shear, $V_e$, to satisfy the basic equilibrium of forces. A similar procedure may be used for other combinations of bending moments and shear force.)

For conditions where the eccentricity is small relative to the dimension $h$, it is reasonable to assume that the relative stiffness associated with the actions $M_1$ and $V_e$ is much larger than the other actions; hence, without being too conservative, the contribution of the moments $M_2$ and $M_3$ may be neglected, which simplifies the equation for $M_1$, to the following:

$$M_1 = M_e - V_e h$$

The required moment capacity, $M_1$, can be computed by taking into account the effect of the slab’s distributed reinforcement that is provided for gravity loads, but is in excess of what is needed to resist seismic load combinations. For cases where the available distributed reinforcement is not adequate to satisfy the required strength, supplemental reinforcing steel should be provided at the eccentric force transfer zone. Figure 7 shows an arrangement of various reinforcing bars perpendicular to section bc; and illustrates the terms used in the following computation for moment $M_1$.

$$M_1 = \varphi F_y \{A_{S1} (j_1.h) + A^* S (j^*.h) \}$$
where, $A_{S1}$ is the available area of the distributed slab reinforcement perpendicular to the section $b_c$ that can be used for seismic load combination, $(j_1.h)$ is its effective moment arm; $A^{*}_S$ is the area of supplemental reinforcement, and $(j^{*}.h)$ is the effective moment arm of the supplemental steel.

**Check Diaphragm Segment Shear Strength.** The slab shear stress demand should be checked for the shear force transfer region adjacent to the wall and, where required, additional reinforcement shall be provided. Two shear force transfer mechanisms should be considered. First, slab shear strength should be evaluated considering the contribution of all available slab reinforcement. The strength reduction factor, $\varphi$, for shear should be taken as 0.75 according to ACI 318-08 9.3.2.3.

$$V_u \leq \varphi A_{cv}\left(\alpha_c \sqrt{f'_c} + \rho_n F_y \right)$$

where, $A_{cv}$ is the net area of the concrete section bounded by the slab thickness and length of the wall, $\alpha_c$ is the ratio of the width to length of the diaphragm segments, which in this case is equal to effective slab width to the length of the wall, and $\rho_n$ is the ratio of distributed shear reinforcement perpendicular to the wall.

Special attention must be given to $A_{cv}$ when the vertical seismic force-resisting member is not continuously connected to the diaphragm. For example, for an exterior wall that is 25 ft long, but is located adjacent to a 10 ft wide stair opening, then the length used in calculating the shear area is 15 ft.

**Check Shear-Friction at Wall-to-Slab Interface.** The strength of transfer mechanism by shear friction at the face of the supporting wall and/or frame should be checked. For this mechanism the potential sliding plane should be identified; for most practical design cases the potential sliding plane is taken as the vertical plane at the interface between the wall and the slab. The shear-friction reinforcement can include all reinforcement that crosses this plane, as long as it is not used to resist direct tension. Hence, the area of the required shear transfer reinforcement, $A_{VF}$ per foot of wall length is calculated as:

$$A_{VF} = \frac{V_n}{\mu * f_y * L_w}$$

where, $L_w$ is length of interface between the wall and the slab.

**Other Considerations for Collector Design**

**Using Gravity Slab Reinforcement.** The collector design procedure in these examples assumes that a portion of seismic collector load is resisted by the shear strength along the wall interface with the floor slab. For this load path the slab longitudinal reinforcement parallel to the shear wall can be used to transfer the balance of the collector force to the side of the wall.

For design efficiency, a portion of the slab reinforcement that is provided for gravity loads, but is in excess of what is needed to resist seismic load combinations, can be used to resist collector or diaphragm forces, provided it meets the special detailing requirements for seismic force-resisting systems. The seismic requirements are more stringent than those of a typical non-seismic slab to reflect the adverse conditions resulting form seismic load reversals.
The main detailing issues in regard to using slab gravity reinforcement for seismic diaphragm include the following:

- Minimum diaphragm reinforcement ratio and required bar spacing;
- Diaphragm reinforcement splices and development length;
- Symmetric distribution of diaphragm reinforcement at top and bottom of section to resist seismic net axial force without inducing additional slab bending moment.

**Check Local Stress Concentration at face of Wall.** The stress concentration at a face of the SFRS elements should be studied. It is the Seismology Committee's opinion that the local failure will trigger the redistribution of the load and eventually the interface between the SFRS elements and slab will carry most of the load. The local failure will not cause collapse or have significant impact on the load transfer from the strain compatibility of the slab.

**Detailing of reinforcement.** Planar elements such as shear walls and diaphragm slabs have a better post-cracking behavior if the reinforcing is reasonably distributed over regions of high shear and axial stress rather than being concentrated in narrow groups near the edges of these elements. Distributed reinforcing allows the formation of multiple narrow cracks over the stressed region, while the stiffening effect of concentrated group reinforcing results in a few wide cracks with possible localized spalling. Note that the current shear wall provisions allow and encourage the use of vertical reinforcing distributed over the wall section rather than in concentrated boundary elements. Similarly, provision of distributed steel in an assigned effective width of a collector element can result in better post cracking performance than if the collector is made up of large diameter bars in and closely adjacent to the vertical lateral force resisting element.

Using smaller bars in larger amounts to spread over a wide band of slab will result in a better stress distribution than using smaller quantities of big bars. Since a wide slab band is used in collector design, this check seldom becomes critical. This idea is supported from the study of finite element analysis of examples indicating the stress distribution is spread out in a wide band across the slab section. Even with the existence of shrinkage cracks, the reinforcement in the slab keeps the diaphragm inertia force distributed in a relatively wide band until the failure line forms. It is believed that the concentration of bars in collectors changes the force distribution; it attracts the force to the narrow band formed by these bars and causes early overstress in the narrow region.

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**Keywords**

collectors  
concrete slab

**How To Cite This Publication**

In the writer’s text, the article should be cited as: (SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:


Accessible via the world wide web at: [http://www.seaoc.org/bluebook/index.html](http://www.seaoc.org/bluebook/index.html)
Background

According to the 2000 NEHRP Commentary, “experience in past earthquakes has shown that, in general, piping systems are rugged and perform well in strong shaking motions” (BSSC 2001b, section 6.3.11). With respect to property loss caused either by damage to piping and bracing or by water leakage, fire sprinkler piping is similar to other pressurized water piping except that it is distributed more extensively throughout the building interior. Compared to other kinds of piping of similar overhead location and weight, fire sprinkler piping poses a comparable falling hazard, although the diameter of the main distribution lines in a fire sprinkler system are typically heavier than most of the potable water or HVAC water lines in a building. The characteristic of fire sprinkler piping that most sets it apart from other piping in ordinary occupancy buildings is its critical post-earthquake function, and this increases the degree of engineering attention this kind of piping should receive.

A complete fire sprinkler system includes components outside the purview of the structural engineer or building official, such as the underground supply lines and their sources of pressurized water in the local water utility system. Tall buildings may contain firewater storage tanks, and other components of the overall fire sprinkler system include alarms, gauges, and valves. In this article, the scope is limited to the piping itself in the building or industrial structure and its associated supports and seismic bracing.

Earthquake damage to sprinkler piping can take many forms. Outside or beneath the building, underground piping can break or buckle due to settlement or liquefaction-induced ground displacements, and damage elsewhere in the water utility system can prevent sufficient delivery of water volume and pressure. While back-up utilities are outside the scope of this article, some essential facilities such as Veterans Administration hospitals have been provided with their own on-site emergency water supplies to contend with utility outages, and such redundancy may be part of the scope of a performance-based design project. Inside the building, vertical pipes (risers) can break under large interstory drifts in the building. Hangers supporting the weight of the pipe can unseat from their attachment points. Fasteners connecting the hangers to the building structure can pull out under seismic loading. Sprinkler heads can break upon impact with adjacent structural or nonstructural components, such as ceiling panels. Couplings and pipe fittings can break or leak. Piping crossing separation joints that is not detailed for differential movement can be ruptured, as can pipes that are unintentionally restrained at locations where they pass through walls. Nearly all of these failure modes have been observed in past earthquakes, resulting in impairment of sprinkler systems and costly leaks, with the first well-documented report on such damage by Ayres, Sun and Brown (1973) on the 1964 Alaska Earthquake. Fire sprinkler piping damage in the US has been documented for several other earthquakes, including the 1971 San Fernando Earthquake (Ayres and Sun 1973); the 1989 Loma Prieta Earthquake (NFPA 1990); and the 1994 Northridge Earthquake (Ayres & Ezer Associates 1996), (Fleming 1998), (FSAB 1994), (Todd et al. 1994), (Reitherman and Sabol 1995). Extensive damage to fire sprinkler systems and resulting lack of functionality in the 1995 Great Hanshin (Kobe) Earthquake is documented in Sekizawa et al. (1998).

Design Issues

Piping that is hung from the floor or roof above and that is inadequately braced can sway under earthquake excitation and experience large displacements relative to its support points. This can cause impact damage to the piping and the sprinkler heads. Several cycles of large displacements can also cause the hangers to break or unseat from their supports, resulting in partial collapse of the sprinkler system. These types of damage are minimized by bracing. The pipe remains free, however, to vibrate between the brace points, and some braces may have little rigidity.

Braces must be designed for seismic loads generated by the vibrating sprinkler pipe and the weight of the contained water. Braced piping experiences smaller relative displacements but larger forces than unbraced piping. The fittings
and couplings in braced piping must transfer larger moments from one length of pipe to the next; they can break under repeated loading. Threaded couplings are especially vulnerable to cyclic loading (FSAB 1994), and this earthquake experience is reflected in Section 13.6.8.3 (3) of ASCE 7-05, which stipulates that such materials are to be designed using 70 percent of its minimum specified yield strength, rather than 90 percent, if Seismic Design Category D, E, or F applies. Structural fasteners must transfer the loads from supports and braces to an adequately designed supporting structure. Braced pipes crossing seismic separation joints between adjacent parts of the building are susceptible to damage, unless provided with flexible separation assemblies capable of absorbing the relative motion. A braced system can still be damaged by impacts from adjacent unbraced nonstructural systems such as suspended ceilings. Even if typical suspended ceiling bracing is present, ceiling movement may be excessive in comparison with the stiffer piping.

General design strategies for seismic protection of piping systems include:

- Braces to control relative displacements
- Flexible connections to relieve stresses at critical locations
- Clearance in ceilings around sprinkler heads to minimize impact damage to piping and sprinkler-heads, or use of flexible vertical pipe products connecting sprinkler heads to lateral lines
- Increasing the lateral stiffness of suspended ceiling bracing.

These basic design strategies have historically been implemented through prescriptive “design by rule” approaches. Systems so designed have generally performed well, although thorough and documented performance assessments are rare, and even isolated failures can be costly. Design features intended to improve performance that have been implemented in areas of expected strong shaking have included:

- Steel braces capable of resisting tension and compression, with the braces and attachments designed for a horizontal force equal to at least half the weight of the tributary length of water-filled pipe (see Code Approaches and Interpretations below)
- In single-story buildings, braces at the top and bottom of risers and at flexible couplings
- Braces on all mains
- Lateral braces at changes in direction and at ends, and not exceeding 40 ft spacing in straight runs
- Longitudinal braces at changes in direction and at ends, and not exceeding 80 ft spacing in straight runs
- Adequate clearance of pipes through floors
- Adequate clearance around sprinkler heads
- Vertical upward restraint at ends of branch lines
- Retaining straps where C-clamps are used to attach hangers to the structure. For a C-clamp on the bottom flange of a wide-flange beam, this may take the form of a strap extending the width of the beam from the clamp on one side across the bottom of the flange to the other side, keeping the clamp snug against its flange and preventing it from “walking” under cyclic loading
- Restraint of equipment, racks, suspended ceilings, and other nonstructural components that could affect sprinkler piping.

**Code Approaches and Interpretations**

ASCE 7-05 section 13 provides general design requirements for nonstructural components. Section 13.6.8 references NFPA 13 (1999) as an appropriate design guide for fire protection piping, but the designer must demonstrate separately that the general force and displacement requirements of Section 13.3.1 and 13.3.2 of ASCE 7-05 are also met. Additional seismic bracing requirements for fire sprinkler systems for Seismic Design Categories D, E, or F are given in Section 13.6.8.3. 1997 UBC section 1632.5 makes a similar requirement for nonstructural protection designed by alternative standards.

The 2003 IBC generally adopts ASCE 7-02 for the design of nonstructural seismic protection, but section 1621.1.1 excludes ASCE 7 section 9.6.3.11.2, which would have made NFPA 13 consistent with the balance of the IBC and ASCE 7 nonstructural provisions. It is expected that the 2006 IBC will adopt ASCE 7-05 for the design of nonstructural seismic protection.
A Technical Interim Amendment (TIA) to the 2002 edition of NFPA was issued by NFPA, and the 2007 edition of NFPA 13 (NFPA 2006) published in the latter half of 2006 aligns NFPA 13 requirements with the 2000 NEHRP Provisions and with ASCE 7-05. This updated 2007 edition of NFPA 13 implements the following two main changes, with which the Seismology Committee generally concurs.

1. It clarifies that the seismic design loads should be computed in accordance with local building codes.

2. It updates the capacities to be used for generic fasteners.

Until the updated 2007 edition of NFPA 13 is adopted by the building code, the Seismology Committee position is that design loads should be computed per ASCE 7-05 section 13.6.8, including section 13.6.8.2 for Seismic Design Category C and 13.8.6.3 for Seismic Design Category D, E, or F. Specifically, the design load procedure in the 2002 edition of NFPA 13, which is based on half the weight of the water-filled pipe, is not appropriate.

With respect to fasteners, the Seismology Committee position is that the tabulated design values of NFPA for wood, concrete, and masonry are unconservative and that fastener capacities should be determined in accordance with the appropriate materials chapter of the governing building code.

With regard to existing building evaluations, the engineer must take into account the changing seismic installation requirements of NFPA 13 over the years. While the standard dates back to 1896 and was the central issue that caused the formation of the National Fire Protection Association that year, the inclusion of seismic bracing requirements extends back only to the 1939 edition of NFPA 13. Only as of the 1940s in some areas of the Western United States was such “sway bracing” installed. Because of insurance industry requirements, some seismic bracing may be found in older buildings that otherwise received no nonstructural protection, because the building code of the day did not yet include seismic requirements for nonstructural components. NFPA 13 seismic bracing requirements have evolved over the years, increasing in specificity and required level of earthquake resistance, resulting in a variety of as-built conditions that may be encountered, even in a single building that has been remodeled or enlarged at various times.

**Recommended Research**

Research is needed to determine the strength and deformation capacity of sprinkler piping components, both braced and unbraced, under cyclic loading, as well as consideration of the behavior of related components such as suspended ceilings. The findings will support system models to predict performance under different levels of shaking—information that is needed to implement performance-based design of these critical nonstructural components. Research is also needed to support estimates of amplitude, frequency, and cyclic-demand parameters at different floor levels within buildings.

A protocol exists for testing seismic brace components under monotonic loading (UL 1995). A relatively new cyclic testing protocol has been proposed for testing brace components under cyclic loading (FM Global, 2001) and used in a series of tests (Malhotra et al. 2003). Shake table testing of piping at the University of Nevada at Reno, although not specifically representative of fire sprinkler piping, is reported in Maragakis et al. (2003) and Maragakis et al. (2005). The in-progress Federal Emergency Management Agency (FEMA) Performance-Based Seismic Design Guidelines (ATC-58) includes tasks related to the definition of engineering demand parameters for nonstructural components and has developed testing protocols for acquisition of data that can support the development of fragility curves (Bachman et al. 2004). Research using those protocols is outside the scope of that FEMA-funded project.

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**Keywords**
- fire sprinklers
- nonstructural piping
- water leakage

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Foundation seismic design and detailing provisions are explicitly contained in both ASCE 7-05 and the 2006 International Building Code (IBC) (2007 CBC). There are subtle differences among these provisions. ASCE 7-05 splits the foundation seismic design and detailing requirements between Section 12.13 for general requirements (such as pile anchorage, pile splices and foundation ties) and the materials-specific Chapter 14. The IBC 2006 contains all of those requirements in Sections 1808 through 1811. Both ASCE 7-05 (Section 12.13.6) and the 2006 IBC (Section 1808.2.23.2) reference the provisions of ACI 318-05 Section 21.10 for Seismic Design Categories (SDC) D, E, and F. For Seismic Design Category C, limited seismic detailing is required. There are no special foundation design and detailing requirements for Seismic Design Categories A and B.

The provisions for evaluating potential geologic and site-specific hazards, including ground slope instability, liquefaction, differential settlement, and surface displacement due to faulting or seismically induced lateral spreading, are contained within ASCE 7-05 Section 11.8 and 2006 IBC Section 1802.2.

With the adoption of the 1997 seismic hazard mapping (revised in 2002) developed by the US Geological Survey (Frankel, et al, 2002), based upon known faults and spectral acceleration contours contained in ASCE 7-05 and the 2006 IBC, there are now significant geographic areas in central and southeast California which are designated as SDC C. As noted above, only limited seismic detailing requirements pertain to this Seismic Design Category, whereas under the 1997 UBC, these areas where classified as Zone 3 and the special seismic detailing provisions were applicable. The relaxation of the seismicity level in areas in some areas of California and, consequently, the relaxation of requirements for structural detailing, will in certain cases result in a detrimental reduction of foundation ductility and strength from that required by the 1997 UBC and prior California building codes.

In addition, since the strength design load combinations in ASCE 7-05 and IBC 2006 contain the effects of vertical acceleration in the earthquake loads (causing a reduction in net vertical downward loads), the strength of vertically oriented elements that resist lateral forces may be underestimated. With this effect, the strength differential between the foundation system and the superstructure is increased, potentially forcing more ductility demand into the foundation system. Increasing the code level earthquake force for design of foundations may be one option to offset this effect. At this time there are no definitive recommendations.

**ASCE 7-05 and 2006 IBC**

The basis for the foundation seismic design provisions in ASCE 7-05 and 2006 IBC originally comes from the 2000 International Building Code (IBC). The seismic provisions in the 2000 IBC foundations chapter were principally the result of SEAOC Seismology Committee code change proposals to the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 2000) adopted in the development of the IBC. ASCE 7-05 also incorporates substantial changes from Chapter 7 of the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA, 2003). There are several subtle differences between ASCE 7-05 and the 2006 IBC provisions for foundation design, which result in different design loads and detailing requirements, as discussed below. In addition, micropile design provisions are included in the IBC 2006, but not in ASCE 7-05.

The following two sections summarize seismic design and detailing requirements for SDC C and SDC D through F.
Seismic Design Category C

The lowest Seismic Design Category for which requirements for seismic design of foundations pertain is SDC C. General geotechnical requirements given in ASCE 7-05 Section 11.8 and IBC 2006 Section 1802.2 include a geologic hazard investigation (when required by the authority having jurisdiction) for slope instability, liquefaction, lateral spreading, and surface rupture; and ties between individual pile caps, piles, drilled piers, and caissons for 0.1 $S_{DS}$ times the foundation component load, unless other means of equivalent restraint are provided.

General seismic design and detailing requirements for Seismic Design Category C include developing the pile reinforcing into pile caps for tension as well as compression, as in the case of pile uplift; termination of spirals and transverse tie reinforcing using seismic hooks; excluding the use of bare structural steel sections for axial tension anchorage using concrete bond only (short embedment lengths relying upon concrete bond to the bare steel section is considered unreliable for cyclic loading under seismic forces); and provisions to maintain the location of the special transverse reinforcing for driven piles considering pile cut-off due to variable pile driving depths. In addition, under the IBC 2006 only, pile splices are required to develop the full strength of the pile unless the demands from the overstrength seismic load combinations are used to design the splice. Pile group effects, both for lateral and vertical load, and pile flexibility relative to the soil for lateral loads, must also be considered. Please refer to Seismic Design Categories D, E, and F below for more information on these particular provisions.

Specific seismic design requirements for each pile material are located in Chapter 14 of ASCE 7-05 and Sections 1809 through 1811 of the 2006 IBC. In ASCE 7-05, concrete piles are specifically addressed in Section 14.2.3. For uncased concrete piles, a very low percentage of longitudinal reinforcing is required as a minimum in the top one-third of the pile or a minimum length of ten feet below ground, but not less than what the analysis dictates. Uncased concrete piles are intended to include cast-in-place piles, auger-cast piles, caissons, and piers. Transverse confinement reinforcing is required within three pile diameters of the bottom of the pile cap where inertial interaction with the structure is expected to produce a plastic hinge in flexure. This transverse reinforcing is closely spaced to provide a significant increase in concrete compressive stress and strain within the core and to prevent premature longitudinal bar buckling for strains exceed the yield strain, in effect to increase the ductility of the section. For the remaining length of the pile reinforcing, the transverse reinforcing requirements are nominal and the strength of the pile is relied upon. These requirements in the remaining length of the pile should permit the longitudinal reinforcing to reach its yield stress and the concrete to reach the 0.003 strain limit without failure.

For precast non-prestressed concrete piles, the minimum longitudinal reinforcing steel ratio complies with that for reinforced concrete columns of the superstructure. Transverse reinforcing requirements are similar to cast-in-place concrete piles with the exception that beyond the three pile diameters (or cross sectional dimension in the direction of seismic forces for rectangular piles) of special confinement transverse reinforcing, the ties or spirals should not exceed eight inches on center.

For precast prestressed concrete piles, a minimum transverse spiral reinforcing ratio is required and an equation is provided for the upper portion of the pile where a plastic hinge in flexure is expected to occur due to inertial interaction with the superstructure. The basis for this requirement comes from the PCI Piling Committee Report (PCI 1993). This equation is also used as a lower bound for transverse spiral reinforcing for reinforced concrete columns of the superstructure in ACI 318-05 as equation (21-2). The transverse reinforcing requirements were based on existing formulas which approximated the amount of transverse reinforcing thought to accommodate the estimated imposed curvatures on the cast-in-place foundation elements. The formulas did not take into account the effects of axial load on the ductility of the pile section, and the spiral reinforcing formula did not consider the size of the section. To account for these significant effects, the 1999 SEAOC Blue Book (SEAOC 1999, p. 63), recommended that the transverse reinforcing be a function of axial load, which is reflected in equation 302-1 of the 1999 Blue Book. The transverse spiral steel ratio determined by this formula has been substantiated by some static testing in the 1970s and early 1980s (Sheppard 1983) to increase the ductility of precast prestressed concrete piles to accommodate those curvatures near the pile cap under moderate ground motions. Beyond the pile region
where the special transverse spiral reinforcing occurs, a minimum of one-half of that ratio is required. This amount is thought to provide sufficient strength for any pile-soil kinematic interaction that may occur.

For micropiles, a permanent steel pipe or casing is required for 120% of the flexural length. The pipe or casing provides concrete confinement and shear capacity for an otherwise unconfined and over-reinforced concrete section. Further discussion on using this pile system in seismically active regions is given in the SEAOC Seismology Committee white paper “Micropile Construction, Design and Code Provisions” (SEAOC 2005).

There are no requirements for seismic design of structural steel piles in SDC C.

**Seismic Design Categories D, E, and F**

In the general geotechnical requirements given in ASCE 7-05 Section 11.8 and IBC 2006 Section 1802.2, the geotechnical report must address lateral earth pressures on retaining and basement walls due to ground motions. For cases of potential liquefaction and large soil strength losses, mitigation measures must be discussed in the geotechnical report. The potential for liquefaction and strength loss is evaluated at the site Peak Ground Acceleration from a site-specific design earthquake ground motion study or may be assumed to be $S_0/2.5$ in accordance with ASCE 7-05 or $S_{as}/2.5$ using the IBC 2006. In addition, under ASCE 7-05 Section 12.13.6.2 and IBC 2006 Section 1805.4.2.2, individual spread footings are required to be tied together in soil site classes E and F where soft or weak soils may result in significant independent lateral footing movements that are detrimental to the superstructure performance.

General design and detailing provisions for all piles and grade beams require that they be designed to resist curvatures from earthquake ground motions (soil-structure kinematics or free-field displacement interactions) and structure response (inertial effects). Curvature multiplied by a fixed length, such as a plastic hinge length, result in rotations, which are a more commonly used unit measurement by structural engineers. Interaction from ground motions must be considered both with respect to kinematic effects (soil movements within the pile length) and inertial effects (forces and moments from the superstructure imposed on the pile top). The general provisions section in the IBC 2006 and the uncased concrete pile provisions in ASCE 7-05 Section 14.2.3 also provide the special transverse confinement reinforcing requirements for concrete piles in soil site classes E, F, and liquefiable soil sites. In essence, transverse reinforcing requirements for columns of Special Moment Frames are prescribed for concrete piles for seven pile diameters (or cross sectional dimension in the direction of seismic forces for rectangular piles) from the bottom of the pile cap and at the interfaces of soft-to-medium stiff clay or liquefiable soil. It is expected that significant kinematic interaction will occur at the interfaces of these different soil strata resulting in potential plastic hinging at that location. The seven-diameter length also considers the effect of the case where the inertial interaction extends to a significant depth. These special transverse reinforcing requirements are not applicable to precast prestressed concrete piles where soil site classes E, F, and liquefiable soils are considered in the provisions. Explicit calculation of confinement reinforcing considering curvature ductility is provided in the specific section for precast prestressed concrete piles.

Batter piles and their connections are required to resist the forces and moments resulting from the special seismic load combinations, or essentially $Q_e$, $Q_h$ forces, where $Q_e$ is the system overstrength factor and $Q_h$ is the effect of horizontal seismic forces. The poor performance of batter piles in wharf structures in past earthquakes was the primary motivation for this provision. Historically they have performed poorly due to material deterioration, neglecting the pile continuity to the pile cap, poor configuration, and poor detailing.

ACI 318-05 contains Section 21.10 on foundation design and detailing for the higher seismicity regions. These provisions are brought into ASCE 7-05 and IBC 2006 by reference, except when they are modified by, or in conflict with, the provisions of the building code. Section 21.10 requires grade beams and beams in mat footings that are subject to forces and moments from columns that are part of the seismic force-resisting system to be designed as ductile Special Moment Frame beams. An exception to this provision is provided in ASCE 7-05 and IBC 2006.
where the grade beams have the strength to resist the overstrength seismic load combinations, or essentially $\Omega_o Q_E$ forces. This provision recognizes that foundation grade beams can be constructed to be very large quite easily and the designer can take advantage of that fact. The length of special confinement transverse reinforcing for piles in ACI 318-05 conflicted with that specified in the provisions of ASCE 7-05 and the IBC 2006, therefore compliance with that ACI provision is not required. And since there are separate precast prestressed concrete pile provisions that have considered soils that may not provide lateral support, there was no need to have them comply with that particular provision in ACI 318-05.

Anchorage of piles to the pile cap is also addressed in the general requirements. The anchorage to the pile cap must consider the interaction from tension uplift and bending from the fixity of the pile-to-pile cap connection. Where the pile is required to resist uplift forces or provide moment resistance, a series of capacity checks are required and the lesser of the capacity or overstrength demand under $\Omega_o Q_E$ forces may be used for design. In the case of uplift, the tensile capacity of the longitudinal reinforcing or the structural steel section, and 1.3 times the pile uplift capacity limited by the soil strength is checked. For moment resistance or rotational restraint, the axial, bending and shear nominal strength capacity of the pile must be checked against the overstrength demand under $\Omega_o Q_E$ forces. These provisions are intended to prevent the failure of the anchorage of the pile to the pile cap where such an event would result in loss of the pile-soil interaction that dissipates the earthquake input energy and resists overturning and sliding.

Pile splices are required to develop the strength of the pile, but need not exceed the forces and moments from the overstrength seismic load combinations, or essentially $\Omega_o Q_E$ forces. The significance of splices is considered in the same context as the anchorage.

Provisions are also provided for pile-soil analysis procedures. Pile moments, shears, and deflections must be derived from the interaction of the shaft and soil. Where the pile is considered rigid with respect to the soil, simplified analytical techniques can be used (Czerniak, 1957). Piles with an embedment-to-diameter ratio less than or equal to six may be considered to be rigid. Where the pile is flexible when compared to the soil, rigorous procedures are required to obtain realistic demands. Computer programs, such as LPILE (Reese and Wang 1997) or COM624 (Reese 1990), are available that consider the nonlinearity of the soil and the elastic stiffness of the pile for these cases.

Pile group effects on lateral and vertical loading must be addressed. The provisions require that pile group effects on lateral load capacity and displacement be included when the pile center-to-center spacing is less than eight pile diameters (or the cross-sectional dimension in the direction of the seismic forces for rectangular piles). Depending on the reference used, the lateral load capacity reduction factors or “p-multipliers” typically range from 0.4 to 1.0 depending on the pile spacing and location. Pile group effects on vertical loading do not have to be included unless the pile spacing is less than three pile diameters, which seldom occurs.

Concrete pile detailing requirements are located in ASCE 7-05 Section 14.2.3 and in IBC 2006 Sections 1808 through 1810. For uncased concrete piles, a nominal amount of longitudinal reinforcing is required, which is one-half of the minimum that is required for the columns of the superstructure. Uncased concrete piles are intended to include cast-in-place piles, auger-cast piles, caissons, and piers. The longitudinal reinforcing must extend a minimum of one-half of the pile length, as opposed to one-third in SDC C, and throughout the flexural length for SDC D, E, and F. The flexural length is defined as that length of the pile where the moment demand does not exceed the concrete section cracking moment multiplied by the resistance factor 0.4. Where reversing moments occur in the pile this should be interpreted to mean the lowest point in the pile where moments occur. Transverse confinement reinforcing from the Special Moment Frame provisions provides a substantial ductility increase over that required for SDC C. The transverse or confinement reinforcement detailing required by ASCE 7-05 is substantially greater than that required by the IBC 2006. ASCE 7-05 requires transverse reinforcing conforming to ACI 318 Sections 21.4.4.1 through 21.4.4.3 for the “minimum reinforced length” of the pile (the greatest of the following: one-half of the pile length, 10 feet, the flexural length, or three pile diameters). The IBC 2006 for site classes A through D
requires only one half of ACI 318 Section 21.4.4.1(a) volumetric ratio for transverse or confinement reinforcing placed for a length of three times the least dimension of the pile section. Where the “least pile dimension” is used to determine the length of the confinement reinforcing, it should be interpreted to mean the pile dimension in the direction of the seismic forces. Where site class E or F, or liquefiable soils, occur, the reduction from ACI 318 Section 21.4.4.1(a) volumetric ratio of transverse or confinement reinforcing is not permitted. In addition, the special transverse reinforcing must extend at least seven pile diameters from the bottom of pile cap in those site classes. Beyond the transverse confinement reinforcing regions or the “minimum reinforced length” in the case of ASCE 7-05, one-half of that for Special Moment Frames is required with larger spacing, which recognizes the anticipated reduction in demand beyond the regions of potential plastic hinging.

Precast concrete piles are required to have the transverse confinement reinforcing from the Special Moment Frame column provisions throughout the pile length for soil site classes E and F. For the other soil site classes, the transverse reinforcing requirements are similar to uncased concrete piles.

Precast prestressed concrete piles have seismic design provisions that do not reference the transverse confinement reinforcing requirements for Special Moment Frame columns. The provisions are based on the PCI Piling Committee Report (PCI 1993) for the higher seismic zones. Circular spiral and rectangular hoop transverse reinforcing ratios take into account the curvature ductility of the section based on the compressive axial load in the pile. Circular spiral transverse reinforcing requirements consider the core capacity of the section after the loss of the cover. The requirements have been reduced substantially from that required for columns of Special Moment Frames. This reduction in transverse reinforcing steel ratio is larger for small axial loads. However, it can be negligible for large axial loads and can reduce the ductility of the section, because more transverse reinforcing is required to maintain the ductility level. The straight 45% reduction for circular spirals, not considering curvature ductility, is justified by cyclic testing of prestressed concrete piles in New Zealand (Park and Hoat Joan 1990) and the United States (Budek, et. al. 1997). The transverse confinement reinforcing length is greater than the other concrete pile types because it was not changed from the PCI Piling Committee Report, and the special confinement spiral reinforcing ratio was considered nominal enough that the extent of that spiral reinforcing was not considered to be a severe penalty.

Micropiles are only permitted as an alternative system in Seismic Design Categories D, E, and F.

ASCE 7-05 addresses seismic design and detailing of steel piles in Section 14.1.8 and for the IBC 2006, Section 1810.6.4.1. The applicable requirements to steel H piles are addressed implicitly in the IBC 2006 by general reference to AISC 341-05 for structural steel seismic force resisting systems. AISC 341-05 contains seismic design and detailing requirements for steel H piles in Section 8.6. The basis for the steel H pile requirements are cyclic tests done at the University of California, Berkeley, in 1997. A minimum 10% of the pile axial capacity has to be used to design the pile-to-pile cap connection unless it can be shown the demands from the special seismic load combinations, or essentially Q_o Q_e forces, are less, or that there is no pile uplift required under the design seismic forces.

**Foundation Design Approaches**

The SEAOC Seismology Committee had major input into the original 2000 IBC seismic provisions, but significant relaxations were made to the “capacity design” approach in that document and in what is now the ASCE 7-05 provisions, the IBC 2006 provisions, and the 2003 NEHRP provisions. The philosophy of the ASCE 7-05 and IBC 2006 foundation design provisions relies more on design for the “demands” rather than for the “capacity” of the superstructure. This also applies to the components of the foundation elements, such as splices and anchorage to the pile cap or grade beam. For foundation components for which survivability is a concern, the forces resulting from the special seismic load combinations, or essentially Q_0, Q_e forces, are considered the maximum demands rather than the forces resulting from the capacity of the seismic force-resisting system elements of the superstructure. What this
philosophy neglects is the potential for areas of overstrength in the superstructure seismic force-resisting system, either due to economy or convenience of design or to sizes selected for construction purposes. This may lead to additional damage and rocking in the foundations and differential deformation between seismic force-resisting elements and the gravity load-carrying system. The 1999 SEAOC Blue Book provisions (SEAOC, 1999) are less comprehensive than the other documents but include more of a “capacity design” approach.

**Strength Design Methodologies**

A limit state design or strength design method for the soil-to-foundation interface for seismic forces has not yet been fully established. However, a framework for one such approach has been investigated by the Ad Hoc Foundations Committee of the SEAOC Seismology Committee. The results of that study have been published in the 2001 SEAOC Convention Proceedings (SEAOC 2001). Another framework for strength design is introduced into the Appendix to Chapter 7 in the 2003 NEHRP *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 2003).

**References**


**Keywords**
caissons, foundations, piles, pile caps, piers, soil-foundation interaction

**How to Cite This Publication**
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s list of cited references, the reference should be listed as:

Introduction
A geotechnical (foundation and soils) investigation is performed to evaluate the soil, rock, ground water, geologic, and seismologic conditions that potentially affect the design and construction of a proposed development or improvement that will include structures. Specifically, the geotechnical investigation provides recommendations for design of foundation systems and subterranean structures, earthwork and excavation, ground water control, design of paving and flatwork, slopes (including construction and stabilization), and for other soil work related to the project. In seismically active areas, the geotechnical investigation evaluates the hazards associated with geologic and seismologic conditions that may significantly impact the project. The geotechnical report needs to address these hazards and provide design recommendations to mitigate or account for these hazards.

Before beginning the geotechnical investigation, the geotechnical/geologic consultant should be provided with as much information about the project as possible, including site plans of the development showing locations and elevations of the structures, topographic surveys, structural details including type of construction, structural loads, and any unusual structural features. If the type of construction and the structural details are not known because the project is in an early schematic design phase, it is helpful for the design professional to provide the geotechnical/geologic consultant with best estimates of structural loads, considering the range of possible types of construction. This information will allow the scope of the geotechnical investigation to be tailored to the specific project and provide recommendations that are appropriate for the structures under consideration.

The geotechnical/geologic consultant should be made aware of any unusual structural features. For example, when new structural technologies are being considered in the proposed buildings, such as the use of seismic isolation or energy dissipation devices (dampers) it will likely require dynamic structural analyses to be performed. This may require that the site be better characterized in order to provide site-specific ground motions. Another example of an unusual structural feature is where there will be heavy column loads immediately adjacent to much lighter loads, creating the possibility of large differential settlements between the two columns. Another example of unusual structural features is where high uplift loads under seismic loading are anticipated. Knowledge of these and similar unusual conditions at the beginning of the project will allow the geotechnical/geologic consultant to address these special concerns and potentially avoid supplemental work that can result in delay and greater expense.

When projects proceed to final design, geotechnical/geologic consultants should be afforded the opportunity to review the plans to see that the intent of their recommendations has been incorporated into the plans and specifications, and to see if there are significant changes in the project. Such changes may include location, size, structural loads, foundation systems and excavation depth. If there are significant changes to the project, modifications to the recommendations in the geotechnical report may be needed, and in some cases, additional geotechnical explorations may be warranted.

An insufficient or inadequate geotechnical investigation, improper interpretation of results, or a failure or inability to properly present results in clear, understandable language may contribute to an inappropriate design, unnecessary delays in construction, costly change orders, use of substandard materials, damage to the site and possibly adjacent sites, post-construction remedial work, or even failure of a structure and subsequent litigation.

Being overly conservative in geotechnical recommendations can seemingly provide a safe design. However, such practices can lead to waste and excessive cost, require that structures be designed for unrealistic forces, or even economically threaten or terminate the project. Excessive excavation and replacement due to excessive conservatism
can also lead to potentially greater impact to adjacent properties. As there is an ever-growing competitiveness in the engineering field, there is a tendency for geotechnical consultants to be selected not on the basis of qualifications, but on price. This can result in budgets for investigation being reduced, which translates not in reduction of the scope of work but rather in the effort expended to meet that scope of work. Also, because of the generally litigious nature of society, these factors sometimes combine to encourage recommendations with a greater degree of conservatism to avoid failures and possible legal actions. The selection of geotechnical services based solely on low cost may result in the work being awarded to less experienced firms or individuals that would be more conservative based on their lack of experience.

**Changing Practice**

The general practice in the production of geotechnical investigations varies from region to region in the United States. While subsurface explorations (generally consisting of drilled bore holes) are conducted in all regions, the type of explorations and the type and frequency of soil sampling varies. In some parts of the United States, standard practice for soil sampling for ordinary projects may be limited to in-situ Standard Penetration Tests (SPT), where the blow counts (N-values) are recorded. The geotechnical recommendations are then based on empirical relationships between soil capacity and/or soil strength and compressibility properties with the N-values and general soil classifications, along with engineering judgment based on similar projects and loadings on similar soils (see Coduto 2001). More critical projects or those of greater scale should include laboratory testing of soils for classification and determination of engineering properties.

In parts of the country where there is significant soil variability or where soil stability under seismic loading is a consideration, the general practice may include more extensive soil sampling with the recovery of relatively undisturbed soil samples in addition to Standard Penetration Tests. Soil properties including shear strength and compressibility may be determined by laboratory testing of the “undisturbed” samples and/or remolded samples of the soils. Investigations may also need to evaluate seismic criteria, such as Site Class, or hazards, such as liquefaction. Additional tools such as seismic velocity profiling by geophysical testing methods or cone penetration tests (CPTs) may also be performed, as well as dynamic testing of soil samples. The geotechnical recommendations are then developed from calculations using the soil properties determined from the field investigation and laboratory testing.

Until recently, seismic design considerations in geotechnical reports were generally only presented for projects in the most seismically active regions of the country. However, as the newer seismic provisions are being developed and adopted that increase seismic design levels, more regions of the country are considered as having greater seismic potential and will be required to meet the new seismic provisions. Thus, geotechnical reports may need to address seismic hazards in addition to the normal static design considerations in those regions formerly thought to be less seismically active. Seismic hazards such as liquefaction are required by the International Building Code (IBC) to be evaluated for projects with a Seismic Design Category of C or higher.

As the practice of civil engineering encompasses many varied disciplines, geotechnical investigations should be performed by licensed, qualified civil engineers with special education, training and demonstrated experience in geotechnical engineering. State licensing and registration regulations for geotechnical engineering professionals vary. In California, government-owned K-12 and community college schools and all non-federal hospital projects are subject to review by the State of California Division of the State Architect and the Office of Statewide Health Planning and Development, respectively, and those agencies require that geotechnical investigations be conducted by registered Geotechnical Engineers. As the evaluation of seismic hazards also involves geology and earth sciences, geotechnical investigations that include detailed seismic hazard evaluations should also be conducted with qualified geologists or earth scientists. In California, these individuals should be Certified Engineering Geologists.

Beginning with the 1997 NEHRP Provisions, there is a departure from earlier seismic provisions. Previously, seismic ground motion hazards for standard structures were defined as having a 10 percent probability of exceedance in 50 years, or a mean recurrence interval of approximately 475 years. It was intended that this definition would provide a uniform likelihood throughout the country that the design ground motion would not be exceeded. However, it did not provide a uniform margin against failure for structures designed for that ground
motion in all regions of the country. The approach adopted in the 1997, 2000 and 2003 editions of the NEHRP Provisions is intended to provide a similar margin against collapse at the design ground motion in all regions of the country. To accomplish this aim, ground motion hazards were related to the Maximum Considered Earthquake ground motions, which are based on a set of rules that depend on both probabilistic and deterministic site-specific ground motions.

The design earthquake ground motion is taken as two-thirds of the Maximum Considered Earthquake ground motion. As stated in the Commentary to the 2003 NEHRP Provisions on ground motion (BSSC 2004), the maximum considered earthquake ground motion in most regions of the U.S. is defined with a uniform likelihood of exceedance of 2 percent in 50 years, or a mean return period of about 2,500 years. In regions of high seismicity, such as much of California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well-defined fault systems. The ground shaking associated with a 2 percent probability of being exceeded in 50 years would be much larger that that which would be expected based on characteristic earthquakes occurring on average every few hundred years on these known active faults. In these regions, NEHRP considers it more appropriate to determine the Maximum Considered Earthquake ground motion based on the characteristic earthquakes of these defined faults. The median estimate of ground motion for the characteristic event is multiplied by a factor of 1.5 to determine the Maximum Considered Earthquake ground motion. Because of this change in the way design ground motions are determined in the newer NEHRP Provisions, the design earthquake ground motion will generally be higher than under previous provisions in many regions outside of the higher seismic regions of California and some other Western states, and seismic design practices will need to be incorporated into more buildings and structures. This will require that geotechnical reports provide more information related to seismic hazards and recommendations related to seismic design.

Where geotechnical and/or geological reports contain specialized seismic design information or where substantial professional judgment needs to be applied, peer review of the geotechnical and/or geological reports may be useful to verify the adequacy of the methods used and the reasonableness of the assumptions made. Such dialogues can lead to eventual development of seismic standards of practice in these areas.

**Code Approaches**

The ASCE 7-05 earthquake provisions are based on the 2003 NEHRP provisions. Foundation design requirements are found in Section 12.13, which addresses only those foundation requirements related to seismic resistant construction. ASCE 7-05 assumes that the foundation investigation will comply and address normal non-seismic requirements dealing with extent of investigation, treatment of fill, slope stability, subsurface drainage, settlement control, basement and retaining walls, and foundation requirements. Section 12.1.5, which applies to all Seismic Design Categories, requires that the capacity of the foundation soils be capable of resisting loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil during the earthquake. Section 11.4 provides the procedures for determining the Maximum Considered Earthquake and design earthquake ground motion accelerations and response spectra. These provisions are based on the 2003 NEHRP provisions (BSSC 2004). Although not required in the provisions, the site class, based on definitions in Table 20.3-1, should be determined by the geotechnical/geologic consultant. In Section 11.8.2, an investigation for structures with Seismic Design Category C or higher should also include the potential hazard due to slope instability, liquefaction, lateral spreading, and surface rupture, and the report should include appropriate recommendations for foundation design or other measures to mitigate the effects of these hazards. The geotechnical/geologic consultant will need to have a full understanding of the proposed project, as discussed earlier in this article, to include the information necessary for design. The design professional will need to inform the geotechnical/geologic consultant of the Seismic Design Category of the project to determine the need for seismic hazards evaluation. Some small projects (such as less than 1,500 square feet), or where a previous geotechnical report including seismic hazards evaluation was written for an adjacent project, may not require a re-evaluation of the seismic hazards.

Chapter 7 of the 2003 NEHRP Commentary provides a discussion of the procedures for evaluating the hazards listed in ASCE 7-05 Section 11.8.2. The information in the Commentary relies upon references that generally predate the Loma Prieta and Northridge earthquakes and does not make references to later developments and advances. The 2000 NEHRP Commentary is based in large part on the Appendix of California Division of Mines and Geology
Special Publication 42 (now California Geological Survey) Special Publication 42, 1988 Revision (Hart 1988). Special Publication 42 has been revised several times since 1988 and the current revision is dated 1997 with 1999 supplements (Hart and Bryant 1999). The same basic information is contained in Note 49 published by the California Geological Survey (2002).

The NEHRP Commentary also provides some procedures for evaluation of liquefaction and slope instability. Its references date to the 1970s and 1980s. Later guidelines for evaluating and mitigating seismic hazards due to liquefaction and landsliding were published by the California Division of Mines and Geology (1997). The current state of practice in evaluating and mitigating liquefaction hazards can be found in Youd and Idriss (1997), Youd et al. (2001), and Martin and Lew (1999). New guidelines for evaluating landslide hazards have been developed by Blake, Hollingsworth and Stewart (2002). The 2007 California Building Code (CBC) suggests that liquefaction evaluation be performed using a peak ground acceleration (PGA) equal to \( S_{DP}/2.5 \) (Section 1802.2.7). However, the magnitude associated with this level of shaking is not defined. The liquefaction hazard could be evaluated by determining the magnitude associated with the design-level ground motion. This could either correspond to the deterministic earthquake or the predominant magnitude from the probabilistic earthquake that were used in developing the design level ground motion. If the probabilistic earthquake governs at short periods in the design level ground motion, as an alternative procedure a probabilistic site hazard analysis could be performed to obtain the Magnitude-7.5-weighted PGA, then divide that PGA by 2.5. It is also implied by Section 1802.2.7 that the design level PGA for other seismic evaluation needs can be taken as \( S_{DP}/2.5 \).

The CBC is based primarily on the 2006 International Building Code (IBC) and presents requirements for a foundation investigation in Section 1802. The CBC requires the evaluation of the Site Class at the building site to determining the seismic loadings. This requirement is similar to those in the 2003 NEHRP and ASCE 7-05 provisions. The classification is to be based on observation and any necessary tests disclosed by borings or excavations made in appropriate locations. The CBC also states that additional studies may be necessary to evaluate soil strength, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness. In addition, the expected and differential settlement is to be reported. For structures with CBC Seismic Design Category C or higher, the potential for slope instability, liquefaction, and surface rupture due to faulting or lateral spreading must be evaluated during the geotechnical investigation. For structures with CBC Seismic Design Categories D or higher, lateral pressures on below-grade structures due to earthquakes and the potential consequences of liquefaction must be evaluated. The effects of adjacent loads are to be included in foundation design where footings are placed at differing elevations. The investigation is also to address provisions for control and drainage of surface water around buildings. For certain types of projects, or where a detailed seismic hazards evaluation is warranted, engineering geologic reports addressing geologic hazards as well as geotechnical and supplemental ground-response reports may be needed.

When recommending seismic lateral earth pressures on below-grade structures, the recommendations should be developed with an understanding of the stiffness of the structure, the height of the structure, the stiffness and type of earth material, the potential movement of the wall, unbalanced earth pressures from one side of the structure to the other (such as with sloping ground), and appropriate seismic parameters. In general, the seismic parameters used in a seismic lateral earth pressure evaluation are significantly less than the peak ground acceleration, and usually account for a reduction due to equivalent uniform loading (as opposed to peak loading), and for a reduction to reflect the seismic coefficient (average acceleration of the block of earth behind the structure).

The application of the building code (such as the CBC, IBC or other model codes) in practice is usually one where conservative design values are provided in the absence of a geotechnical or foundation investigation. The presumptive design capacities for footings (such as bearing capacity and lateral bearing) are usually conservative for the soil types listed and generally would not result in economical design for major or complex structures with heavy foundation loads. It should be noted that the presumptive code values consider only strength and not settlements, deformations, or displacements. The presumptive code values also do not adequately address the capacities of the soils for seismic loading. Design loadings may similarly be conservative in the absence of a geotechnical investigation. A site-specific ground motion study may provide better estimates of the ground motions for design
than the ground motions determined from the building codes. The conservative of the building codes inherently encourages the use of geotechnical investigations to provide realistic and reasonable design values.

Geotechnical reports must contain and address increasing amounts of seismic and geologic information to provide the necessary design recommendations for design of buildings and structures. Geotechnical and geologic consultants must continue to educate themselves in identification and mitigation of seismic hazards. Structural engineers will need to have a basic understanding of geotechnical issues that may impact their projects so that they are able to recognize and discern whether adequate exploration and analysis has been performed and whether the proposed solutions or mitigations are reasonable and appropriate. The design professional should evaluate if the geotechnical/geologic consultant has the proper knowledge and experience in seismic issues to provide recommendations that are state of the practice without being overly conservative.

Geotechnical parameters must also be developed with an understanding of how they will be applied. Sometimes allowable values are provided when ultimate values would be more suitable. Also, geotechnical professionals should be aware that some design values such as lateral earth pressure are provided as a strength-based design value that is identical to the recommended design value; this is the case where parameters are computed with no factor of safety applied to the value.

The geotechnical/geologic consultant should be aware of additional requirements that the local reviewing agency or jurisdiction may have in addition to the model code provisions. For example two jurisdictions that have formal manuals for the preparation of geotechnical reports are the County of Los Angeles (LADPW 2001) and the City of Santa Monica, California (SMDBS 2002). The California Geological Survey (2007) has published its Note 48, which serves as a checklist for review of engineering geology and seismology reports for public schools, hospitals and essential services buildings. These manuals and checklists provide the minimum standards and recommended format for geotechnical investigation reports submitted to these agencies. These types of documents can provide insight into the geotechnical review process, the minimum standards used in the review, and project approval process for these agencies and thus may produce more consistent, reliable reports. These manuals should not be intended to specify specific engineering methods or scope of studies or to supplant the engineering judgment of the professionals performing the investigations.

**New Thinking**

With an increased emphasis on performance-based engineering, especially with regard to behavior of structures under extreme loading conditions including earthquake, traditional geotechnical methods of analysis may not provide sufficient or adequate information for the foundation design or analysis methods required to model foundation behavior and capacities. Performance based geotechnical analyses will not be governed by traditional factors of safety and arbitrary allowable increases for extreme load conditions. Performance-based geotechnical analyses will most likely be deflection or deformation controlled rather than load or “capacity” controlled.

Statistical methods for understanding the risks and likely outcomes of design may become more useful. This may be especially useful where multiple variables may interact. Such methodologies are currently used in probabilistic seismic hazard analyses where multiple scenarios can be modeled using decision tree analysis techniques. A possible application of this technique may be the likelihood for liquefaction when there is variability in the ground shaking as well as variability in the ground-water level.

The pace of research and new developments in earthquake resistant design has increased greatly since the 1994 Northridge Earthquake. Previously unknown potentially active or active earthquake faults will likely be found, and further understanding of seismic wave propagation through basin deposits may alter the current practice. Design professionals must not be content to sit still and continue to rely upon the past for the answers to the problems of the future. It is incumbent upon the profession to be aware of the latest research, continue to advance in knowledge, and use new technology and analysis techniques as they become proven as effective.
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Keywords
design earthquake
foundations
geologic hazards
geotechnical engineering
site characteristics
soil classification

How To Cite This Publication
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Design Overview

An overview of the design of piles, pile caps, and grade beams under ASCE 7-05 and the 2006 IBC (2007 CBC) is provided in Article 7.01.001, Foundation Design Overview. That article explains that the 2006 IBC is based upon provisions initiated by the 2000 NEHRP/2000 IBC provisions. ASCE 7-05 also incorporates substantial changes from Chapter 7 of the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. The 1999 Blue Book provisions (SEAOC 1999) do not cover detailing provisions in as much depth as ASCE 7-05 does. However, the 1999 Blue Book provisions are still relevant to the design of piles, pile caps, and grade beams, particularly where a capacity design approach is taken.

A performance-based design approach for pile foundations has not yet been addressed by any of the above codes, standards, or guidelines. However, researchers at the University of California, San Diego did propose performance-based design criteria for specific pile types (Silva, Seible, Priestley, 1997). This work can be used as a basis for the performance-based design of other piles and for pile connections where progressive cyclic testing has been done to establish various damage levels versus deformation.

The design force level for the vertical system that resists lateral forces and for the foundation (except the superstructure-to-footing connection) are the same. This has been and still is a recognized area requiring research and code changes (Harden, Hutchinson, and Moore 2006), and it is therefore uncertain as to where the majority of the inelastic deformation occurs. To address this uncertainty and to prevent excessive concentration of inelastic deformation in the foundation system, a BSSC Issue Study (BSSC 2007) has recommended that the foundation design force be higher than the code level design force where there is a high system ductility (high R value). This does not preclude nonlinear behavior in the foundation system, it simply endeavors to limit the ductility demand level imposed on the foundation system. This article recommends, in addition to the philosophical approach of limiting the ductility demand on the foundation system, that a capacity design approach be applied to preclude the formation of any non-ductile mechanisms. The following outlines an approach for the design of the pile cap and reinforced concrete piles. The approach can be extended to other forms of deep foundations and to shallow foundations.

Pile Cap Design

The above references have a limited number of seismic design and detailing provisions for pile caps. If the pile cap should yield or fail in shear before developing the strength of the piles, then the displacement capacity of a yielding foundation system may be compromised. Such a pile cap mechanism can occur due to joint shear stresses from column or wall pier bending moments or from under-reinforcement for bending moments necessary to transfer the overturning forces to the piles. Joint shear failure in footings has occurred prematurely under large-scale testing of column to footing connections (Xiao, Priestley, and Seible, 1996). This type of mechanism lacks the ductility capacity to achieve the performance intended by the code. The exception to this is where flexural yielding of a grade beam or pile cap occurs first. The component in this scenario should be treated as a flexural element and either meet the detailing requirements of ACI 318-05 Section 21.3 or be demonstrated to afford sufficient ductility.

Below are suggested strength and ductile detailing provisions for the pile cap as a flexural member and an evaluation approach to prevent pile cap joint shear failure before flexural yielding of the pile cap.
Pile Cap Seismic Design Provisions

Unless explicitly detailed for ductility, the pile cap should be sized and reinforced to develop the capacity of either the vertically oriented system that resists lateral forces or the pile capacities. This is applicable to both overturning and lateral forces. The pile cap should be designed to transfer the forces of the vertical lateral force-resisting system to the piles, in particular the joint formed between structural components. The joint(s) should be reinforced and detailed to resist the diagonal tension stress induced by the pile, column, or structural wall using the following provisions.

1. The pile caps shall have continuous top and bottom longitudinal reinforcing using a steel ratio not less than 0.0020 for each layer and terminated with seismic hooks in accordance with ACI 318 Section 21.1 at the edges. For pile caps not containing shear reinforcement, the top and bottom reinforcing layers of the pile cap shall be connected by not less than the equivalent of #5 vertical bars at 24 inches on-center each way with 90 degree hooks on one end and seismic hooks on the other end alternated.

2. For piles located at pile cap edges, the distance of pile centerline away from the cap or grade beam edges shall be the greater of the diameter or least dimension of the pile, but not less than 20 inches unless transverse reinforcing consisting of spirals, with a volumetric steel ratio of not less than 0.010, confines the pile or pile longitudinal reinforcing throughout its development length into the pile cap.

3. The diagonal tension stress, \( f_t \), within the pile cap induced by the plastic moment demand from the column, structural wall or pile shall not exceed 3.5 \( \varphi \sqrt{f'c} \), where \( \varphi = 1.0 \). The axial compression force in the column, structural wall or pile may be used to reduce the diagonal tension stress using the relationship

\[
f_t = \frac{f_a}{2} - \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jv}^2}
\]

where \( v_{jv} \) is the average shear stress on the joint area, \( f_a \) the average effective axial compressive stress at the mid-depth of the pile cap and \( f_t \) is negative for tension.

\[
f_a = \frac{P}{A_{eff}}
\]

where the effective \( A_{eff} \), over which the total axial load \( P \) at the column, wall or pile is distributed, is found from a 45 degree spread of the zone of influence for a rectangular section

\[
A_{eff} = (B_c + d_f)(D_c + d_f)
\]

and for a circular section

\[
A_{eff} = \pi (D_c + d_f)^2/4
\]

where \( D_c \) is the overall section depth of the rectangular section or the diameter of the circular section and \( B_c \) is the width of the rectangular section and \( d_f \) is the effective depth of the footing. The effective joint width and depth shall not exceed the pile cap dimensions.

The vertical joint shear \( V_{jv} \) may be taken as the plastic moment of the column, wall, or pile divided by its effective moment arm force couple. Where a pile and column align, the joint shear shall be based on the summation of the internal actions of each component. The effective joint shear stress is taken as
\[ v_{jy} = \frac{V_p}{b_{jy}d_c} \]

where the effective joint width, \( b_{jy} \), is taken for rectangular sections as

\[ b_{jy} = B_c + D_c \]

and for circular sections as

\[ b_{jy} = \sqrt{2} D_c \]

**Exception:** Where grade beams, separate from the pile cap, are designed to develop the moment strength of the column or wall above, the pile cap need not be designed and detailed to resist the plastic moment capacity of the column or wall.

**Limit State analysis of Concrete Piles for Lateral Seismic Forces**

A simplified limit state analysis of fixed-head concrete piles for seismic lateral loading has been developed by Song, Chai, and Hale (2005), which can be used for a capacity design approach. This analysis has been rearranged and a condensed version reproduced in this article. The method is based on characterizing the response of the pile for a selected number of limit states and applies to a fixed-head pile subject to lateral inertial loading at the pile head. The model assumes sequential yielding of the pile under progressive lateral deformation until a plastic mechanism is fully developed. Idealized ultimate soil pressure distributions are assumed in the ultimate state. It is assumed in the methodology that the pile shear capacity is sufficient to develop the plastic moment capacity of the section. Similar procedures can be used to develop a simplified limit state analysis for pinned-head piles (Chai 2002) and steel piles. Figure 1 shows the progressive limit states of the pile, where uniform flexural strength is assumed along the length of the pile. The first-yield limit state occurs when the flexural strength of the pile equals the moment demand.

![Figure 1](https://example.com/figure1.png)

**Figure 1.** Limit states of Fixed-head Concrete Piles under Lateral Seismic Loads
(from Song, Chai, and Hale, 2004)
occurring at the pile head. Further displacement beyond the first-yield limit state results in the formation of the first plastic hinge at the pile cap. Upon even further displacement, the bending moment redistributes to a lower level in the pile forming a second plastic hinge at the depth \( L_m \). Inelastic rotation in both hinges occurs upon further displacement until the pile reaches its ultimate limit state. The ultimate limit state is taken to occur when the pile section curvature capacity is exhausted in the first plastic hinge of the pile, or where the pile section curvature capacity is something less than that of the second plastic hinge where it may control the ultimate limit state. In other words, the second plastic hinge curvature capacity may become exhausted before that of the first plastic hinge if lesser confinement of concrete is provided.

In this method, the lateral soil-pile stiffness is based on the elastic response of the soil-pile system. For a cohesive soil, the soil reaction is proportional to the lateral deflection of the pile, or \( k_h y \), where \( k_h \) is the constant for horizontal subgrade reaction and \( y \) is the lateral deflection. The initial lateral stiffness can be derived from the applicable differential equation (Poulos and Davis 1980) to produce

\[
K_1 \equiv \frac{V}{\Delta} = \sqrt{\frac{2}{E}} \frac{E l_C}{R_e^3}
\]

where \( R_e = \sqrt{\frac{E l_C}{k_h}} \) is the characteristic length of the pile in cohesive soil and \( E l_C \) the effective flexural rigidity of the pile. The force-deformation curve for the pile-soil system, assumed to be tri-linear in characterization, is shown in Figure 2. The reduced lateral stiffness beyond the first yield limit state is derived as

\[
K_2 \equiv \frac{V - V_y}{\Delta - \Delta_{y1}} = \frac{E l_C}{\sqrt{2} R_e^3}
\]

and the rotation of the first plastic hinge \( \theta = (\Delta - \Delta_{y1})/\sqrt{2 R_e} \). The pile-head displacement and lateral force at the first-yield limit state is taken as \( \Delta_{y1} = M_u R_e^2 / E l_C \) and \( V_y = K_1 \Delta_{y1} = \sqrt{2} M_u / R_e \). The term \( M_u \) is the expected ultimate moment capacity of the section.

For lateral strength estimation, the ultimate soil pressure distribution recommended by Reese and Van Impe (2001) is assumed. Using the assumed ultimate soil pressure for cohesive soils, the normalized depth of the second plastic hinge may be determined by solving the equation for \( L_m^* \) and for the actual depth \( L_m = L_m^* D \).

\[
0.5 L_m^* + 1.5 L_m^* \frac{3}{\Psi_r} = M_u^* \quad \text{for} \quad L_m^* \leq \Psi_r
\]

\[
2.75 L_m^* - 0.75 \Psi_r^2 = M_u^* \quad \text{for} \quad L_m^* > \Psi_r
\]

where \( M_u^* = M_u / (s_u D^3) \), \( \Psi_r \) is the critical depth coefficient which is equal to \( 9 s_u / (\gamma' D + 2\sqrt{2} s_u) \), \( D \) is the diameter of the pile, \( s_u \) is the undrained shear strength of the soil and \( \gamma' \) the effective unit weight of the soil. The normalized lateral strength at which the second plastic hinge is formed can be found using the equations.
where the actual lateral strength of the soil-pile system is \( V_u \equiv V_u^* s_u D^2 \).

For a cohesionless soil, the soil reaction can be assumed to be proportional to a modulus of horizontal subgrade reaction which increases with depth, i.e. \( n_h \rho \), where \( n_h \) is the constant rate of increase in the modulus of horizontal subgrade reaction and is defined as \( k_u/x \). The initial lateral stiffness can be derived from the applicable differential equation (Matlock and Reese 1960) to produce

\[
K_1 \equiv \frac{V}{\Delta} = 1.08 \frac{EI_e}{R_n^3}
\]

where \( R_n = \sqrt[3]{EI_e/n_u} \) is the characteristic length of the pile in cohesionless soil and \( EI_e \) the effective flexural rigidity of the pile. The force-deformation curve for the pile-soil system, assumed to be tri-linear in characterization, is shown in Figure 2. The reduced lateral stiffness beyond the first yield limit state is derived as

\[
K_2 \equiv \frac{V - V_y}{\Delta - \Delta_{y1}} = 0.41 \frac{EI_e}{R_n}
\]

and the rotation of the first plastic hinge is \( \theta = (\Delta - \Delta_{y1})/1.5R_n \). The pile-head displacement and the corresponding lateral force at the first-yield limit state are taken as \( \Delta_{y1} = M_u R_n^2/EI_e \) and

\[
V_y = K_1 \Delta_{y1} = 1.08 M_u / R_n
\]

respectively. The term \( M_u \) is the expected ultimate moment capacity of the section, same as before.

Using the assumed ultimate soil pressure pattern for cohesionless soils, the normalized depth of the second plastic hinge may be determined using the equation \( L_m^* = \sqrt{2 M_u^*} \) and the actual depth

\[
L_m = L_m^* D,
\]

where \( M_u^* \equiv M_u / (K_p \gamma' D^4) \), \( K_p \) is the coefficient of passive soil pressure, which depends on the friction angle of the soil, \( \gamma' \) the effective unit weight of the soil and \( D \) is the diameter of the pile. The normalized lateral strength associated with the second plastic hinge may then be taken as \( V_u^* = 1.5 L_m^*^2 \) or the actual strength of

\[
V_u \equiv V_u^* K_p \gamma' D^3
\]

The first-yield limit state occurs at a displacement of

\[
\Delta_u = \frac{M_u}{\Delta_{y1}}
\]

Figure 2. Idealized Force-Displacement Curve for Fixed Head Piles

(from Song, Chai, and Hale, 2004)
$\Delta y_1$ (as given above) and the second yield limit state at $\Delta y_2$. A lateral displacement beyond $\Delta y_2$ is characterized by a perfectly plastic force-deformation curve. An equivalent idealized elasto-plastic yield deformation is considered to be related to $V_u$ at a yield displacement of

$$\Delta y = \frac{V_u}{K_1}$$

The lateral displacement at the second yield limit state is taken as

$$\Delta y_2 = \frac{V_y}{K_1} + \frac{V_u - V_y}{K_2}$$

where all terms are defined as above.

The lateral displacement of the pile head from the first plastic hinge formation to the second plastic hinge formation corresponds to $\Delta p^*$ and associated plastic rotation $\theta p^*$ as

$$\Delta p^* = \Delta y_2 - \Delta y_1 = \frac{V_u - V_y}{K_2}$$

$$\theta p^* = \frac{\Delta p^*}{\eta L_m}$$

where $\eta = \sqrt{2} R_c / L_m$ for cohesive soils and $\eta = 1.5 R_u / L_m$ for cohesionless soils, and the other terms are as defined above.

The lateral displacement of the pile head from $\Delta y_2$ to $\Delta u$ (the ultimate displacement based on the ductility of the hinges) corresponds to $\Delta p^{**}$ and a plastic rotation of $\theta p^{**}$ for both hinges as shown in Figure 1. Based on the first plastic hinge capacity, $\Delta p^{**}$ and $\theta p^{**}$ can be found by

$$\Delta p^{**} = (\phi_{u1} - \phi) L_{p1} L_w$$

$$\theta p^{**} = (\phi_{u1} - \phi) L_{p1}$$

for $\phi_{u1} \geq \phi \geq \phi_y$

where $L_{p1}$ is the equivalent plastic hinge length for the first hinge, $\phi_{u1}$ is the ultimate curvature of the first plastic hinge which may be obtained from a moment-curvature analysis of the section, and $\phi$ is the curvature in the first plastic hinge at the lateral displacement of $\Delta y_2$. The term $\phi$ may be found from

$$\phi = \phi_y \left[ 1 + \frac{K_1}{K_2} \frac{\beta L_m}{\eta L_{p1}} (1 - \alpha) \right]$$
where $\alpha = V_y / V_u = \Delta_y / \Delta_y$ and $\beta = \Delta_y / (\phi_y L_m^2)$ and $\eta = \sqrt{2} R_e / L_m$ for cohesionless soils. $\phi_y$ is the idealized yield curvature taken from a moment-curvature section analysis. $\Delta_p^{**}$ and $\theta_p^{**}$ are related by $\theta_p^{**} = \Delta_p^{**} / L_m$.

Based on the second plastic hinge capacity, $\Delta_p^{**}$ and $\theta_p^{**}$ can be found by

$$\Delta_p^{**} = (\phi_{u2} - \phi_y) L_{p2} L_m$$

$$\theta_p^{**} = (\phi_{u2} - \phi_y) L_{p2} L_m$$

for $\phi_{u2} \geq \phi_y$

where $L_{p2}$ is the equivalent plastic hinge length for the second hinge. Based on prior research, $L_{p2}$ may be taken as equal to $D$. The term $\phi_{u2}$ is the ultimate curvature of the second plastic hinge, and $\phi_y$ is the idealized yield curvature, both of which may be determined from a moment-curvature section analysis. The plastic displacement and plastic rotation, $\Delta_p^{**}$ and $\theta_p^{**}$, are related by $\theta_p^{**} = \Delta_p^{**} / L_m$.

Song, Chai and Hale (2004) have also developed equations to determine the curvature ductility demand under a certain design displacement ductility, which are not included here but may be useful to the reader.

References


Keywords
piles, pile caps, grade beams, limit state design

How to Cite This Publication
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(SEAOC Seismology Committee 2008)

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Example Assumptions

This article provides a worked analysis example to accompany Blue Book Article 7.02.02, “Limit State Design of Piles, Pile Caps, and Grade Beams” (SEAOC Seismology Committee 2008). The limit state response of a fixed-head concrete pile will be assessed for a soft, cohesive soil, classified as soil profile type S6. The 22-in-diameter pile structural section consists of 8 #7 longitudinal bars of Grade 60 with #5 spirals at a 3-1/2 inch pitch ($\rho_s = 0.021$) throughout the length of the pile. The pile length is 63 feet.

The following properties result from a moment-curvature analysis of the section.

$\phi_y = 0.00026$ rads/in (idealized)
$\phi_{\mu 1} = \phi_{\mu 2} = 0.0043$ rads/in
$EI_e = 83438$ k-ft$^2$
$M_u = 261.7$ ft-k (expected ultimate moment capacity idealized)

The following soil properties were used.

$\gamma' = 110$ lbs/ft$^3$
$s_u = 430$ lbs/ft$^2$
$k_h = 28.87$ k/ft$^2$

The following parameters are generated from the above soil and structure properties.

$D = 1.83$ ft. = 22 inches
$R_e = \sqrt[3]{\frac{EI_e}{k_h}} = 7.33$ ft
$\psi_r = \frac{9s_u}{(\gamma'D + 2\sqrt{2}s_u)} = 2.73$

The lateral soil-pile stiffness is then taken as:

$K_1 = \sqrt{2 \frac{EI}{R_e^3}} = 297$ k/ft
$K_2 = \frac{EI}{\sqrt{2} R_e^3} = 149$ k/ft

The lateral force required to develop the first plastic hinge is:

$V_y = \sqrt{2 \frac{M_u}{R_e}} = 50.5$ k

The depth to the second plastic hinge may be found as follows.

$M^* = M_u / (s_u D^3) = 99.3$

assume $L^* > \psi_r$

and then solve for $L^*$ using
\[2.75L_m^2 - 0.75\phi^2 = M^*\]

\[L_m = 6.17 > 2.73\] and therefore \[L_m = L_m^* D = 11.3\text{ ft}\]

The lateral strength of the soil-pile system which develops both plastic hinges is

\[V_u^* = 11L_m^* - 4.5\phi = 55.6\]

\[V_u^* = V_u^* s_n D^2 = 80\text{ k}\]

The lateral displacements may be obtained as follows

Idealized yield displacement \(\Delta_y = V_u/K_1 = 0.269\text{ ft.} = 3.2\text{ in}\)

First yield limit state displacement \(\Delta_{y1} = M_u R_s^2 / E I_s = V_u/K_1 = 0.17\text{ ft.} = 2.0\text{ in}\)

Displacement at the second yield state limit \(\Delta_{y2} = V_u/K_1 + (V_u - V_y)/K_2 = 0.37\text{ ft.} = 4.4\text{ in}\)

The plastic rotation and plastic lateral displacement for the first plastic hinge at the onset of the second plastic hinge formation is as follows.

\[\Delta_p^* = (V_u - V_y)/K_2 = 0.20\text{ ft.} = 2.4\text{ in}\]

\[\theta_p^* = \Delta_p^* / \eta L_m = 0.019\text{ rads}\]

where \(\eta = \sqrt{2} R_s / L_m = 0.92\)

The total plastic lateral displacement, \(\Delta_p\), and the plastic rotation in the second hinge, \(\theta_p^*\), as limited by the second plastic hinge are:

\[\Delta_p = \Delta_p^* + \Delta_p^* = 14.4\text{ in}\]

\[\theta_p^* - (\phi_{x2} - \phi_p) L_{p2} = 0.088\text{ rads}\]

where

\[\Delta_p^* = (\phi_{x2} - \phi_p) L_{p2} L_m = 12.0\text{ in}\]

and plastic hinge length for the second hinge has been taken as \(L_{p2} = D\).

The total plastic lateral displacement \(\Delta_p\) as limited by the first plastic hinge is

\[\Delta_p = \Delta_p^* + \Delta_p^* = 9.7\text{ in}\]

where

\[\Delta_p^* = (\phi_{x1} - \phi_l) L_{p1} L_m = 7.3\text{ in}\]
The plastic hinge length of the first hinge, $L_{p1}$, is 18 inches (as calculated from $0.3f_yd_b$, where $f_y$ is in ksi units and $d_b$ is in inch units), and

$$\phi_1 = \phi_y \left[ 1 + \frac{K_1}{K_2} \frac{\beta L_{pl}}{\eta L_{pl}} (1 - \alpha) \right] = 0.0013 \text{ rads/in.}$$

where $\beta = \frac{\Delta_y}{\phi_y L_{pl}^2} = 0.67$ and $\alpha = \frac{V_p}{V_u} = \frac{\Delta_y/\Delta_p}{\phi_{pl}} = 0.63$ and $\eta$ as previously defined.

The total plastic rotation in the first hinge is therefore

$$\theta_p = \theta_p^0 + \theta_p^{\infty} = 0.071 \text{ rads}$$

where

$$\theta_p^{\infty} = (\phi_{pl} - \phi_1)L_{pl} = 0.052 \text{ rads}$$

and $L_{pl} = 18$ inches as given previously.

References


Keywords
foundations
piles
pile caps

How to Cite This Publication

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

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Introduction

The load and deformation characteristics of the structural and geotechnical (soil) components of the foundations of structures can affect, and in some cases dominate, seismic response and performance. Recognizing this important fact, many structural engineers have included representations of foundation strength and stiffness in their seismic analysis models for many years. This was especially the case for those engineers conducting sophisticated analyses in the nuclear power plant industry.

Today's widely available analytical methods and tools allow all structural engineers to include soil-foundation-superstructure interaction in their projects. Performance-based engineering documents adopt detailed procedures that facilitate the inclusion of the foundation in the global structural model for both elastic and inelastic seismic analyses. See for example FEMA 440 (FEMA 2005) on nonlinear static procedures; ATC 40 (ATC 1996) on the evaluation and retrofit of concrete buildings; and ASCE 41-06 (ASCE 2007), which deals with seismic rehabilitation for various kinds of buildings and supersedes the earlier FEMA 356 (2000), which in turn was preceded by FEMA 273 (FEMA 1997).

Soil-structure interaction

The modeling of the soil and structural parts of foundations inherently accounts for the interaction of the soil and structure. There are three primary categories of soil-structure interaction (SSI) effects. These include: (1) a soil and foundation flexibility effect, an introduction of flexibility to the soil-foundation system resulting in a change in the stiffness of the lateral-force resisting elements, which lengthens the fundamental response period of the model; (2) a foundation damping effect, dissipation of energy from the soil-structure system through radiation and hysteretic soil damping; and (3) a kinematic interaction effect, the filtering of the dynamic characteristics of ground shaking transmitted to the structure.

Such effects can be quantified on a global building basis or explicitly on individual foundation components. All three effects, approximated on a global basis, for the equivalent lateral force or modal response spectrum analysis procedures, can be accounted for by using ASCE/SEI 7-05 Chapter 19 (ASCE 2006). Chapter 19 - Soil Structure Interaction for Seismic Design should not be used in conjunction with flexible foundation modeling in the global structural model. Also, this procedure was developed assuming no loss of contact between the soil and the foundation, i.e. no rocking, and therefore may have limited applicability.

Figure 1 illustrates options for structural analysis models, starting with a traditional fixed base assumption.
Fixed Base Assumption. Figure 1a illustrates the assumption that the superstructure is mounted on a completely fixed base and subjected to free field motion (FFM). There is no soil-structure interaction with this assumption. Assuming elastic response, the total force imparted to the structure when subjected to shaking is controlled by its fixed-based period and initial damping (usually 5%). Since the foundation is completely rigid, all deformations in response to the ground motion take place in the structure. The distribution of elastic forces to the individual seismic force-resisting elements depends only on their relative stiffnesses within the superstructure, determined with fixed foundation stiffness. The total stiffness of the superstructure controls the maximum elastic displacement. For inelastic behavior in response to stronger shaking, the sequence of yielding and inelastic behavior occurs entirely in the superstructure. If the shaking is sufficiently large, eventually an inelastic lateral mechanism will form within the superstructure. The strength and energy dissipating characteristics of this mechanism significantly influences the maximum inelastic displacement.

Soil/foundation flexibility. Figure 1b illustrates the incorporation of foundation and soil flexibility into structural modeling. This approach explicitly models both the structural (e.g., spread footing, pile) and geotechnical (soil) components of the foundation. The result is that the response of the overall structural system includes the effects of deformations in the structural and geotechnical parts of the foundation. These deformations are sometimes referred to as an inertial SSI effect. These improvements in modeling can lead to significant departures from fixed based analytical results. Compared with the fixed base modeling approach, the predicted period of the structure lengthens and the elastic distribution of forces among various elements changes. In inelastic models, the sequence of inelasticity and the modes of inelastic behavior can change, and foundation mechanisms (e.g. rocking, soil bearing failure, pier/pile slip) can be directly evaluated and considered. All of these effects result in more realistic evaluation of the actual probable structural behavior.
Including foundation flexibility explicitly in the model of the superstructure essentially quantifies the soil-foundation-superstructure element interaction and displacements. It also quantifies the effects of that flexibility on the seismic force-resisting system and portions of the structure locally connected to, but not part of, the seismic force-resisting system. This is exemplified when new building code design provisions have been used with a fixed base for certain seismic force-resisting systems (typically stiff or single bay systems) and checked using nonlinear analyses with foundation flexibility modeled. The analyses of these systems will typically show significantly reduced demands or plastic hinging of the superstructure seismic force-resisting system elements due to rocking of foundations. However, other components, such as continuous gravity beams and beam or slab collectors, or other elements, either part or not part, of the seismic force-resisting system will sustain increased shear forces, moments and rotational deformation demands from rocking shear walls or frames.

**Kinematic Interaction.** Figure 1c illustrates the filtering effects that soil-structure interaction can have on the character and intensity of ground motion experienced by the structure. For the fixed base model, the free field motion (FFM) is the theoretical movement of a single point on the surface of the ground assuming that there is no structure near it. Kinematic interaction results from the presence of relatively stiff foundation elements on or in soil that causes foundation motions to deviate from free-field motions. Base slab averaging and embedment effects can cause these changes. The base slab averaging effect can be visualized by recognizing that the motion that would occur in the absence of a structure is variable within and below the footprint of the building. Placement of a foundation slab across these variable motions produces an averaging effect in which the foundation motion is always less than the localized maxima that would have occurred in the free-field. The embedment effect reflects the fact that the discontinuity at the ground surface intensifies motion and the depth that a structure extends below the surface diminishes this motion. Both base-slab averaging and embedment modify the character of the foundation-level motion (sometimes called the foundation input motion, or FIM) from the free-field motion (FFM). Although the physics of kinematic interaction are independent of the structure above the foundation, the effects transmitted to the superstructure are highly dependent on its period. In fact, the effects can be visualized as a filter applied to the high-frequency components of the free-field ground motion. The impact of those effects on superstructure response will tend to be greatest for short-period buildings (see Figure 2).

**Foundation damping.** Figure 1d illustrates foundation damping effects which, in addition to soil/foundation flexibility, are a further manifestation of inertial soil-structure interaction. Foundation damping results from the relative movements of the foundation and soil, and is associated with radiation of energy away from the foundation and hysteretic damping within the soil. The foundation damping is linked to the foundation plan geometry, embedment, and the lengthening of the structural period from the fixed to flexible base conditions (see Figure 3). Foundation damping can be combined with the conventional initial structural damping to generate a revised
damping ratio for the entire system including the structure, foundation, and soil. This system damping ratio then modifies the initial ground motion (e.g. FFM or FIM) imparted to the structural model as seismic shaking demand. The result is normally an effective decrease in the spectral ordinates of ground motion experienced by the structure.

**Summary of current codes and standards related to foundation modeling**


ASCE 7-05 Modeling Criteria, Foundation Modeling, Section 12.7.1 permits the use of a fixed base in the structural analysis. Where foundation flexibility is considered, the procedure given in Chapter 19 - Soil Structure Interaction for Seismic Design or Section 12.13.3 – Foundation Load-Deformation Characteristics may be used.

**ASCE 7-05 Chapter 19 - Soil Structure Interaction for Seismic Design.** This procedure has existed in the NEHRP Provisions for a long time, but appears not to have been used very often in practice. This method of including foundation flexibility may be used only for the equivalent lateral force or modal response spectrum analysis procedures, which are both linear. The NEHRP Commentary aligning with this section comprises a good summary of the basis of the procedures and SSI effects generally. The SSI procedures account for inertial interaction effects with a reduction in the elastic base shear used for design as reflected in Equations 19.2-1&2. This source of this reduction can be illustrated by combining the two equations as follows:

\[
\tilde{V} = V - \Delta V = C_s W - \left[ C_s - \tilde{C}_s \left( \frac{0.05}{\beta} \right)^{0.4} \right] \tilde{W} = C_s \left( W - \tilde{W} \right) + \tilde{C}_s \left( \frac{0.05}{\beta} \right)^{0.4} \tilde{W}
\]

where:

- \(C_s\) = the seismic response coefficient computed from Eqs. 12.8-2, 12.8-3 and 12.8-4 using the fundamental natural period of the fixed-base structure as specified in Sec. 12.8.2,
- \(\tilde{C}_s\) = the seismic response coefficient computed from Eqs. 12.8-2, 12.8-3 and 12.8-4 using the effective period of the flexibly supported structure defined in Sec. 19.2.1.2,
\( \tilde{\beta} \) = the fraction of critical damping for the structure-foundation system determined in Sec. 19.2.1.2, and 

\( \overline{W} \) = the effective gravity load of the structure, which shall be taken as 0.7\( W \), except that for structures where the gravity load is concentrated at a single level, it shall be taken equal to \( W \).

The reduction in base shear can only be applied to the mass of building associated with the fundamental period of vibration, so the term \( C_i (W - \overline{W}) \) represents the portion that is not subject to reduction. The term \( \tilde{C}_i \) is calculated using a period that accounts for the flexibility of the base. The revised period is a function of the translational and rotational stiffnesses of the foundation, \( K_t \) and \( K_{\theta} \), computed by principles of foundation mechanics (see the Commentary) using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The spectral acceleration associated with this adjustment for period lengthening is further reduced by the term \( \left( \frac{0.05}{\beta} \right)^{0.4} \) that accounts for the increase in system damping (over the 5% normally assumed for fixed base) caused by the foundation damping, \( \tilde{\beta} \).

**ASCE 7-05 Section 12.13.3 – Foundation Load-Deformation Characteristics.** This procedure permits a linear force-deformation behavior of foundations represented by an equivalent linear stiffness using soil properties that are compatible with the soil strain levels associated with the design earthquake motion or, in other words, the secant stiffness of the foundations. This particular section implies an alternative flexible foundation methodology to that in Chapter 19. This flexible foundation methodology is also permitted for the seismic response history procedures in Chapter 16. The intent of this provision is to permit the use of discrete soil springs, and other springs, including the structural section of the foundation, to model the flexibility of the foundation system in the global structural model. Due to deformation incompatibility between a fixed base superstructure model and a separate flexible foundation model, such separate models are not considered to quantify any soil-structure interaction effect.

The basis for this provision comes from the NEHRP Provisions for Seismic Regulations for New Buildings and Other Structures (BSSC, 2003), Appendix to Chapter 7, Section A7.2.3. The Appendix to Chapter 7 attempts to develop a framework or methodology for the ultimate strength design of foundations and foundation load-deformation modeling for both linear and nonlinear analysis procedures. Further detailed recommendations and illustrations that are not included in ASCE 7-05 may be obtained from that document and the associated NEHRP Provisions Commentary for Appendix to Chapter 7.

The equivalent linear soil stiffness is determined using the soil shear modulus and shear wave velocity for the soil beneath the foundations. The geotechnical parameters to obtain shear modulus, \( G_0 \), and shear wave velocity, \( v_{so} \), are obtained through soil laboratory testing at small soil strain levels. The laboratories generally cannot test to large strain levels. The \( G_0 \) and \( v_{so} \) at small strain levels are extrapolated to large strain levels commensurate with the magnitude of ground motion through the use of ASCE 7-05 Table 19.2-1. Reduction factors are identical to Table 10-5 of ATC-40, but deviate from those given in FEMA 356 and ASCE 41, thereby obtaining large strain \( G \) and \( v_{s} \) for foundation stiffness computation.

**ASCE 41-06 Section 4.5 Kinematic Interaction and Foundation Damping Soil Structure Interaction Effects.** Kinematic interaction and foundation damping effects may be considered under the foundation design basis given in ASCE 7-05 Section 12.1.5 Foundation Design. These effects may be applied to structures even when the flexibility of the foundation is explicitly modeled.

The ASCE 41 provisions for kinematic interaction and foundation damping are taken from FEMA 440 (ATC, 2005). FEMA 440 addressed only the nonlinear static analysis procedure. ASCE 41 has expanded the provisions to all analysis procedures in ASCE 41.
Kinematic interaction effects include Base Slab Averaging and Embedment. These effects result from modification of the ground motion waves by the foundation system. To take advantage of this interaction the foundation components must be laterally connected and meet specific soil site classifications. For Base Slab Averaging, the building should have rigid diaphragms. For Embedment interaction, the building should have a basement. Both kinematic interaction effects are applied as reduction factors on the site response spectra. Kinematic interaction will typically reduce the spectral ordinates in the acceleration (or short) period range of that response spectrum. Where the response history analysis procedure is used, it is implied that the time history records are scaled to the reduced spectral acceleration ordinates resulting from the kinematic interaction.

The foundation damping effects in ASCE 41 Section 4.5 are the result of global radiation damping, not hysteretic soil damping at the component level. There are several restrictions in the use of foundation damping based upon vertical lateral force resisting element spacing, soil shear stiffness profile and location of soft soil layers. The foundation damping effects are applied as a damping factor $\beta_o$, where the spectral acceleration ordinates of the response spectrum are divided by $\beta_o$. The $\beta_o$ damping factor consists of the viscous damping of the structure $\beta$ prior to yield plus the damping due to the radiation damping of the soil $\beta_r$. The $\beta$ value is modified by the period lengthening due to the anticipated ductility demand on the structure. It is not clear in ASCE 41 whether this methodology is appropriate for the nonlinear response history procedure. The SEAOC Seismology Committee recommends that if foundation damping is used in conjunction with the nonlinear response history procedure, then the period-lengthening effect on the $\beta$ value (due to ductility demand on the structure) be disregarded since the nonlinear response history analysis will explicitly take care of that effect. Again, where the response history analysis procedure is used, it is implied that the time history records are scaled to the reduced spectral acceleration ordinates resulting from the radiation damping.

**Foundation Component Modeling**

For shallow bearing footings considered rigid with respect to the soil and where detailed actions in the structural section are not needed, single node point uncoupled soil spring stiffnesses may be generated using elastic-based equations given in or ASCE 41 Section 4.4.2.1.2 or FEMA 356. This type of spring addresses only the soil stiffness as a function of footing geometry and embedment. The directional (vertical and horizontal translational, and rotational) spring components are uncoupled since there is no interaction or influence that alters the response of the other springs when any spring is activated. These elastic-based stiffness equations assume full footing contact with the soil at all times. In this classic procedure, each directional soil spring stiffness at the ground surface is first calculated and then multiplied by a correction factor for the embedment. The soil spring stiffness components are combined and usually placed within a 6 x 6 stiffness matrix for input into each flexible support node in three dimensional analysis computer programs. The horizontal translation stiffness may be derived using FEMA 356 or ASCE 41 Figure 4-6 Passive Mobilization Curve, although the applicability of this curve to a specific soil type is not given. Alternatively, the passive soil resistance force-deformation curve given in the SEAOC AD Hoc Foundation Committee report (SEAOC, 2001) may be used for site class C and D soils. Where pile foundations are used, vertical and rotational stiffness of the pile group may be derived using equations in ASCE 41 Section 4.4.2.2 or FEMA 356. The computer program LPILE (Ensoft, 2004), or its generic counterparts, may be used to quantify lateral stiffness due to pile-soil interaction. Where there are pile groups, reduction factors or “p-modifiers” need to be applied to the lateral stiffness when the pile spacing is less than approximately 6 to 8 pile diameters. This reduction factor accounts for “shadowing” effects of the closely spaced piles. The soil bearing capacity immediately below the pile cap bottom is typically neglected, due to the potential of soil settlement in this region.

For shallow bearing footings that are considered flexible with respect to the soil, the structural section and stiffness of the footing is explicitly modeled with vertical multiple parallel translational soil springs distributed across the footing length, as illustrated in Figure 1 b. This modeling technique will permit recovery of actions on the structural section of the footing and should be used where footing structural section failure is anticipated. The properties of the soil springs may be calculated using the equations given in ASCE 41-06 Section 4.4.2.1.3. This method may also be used for irregularly shaped footings or when the foundation rotational stiffnesses are not calculated. Unless nonlinear elastic soil springs are used, which do not resist tension forces, the structural section of the footing is considered to be in contact with the soil at all times. Under large overturning or differential vertical forces on
continuous footings, there may be tension on the linear soil springs. Significant tension on the soil springs does not represent an admissible state for the shallow footing modeling soil-structure interaction. Under static loading, the soil springs with tension can be progressively deactivated to obtain an admissible state of compression only forces in the springs. Performing a separate analysis to account for foundation flexibility, such as a beam on elastic foundation analysis, by applying reactions from the fixed base superstructure analysis, will lead to deformation compatibility errors between the foundation and superstructure.

Typically, it is not feasible to model all shallow bearing footings with multiple vertical springs for the purposes of quantifying all modes and locations of structural section and soil failure, such as out-of-plane flexure and shear failures of continuous spread footings. Therefore, vertical springs placed intermittently along the longitudinal axis of continuous footings and at the center of non-moment-resisting square spread footings are typically used in modeling, as shown in Figure 1 b. Where the structural section and soil are combined into one spring, the post-elastic behavior will depend upon the relative strengths of the soil, footing flexural capacity, and footing shear capacity.

**Soil.** The soil capacity will control when the soil bearing pressure at the strength of the structural section of the footing is greater than the ultimate soil bearing pressure. For competent soils, the soil force-deformation curve is typically considered elastic-perfectly plastic. A limit for the ultimate soil deformation should be established by the geotechnical engineer in concert with the structural engineer for the performance-level requirements of the structure.

**Footing Flexural Capacity.** The flexural strength of the footing, either in- or out-of-plane, will control when the soil bearing pressure at the flexural strength is less than the ultimate soil bearing pressure and the soil bearing pressure at the shear strength of the footing. For reinforced footings, the structural section plastic hinge rotation limit will govern the deformation limit of the spring. For the Collapse Prevention performance level, when the flexural hinge exhausts its plastic rotation capacity for the out-of-plane footing condition, the residual footing capacity can be based upon the wall or grade beam thickness at the ultimate soil bearing pressure.

**Footing Shear Capacity.** The shear strength of the footing, either in- or out-of-plane, will control when the soil bearing pressure at the footing shear strength is less than the ultimate soil bearing pressure and the soil bearing pressure at the flexural strength of the footing. For the Collapse Prevention performance level, where shear failure in the out-of-plane footing condition occurs, the residual footing capacity can be based upon the wall or grade beam thickness at the ultimate soil bearing pressure.

For pile foundations, the pile cap structural section can usually be considered rigid, due to the span relative to its depth (span/depth ratio of approximately 2). Where the foundation is considered fixed in the horizontal translational direction, each pile can be modeled by using one vertical translational spring, assuming that the spring force-deformation characteristics will account for both the pile structural section and soil behavior. Where the flexibility in the horizontal translational direction is considered, the lateral stiffness based upon pile-soil interaction and passive soil resistance pressure (as discussed above) on the vertical face of the pile cap should also be included in the model. Where the pile structural section and the soil are modeled as one vertical spring, the following relative strengths should be considered:

- **Under pile uplift**, the cohesion or friction of the soil will control when the structural pile section capacity under combined bending and tension is greater than the cohesion or friction force on the surface of the pile. For new construction, soil cohesion or frictional resistance greater than the pile structural capacity should be avoided since pile structural damage under strong ground motions cannot be easily verified nor repaired. Where the pile connection to the pile cap has less strength than the pile combined bending plus tension capacity and the cohesion or frictional force on the pile, then the pile connection to the pile cap will control.

- **Under pile downward loading**, the cohesion or friction of the soil and/or end bearing capacity will control when the structural pile section capacity under combined bending plus compression is greater than the soil capacities. Again for new construction, the case where the structural pile capacity is less than the soil
capacity should be avoided. It is desirable that the yielding and energy dissipation occur in the soil rather than the structural pile section.

Piles can also be explicitly modeled as structural elements with multiple p-y and t-z soil springs attached to the pile along its longitudinal axis. This modeling technique separates the soil from the structural pile section behavior permitting a more detailed analysis and evaluation of failure modes. The pile structural section can then be modeled with nonlinear inelastic elements to account for plastic hinging in the pile under large lateral displacements. Such detailed pile-soil models are usually done outside of the global foundation-superstructure model or accomplished in two-dimensional global models only. Force-deformation curves for nonlinear inelastic fixed head piles with soil interaction under lateral loads may also be derived from a simplified limit state analysis procedure given in Blue Book Article 7.02.020 Design of Piles, Pile Caps and Grade Beams (SEAOC Seismology Committee 2008).

**Keywords**
- foundation modeling
- foundations
- free-field motion
- soil-structure interaction

**References**


ASCE, (2007), *Seismic rehabilitation of existing buildings, ASCE 41-06*, Structural Engineering Institute, Reston, VA.


**How To Cite This Publication**

In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:
**System Issues**

It is important to recognize the limit state of a steel SFRS (seismic force-resisting system), which may take the form of a moment frame, braced frame, or steel plate shear wall, so that the connections to the foundation are compatible with that system.

The initial yielding in a moment frame system often will be located at the base connection unless explicitly designed otherwise. On the other hand, only in special circumstances should the connection of a steel frame to the foundation be designed to yield, due to the difficulty of ensuring that ductile yielding behavior will occur in this usually complex connection. In general terms, the foundation connection of moment frames should either be capable of accommodating the required hinging rotations or be strong enough to induce yielding in the column.

The connection of an Ordinary Concentrically Braced Frame (OCBF) to the foundation should have a significant amount of overstrength, while the connection of a Special Concentrically Braced Frame (SCBF) to the foundation should develop the tensile strength of the brace. The foundation connection of an Eccentrically Braced Frame (EBF) should be designed for the larger of the amplified seismic force or the summation of the expected nominal shear strength of all the links above, similar to the column design per AISC 341, Section 15.8.

**Code Issues**

Sections 14.1.2 and 14.1.3 of ASCE 7-05 require steel structures to be designed in accordance with AISC 341. The connection of the superstructure to the foundation is discussed in the specifications and commentary of AISC 341. Anchor rods are to be designed and detailed per ACI 318-05, Appendix D, as modified by AISC 341, Section 8.5, and ASCE 7-05 Sections 14.2.2.17 and 14.2.2.18. AISC 341 also has a general discussion on column bases in the Commentary (Section C8.5) for Section 8.5, which is summarized below.

**Column Bases**

Column bases must have adequate strength to permit the expected ductile behavior for which the system is designed in order for the anticipated performance to be achieved.

Column bases are required to be designed for the same axial forces as those required for the members and connections framing into them. If the connections of the system are required to be designed for the amplified seismic loads or loads based on member strengths, the connection to the column base must also be designed for those loads.

System requirements are described in the AISC 341 Sections listed below.

**Moment Frame Systems**

* For Special Moment Frames (SMFs) per Section 9.2, and Intermediate Moment Frames (IMFs) per Section 10.2, beam-to-column joints and connections used in the seismic force-resisting system are required to sustain prescribed interstory drift angles and provide prescribed flexural and shear strengths related to the beam strengths. Thus it is implied that the Section 8.5 requirements can be met by showing that the frame-to-foundation connection is capable of sustaining the prescribed interstory drift angles without failure or be sufficiently strong to not yield. Note that IMFs are not permitted in Seismic Categories E and F, per ASCE 7-05.
• For Ordinary Moment Frames (OMFs), the connection to the foundation should be designed for equivalence with Section 11.2. OMFs are not permitted in Seismic Categories D, E, and F (see exceptions in ASCE 7-05).

• For Special Truss Moment Frames (STMFs), the connection to the foundation should be designed for equivalence with Section 12.4. STMFs are not permitted in Seismic Category F (see exceptions in ASCE 7-05).

**Braced Frame Systems**

• For SCBFs, the connection to the foundation should be designed for equivalence with Section 13.3 which states in part: The required tensile strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:
  
  o The expected yield strength, in tension, of the bracing member, determined as \( R_y F_y A_g \) (LRFD) or \( R_y F_y A_g / 1.5 \) (ASD), as appropriate.
  
  o The maximum load effect that can be transferred to the brace by the system as indicated by analysis.

• For OCBFs, the connection to the foundation should be designed for equivalence with Section 14.4. OCBFs are not permitted in Seismic Category F (see exceptions in ASCE 7-05).

• For EBFs, the connection to the foundation should be designed for equivalence with Section 15.1.

• For buckling-restrained braced frames (BRBFs), the connection to the foundation should be designed for equivalence with Sections 16.3a and 16.5b.

**Anchor rods**

To differentiate between steel-to-steel structural bolting applications and steel-to-concrete anchorage, AISC has adopted the terms “anchor bolt,” which applies only to the steel-to-steel case, and “anchor rod,” which applies to the steel-to-concrete case. The most common connections of steel braced frames to foundations involve anchor rods cast in the concrete. Headed anchor rods or threaded rods with heavy hex nuts are generally utilized to provide uplift resistance. The failure modes (limit states) of anchor rods are defined in ACI 318-05 Appendix D, Section D4.1. Whenever feasible, anchor rods should be sized or embedded sufficiently to preclude a brittle concrete pullout failure. The head of the anchor rod provides sufficient anchorage and the use of an additional plate washer does not add significantly to the anchorage.

ASTM A307 steel for anchor rods and A36 steel for threaded rods is readily available, as is A449 high-strength anchor rods (up to 120 ksi minimum tensile strength). F1554 is a specification that specifically covers anchor rods. ASTM F1554 rods may be hooked, headed, or threaded with nuts. Three strengths are available, and the specification covers the use of galvanized and weldable material.

There is extensive literature on designing anchor rods embedded in concrete to account for the combined effects of tension and shear. ACI 318-05, Appendix Section D.7 should be used to check the concrete for these combined effects. The shear force causes a bearing failure near the concrete surface and translates the shear load on the anchor rod into an effective tension load by shear friction (Shipp and Haninger 1983). However, these approaches do not take into account the detailing commonly utilized for braced frame anchorage, with oversize holes and the rod passing through either a grout bed or a grout pocket. ACI 318-05 does allow the use of built-up grout pads, with a reduction of 20 percent in the nominal shear strength of the anchor rods (Appendix D.6.1.3), although no supporting commentary is supplied. It is not recommended to use anchor rods for the transfer of large shear loads. The use of alternatives such as shear lugs or direct shear transfer are recommended, as discussed below.
If adequate edge distance is not available, for instance for a connection on top of a wall, rod tension may be resisted by using threaded reinforcing bars that extend deep enough into the confined region of the wall to lap with the wall reinforcing. For a high-strength application of this type, threaded bars (ASTM A722) may be utilized. A special case is where the anchor rods are expected to yield in tension. Adequate concrete embedment is required, but it is also required that the rods have a consistent cross section, that is, upset threads or continuously threaded over the strain length, and that they be debonded from the concrete for some length to provide an adequate strain length. Details that extend the anchor rods up the side of the column flange and provide a bearing bracket on the face of the flange, which engages the anchor rod nut, also will provide a strain length and are particularly effective for transferring large forces.

Holes for anchor rods in base plates larger than standard rod holes may need to be considered in the design if steel templates are not used to place the anchor rods. Maximum baseplate hole diameters and minimum plate washer sizes are indicated in Table 14-2 of AISC 360-05, Steel Construction Manual. A hardened washer will usually be adequate to transfer tension forces across a 3/8” hole, but a thicker plate washer will usually be required for larger gaps. Also, if shear is to be transferred directly through the rods, and base plate movement is to be prevented, then a welded plate washer is required.

Rod bending may need to be addressed if the base plate is installed above the concrete surface and the grout is unconfined. Since a round shape is an inefficient bending member, only relatively small shear forces can be transferred by rods. If the base plate and its grout bed are confined in concrete, then a bearing method may be utilized at the edges and below the bottom of the base plate. If adequate pretensioning of high-strength anchor rods is done then it may be possible to use shear friction at the bottom of the base plate. An alternative approach is to acknowledge that localized yielding in the anchor rods will occur and to limit the shear force to the rod such that the rod deformation will be less than the strain limit of the rod.

**Recommended Shear Transfer Mechanisms for High Shears**

AISC 341-05, Commentary, C8.5b, provides guidance on the design of high-shear transfer mechanisms.

Shear lugs welded to base plates are often utilized when a direct transfer of shear to the foundation concrete is needed. Where the base plate itself is embedded, direct bearing of the edge of the base plate is adequate for lower loads. The lug can be designed as a cantilever through the depth of the grout pocket as required to resist the shear by bearing on the concrete. At the base of the shear lug a shear plane crack can be hypothesized parallel to the base plate and the anchor rods or other reinforcing crossing that plane checked per the ACI 318 shear friction provisions. The shear lugs can also be detailed to enhance the bending rigidity of the base plate for resisting uplift loads. Disadvantages of shear lugs include possible interference with grade beam reinforcing bars and possible incomplete placement of grout in the vicinity of the lug. Uncertainty regarding the second point can be reduced by adding small holes in the base plate so that the grout is visible.

Shear studs welded to the bottom face of the base plate can substitute for anchor rods, if a grout pocket detail is used. This obviously eliminates the need for a weld washer and mitigates the grout placement issue associated with shear lugs. Shear studs may be designed utilizing PCI and AISC provisions.

An effective direct shear transfer to the slab is the use of steel shape or reinforcing bar drags welded directly to the brace gusset and/or base plate. An alternative approach is to provide a reinforced recess in the slab so that direct bearing of the column on concrete may be utilized for shear transfer. In either case, reinforcing dowels adequate to transfer shear between the slab and the foundation system are required.

**References**

Key Words
anchor bolts, anchor rods, foundation connections, steel column base

How to Cite This Publication
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s list of cited references, the reference should be listed as:

Adoption Status
The current standard for earthquake design of steel structures is AISC 341-05, the 2005 AISC (American Institute of Steel Construction) Seismic Provisions for Structural Steel Buildings (AISC 2005a). The 2005 AISC Seismic Provisions document was written to be consistent with ASCE 7-05.

ASCE 7-02, the 1997 UBC, and the 2001 CBC all cite older versions of this AISC document as reference standards:
- 1997 UBC sections 2210 and 2211 revise the 1992 provisions for Load and Resistance Factor Design (AISC1992), and sections 2212 and 2213 retain the historic provisions of the UBC for Allowable Stress Design.
- For structures regulated by local building officials, the 2001 CBC matches the 1997 UBC, except as modified by locally adopted amendments.
- The 2003 IBC (section 2205.2.2 and 2205.3.1) adopts the 2002 AISC provisions as its earthquake design standard for steel and composite construction.
- The 2006 IBC section 2205.2, and ASCE 7-05 section 14.1.1 adopt the 2005 AISC provisions as their earthquake design standards.

The SEAOC Seismology Committee recommends the 2005 AISC Seismic Provisions for use by California engineers and for adoption as an alternative standard by local building officials. Absent a local amendment referencing the 2002 or 2005 Seismic Provisions, an engineer may request a variance from the building official in order to use the current standard.

Local adoption. Some California jurisdictions, notably those participating in the Tri-Chapter Uniform Code Adoption and Interpretation Program (Tri-Chapter 2002) in the San Francisco Bay Area, and the City of Los Angeles, have amended the California Building Code to incorporate the 1997 AISC Seismic Provisions through Supplement No. 2 (AISC 1997; AISC 1999a; AISC 2000). Their amendments typically delete CBC Chapter 22 Division V and replace Division IV with slightly modified versions of AISC Seismic Provisions Parts I and III. In addition, these local amendments have modified CBC Table 16-N to make the seismic design parameters for various moment-resisting frame systems consistent with the 2003 NEHRP Provisions (BSSC 2004).

With revisions to Chapter 22A, the CBC has essentially made the same update of reference standards for buildings regulated by State of California agencies including the Office of Statewide Health Planning and Development, the Division of the State Architect, and the Building Standards Commission (ICC 2004a; 2004b).

Historical Background
The SEAOC Seismology Committee had the lead role in developing UBC seismic provisions for steel structures from the 1950s through the 1980s, and NEHRP led the more recent nationalization process. To support the NEHRP effort, the American Institute of Steel Construction began the development of a set of seismic design provisions specifically for steel buildings. The first edition was developed under the direction of Professor Egor Popov and published in 1992 (AISC 1992). The 1992 AISC Seismic Provisions were similar in scope and content to the UBC
provisions developed by the SEAOC Seismology Committee, but they were in Load and Resistance Factor Design (LRFD) rather than Allowable Stress Design (ASD) format.

Damage in the 1994 Northridge earthquake (SAC Joint Venture 2000i) prompted a significant effort to update seismic design provisions for steel structures, especially welded moment-resisting frames. The Federal Emergency Management Agency (FEMA) organized and funded a program to develop new guidelines for the design and construction of moment-resisting frame structures throughout the United States, as well as for the inspection, evaluation, repair, and upgrading of existing ones. The program was executed for FEMA by the SAC Joint Venture, a partnership of SEAOC, ATC, and CUREE. Late in 2001, FEMA published four volumes of recommended technical criteria numbered FEMA 350 through FEMA 353 (SAC Joint Venture 2000a-d). Accompanying them were six “State of the Art Reports” published as FEMA 355A through FEMA 355F that explain and support the recommendations (SAC Joint Venture 2000e-j). Reports from the more than sixty analytical and experimental studies performed by the SAC Joint Venture are available through ATC at www.atcouncil.org.

Following publication of the FEMA series of documents, the SEAOC Seismology Committee conducted a detailed review of FEMA 350 along with many of the background documents. The results of this review are presented in *Commentary and Recommendations on FEMA 350* by the SEAOC Seismology Committee (SEAOC Seismology Committee 2002).

As the FEMA project moved forward, so did the AISC Seismic Provisions, which were almost completely re-written in 1997, with additional major modifications in 1999 and late in 2000 (AISC 1997; 1999a; 2000). The changes reflected the post-Northridge research results. The 2002 Seismic Provisions (AISC 2002) incorporate the recommendations in FEMA 350 and FEMA 353 and many of the findings described in FEMA 355.

Two systems that were initially developed and incorporated into the 2003 NEHRP Provisions (BSSC 2004) are the Buckling Restrained Braced Frame (BRBF) and the Steel Plate Shear Wall (SPSW). Both of these systems are included in the 2005 AISC Seismic Provisions. The BRBF system, was originally developed in Japan, and has recently been used on a number of projects in the USA on the West Coast. This system relies on a brace element that is restrained from overall member buckling, thereby significantly increasing the energy dissipation of the system over that of a traditional CBF (concentrically braced frame) system. The seismic design provisions for the BRBF system were developed by a task force comprised of members from the SEAOC Seismology Committee and AISC TC9 – Seismic Design. As with eccentrically braced frame (EBF) systems, the provisions intend to ensure that the connections and other members in the BRBF system remain essentially elastic at the full capacity of the bracing elements. Connection design requirements recognize the fact that the braces are likely to be stronger in compression than tension. It should also be noted that because of the better energy dissipation characteristics of the bracing elements in BRBFs, the bracing configuration limitations are not as strict as those imposed on special concentrically braced frames (SCBFs).

The use of the SPSW system in buildings in highly seismic regions dates to the early 1970s. Renewed interest in this system developed in the 1990s in Canada as the result of a series of research projects at the University of British Columbia and the University of Alberta (Driver et al. 1997; Thorburn et al. 1983; Timler and Kulack 1983). The National Building Code of Canada has design provisions for this system based on the results of this research. Additional research on this system is presently being conducted at the University of California at Berkeley. The design provisions were initially developed by a BSSC TS6 – Structural Steel task committee based on both the Canadian and American research efforts. Like other systems, the other elements in the frame are designed to remain essentially elastic for the capacity of the thin web plates. Limitations on configuration, width-thickness ratios, and other details are provided to be consistent with the successful test results.

**The 1997 AISC Seismic Provisions**

In coordination with, and with significant input by the SEAOC Seismology Committee, the 1997 Seismic Provisions (AISC 1997) were completed as a joint effort of AISC and BSSC subcommittees. One benefit of this coordination is
that it allows the AISC provisions to presume, without duplication, the seismic design parameters $R$, the seismic response modification factor, and $C_d$, the deflection amplification factor, that are developed for the NEHRP Provisions and later adopted into ASCE 7 and the model codes. Another benefit is that it facilitates adoption. For example, the provisions were adopted by reference into the 1997 NEHRP Provisions (BSSC 1998) without modification.

Part I of the AISC provisions, in LRFD format, updated design requirements that apply to all structural systems (such as those regarding materials, welded and bolted joints, columns, and column splices) and added new requirements for specific seismic force-resisting systems. Part III paralleled Part I, but in ASD format. This part was included in the provisions to ease the transition from working stress to strength seismic design. Part II of the provisions covered composite steel and reinforced concrete structures.

The 1997 provisions incorporated many of the early findings of post-Northridge studies. Three new systems were introduced: Intermediate Moment Frames, Special Truss Moment Frames, and Special Concentrically Braced Frames. Quality assurance requirements for seismic systems were significantly expanded. Finally, Appendix S for the testing of steel moment-resisting connections was provided to assist engineers engaged in project-specific testing. This appendix was necessary because, in the wake of Northridge earthquake damage, the 1997 UBC (in section 2210, item 8) (ICBO 1994) called for “approved cyclic testing results.”

The 1997 AISC Seismic Provisions also:
- Added ASTM A913 steel as an acceptable material (Part I, section 2)
- Specified system-specific overstrength factors (Table I-4-1)
- Recognized variations between nominal yield strength and expected yield strength (Part 1, section 6.2), which is important where the provisions require comparison of member strengths to control the location of inelastic deformations
- Provided requirements for the design of bolted joints and specified that load may not be shared between welds and bolts in the same line of action (Part I, section 7.2)
- Required that all complete penetration welds in the seismic system be made with materials that have a required Charpy v-notch toughness of 20 ft-lbs at −20°F (Part I, section 7.3b)
- Required base material Charpy v-notch toughness of 20 ft-lbs at 70°F (Part I, section 6.3)
- Increased the design loads for column splices (Part I, section 8.3a)
- Required demonstration of moment connection inelastic rotation capacity with full-scale tests (Part I, section 9.2a).

Supplement No. 1. Recognizing the rapid and significant developments in post-Northridge research and practice, the AISC Specifications Committee committed to keep the Seismic Provisions as current as possible with frequent supplements. The first of these (AISC 1999a):
- Added ASTM A992 steel as an acceptable material for rolled shapes (Part I, section 2)
- Required all welds in the seismic force-resisting system to be made with notch tough materials as specified in the 1997 Provisions (Part 1, section 7.3b)
- Highlighted potential problems with low toughness materials in the “k-area” of rolled shapes (Part I, section 16)
- Recognized the test loading protocol developed by the SAC Joint Venture (Part I, section CS6)
- Clarified the configuration of gusset plate details in Special Concentrically Braced Frames (Part I, section C13.1).

Despite these updates, most California jurisdictions in 1999 were using the 1998 California Building Code, which was based on the 1997 UBC, which in turn had adopted only the 1992 AISC Seismic Provisions. One significant revision motivated by Northridge had been implemented as an emergency change in 1994 (ICBO 1994), but other information incorporated into the AISC Seismic Provisions would not filter down into the building code for several years. As noted above, the 2001 CBC, still in use as of mid-2006, still adopts the 1992 AISC Seismic Provisions with only a few modifications.
Supplement No. 2. The second supplement to the 1997 provisions (AISC 2000) attempted to incorporate many of the final recommendations generated by the SAC Joint Venture. Among other things, this supplement:

- Added requirements to avoid material discontinuities created by fabrication or erection errors, the placement of welded shear studs, or the attachment of other construction in the plastic hinging zone, all of which can lead to premature fracture (Part I, section 7.4)
- Changed the connection test acceptance criteria from inelastic rotation to interstory drift angle and modified the testing appendix (Part I, section 9.2a and Appendix S)
- Revised the requirements for panel zone shear strength in Special Moment Frames, such that excessively weak panel zones would be avoided (Part I, section 9.3a)
- Tightened the column width-thickness ratio and lateral bracing requirements where column inelasticity is possible, recognizing that limited column hinging can not be precluded for moment-resisting frames unless the columns are significantly stronger than the beams (Part I, section 9.4b)
- Redefined Intermediate Moment Frame systems to be more consistent with the tested connection system previously defined as part of Ordinary Moment Frames (Part I, section 10)
- Redefined Ordinary Moment Frame systems and further limited their use (Part I, section 11)
- Further limited the use of Ordinary Concentrically Braced Frames, reflecting their limited ductility capacity (Part I, section 14.2).

The 2002 AISC Seismic Provisions

In 2002, recognizing the breadth of changes made since 1997, the AISC Seismic Provisions were republished in their entirety. The 2002 Provisions (AISC 2002) incorporated the results of the SAC Joint Venture that had been published as FEMA 350 in 2000. In addition, the provisions were modified for consistency with ASCE 7-02. This would allow the document to be incorporated by reference into the IBC and NFPA (National Fire Protection Association) model codes, improving uniformity of practice and limiting the divergence between the two model code organizations.

For consistency with FEMA 350 and FEMA 353, the 2002 Seismic Provisions include new requirements for:

- Toughness of filler metals in certain welds (Part I, section 7.3)
- Moment frame column splices (Part I, sections 7.3, 8.4, and 9.9)
- Beam web-to-column connections in special moment frames (Part I, section 9.2a)
- Weld access holes in ordinary moment frames (Part I, section 11.2)
- Pre-qualification of moment connections, to be administered by an AISC committee established specifically for this purpose (Part I, Appendix P).

Other new requirements address:

- Chords and collectors, now subject to the provisions as seismic force resisting system components (Part I, Glossary and section 1)
- Member slenderness ratio, in coordination with the LRFD specification (Part I, section 8.2)
- Column base design demands (Part I, section 8.5)
- Lateral bracing requirements for special moment frame beams (Part I, section 9.8)
- Slenderness ratios in steel H-piles subjected to seismic demands (Part I, section 8.6).

The 2005 AISC Seismic Provisions

Consistent with the changes to the main design specification, the 2005 Seismic Provisions (AISC 2005a) combine ASD and LRFD into a single specification. As such, Part III in previous editions (which addressed ASD) of the Seismic Provisions has been absorbed into Part I. In addition to adding provisions for BRBF and SPSW systems, a number of significant technical modifications occurred. These include the following:

- Clarifying the scope of structures to be covered include “building-like non-building structures”
- Clarifying that all steel buildings designed with an $R$ factor of greater than 3 must comply with the Seismic Provisions (see discussion below)
Adding new requirements to delineate the expectations for structural design drawings and specifications, shop drawings, and erection drawings

Adding new ASTM (American Society for Testing and Materials, now ASTM International) materials specifications that are commonly used in the metal building industry

Adding R values for all materials to be used in determining susceptibility of connections to fracture failure modes

Relaxing the limitations of oversize holes in bolted joints

Defining a new term “Demand Critical Welds” with associated additional quality and toughness requirements; for each system, welds considered to be demand critical are defined

Defining a new term “Protected Zone” to ensure that regions of members expected to be subjected to significant inelastic strains are not disturbed by erection or fabrication discontinuities; for each system, the areas that are considered to be protected zones are defined

Extending the requirements on splices in columns that are not part of the seismic force-resisting system to all systems, not just moment frames

Improving the provisions related to the design of column bases

Making the stability bracing requirements more consistent throughout the document

Added references to the new AISC Standard on Moment Connection Prequalification (ANSI/AISC 358) as one means for SMF (Special Moment Frame), IMF (Intermediate Moment Frame) and EBF (Eccentrically Braced Frame) (link-to-column) connection acceptance; ANSI/AISC 358 is also adopted into ASCE 7-05 with Supplement No. 1 by reference

Decreasing the column splice shear capacity requirements for SMF systems

Increasing the stability bracing requirements for IMF systems

Clarifying that connections meeting the requirements for SMF or IMF systems are also acceptable for OMF (Ordinary Moment Frame) applications

Increasing the requirements on SCBF systems that employ high K/U,r ratio braces

Reducing the connection force demand on OCBF (Ordinary Concentrically Braced Frame) bracing to allow the use of the Amplified Seismic Load factor

Eliminating the requirement to design all members in OCBF systems for the Amplified Seismic Load; this was done in concert with a commensurate reduction in the R factor for this system in ASCE 7-05

Adding the special bracing configuration requirement checks for V and inverted V bracing configurations in OCBF systems

For BRBF systems, increasing the deformation that the system is required to accommodate from 1.5 to 2.0 times the Design Story Drift

Adding a strong column-weak beam check for frames in SPSW systems

Significantly improving the provisions related to quality assurance and quality control to address many of the issues identified in FEMA 353

Adding new requirements for welded joints based on FEMA 353 recommendations, including the following:

• Qualifications for inspection personnel
• Acceptable non-destructive testing procedures
• Intermixing of filler metals
• Acceptable limits of diffusible hydrogen in filler metal
• Maximum wind speeds for gas-shielded welding
• Limitations on maximum interpass temperatures
• Proper application of weld tabs
• Additional requirements for Demand Critical Welds related to welding processes, filler metal packaging and exposure, and the use of tack welds
• Made changes to Part II to be consistent with the modifications to Part I and changes to ACI 318.

2005 AISC Prequalified Moment Frame Connections
Appendix P in the 2002 AISC Seismic Provisions established the rules for pre-qualification of moment connections. Since the publication of the 2002 Provisions, AISC has established a Connection Pre-qualification Review Panel (CPRP) to implement the requirements of Appendix P in an ANSI (American National Standards Institute) consensus process. This committee, following the requirements of Appendix P, wrote AISC 358 in 2005 (AISC 2005c) that provides the pre-qualification of Reduced Beam Section (RBS) and some end-plate moment connections. This document is referenced in the 2005 AISC Seismic Specification as an acceptable means of qualifying moment connections. It is adopted by reference in ASCE 7-05, Supplement No. 1, and 2006 IBC.

These prequalified connections are largely based on the results of the SAC Joint Venture project. To better understand the performance and behavior of these connections, review FEMA 350 (SAC Joint Venture 2000a), FEMA 353 (SAC Joint Venture 2000d) and the supporting State of Art Reports (SAC Joint Venture 2000 e-j). More insight on connection performance may be gained from reading Commentary and Recommendations on FEMA 350 by the SEAOC Seismology Committee (SEAOC Seismology Committee 2002).

**Future Development**

Ten years after the initial research motivated by the 1994 Northridge and 1995 Kobe earthquakes, special efforts are still required to keep seismic design provisions for steel structures current. In the United States, the task will be shared by AISC, which will have primary responsibility for the development of the specific seismic design provisions, and BSSC, which will develop seismic design parameters for new systems.

The AISC TC 9 committee develops specific code provisions to be balloted through the main AISC Specifications committee. As a consensus activity accredited by ANSI, this balloting will result in a standard document eligible for adoption by any model building code.

The BSSC TS6 subcommittee works on the introduction of new seismic force-resisting systems and the proper and consistent application of design parameters. It publishes its recommendations in the NEHRP Provisions for review by the professional community and for provisional use, usually peer-reviewed. As experience is gained, draft provisions are refined for incorporation into the AISC Seismic Provisions, which are then adopted by building codes. Two steel systems included in the 2003 NEHRP Provisions (BSSC 2004) are Buckling-Restrained Braced Frames (section 8.6) and Special Steel Plate Walls (section 8.7).

**Welding.** Many of the design improvements incorporated into FEMA 350 and the 2002 AISC Seismic Provisions rely on proper welding. With its publication in 2000, FEMA 353 (SAC Joint Venture, 2000d) updated the state of practice, but its recommendations have not yet been incorporated into standards or codes. In mid-2006, the American Welding Society published a new document, AWS D1.8 (AWS 2006) that adopts the essential recommendations of FEMA 353. The welding provisions in the 2005 AISC Seismic Provisions, which incorporate much of the critical material in an appendix (Appendix W), were included as interim requirements pending publication of AWS D1.8.

**Specifications.** The AISC Seismic Provisions are intended to be used with the *AISC Specifications for Structural Steel Buildings* (AISC 2005b). The main specification applies to all structural steel elements. The seismic provisions add requirements for the design and fabrication of elements of the seismic force-resisting system. Generally, the seismic provisions defer to the main specification for the definition of member and connection capacities.

AISC introduced its first edition of *Load and Resistance Factor Design Specification for Structural Steel Buildings* in 1985, with updates in 1994 and 1999 (AISC 1994; 1999b). However, the design community has not fully embraced the LRFD method of design, and many engineers continue to use the Allowable Stress Design (ASD) Specification, last updated in 1989 (AISC, 1989). The continued use of two design methods has at times resulted in confusion and design inconsistency. The position of the Seismology Committee is that neither method is necessarily superior for the design of typical seismic force-resisting systems, since both methods are based primarily on elastic material response. For earthquake design, where substantial inelasticity is both expected and relied upon, special seismic design provisions are of greater significance than any differences between ASD and LRFD.
The 2005 AISC Specification for Structural Steel Buildings (AISC 2005b) incorporates in a single specification both ASD (which now stands for Allowable Strength Design) and LRFD in a “side-by-side” format. A nominal strength is given for each limit state, followed by an LRFD resistance factor and an ASD factor of safety. The required strengths come from ASCE 7 factored load combinations for either LRFD or ASD, depending on the method used. With these changes, the only difference between the LRFD and ASD methods of strength design will be on the required strength side of the equation. LRFD is based on factored load combinations ASCE 7-02 section 2.3.2, ASCE 7-05 section 2.3.2, or 1997 UBC section 1612.2), and ASD is based on service load combinations (ASCE 7-02 section 2.4.1, ASCE 7-05 section 2.4.1, or 1997 UBC section 1612.3).

References


Overview of Steel Design Requirements


SEAOC Seismology Committee (2002). *Commentary and Recommendations on FEMA 350*, January 2002, Structural Engineers Association of California, Sacramento, CA. (see [http://www.seaoc.org/Pages/committees/seismology_post.html](http://www.seaoc.org/Pages/committees/seismology_post.html))


**Keywords**
steel braced frame
steel plate shear wall
steel moment frame
steel seismic design
welding

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Background

The staggered truss system was developed at MIT in the 1960s (Scalzi 1971). Its arrangement of story-deep trusses in a staggered pattern allows large column-free areas and low floor-to-floor heights. With fewer columns than other steel framing systems, staggered truss frames can also offer faster fabrication and erection schedules and reduced foundation costs (Wexler and Lin 2003).

Most staggered truss systems are in areas of low seismic hazard. Because of the apparent benefits of the system, AISC, structural steel contractors, and others have expressed interest in using the system in California. The American Institute of Steel Construction (AISC) has published a design guide with a chapter on seismic applications (Wexler and Lin 2003). The following statement appears in its introduction:

One added benefit of the staggered-truss framing system is that it is highly efficient for resistance to the lateral loading caused by wind and earthquake. The stiffness of the system provides the desired drift control for wind and earthquake loadings. Moreover, the system can provide a significant amount of energy absorption capacity and ductile deformation capability for high-seismic applications.

In contrast to these assertions, the SEAOC Seismology Committee position is that the staggered truss system is not addressed as a seismic force-resisting system in ASCE 7-02 Table 9.5.2.2 or in ASCE 7-05 Table 12.2-1 and that it is an “undefined structural system” per 2001 CBC section 1629.9.2 and subsequent editions. Therefore, pending review of substantiating cyclic test data and analytical studies, the Committee recommends against use of the staggered truss system as a seismic force-resisting system in ASCE 7-02/05 Seismic Design Categories (SDCs) C through F and in 2001 CBC Seismic Zones 3 and 4. While SDCs C-F effectively cover all of California, the “substantiating test data” requirements of ASCE 7 and the 1997 UBC apply to all SDCs and Seismic Zones.

Description of the System

The staggered truss system is contemplated for buildings from 6 to 25 stories tall (Wexler and Lin 2003). Its benefits are most apparent in regular buildings with rectangular floor plans. The system consists of full story-deep trusses spanning the transverse direction of the building; truss spans are typically 60 feet. From one story to the next, the trusses are horizontally offset by one column bay (typically 20 to 30 feet) so that the truss locations are staggered up the height of the building. See Figure 1. The stagger is typically of a uniform dimension and symmetric in plan. Floor diaphragms are typically precast planks spanning from the bottom chord of one truss to the top chord of the adjacent truss. Exterior columns support the ends of the truss and provide frame columns for the lateral force-resisting system in the longitudinal direction of the building. To maximize the architectural benefits of the system, there are frequently no continuous interior columns.

Each truss acts as a braced frame in the transverse direction. A Vierendeel panel is often provided at the midspan of the truss to accommodate passageways. Under transverse seismic loads, the Vierendeel panel would be subject to high deformations (much like the similar panel in a special truss moment frame) and would therefore have to be designed to dissipate energy through flexural yielding. The trusses resist transverse shear, overturning forces, and interstory drift, and the floor diaphragm acts as a load path element between adjacent trusses. The longitudinal lateral force-resisting system is typically a perimeter moment frame or braced frame.
Response to Earthquake Loads
Acceptable earthquake performance of a staggered truss system will be limited by the following attributes:

- The lateral and gravity force-resisting systems of the building are one and the same. Every gravity-resisting truss and column is also integral to the transverse lateral system.

- Long transverse spans limit the ability of the system to redistribute gravity loads in the event of a column failure.

- The ground story is usually much more flexible than the floors above. Customarily, moment frames replace the staggered truss elements at the ground story.
Diaphragms are critical to the lateral load path of this structural system, transferring relatively high forces between vertical elements. This is especially true at lower stories, where the diaphragm and diaphragm-to-truss connections must transfer nearly the entire base shear from one story to the next.

This last point about the diaphragms might be unique to the staggered truss system. The floor diaphragms are required to participate in the lateral system as fully as the trusses and columns. Model earthquake design codes, however, assign parameters such as $R$, $C_d$, and $\Omega_0$ without thorough consideration of diaphragm ductility and modes of inelasticity. Use of design parameters from moment-resisting frames, braced frames, or special truss moment frames for a staggered truss system would be inappropriate.

The assertions by Wexler and Lin quoted above are apparently based on elastic analysis results and theoretical response estimates by Goel et al. (1973) and perhaps by an inappropriate extrapolation of inelastic behavior modes expected in special truss moment frames (Basha and Goel 1994). Additional information may be found in Scalzi (1971), Goel et. al. (1973), Gupta and Goel (1972), and Hanson and Berg (1974).

**Recommended Research**
The Seismology Committee is not aware of any recent completed testing of the staggered truss system for use as a seismic force-resisting system. AISC and others are investigating the feasibility of the staggered truss system for areas of high seismicity Testing and analysis are expected to focus on sources of inelasticity, diaphragms, and diaphragm-to-truss connections.

The development of eccentrically braced frames and special moment-resisting frames perhaps offers examples for proponents of the staggered truss to follow. Specifically, ASCE 7-02 section 9.5.2.2 and ASCE 7-05 section 12.2.1 give requirements for qualifying an “undefined” seismic-force-resisting system. The Seismology Committee expects that adequate testing and analysis will need to address at least the following design and performance issues:

- Identification of predictable inelastic mechanisms
- Design forces and deformations in yielding Vierendeel panels and adjacent truss members
- Design forces related to diaphragm-truss interaction, considering expected strength, stiffness, and ductility
- Force distribution and inelasticity in precast diaphragms and topping slabs under high in-plane forces
- Force distribution and inelasticity in diaphragms under vertical displacements related to truss deflections and link deformation
- Design of diaphragm-to-truss connections, considering cyclic loading and diaphragm or truss overstrength
- Column design forces and ductility demands, considering dynamic truss-column interaction and sharing of columns by lateral and transverse systems
- Vulnerability of the gravity system to failure of seismic-force-resisting members
- Effects of openings and discontinuities in highly loaded diaphragms
- Disproportionate effects of atypical and irregular building configuration
- Axial and flexural interaction in truss chords, diagonals, and connectors.

In addition, because the system’s load path involves an out-of-plane offset at every floor level, testing must consider the interaction of yielding (and possibly degrading) diaphragms, trusses, and connections, as opposed to just the behavior of individual components. Even the testing of an entire truss frame would not capture the essential aspect of shear transfer between adjacent frames.

**References**


**Keywords**
staggered truss frames

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Introduction

Buckling-Restrained Braced Frames (BRBFs) are a special class of concentrically braced frames (CBFs) in which overall brace buckling is precluded at expected force demands of the brace (Clark et al., 1999). Just as with other concentrically braced frames, the centerlines of BRBF members that meet at a joint intersect at a point (or with minor eccentricities) to form a complete vertical truss system that resists lateral forces. The behavioral qualities of BRBFs, such as braces with positive post-yield stiffness, lack of strength degradation, and large repeatable hysteretic loops, make them very attractive for use in high seismic zones (López 2001; Shuhaibar et al. 2002). Analytical and experimental studies of BRBFs (Lee et al. 2000; López et al. 2002; Sabelli 2001) have confirmed their desirable characteristics.

The main component of BRBFs is the brace, which is known as a Buckling-Restrained Brace (BRB). The BRB gets its name for its ability to resist compressive loads higher than its core plate axial buckling strength when considered as unbraced along its length. The idea behind BRBs is simple, namely, to provide a buckling-restraining mechanism separate from the load-resisting portion of the brace (the steel core) such that buckling of the core is limited to very small amplitudes. By limiting out-of-plane movement the core is able to yield in compression, and even sustain compressive strains well in excess of its yield strain.

Possible BRB types are too many to fully describe. López (2001) and Uang and Nakashima (2004) discuss different BRB types. In the US, proprietary BRBs used in construction projects as of mid 2003 included Unbonded Brace™ manufactured by Nippon Steel Corporation (http://www.unbondedbrace.com/), CoreBrace’s buckling-restrained braces manufactured by CoreBrace (http://www.corebrace.com/), and POWERCAT™ braces manufactured by PKM Steel (http://www.starseismic.net/).

Expected Behavior

Because of their efficiency in compression and their manufacture to required steel core areas, BRBs are normally not designed with inherent overstrength. At small interstory drifts (less than 0.50%) BRBs will experience axial yielding either in tension or in compression (Huang et al. 2000; Iwata et al. 2000; Uriz 2005). In contrast to the behavior of BRBFs, Special Concentrically Braced Frame (SCBF) and Ordinary Concentrically Braced Frame (OCBF) braces are expected to buckle under compressive forces from the design earthquake, leading to poorer hysteretic inelastic behavior and larger drifts than would result under pure axial yielding of the braces. After initial yielding, braces in BRBFs will dissipate energy and not experience strength degradation. As a result, even though a BRBF will have a smaller elastic stiffness than an SCBF, because the SCBF has a larger brace area due to local and global brace slenderness requirements, the inelastic drift of the BRBF will be comparable or lower than that corresponding to a SCBF (Sabelli et al. 2003). SCBFs utilizing braces made of Hollow Structural Sections (HSS) will experience even larger inelastic drifts than BRBFs because of the HSS susceptibility to fracture (Sabelli 2001). BRBs, on the other hand, are not likely to fracture when subjected to the inelastic cycles associated with the design earthquake. Sabelli et al. (2003) performed a statistical evaluation of BRBFs subjected to a suite of ground motions associated with mean and mean-plus-one-standard-deviation peak interstory drift ratios. They compared the results to those similarly obtained for SCBF (Sabelli, 2001) and Steel Moment Frame (SMF) model buildings (MacRae 1997). The researchers concluded that "the behavior of the frames with the buckling-restrained braces is comparable and often better than that associated with conventional concentric braced frames and moment frames."

BRBs have been shown to perform adequately to the high strains associated with code-allowed interstory drifts (López et al. 2002; Merritt et al. 2003a, 2003b; SIE 1999, 2001, 2003; Uriz 2005). Further, low-cycle fatigue tests performed on BRBs (Merritt et al. 2003a, 2003b; SIE 1999, 2003) have demonstrated the robustness of the BRB as a seismic component. Given that Sabelli (2001) showed that during a seismic event the sigma plus one standard
deviation cumulative plastic demand value of a BRB in a model building could be approximated as 140 and that most braces have been tested to several times that value (Iwata et al. 2000, Merritt et al. 2003a, 2003b; SIE 1999, 2001, 2003; Uriz 2005), SEAOC Seismology Committee is of the opinion that high-amplitude (approximately 1.0% axial strain), low-cycle fatigue is not a concern for the successful BRB concepts tested so far.

The response of the frame surrounding the BRB to the interstory drifts allowed by code is not as robust as that of the BRB. Gusset plates, columns, and beam-column connections exhibited deformation demands requiring ductile detailing (López et al. 2002). While the behavior exhibited by the frame surrounding the BRB can also be expected of frames surrounding HSS and W (wide flange) braces, (López et al. 2002), SEAOC Seismology cautions the design structural engineer to exercise care when detailing those components that have shown sensitivity to the interstory drifts allowed by code, or to limit the interstory drifts to less than code values where it cannot be determined what effects the gusset plate detailing will have.

**BRBF Performance and Braced Frame System Selection**

Structural engineers have selected BRBFs when an owner was interested in better-than-code performance in a braced frame building. As compared to SCBFs, BRBFs would not exhibit out-of-plane buckling of the braces, brace strength and stiffness degradation, and large flexural demands on chevron-configuration beams (Aiken and Kimura 2001). The same as Eccentrically Braced Frames (EBFs), BRBFs concentrate the majority of the energy dissipation in a system component intended for that role -- link beams for EBFs and braces for BRBFs. BRBFs are different from EBFs in that they possess an energy-dissipating element that does not resist gravity loads and is more easily repairable after an earthquake. While the energy dissipater in BRBFs, the brace, can endure displacement protocols in excess of those associated with the design earthquake (refer to the recorded cumulative plastic ductility demands in Merritt et al. 2003a, 2003b; SIE, 1999, 2001, 2003a; Uriz 2005), the energy dissipater in EBFs, the link beam, cannot endure displacement protocols in excess of those associated with the design earthquake (see experimental results in Arce 2002 and proposed revised displacement protocol in Richards and Uang 2003). Clearly, the performance of BRBFs stands out when compared with other conventional braced frame systems. And, since its hysteretic loops are as stable and large as those of SMFs, and its energy-dissipating element does not resist gravity loads, they are considered to be one of the best available conventional lateral force-resisting choices for steel systems.

While a standard code-designed BRBF with adequate details may perform better than other conventional code systems, the potential exists for BRBFs to be designed for higher-than-code-required performance. Wada et al. (1994) and Connor et al. (1997), and other researchers not listed here, describe procedures by which a backup moment frame can be used in conjunction with a BRBF and be designed to remain essentially elastic after the design earthquake so as to center the building. As of 2006, several hospital buildings in California have been designed and are under construction based upon an Immediate Occupancy performance level using buckling-restrained braced frames with moment-resisting beam-to-column connections acting as a secondary seismic force-resisting system. This article focuses on BRBFs designed for code-type expected behavior, and thus this topic of a combined BRBF-moment frame system is outside our scope.

**Past US Practice**

The concept of using a BRB within a building frame was refined in Japan during the 1980s. Uang and Nakashima (2004) estimated as of 2003 that there were about 250 buildings in Japan utilizing BRBFs. At that time, the design of BRBFs was not specifically codified in Japan. The design of BRBFs was not formally regulated in the US until ASCE 7-05 (ASCE 2005) and AISC-Seismic, ANSI/AISC 341-05, (AISC, 2005) were published. Before the 2005 referenced standards and starting in the fall of 1999, the seismology steel subcommittee of the Structural Engineers Association of Northern California (SEAONC) embarked on the task of developing a set of recommended provisions for design of BRBFs. The results of this task led to the SEAONC Recommended Provisions (SEAONC, 1999-2001). Earlier BRBF applications were characterized by structural engineers defining their own design methodology and having the peer reviewer concur with their approach. The first US application of a BRBF design involved the peer-reviewed design of a laboratory facility in Davis, California in 1999 (Clark et al. 1999). Also in
that year uniaxial testing of a BRB (SIE 1999) and presentation of a design procedure (Clark et al., 1999) took place. As all of the known projects are located in seismically-active areas, BRBFs were chosen for their seismic resistance properties, with about a quarter of them representing retrofit of existing buildings. Aiken and Kimura (2001) provide additional details on the past use of BRBFs in the United States.

**Design Issues**

As was described previously, BRB concepts and products are too many to list and describe in this article. In California, there are three commercially available BRB types – all proprietary. Several of the available BRB manufacturers have undergone a developmental testing phase where variations in their proprietary buckling-restraining mechanisms were studied as well as a qualifying testing phase whereby different brace sizes were tested to the requirements of the SEAONC Recommended Provisions or similar pre-published versions of AISC Seismic 2005 (Appendix T) BRB loading protocols. As a result, there are now several reports (Merritt et al. 2003a, 2003b, 2003c; SIE 1999, 2001, 2003) containing experimental data available to the design structural engineer. From such a compendium of experimental results, the design engineer can select the BRB type that complies with all the applicable requirements of the provisions and the strength and deformation demands of the actual building project. The Seismology Committee recommends that the only BRB systems the structural designer should select will have been tested for their full range of brace axial capacities, including short and long brace lengths as applicable, including connections for these requirements, or tested to AISC Seismic 2005 (Appendix T) requirements, or to requirements in excess of these. The International Code Council has published AC238 (ICC 2003), which also gives the authority having jurisdiction additional acceptance criteria for design and testing of BRB and BRBFs.

More published data (Black 2004; Iwata et al. 2000; López et al. 2002; Staker and Reaveley 2002; Uriz 2005) and unpublished experimental data exists. The design engineer is encouraged to contact brace manufacturers for obtaining data not referenced in this article.

Available to the structural engineer performing a BRBF design is design literature including the following: Clark et al. 1999; López 2001; López et al., 2002; Tremblay et al. 1999; Wada et al. 1994; Black 2004), and most notably the Steel Tips “Seismic Design of Buckling Restrained Braced Frames” (SSEC, 2004). Regardless of the reference material consulted and in addition to using the AISC 341-05 (AISC Seismic 2005) provisions, SEAOC Seismology recommends that structural engineers pay extra attention to the following design issues.

- **Acknowledge the range of yield stress in steel core material within a single material standard specification.** As described by López (2001), one of the characteristics of structural plate steel is that the actual yield stress measured can vary widely from the specified nominal value. Steel used in the fabrication of the steel core of buckling-restrained braces is no exception. BRBFs, as opposed to other types of concentric braced frames, are more sensitive to variations in yield stress of plate material when the sizing of the braces is governed by strength demands at code-level forces. AISC Seismic 2005, mirroring provisions for all other structural steel systems, require that brace sizes be chosen based on the minimum specified yield strength. However, the option is given in AISC Seismic 2005 to use the actual yield stress of the steel core for design based upon coupon tests. This will typically result in smaller core plate areas than using the minimum specified yield stress from the material standard. If such an option is taken the variability of yield stress must be limited in the BRB design specification and, in addition, a material testing program established to assure that the material yield stress, both minimum and maximum, assumed in the design is actually provided in the building. The current proprietary brace manufacturers are aware of the variability in the steel plate yield stress and should be consulted in establishing such yield stress limits for a project.

- **Beam-column joint fixity should be incorporated in analyses.** As demonstrated analytically by Richards (1986), experimentally by Gross (1990), and acknowledged in analytical studies by Sabelli (2001), the presence of large gusset plates at beam-column joints creates a de-facto rigid frame within the braced bay whether one intends this moment resistance or not. Once the frame beam is connected to the column with the required welds to
transfer the high collector loads, more restraint is added to the connection. Therefore, it is more accurate to acknowledge the joint fixity in analyses, element proportioning, ductile detailing, and overstrength considerations (López 2002). The need to acknowledge beam-column joint fixity is applicable to all concentrically braced frame systems and not exclusive to BRBFs. It is repeated here for emphasis because of the results of the experimental studies conducted on a class of BRB at UC Berkeley (Uriz, 2005). When sizing BRBs to resist the design base shear, it is acceptable to model beam-column joints with large gusset plates as 100% fixed and to size the braces for the load that they resist in proportion to their stiffnesses when compared to the frame. Further recommendations are given below for detailing beam-column-gusset plated joints.

• Maximum brace strains and interstory drift ratios should be kept below maximum successfully tested values. The applicability of the SEAONC Recommended Provisions limits the maximum brace strain allowed in a design to less than or equal to values from cyclic test specimen results. The requirement is repeated here for emphasis. For designs conducted using linear code analysis methods, AISC Seismic 2005 requires that the brace have the capability to resist local and overall buckling at 2.0 times the design story drift. This requirement should be interpreted to mean the deformation capacity necessary to accommodate the interstory drift of 2.0 times the design story drift without exhibiting strength deterioration. It is convenient and typical to quantify the BRB deformation capacity in terms of axial strain in the yielding portion of the core plate. Since frame geometry and deformations, and brace beam vertical deformations in the case of chevron configurations, can be significant, it is not appropriate to use interstory drift as the sole deformation criterion for brace qualification. AISC Seismic 2005 uses a factor of 2.0 on the design story drift as an approximation to the inelastic axial deformation the BRB will need to accommodate as a result of frame geometry and frame deformations. For the same design story drift and identical installed angle, short BRBs will have larger strain demands. Chevron configurations may further increase BRB strain demands caused by the vertical deflection of the brace beam due to unbalanced BRB compression/tension capacities. It is further emphasized here that brace strains are not the only variable determining applicability of test results to the design in an actual building project. There are other variables such as member sizes, brace angles, and brace end connections. It is possible for a successfully tested brace to be subjected to high axial strains at an angle and end connection conducive to favorable flexural strains that would not represent actual project conditions because of the angle, length, and end condition of the brace in the actual building project. Neither the Provisions nor applicable codes cover what happens to the frame surrounding the brace at 2.0 times the design story drift. Experimental data reveals (López et al. 2002 and Uriz 2005) that at such interstory drifts the successful behavior of any concentrically braced frame system is very sensitive to the detailing employed.

• Acknowledge the variability of the strength characteristics of the brace. This variability is considered by AISC Seismic 2005 as the “adjusted brace strength” and is essentially the overstrength of the brace. This strength variability accounts for \( \beta \), the difference between the maximum tension strength versus the maximum compression strength, and also accounts for \( \omega \), the strain hardening characteristics measured between the maximum tension strength at the target brace deformation (taken as two times the design story drift by AISC Seismic 2005) and the monotonic yield stress of the material. Cyclic testing of BRBs with a core plate yield stress of approximately 40 ksi has shown an increase in tension/compression capacity of approximately two times its initial yield strength when brace strains in the yielding core length approach or exceed 3%. When chevron brace configurations are used, the brace beam should be designed for the compression/tension strength variability, \( \beta \), of the BRB as required by AISC Seismic 2005, 16.4. AISC Seismic 2005 requires that, in addition to the normal code required checks, BRBF columns be checked for an axial load that is the lesser of the load from the adjusted brace strength or the maximum force that can be delivered by the system. This requirement is not the product of issues raised by recent analytical or experimental results. During the writing of the SEAONC Recommended Provisions, when comparing BRBF requirements to those of other braced frame systems, a majority of the SEAONC ad-hoc committee felt that frame columns played such an important role in the adequate performance of the system that it was necessary to include such requirements within the BRBF provisions. A minority of members unsuccessfully argued that frame columns play an important role in all braced frame systems, not only the BRBF, and that the proposed requirements are better suited for inclusion in
Section 8 of AISC-Seismic. It was argued that such system-specific requirements would give the wrong impression regarding the satisfactory performance of BRBFs in analytical studies (Lee et al. 2000; Sabelli 2001). The exception, to use the maximum force that can be delivered by the system, is intended to limit the axial load demand computed, but to truly take advantage of the exception engineers would be required to use nonlinear analysis methods. As a result, compliance with such a provision is more burdensome for engineers with no training or design fee to perform such nonlinear analysis tasks, and this situation would arise all because of poor design requirements for other braced frame systems.

- BRBF beams are designed for an axial load demand corresponding to the maximum force experienced by the braces connecting to it and also considering any collector member loads. This requirement of AISC Seismic 2005 is intended to ensure that at the brace forces associated with the design story drift (2.0 times the design story drift for code linear analyses), BRBF beams have enough axial design strength to preclude overall flexural buckling. At the design story drift, BRBF beams are expected to experience flexural yielding because of the concurrent action of beam-column joint rotation and high axial loads. Flexural yielding of BRBF beams should not affect the positive seismic behavior of BRBFs as long as the beams are compact per AISC-Seismic Table I-8-1 and are adequately braced, directly or indirectly, such that lateral torsional buckling is precluded.

- Beam-column joints of BRBF beams subjected to flexural yielding should incorporate details with rotational capacities in excess of the computed demand at the design story drift (2.0 times the design story drift for code linear analyses). SEAOC Seismology recommends that beam-column joints of all braced frame systems be detailed with ductile characteristics for their range of intended use. This may require the use of Complete Joint Penetration (CJP) welded connections of beam flange to the column, such as the WUF-W in FEMA 350 (FEMA 2000). However, the recommendation of the SEAOC Seismology Committee is found here and not elsewhere because the experimental results upon which the recommendation is based are for a BRBF (López et al. 2002, Uriz, 2005). Recent full-frame cyclic testing with BRBs (López 2002) has shown the detrimental effects of this particular connection on the brace beam at approximately 2% interstory drift. Brace beam fracture at the interior end of the gusset plate occurred at this interstory drift. In consideration of this effect, a compact gusset plate would be a better choice for BRBFs. Since the BRB does not buckle, the 2t distance between the end of the BRB and the beam-column restraint need not be provided in the gusset plate. Where it is expected that the BRBF will exceed a 2% story drift the Seismology Committee recommends further study into ductile detailing to mitigate this vulnerability.

- For a given direction under consideration, the story strengths of the BRBF system should be graduated up the height of the building, which will promote a more even distribution of yielding and energy dissipation. This will also mitigate the tendency for single-story mechanisms near the base to occur under strong ground motions, because yielding occurs at a very small drift, particularly if non-moment-resisting beam-to-column connections are used. Depending on the building configuration, the graduation of the story strengths could be accomplished by adjusting the strength of individual frames or by adding frames in lower levels of the building. Kinematic and isotropic strain hardening in BRBs has been shown to be significant during cyclic testing (Uriz 2005, SIE 1999, 2001; Black 2004). Therefore, a BRBF system appropriately proportioned up the height of the structure, even with “pin-” connected assemblies, would experience transfer of forces to, and brace yielding in, the upper stories. In contrast, the hysteretic behavior of conventional buckling braces (with slenderness ratio assumed to be roughly 50-70) produces a strength degradation under repeated cyclic loading (Uriz 2005), resulting in a propensity for story mechanism formation. This is further exacerbated by the discrete size of bracing members typically utilized in SCBF construction, where two stories may use identical brace sizes, further concentrating seismic loads on one floor.

**Quality Assurance and Quality Control**

After a particular BRB type is selected as the supplier in a project, the brace manufacturer should submit for approval its Quality Assurance plan. The brace manufacturer should also be required to fabricate the production
braces with the same or better quality control processes and procedures that were employed in the fabrication of the tested braces. In addition, the core plate steel specification (e.g. ASTM) and grade used for the tested braces should be the same as that being provided in the production braces. The main quality assurance issues that have surfaced are the need and type of special inspection of the production braces and the certification by the brace manufacturer of the quality of the production braces. There are good arguments on both sides of the issue for either providing periodic inspection or no inspection at all of the production braces during fabrication. A decision on the amount of special inspection required would have to be made on a project-by-project basis as permitted by the Authority Having Jurisdiction. Because all BRBs fabricated so far are proprietary, brace manufacturers have expressed concerns regarding outside parties learning their particular manufacturing techniques. Depending on the sophistication of the special inspection outfit, structural engineers may be better suited to perform periodic visits to the manufacturer’s plant to provide quality assurance on the owner’s behalf. This type of arrangement has occurred in certain projects with the approval of the Authority Having Jurisdiction. This service would be considered outside of the normal scope of structural engineering services provided to projects. Brace manufacturers and structural engineers agree that the production braces must attain at a minimum the same quality as the successfully tested braces. In some instances, owners have required a statement to that effect certified by an officer of the brace manufacturing company.

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ASCE (2005). ASCE 7-05, Minimum design loads for buildings and other structures, American Society of Civil Engineers, Reston, VA.


**Keywords**

braced frames, buckling-restrained braced frames, concentrically braced frames, eccentrically braced frames

**How to Cite This Publication**

In the writer’s text, the article should be cited as:
(SEAOC Seismology Committee 2008)

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### Background

A Truss Moment Frame (TMF) is a building framing system that is used for relatively long bay widths. This framing system provides higher lateral stiffness with relatively less weight as compared to moment framing systems with solid beams. Previous editions of the Uniform Building Codes allowed the use of trusses as a Special Moment-Resisting Frame (SMRF) as indicated in Section 2211.7.6, which stated “Trusses may be used as horizontal members in SMRF if the sum of the truss seismic force flexural strength exceeds the sum of the column seismic force flexural strength immediately above and below the truss by a factor of at least 1.25.” In other words, this allowed the strong truss-weak column concept in seismic zones.

During the 1985 Mexico Earthquake, the Pino Suarez complex of tall buildings suffered extensive damage in the form of yielding and buckling in the columns and truss members. One building 21-stories in height collapsed onto an adjacent 14-story building in the complex. That experience pointed out the need to re-evaluate the provisions in the UBC. To better understand the seismic response of truss moment frames, research was conducted at the University of Michigan in the late 1980s (Itani and Goel 1991). The results of this research showed that the vertical members of the truss moment frame significantly affect lateral strength and stiffness. Buckling of these vertical members will cause a sudden loss of lateral strength and stiffness and will lead to undesirable seismic response. Those early studies eventually resulted in the development of formal design specifications for Special Truss Moment Frame (STMF) systems.

The STMF is a seismic load-resisting system that consists of trusses as horizontal members and specially designed segments that are expected to withstand large cyclic deformation during seismic events. This system allows detailing for controlled damage in the special segments of open web trusses. During lateral loading, the truss will be subjected to constant shear and varying axial forces in the chord members. The maximum axial forces in the chord members occur near the ends of the truss, while the minimum forces occur in the middle zone of the truss. Therefore, the special segment is located in the middle of the truss to minimize the adverse effect of the axial forces in the chord members. The shear in the special segment is resisted through axial forces in the diagonal members and flexural shear in the chord members. If the diagonal members are not present in the special segment, then the entire seismic shear is resisted by flexural shear of the top and bottom chord members.

Experimental and analytical investigation in the late 1980s through the 1990s (Itani and Goel 1991; Basha and Goel 1994) led to the development of the STMF seismic design criteria. Building seismic specifications and codes adapted these criteria as early as in the 1997 UBC (ICBO 1997), and subsequent codes and standards have included similar seismic provisions (ICC 2003, 2006; AISC 1997, 2002, 2005). Several buildings in seismic zones have been designed and constructed (Dusicka et al. 2002). The basic concept of the STMF is to design a special segment inside the truss as a fuse to dissipate the input energy in a ductile manner, while the other members outside the special segment stay in the elastic range. The Vierendeel special segment in an STMF has the advantage that it can concentrate the inelastic behavior in the truss chords at the ends of the special segment. This STMF configuration is also preferred by many structural engineers because ducts and other nonstructural components can pass through this opening. Under lateral loading, the ends of the special segment are subjected to double curvature. The flexural moments add to produce shear in the special segment. The section at the ends of the special segment will be subjected to significant flexural yielding, and thus they are required to sustain large cyclic strains.

Double-angle sections were originally used for the chord members, and experimental studies showed the ability of these sections to sustain large inelastic strains within the special segment without early fracture or strength degradation, if they were designed according to the seismic provisions of the American Institute for Steel Construction (AISC). In order for the double angles to sustain the large inelastic rotations at the ends of the special segment, the gap between the angles was infilled solid with a vertical plate. However, the moment-of-inertia of the
double-angle sections limits the lateral drift resistance of the system, which requires a large column section to satisfy the code seismic drift limitation, particularly for mid-rise and tall multistory buildings and critical facilities such as hospitals and schools. Therefore, heavy chord member sections such as a W-shape or double channels can be used in the STMF under those demands. Recent experimental investigations (Chao and Goel 2006) has shown that the double-channel section possesses significant inelastic rotation capacity if it is seismically detailed.

Lateral Response of STMFs with Special Vierendeel Segment
The inelastic lateral response characteristics of STMFs can be observed effectively by means of a push-over analysis. For this purpose a 9-story, 5-bay STMF has been designed according to the recently proposed performance-based plastic design procedure of Chao and Goel (2006). The pushover response of this STMF is shown in Figure 1, with the various stages of hinge formation. It is noted that these observations conform to the primary design objective that all of the structural elements outside the special segments are to remain linear and elastic, and that the seismic input energy is dissipated due to the yielding of the chord members in the special segments. However, generally the yielding at the base of the first story columns is inevitable, which is deemed acceptable damage.

Figure 1. Pushover Response of a 9-story STMF

Figure 2 demonstrates the lateral deformation characteristics and desired hinge formation pattern of an STMF system for a 3-story, 3-bay building. Significant forces may develop at the truss-to-column connections.

Figure 2. Lateral Deformation and Hinge Formation of a 3-story STMF
Design of Special Truss Moment Frames

The current AISC Seismic Provisions for Structural Steel Buildings (AISC 2002) provides design considerations and specifications in its Section 12. A series of geometrical limits are imposed to ensure reliable system performance based on available experimental and analytical studies. Accordingly,

- STMFs are limited to span lengths of 65 ft and overall depth of 6 ft;
- Special segments shall be located between the quarter points of the truss span, and length of the special segments are to be between 10-50% of the truss span length;
- The length-to-depth ratio of any panel that is designed as part of the special segment shall be between 0.67 to 1.50;
- Special segments can be of either Vierendeel or X-braced panels; combination of the two is strictly not allowed;
- Chord and diagonal web members shall meet the requirements of AISC Table I-8-1 for the width-thickness ratio for compression elements.

In general, the design of an STMF starts with the determination of required vertical shear associated with the special segments due to the appropriate load combinations. The identical top and bottom chord members of the special segments are then designed for the governing vertical shear. As per AISC Section 12.3, the available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chords members through flexure, and the shear strength corresponding to the available tensile strength. The special segments are designated as “protected zones” and therefore must comply with the requirements of AISC Section 7.4.

Members adjacent to the Vierendeel special segment should be modeled as beam-column elements considering combined bending and axial forces. In selecting the special segment length, short special segments will increase the rotational ductility demands; longer special segments will reduce the lateral stiffness of the structure. There should be no heavy concentrated gravity loads within the special segment, which would change the plastic hinge locations assumed at the ends of the special segment.

Members outside the special segment are required to remain elastic and are designed for forces generated by the fully yielded and strain-hardened chords of the special segment ($V_{ne}$ in Equation 12-1 of AISC) plus the corresponding gravity loads. The required shear and flexural strength (the available strength from the special segment) shall not exceed the corresponding design strengths of the chords and web verticals outside of the special segment. Columns are also to be designed to resist the available shear strength of the special segment ($M_{ICE} \approx \Sigma V_p L/2$ where $L$ is the span length of the truss), thus assuring a strong column-weak beam system (Goel et al, 1998). Recently, a more accurate representation of the expected vertical shear strength has been proposed by Chao and Goel (2006), who also provide a performance-based plastic design procedure that targets a desired damage mechanism and attempts to balance the seismic energy demand with the hysteretic energy dissipation.

Another important consideration in the design and construction of STMFs is the lateral bracing requirements, found in AISC Section 12.6. Top and bottom chord members are required to be laterally braced at the ends of the special segments. In addition, lateral bracing along the entire length of the trusses is required at intervals not to exceed $L_p$ as defined in Chapter F of the AISC Specifications for Structural Steel Buildings (ANSI/AISC 360-05). However, the use of $L_p$ for the determination of the lateral bracing requirements outside the special segments is currently being questioned, and on-going research is expected to address this issue.

Current and Future Research

The built-up double-channel section for chord members in the special segment have not been yet been cyclically tested in a full truss subassemblage configuration as have the built-up double angle chord members. Component built-up double-channel cyclic tests have shown promise of ductile behavior (Chao and Goel 2006) and future full truss subassemblage testing should substantiate the section’s performance in the STMF system.
Recent studies have demonstrated potential alternative detailing of STMFs with energy-dissipating devices (EDD) that exploit the large deformations inside the special segments. This can be achieved by introducing real pins at the ends of the top and bottom chord members as depicted in Figure 3 (Pekcan and Itani 2007).

![Figure 3. Pin-ended Special Vierendeel Segment with Energy-Dissipating Device](image)

This alternative detailing and use of EDDs in STMFs lends itself to Damage Avoidance Design, which makes the elastic analysis procedures truly possible. In general, forces in the load-bearing structural elements and truss members are significantly reduced by a favorable introduction of moment releases in the special segments. Furthermore, with the introduced reliable and improved energy dissipation (damping) capability, the seismic demand is significantly reduced, allowing the selection of lighter structural elements. Another innovative variation of the concept of special truss moment frames was introduced in a design procedure by Dusicka et al. (2002), which combines fuses with the special segments.

Future studies are expected to address primarily the current interest by the academic and practicing communities in seismically resistant and economical systems with predictable response characteristics aligned with a performance-based seismic design methodology. Clearly, the STMF system is a very promising candidate to realize this objective, though the current state-of-the-knowledge lacks findings from large-scale experimental studies. In this respect, full-scale or near full-scale testing of STMF subassemblies or complete structural systems is a critical research area.

**References**


**Keywords**

energy-dissipating devices
special truss moment frame (STMF)
truss moment frame
Vierendeel truss

**How To Cite This Publication**

In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2008)

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Introduction

Concentrically braced frames are vertical truss systems that resist lateral loads in the elastic range primarily through axial forces in members. Members intersect at a point or with small eccentricities that are not a source of inelastic deformation. In the inelastic range, braced frames may involve the flexure of frame members, but the inelastic drift is expected to be mainly a result of brace axial deformation, except in certain configurations that are not recommended.

Until the 1994 UBC, concentrically braced frames had been treated by codes as essentially elastic truss systems. Post-elastic behavior was only considered in prescribing a reduction in calculated brace strengths, which resulted in raising the elastic force capacity of these systems. Subsequent research carried out at the University of Michigan showed that these systems, if they had careful proportioning of members and detailing of connections, could perform in a ductile manner (Astaneh et al. 1985; Hassan and Goel 1991; Goel 1992). These more ductile braced frame systems can achieve tri-linear hysteretic behavior, with the three ranges of behavior being the elastic, post-buckling, and tensile yielding ranges (AISC 2005; Bruneau et al., 1998).

The 1994 UBC acknowledged these findings by establishing two categories of concentrically braced frame systems: Ordinary Concentric Braced Frames (OCBFs), and Special Concentric Braced Frames (SCBFs). The code distinguished between OCBFs and SCBFs through design forces and detailing requirements. OCBFs were designed for large base shears with the expectation of low ductility demands. SCBFs had lower required base-shear capacity and were treated as ductile systems that had to accommodate cyclic excitations into the post-buckling range. The distinction between OCBFs and SCBFs made in the 1997 UBC is somewhat less clear. The force demand for OCBFs remains unchanged, but in the LRFD provisions some ductile detailing requirements were added. The requirements for SCBFs remained unchanged. The differences between the Ordinary and Special types are therefore limited to: slenderness limits, OCBF brace capacity reduction, brace compactness and stitch requirements, permissible configurations, column requirements, and the waiving of certain requirements for one- and two-story OCBFs. General summaries of the most recent code changes regarding OCBF and SCBF systems are shown respectively in Table 1 and Table 2.

The 2005 AISC seismic provisions (AISC, 2005) make a more rational distinction between the two systems. OCBFs are expected to have a higher elastic force capacity (because of the higher design base shear and the prescribed reduction in calculated brace capacity) and to accommodate cyclic buckling of braces in the connection design. Excursions into the tensile yielding range need not be considered. SCBFs are expected to achieve trilinear hysteretic behavior by accommodating cyclic brace buckling and withstanding forces corresponding to the yielding capacity of the braces. The force level corresponding to the yield mechanism determines the maximum forces that elements of the system, such as the connections, are required to resist. As capacity design is used for SCBFs, AISC Seismic 2005 uses an overstrength factor ($R_y$) to account for expected yield strength and strain hardening. SEAOC recommends that designers follow the provisions of AISC-2005 and design SCBFs for trilinear hysteresis.

SCBF requirements for braces are intended to prevent undesirable modes of brace behavior. Analytical studies on bracing systems designed in strict accordance with earlier code requirements predicted brace failures without the development of significant energy dissipation (Tang and Goel 1987; Hassan and Goel, 1991). Brace failures occurred most often at plastic hinges (concentrated areas of curvature and inelastic strain susceptible to local buckling due to lack of compactness); plastic hinges in buckled braces occur at the ends of a brace and at the brace midspan. Analytical models of bracing systems that were designed to ensure stable ductile behavior exhibited full and stable hysteresis without fracture when subjected to the same ground motion records as the previous concentrically braced frame designs. Similar results were observed in full-scale tests by Yang and Mahin (2005), Uriz (2005), Wallace and Krawinkler (1985), and Tang and Goel (1989).
Table 1. Recent code revisions for OCBF systems.

<table>
<thead>
<tr>
<th></th>
<th>AISC SEISMIC 341-02</th>
<th>AISC SEISMIC 341-05</th>
<th>UBC 1997</th>
<th>IBC 2006</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Member and connection demands (non-brace connections) shall include Amplified Seismic Load (See UBC 1997)</td>
<td>• No Amplified Seismic Load demands, but reduced R-value (See IBC 2006).</td>
<td>• R = 5.6; Ω, = 2.2</td>
<td>• R = 3.25; Ωo = 2</td>
<td></td>
</tr>
<tr>
<td>• Braces with KL/r ≥ 4.23 E/Fy shall not be used in V or inverted-V configurations.</td>
<td>• Braces in K, V, or inverted-V configurations must have KL/r ≤ 4 E/Fy .</td>
<td>• Only allowed in “Low Buildings.”</td>
<td>• Steel Design per applicable codes, including AISC 341-05 and AISC 360-05.</td>
<td></td>
</tr>
<tr>
<td>• Brace connection demand = R_y F_y A_g of brace.</td>
<td>• For K, V, and inverted-V configurations, beams (columns for K) must be designed for unbalanced brace forces, R_y F_y A_g vs. 0.3P_n and full gravity loads.</td>
<td>• Must use Amplified Seismic Load: 0.4<em>R</em>E.</td>
<td>• No Amplified Seismic Load.</td>
<td></td>
</tr>
<tr>
<td>• OCBF above Seismic Isolation System: KL/r ≤ 4 E/Fy ; no K frames; continuous beams between columns for V and inverted-V configurations.</td>
<td>• Brace connection demand = R_y F_y A_g; except for the bolt-slip limit state, or if max. system capacity or Amplified Seismic Load demands are used.</td>
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Past observations of performance in earthquakes have revealed that some concentrically braced frames have not achieved ductile yielding due to one of several nonductile behaviors. The nonductile modes can be characterized as problematic brace modes, such as fracture, local buckling, and stitch failure; and problematic connection modes, such as failure in tension, the inability to accommodate brace buckling, and stability problems. Additionally, certain bracing configurations (chevron bracing and K-bracing) are known to be problematic. More information on past performance of braced steel frames can be found in Kato et. al. (1980), Hanson and Martin (1987), Osteraas and Krawinkler (1989), Kelly et al. 2000), Bonneville and Bartoletti (1996), WJE (1998), Naeim, (1997, 1998), Architectural Institute of Japan, (1995), Tremblay et. al (1995).

Though the OCBF category has been preserved in current codes, its use has been severely restricted. OCBF’s have almost none of the proportioning rules that apply to SCBF’s, but they are designed for high base shears. All bracing connections should be designed for the expected tension strength of the brace. OCBF’s are not recommended for seismic design, as their ductility and seismic performance are not well controlled. They may be appropriate for minor structures such as penthouses where they can reliably be designed for the maximum accelerations that the structure below can transmit. By preventing limiting nonductile modes of behavior, SCBF provisions are intended to lead to systems that can develop trilinear hysteresis and significant system ductility. When properly designed and detailed, braced frames can sustain cycles of large inelastic drift without brittle failures. Proper design of CBF systems requires consideration of the geometric configuration, element proportioning, connection detailing and analytical modeling.
Table 2. Recent code revisions for SCBF systems.

<table>
<thead>
<tr>
<th>AISC SEISMIC 341-02</th>
<th>AISC SEISMIC 341-05</th>
<th>UBC 1997</th>
<th>IBC 2006</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Braces shall have KL/r ≤ 5.87 $\sqrt{E/Fy}$</td>
<td>• Braces shall have KL/r ≤ 4 $\sqrt{E/Fy}$, unless columns have available strength for brace capacities, $R_cF_yA_g$.</td>
<td>• R = 6.4; $\Omega_0 = 2.2$ (Uses exact provisions of AISC Seismic 341-92, as follows)</td>
<td>• R = 6; $\Omega_0 = 2$</td>
</tr>
<tr>
<td>• Width-thickness ratios per table I-8-1, and AISC 360-99, table B5.1.</td>
<td>• Width-thickness ratios per table I-8-1 (no changes), and AISC 360-02, table B4.1 (changes to numerous section types, including HSS).</td>
<td>• Braces shall have L/r ≤ 720 / $\sqrt{Fy}$ .</td>
<td>• Steel Design per applicable codes, including AISC 341-05 and AISC 360-05.</td>
</tr>
<tr>
<td>• Alternative requirement of gusset plate to accommodate inelastic post-buckling rotation (no guidelines given as to how).</td>
<td>• Alternative requirement of gusset plate to accommodate inelastic post-buckling deformations (typical condition detailed in commentary).</td>
<td>• Brace compressive strength shall not exceed 0.8$\phi P_n$.</td>
<td></td>
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<tr>
<td>• Beam bracing axial demand (at V and inverted-V config.’s) shall satisfy 2% of $F_yb_ft$.</td>
<td>• Beam bracing (at V and inverted-V config.’s) must meet max. spacing criteria, and axial demand shall satisfy $P_{nc} = 0.02M_C/h_o$, where $M_C$ is the required flexural strength, $C_\alpha$ is a curvature factor, and $h_o$ is the distance between flange centroids.</td>
<td>• Limit net section area to: $A_L/A_g \geq (1.2\alpha P_u^*)/\phi P_n$, where $P_u = brace$ axial demand, $\alpha = fraction of demand across net section, and $\phi P_n = brace$ tension capacity.</td>
<td></td>
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<tr>
<td>• Column splices shall develop nominal shear strength of smaller connecting member.</td>
<td>• Exception for penthouses, ones story structures, and top stories of buildings removed for special beam design requirements for V and inverted-V config.’s.</td>
<td>• Gusset plates required to have a 2$\theta_{gp}$ offset of brace from the theoretical line of bending.</td>
<td>• No unbalanced brace force design requirement for V and inverted-V braced frame beams.</td>
</tr>
<tr>
<td>• No K brace systems allowed.</td>
<td>• Column splices shall develop shear strength, $\Sigma M_{pc}/H$, where $\Sigma M_{pc}$ is the sum of nominal plastic flexural column strengths.</td>
<td>• Beam bracing axial demand (at V and inverted-V config.’s) shall satisfy 1.5% of $F_yb_ft$.</td>
<td>• Beam bracing (at V and inverted-V config.’s)</td>
</tr>
</tbody>
</table>
CBF Configurations
Figure 1 shows common concentric braced frame configurations.

Figure 1. Typical braced frame configurations: (a) diagonal bracing; (b) cross or X bracing; (c) chevron bracing; (d) K-bracing, (e) two-story X bracing, and (e) “zipper” braced frame

Chevron Bracing. Traditional chevron-braced (V- and inverted-V-braced) frames have been shown to have undesirable post-buckling behavior characterized by beam flexure rather than truss action (Figure 2). In the post-buckling range the force in the buckled brace diminishes with additional deformation, and the vertical components of the compression brace and tension brace forces no longer balance; the beam must then resist the unbalanced component. This post-buckling behavior results in a great reduction in the system capacity and in negative system stiffness (Khatib et al., 1988). Thus, traditional chevron-braced frames have not been able to achieve trilinear hysteretic behavior and have been much more susceptible to large displacements and corresponding P-delta effects than have other braced frame configurations.

Figure 2. Expected deflected lateral shape for (a) a strong and stiff beam, and for (b) a beam not designed to carry unbalanced load.

SCBF provisions for chevron-braced frames require a strong beam capable of withstanding unbalanced forces at the intersection of the braces, thereby permitting trilinear hysteretic behavior. Typical bracing members demonstrate a residual post-buckling compressive strength of 30 to 50 percent of the initial buckling strength (Hassan and Goel, 1991). The use of 30 percent of the buckling strength is allowed in the reduction of the calculated beam flexural demand imposed by the tensile yielding of the adjacent brace. The flexural demand should be combined with appropriate gravity loads. The beam strength can include composite slab effects, where appropriate, or the truss behavior of braces and beams at other levels, where the brace configuration permits transfer of the unbalanced force (the two-story X and the zipper configurations, shown in Figure 1). Though not explicitly allowed by the IBC, calculation of beam capacity in this manner is not disallowed and SEAOC Seismology Committee considers this design method consistent with the highest performance goals of SCBFs.

Even with the very strong beams that are typically required for this configuration, beams with sufficient stiffness to prevent negative post-buckling system stiffness cannot be provided for most building cases (Khatib et al. 1988). Remennikov and Walpole (1998) have shown that the system may still be susceptible to large displacements and P-delta effects. Chevron bracing is therefore not recommended except where special configurations are used so that the vertical unbalanced force can be resisted by truss action of braces and beams at other levels. Lateral support of both beam flanges at the brace-to-beam intersection is necessary in order to prevent lateral torsional buckling of the beam due to post-buckling moment demands from the brace (Kim and Goel 1992). Beam-to-brace connections in chevron-braced frames should comply with all strength and detailing requirements, as discussed under “Bracing Connections” below.
Two-story X Bracing and Zipper Bracing. Khatib et al. (1988) studied the two-story X and zipper variants of the traditional chevron-braced configuration. These configurations can achieve trilinear hysteresis and positive post-buckling stiffness with beams not specifically governed by unbalanced load. For those configurations the beam strength requirement is unnecessary and sometimes even detrimental (Sabelli et al. 1998). Properly proportioned variants of the chevron-braced configuration can lead to much greater system ductility. For these variants, beam hinge formation is part of the ultimate yield mechanism, and compact section criteria should be applied. For a more complete treatment of design procedures for zipper-braced frames, see Tremblay (2003). Two-story X frames designed with the same members above and below the intersected beam are likely to experience a concentration of damage and interstory drift at the lower level (Khatib et al. 1988). It is recommended that braces be proportioned to the demand calculated by a linear analysis or proportioned to avoid a single-story mechanism using a nonlinear analysis. The requirements in AISC-Seismic 2005 provide for a minimum strength of the beams to support gravity loads in the event of loss of brace capacities.

K-Bracing. K-bracing is prohibited for seismic application because columns that are subjected to unbalanced lateral forces from the braces in the post-buckling range are susceptible to buckling. Similar to the unbalanced loads on the beam in chevron bracing, columns would be expected to react to very large unbalanced loads, but without the support of a slab. As such, the large unbalanced loads are likely to cause very large in-plane bending of the columns, and may create a gravity load-path instability.

Lateral Force Distribution
For most structures, the vertical lateral force distribution is prescribed in ASCE 7-05 in accordance with the equivalent lateral force procedure. This procedure is predominantly based on first mode behavior, which is generally an acceptable assumption for structures with relatively regular story heights and regular distribution of mass. Alternatively, if a dynamic analysis is performed, the story force distribution is based on modal mass participation. One of the primary concerns for design of any lateral force-resisting system is to prevent a weak story mechanism from occurring. A weak story is predisposed to accumulate large inelastic deformations that may cause the collapse of an entire story. Absent the weak story vulnerability, inelastic deformations could have been distributed over several levels with considerable reduction in inelastic demands. Concentrated weak story behavior not only causes large inelastic deformations at a single story, but also produces considerably less energy dissipation than distributing the deformation over multiple floors. ASCE 7-05 also requires that for any given story the lateral capacity must be at least 65% of the floor above it. For SCBFs, the AISC Seismic Provisions (AISC 2005) further require that at any given brace frame line, 30% to 70% of the braces are oriented such that they are in tension. The flip side to that provision limits requires 30% to 70% of braces in any line to be in compression to prevent the accumulation of inelastic drifts in either direction. This accumulation of inelastic deformations is from the ultimate post-buckling capacity for braces oriented in each direction and the energy required to produce inelastic drift. In tension the braces may inelastically strain harden, thereby maintaining or slightly increasing their capacity, whereas in compression, the remaining buckling capacity is significantly reduced. An exception to this provision is provided if the system is designed for the amplified seismic forces, such that more energy is dissipated by elastic deformation and therefore requiring less inelastic energy dissipation.

With concrete fill on metal deck, the induced story forces are resisted by the braced frames and distributed among the braced frame lines predominantly based on the behavior of a rigid diaphragm. This proposition assumes rigid body motion of the diaphragm with the resistance to motion by each braced frame line based on its relative stiffness for direct shear and its relative stiffness and distance to the center of rigidity for torsional resistance. Clearly for rigid diaphragms, the further away the braced frame line is from the center of rigidity the more efficient it may be in resisting both direct shear and torsional shear. This assumption reduces the degrees of freedom and is generally acceptable for well-distributed braced frame lines in plan in each direction. Where this is not the case or where the braced frame lines are widely spaced apart, semi-rigid diaphragms more accurately model the distribution of the inertial forces to the braced frame lines.
Brace Elements

Braces in SCBFs must withstand cycles of buckling and tensile yielding. Once brace forces are calculated, a member with sufficient compression capacity must be selected, and in that selection the slenderness and cross-sectional shape chosen are key variables. Several aspects of system behavior depend on these variables, including energy dissipation, susceptibility to local buckling, fracture life, post-buckling stiffness, and the ratio of yield strength to buckling strength.

Slenderness. The effects of slenderness on brace hysteresis are important, but no range optimizes all aspects of performance. As shown in Figure 3, the buckling of braces reduces their compression capacity for subsequent cycles (Zayas et al. 1981; Black, Wenger, and Popov 1980). This degradation is most pronounced for slender braces. For an expression of hysteretic stability as a function of slenderness see Remennikov and Walpole (1998). Purely elastic buckling may result in very low system stiffness when the direction of loading is reversed, creating “tension-only bracing” type behavior. For more information regarding tension-only bracing and design see Tremblay (1996). The design community has still not embraced very slender bracing, although preliminary studies show that slender braces designed for compressive loading can increase the amount of energy dissipation for similar drifts. Hysteresis diagrams of intermediate-slenderness braces show that their post-buckling behavior is also not ideal: they lose force after buckling with very little additional deformation (Jain and Goel 1978). This can lead to greatly reduced system stiffness, especially for chevron-braced frames (Khatib et al. 1988). Braces of low slenderness dissipate the most energy per cycle, but are susceptible to local buckling and early fracture (Tang and Goel 1987; Goel and Lee 1992). Low-slenderness braces exhibit the least hysteretic degradation. Braces with slenderness ratios below 30 exhibit stable hysteresis. AISC 2005 now permits designing compression members with slenderness ratios of 200 due to the inherent overstrength in the tension capacity. Research has shown that these systems will perform well under seismic excitation; see Tremblay (2000). However, for braces with an overall slenderness greater than 4.0 $\sqrt{\frac{E}{F_y}}$, the overstrength factor, $\Omega_0$, of 2 in ASCE (2005) is not adequate to account for the effect of the tension overstrength when designing other members, such as columns.

![Figure 3](image-url)

**Figure 3.** Typical hysteretic behavior for slenderness (kl/r) of (a) 40, (b) 80, and (c) 120, from Black, Wenger and Popov (1980)

The effect of “pinched” hysteresis, as shown in Figure 3b and 3c, above, will likely result in concentration of drift in a particular floor. Once interstory forces in a structure drop where braces have failed, surrounding floors remain elastic, amplifying the demand on the structure. Analytical simulations confirm that once inelastic buckling of braces begin, subsequent drift concentration is common; see: Uriz (2005), and Tremblay and Tirca, (2003).

In order to achieve trilinear hysteresis, it is usually necessary to design connections and adjoining elements to withstand forces corresponding to the tensile capacity of the brace. The tensile capacity of slender braces can be much larger than their compression capacity. Since the brace compression capacity is usually determined by code-prescribed loads, the use of slender braces may result in connection design forces that are much larger than code...
demand. If the system is to withstand tensile yielding, the force level of that yielding may need to be controlled. Providing braces with a higher yield strength \((F_yA)\) will improve system performance only if all other components of the system can resist the higher corresponding forces. For example, a low-slenderness pipe brace of low expected yield strength will result in a low ratio of tension capacity to compression capacity; a slender, wide-flange beam of high expected yield strength that buckles about its minor axis will result in a much higher ratio. Filling hollow sections with concrete also lowers the ratio as well as prevents local buckling (Liu and Goel 1988). However, the maximum compressive force that the brace can impose on the connections should include the strengthening effect of the concrete. Popov and Black (1981) determined that the effective length concept is applicable to braces subject to cyclic buckling.

Researchers at the University at Buffalo tested 1/2-scale single-diagonal and single-story X-brace configurations using hollow square tubes and solid rectangular bars. These tests were performed to demonstrate the effect of using channels installed on both sides of the bracing system in the plane of the braced bay to restrain out-of-plane buckling of the members (Celik et al. 2004). These cold formed channels permitted second mode buckling of members when they were installed; however, the increased plastic rotation in the hollow members led to premature fracturing of the braces during earlier cycles when compared to the non-restrained members (ductility of 4 versus 6). The cold-formed channels were sized to restrain the hollow sections from buckling, and as a result, substantially strengthened and stiffened the assembly. Typical failure mechanisms for the braces involved plastic hinges, eventually leading to local buckling and fracture at the local buckle. The hollow unconstrained brace fractured at a drift of 2.8% where the constrained brace fractured at a drift of roughly 1.9%.

The computed slenderness should reflect the expected buckling length with the effects of end restraint included. Use of the centerline-to-centerline distance will result in lower brace capacities in compression, which is conservative for brace design. However the compression demand on gusset connections and stitch plates will then be unconservative. It may be prudent to envelope the design using upper and lower-bound end restraint assumptions.

Numerous experimental and analytical studies have confirmed that the effective length for pinned-end braces in cross-braced frames can be taken to be the brace half-length (DeWolf and Pelliccione 1979; El-Tayem and Goel 1986). These results are valid for systems with brace-to-brace connections that provide flexural continuity but relatively little rotational restraint in the plane of buckling, for example, out-of-plane buckling of torsionally flexible sections (Sabelli and Hohbach 1999) for the effects of rotational restraint. Cross-braced frames without that flexural continuity have not been studied as thoroughly, but the upper and lower bounds of effective length can be determined.

It can be shown that the effective length of out-of-plane buckling of the discontinuous brace is the half-length; for out-of-plane buckling of the continuous brace it lies between the length and the half-length. Those bounds should be used conservatively. The upper bound of effective length should be used for determining brace capacity, and the lower bound for determining the maximum compression demand that the brace can impose on connections and adjoining members. It should be noted that there is the potential of forming a premature compression yielding mechanism, which involves twisting of the continuous brace and hinging in the center splice plate on both sides of the continuous brace. The use of torsionally stiff sections is therefore recommended. Effective length factors for use in the design of cross-braced frames are presented in Table 3. Illustrations of lengths and end conditions for use with the table are shown Figures 4, 5, and 6.
### Table 3. Effective Length Factors for Cross-Braced Frames.

<table>
<thead>
<tr>
<th>Splice Condition</th>
<th>Fixed In-Plane</th>
<th>Fixed Out-of-Plane</th>
<th>Fixed In-Plane</th>
<th>Fixed Out-of-Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Brace</td>
<td>Continuous Brace or Spliced Brace</td>
<td>Continuous Brace</td>
<td>Spliced Brace</td>
</tr>
<tr>
<td></td>
<td>End Condition 3</td>
<td>Theoretical K value</td>
<td>Recommended K value</td>
<td>Theoretical K value</td>
</tr>
<tr>
<td>Fixed Out-of-Plane</td>
<td>0.70&lt;sup&gt;6&lt;/sup&gt;</td>
<td>0.8</td>
<td>Note 9.</td>
<td>(1.0)&lt;sup&gt;10&lt;/sup&gt;</td>
</tr>
<tr>
<td>Pinned Out-of-Plane</td>
<td>1.00&lt;sup&gt;6&lt;/sup&gt;</td>
<td>1.0</td>
<td>Note 9.</td>
<td>(2.0)&lt;sup&gt;10&lt;/sup&gt;</td>
</tr>
<tr>
<td>Fixed In-Plane</td>
<td>0.54&lt;sup&gt;11&lt;/sup&gt;</td>
<td>0.7</td>
<td>0.54&lt;sup&gt;11&lt;/sup&gt;</td>
<td>0.7</td>
</tr>
<tr>
<td>Pinned In-Plane</td>
<td>0.80&lt;sup&gt;11&lt;/sup&gt;</td>
<td>0.9</td>
<td>0.80&lt;sup&gt;11&lt;/sup&gt;</td>
<td>0.9</td>
</tr>
</tbody>
</table>

**Notes to Table 3:**
1. Values apply to frames in which braces are of the same length and cross-section.
2. Values apply to frames in which the brace tension force is approximately equal to the compression force in the opposed brace. Where brace compression due to gravity loads exceeds 15% of the computed buckling strength, values may be unconservative. See applicable references for adjustments.
3. See Figure 5 for examples.
4. See Figure 6 for examples.
5. Values combine theoretical values with increases in effective length recommended in AISC LRFD Table C-C2.1.
6. Recommended values are appropriate for brace design; for connection design, the higher theoretical values should be used to determine the required compression strength.
8. See Kitipornchai and Finch (1986); el-Tayem and Goel (1986); Picard and Beaulieu (1987).
9. Recommended values for this splice condition are conservative and are not meant to preclude designers from calculating lower values. See note 9.
Figure 4. Brace lengths to be used with effective length factor in Table 3.

Figure 5. Gusset connections providing differing end-fixity conditions.
Use of different sizes for the continuous and discontinuous braces is not recommended, as this may result in the accumulation of inelastic drifts in one direction. Brace-to-brace connections in cross-braced frames should comply with all strength and detailing requirements discussed under “Bracing Connections.”

**Cross Section.** During design, three important aspects of a brace’s cross section should be considered, notably the geometric shape, material, and the thickness of compression elements within the section. These characteristics have been shown to significantly affect ductility of the brace. Certain section types have been shown to be more susceptible to low cycle, fatigue-induced fracture. Square and rectangular cross section hollow structural steel (HSS) members were shown to be particularly susceptible to fracture due to local buckling behavior of the cross section and, therefore, are not recommended for concentric braced frame applications, unless all the elements of the lateral force resisting system are designed for the elastic code level forces reduced by an $R$ factor, limited to the over-strength component ($R_o$) ignoring the ductility component ($R_d$), or the design addresses compactness and slenderness as discussed herein.

In contrast to square and rectangular HSS, round pipes, round HSS, and wide-flanges exhibited less severe local buckling, leading to better fracture resistance and greater brace ductility (Fell and Kanvinde, 2006). Tests have shown this failure mode to be especially prevalent in rectangular HSS braces with width-to-thickness ratios larger than the prescribed limits (Hassan and Goel 1991; Tang and Goel, 1989; AISC-Seismic 97). Because much research has been focused on square and rectangular cross-section HSS, other important issues remain to be fully answered such as detailing of a wide-flange brace to a beam-column connection. Braces with high width-thickness ratios exhibit relatively poor ductility compared to more compact members. This critical parameter was first prescriptively limited in the 1997 UBC following the 1994 Northridge Earthquake. Width-thickness ratios continue to be limited in AISC 2005 Seismic Provisions. However, recent experimental test results indicate that bracing members that comply with current width-thickness limits fractured at drifts of 2% to 3% at design-level forces (Fell and Kanvinde 2006). Research has shown that a lower width-thickness ratio delays fracture of braces subjected to cyclic loading. Another method to delay the onset of fracture for braces subjected to cyclic loading is to fill HSS with grout. Grout-filled HSS members exhibit more favorable local buckling characteristics, significantly altering the post-yield behavior of these sections (Liu and Goel 1988). Based on the 2007 CBC, HSS braces are required to be filled with grout to ostensibly preclude local buckling. However, the maximum compressive force that the brace can impose on the connections should include the strengthening effect of the concrete. Popov and Black (1981) studied cyclic...
buckling of struts, testing a number of cross-sectional shapes and a range of slendernesses. Their results indicate that, in general, the best behavior (defined by fracture life and fullness of the hysteresis diagram) can be expected of pipes and tubes, if local buckling can be prevented. Next in quality of performance are wide-flange sections, tees, double channels, and, finally, double angles. Yang and Mahin (2005) theorize that pipe sections are generally less susceptible to local buckling than tube sections. Astaneh et al. (1986) have shown that the performance of stitched sections can be improved by using closely spaced, strong stitches. SCBF provisions now incorporate these requirements.

The section shape often implies a particular material and fabrication method, which also affect brace behavior. For example, a popular bracing member is the square HSS section, which typically provides the largest radius of gyration for a given area. These sections are generally conforming to ASTM A500 grade B material (see ASTM A500). This classification is for cold-formed welded and seamless material. Although the ASTM standard has provisions for heat-treated steel (Grade D), this is rarely used in design, as the design strengths are lower, although they may provide better ductility.

Ductility can be increased by hot-forming HSS sections, and even accommodate higher strength capacity, according to specification ASTM A501, which also accommodates a grade B material with 50 ksi yield strength. The grade B material specifies V-notch impact testing and weld-line integrity evaluation to help ensure the most ductile behavior of the material. The minimum elongation requirements are the same as the cold-formed sections. These requirements make the hot-rolled section a more appealing selection for design, because they will likely have more ductile performance in an earthquake. There are cost implications that may factor into design decisions for sections satisfying this specification.

Canadian researchers (Koseteski, Packer, and Puthli 2005) have discovered that the orientation of coupons used for notch toughness testing of internationally produced HSS members may affect test results. For projects requiring specific notch toughness, the exact testing protocol should be specified.

Although the effect of overall brace slenderness needs to be considered to determine the appropriate width-to-thickness ratio for a bracing element as described above, the current provisions may not have section compactness requirements which are sufficient to preclude early local buckling for intermediate kl/r. It is recommended to be very conservative when selecting brace section compactness, and at a minimum use the expected strength in the calculation of the limiting compactness requirement (i.e. $R_y F_y$).

**Built-Up Members.** SCBFs require closer spacing of stitches and higher stitch strength requirements for built-up bracing members in SCBFs than are required for OCBFs. These are intended to restrict individual element bending between the stitch points and consequent premature fracture of bracing members (Aslani and Goel, 1991; Xu and Goel 1990]. Wider spacing is permitted under this exception when buckling is in the plane that does not cause shear in the stitches. Bolted stitches are not permitted within one-fourth of the clear brace length, as the presence of bolt holes in that region may cause premature fractures due to the formation of plastic hinges in the post-buckling range.

**End Connections**

**Bracing Connections.** Since SCBF braces are intended to provide ductility in both tension and compression, connections must accommodate cycles of brace buckling and tensile yielding. SCBF requirements for connections are intended to prevent undesirable connection performance from limiting system performance, caused by nonductile detailing or insufficient strength. Many of the failures reported in concentrically braced frames subjected to strong ground motions occurred in the connections. Similarly, cyclic testing of specimens designed and detailed in accordance with previous provisions for OCBFs has produced connection failures (Astaneh et al. 1985). Although typical practice has been to design connections only for axial loads, good post-elastic response demands that eccentricities be accounted for in the connection design. Good connection performance can be expected if the effects of cyclic post-buckling of braces are considered (AISC 2005, Astaneh et al. 1986, Goel and Lee, 1992, Uriz 2005). It should be noted that some of the issues associated with moment frames (e.g., the toughness of welds and
procedures for complete-penetration welds) need to be considered in order to ensure successful connection performance.

In tension, brace connections are designed to remain elastic for the maximum expected force demand. The lower of two force levels can be used to determine this demand: (a) yield strength of the brace, or (b) the maximum force that the system can deliver. In determining the yield strength of the brace, realistic values for brace area and yield stress should be used. The expected yield strength for all sections is significantly higher than the product of the nominal yield stress and nominal area (see AISC-2005 for the determination of maximum expected yield strength). The second force level is valid where certain braces, such as diaphragm or penthouse braces, are designed to remain elastic while other yield modes govern the system capacity. This condition must be demonstrated by analysis. This force level also is valid for certain configurations, such as the zipper configuration, where tensile yielding is not involved in the formation of a yield mechanism (Khatib et al. 1988). It is assumed that the controlling yield mode of an SCBF building is brace yielding. It is not valid to limit force demands because some other yield mode, such as column buckling, collector failure, or foundation uplift governs. In fact, the former two modes should be precluded; foundation uplift may not prevent inelasticity in the braces even if a static analysis so indicates.

Compression should also be considered in connection design. Thin-webbed beams and columns may require stiffening if the buckling capacity of the brace is to be developed. Note that the true effective length of the brace should be used in determining the maximum compression force to be resisted; the distance from centerline to centerline is significantly longer than the buckling length and will therefore give an unconservatively low design compression force. Buckling of gusset plates is also of concern, especially if a hinge zone is detailed.

Eccentricities of the forces applied at connections should be considered in the design of welds, bolts, gusset plates, etc. For example, it has been recognized by some engineers on the SEAOC Seismology Committee that force eccentricities can occur on the welds of connections of tube or pipe braces to gusset plates. Consideration of these eccentricities on short welds can result in the requirement for significantly greater weld size than would otherwise be derived based on shear forces only.

**Gusset Plates.** Inelastic brace buckling creates plastic hinges in three locations: one at the brace midspan and one at each end of the brace. There have been many observed fractures in tests on connections that failed to accommodate cycles of plastic hinge formation at the ends of braces (Astaneh et al., 1985). SCBF brace connections are designed to accommodate cyclic buckling of braces in one of two ways. First, connections can be designed to provide fixity and withstand the maximum axial load and moment that the brace can deliver, thereby forcing the plastic hinge to occur in the brace. Alternatively, connections can allow the hinge to occur in the gusset plate by providing an unrestrained zone that can tolerate the rotational demands imposed by brace buckling.

Except at the foundation, where brace connections are often buried in concrete, fixity is often difficult to achieve. A common alternative is to provide a hinge zone in the gusset plate to accommodate brace buckling without transferring large moments into the frame; this zone is expected to undergo cycles of large inelastic strains as brace buckling forces the plate to bend, as shown in Figure 5 above. Astaneh et al. (1986) demonstrated that this hinge zone performs in a ductile manner if there is an unrestrained zone perpendicular to the brace axis of width between two and four times the gusset plate thickness. It is recommended that three times the gusset plate thickness be used in detailing, and four times the gusset thickness be used in gusset plate stability calculations, thus allowing for inexactness in construction and weld thickness.

The end restraint provided by gussets determines both the slenderness (and therefore the buckling strength) of braces and the plane of buckling. Typically, hinge zones are provided in the gusset plates to accommodate brace buckling out of the plane of the frame while providing fixity in-plane. Gusset plates can also be oriented to provide hinges for brace buckling in the plane of the frame and fixity out-of-plane.

Care should be given to ensure that brace buckling and gusset plate hinging can be accommodated by all elements of the building. The inadvertent restraint of ductile behavior may cause a nonductile mechanism to control. Gusset
hinge zones may be restrained by a concrete deck, leading to gusset fracture. This can be prevented by moving the hinge zone out of the slab, or by providing block-outs in the slab. In-plane brace buckling can be restrained by wall studs if the frame is hidden in a wall, resulting in unplanned out-of-plane buckling. This can lead to higher-than-expected compression and moment demands on adjoining elements, nonductile modes of end-connection and stitch-plate behavior, and early fatigue fracture of braces due to large plastic moment demand at the hinges.

The detailing of connections with hinge zones leads to gusset plates that are larger than those designed under the 1994 UBC OCBF provisions or under the 1991 UBC. These gusset plates are more susceptible to edge buckling, plate buckling, and stress concentrations at reentrant corners primarily due to the modified geometry. The use of extremely thick gussets to prevent undesirable modes of behavior may result in excessive brace section reduction for slotted braces. Gussets of moderate thickness with stiffeners are recommended. Stiffeners should not intrude into the hinge zone. Rabinovitch and Cheng (1993) verified the applicability of the Whitmore (1952) method and use of the dimension from the gusset edge to the brace end (measured along the brace centerline) as the effective length. Astaneh (1998) gives guidelines for preventing gusset edge buckling. Researchers at the University of Washington (Yoo, et. al., 2008) have concluded that more compact and flexible gusset plates will lead to improved ductility and improved performance of SBCB structures. The new procedures assume an elliptical clearance model for allowing out-of-plane rotation, reducing the required length of the hinge zone while still providing ductile performance.

Net Area in Tension. Brace fracture across the critical section is of concern for bolted connections. It may also be of concern in welded connections of HSS braces, which are typically detailed as a gusset plate that fits into a slot in the walls of the brace member. To facilitate erection, the slot usually is made longer than the extent of the gusset, resulting in a short segment of brace whose net area is less than its gross area. Particularly if very thick gusset plates are used, net section fracture may occur in the brace prior to tensile yielding. Where such slotted connections are used, the brace section should be locally reinforced, and the slots should end in a drilled hole to minimize stress concentrations. Yang and Mahin (2005) studied the effect of net section reinforcement for a small sample of braces under large inelastic cyclic axial strains. They considered loading protocols having symmetric cycles as well as protocols with large near-fault-type pulses favoring either tension or compression. Most of the braces tested were square HSS tube sections, though one was a pipe section. The reinforced braces were designed to meet the net section criteria of the 1993 AISC LRFD provisions by adding a thick plate to each side of the element parallel to the slot. Most of the unreinforced braces failed by brittle tension fracture at the slot end after relatively few inelastic cycles, whereas reinforced braces exhibited much higher ductility and energy dissipation capacity. This difference was especially pronounced in the pulse-type loading protocol favoring tension. The tested pipe section, though unreinforced, was able to complete the loading sequence, however it eventually failed by net section rupture. The better performance of the unreinforced pipe section compared to the unreinforced tube sections could be predicted by the pipe material’s larger ratio of fracture to yield strength.

The slotted connection of braces at the gusset has historically been used for its ease of fabrication and because it maintains a fairly concentric load path. As noted above, during testing, net-section rupture was observed in unreinforced braces with reduced sections at slotted connections to the gusset plate (Shaback and Brown 2003). Brace configurations with this detail exhibited limited ductility. It is recommended to include reinforcement as required by application of AISC 341 Section 13.2b and AISC 360 Section D.2 at reduced sections of the brace, as explained in the SEAOC Seismic Design Manual.

Beams and Columns
Beams or columns of the frame should not be interrupted to allow for continuous braces. This provision is necessary, although perhaps not sufficient, to ensure out-of-plane stability of the bracing system at those locations. Typical practice is to provide perpendicular framing that engages the diaphragm to provide out-of-plane strength and stiffness as well as resistance to lateral torsional buckling of beams where these are intersected by braces (Kim and Goel 1992).

Columns. In the event of a major earthquake, columns in concentrically braced frames can undergo significant bending beyond the elastic range after buckling and yielding of the braces. Even though their bending strength is not
considered in the design process when elastic design methods are used, columns in SCBFs are required to have adequate compactness as well as shear and flexural strength in order to maintain their lateral strength during large cyclic deformations of the frame.

Yield modes of SCBFs almost always entail column rotational demands. Columns must maintain their axial capacity for both gravity and seismic loads and are therefore required to adhere to certain compactness criteria. These areas of concentrated column rotational demand are expected to occur adjacent to beam connections or adjacent to gusset plates if these plates provide significant restraint. Splices should be located away from these areas.

Analytical studies on SCBFs that are not part of a dual system have shown that columns can carry as much as 40 percent of the story shear in the post-buckling stages (Tang and Goel 1987, Hassan and Goel 1991). When columns are common to both SCBFs and Special Moment Frames in a dual system, their contribution to the story shear may be as high as 50 percent. This feature of SCBFs greatly helps in dissipating energy, making the overall frame hysteretic loops full when compared with those of individual bracing members, which are generally pinched (Hassan and Goel 1991, Black et al. 1980).

Columns in concentrically braced frames are anticipated to undergo axial forces well in excess of those calculated using the basic seismic load combinations due to the inherent overstrength of braces. Designers should be aware that very slender braces will have significantly higher overstrength than that implied by the system overstrength factor assigned in ASCE-7 (2.0). Additionally, many steel materials commonly used as braces exhibit higher material overstrength than that assumed in the determination of the overstrength factor. A capacity-design approach to columns in concentrically braced frames is recommended.

In single diagonal bay configurations where two braced bays are placed next to each other (see Figure 7), theoretical column loads in the elastic range for the columns that share both braces are zero if braces in either bay, on each floor, are identical. As mentioned above for a chevron configuration, the actual unbalanced loads due to post-buckling behavior can be quite large. Columns designed only for gravity loading may experience very large compressive loads, potentially overloading the column. It is recommended to design these columns for the same unbalanced seismic forces, and gravity loads, as the beams in chevron configurations.

**Figure 7.** Single diagonal configuration sharing a center column.

**Beams.** The braced frame beams should be capacity designed for axial forces that may be sustained by the braces. The advantage of providing a braced frame configuration that has common brace workpoints such as the two-story X configuration and others shown in Figure 1b, 1d, and 1e, is that brace forces flow directly from one brace to the next. The braced frame beam only needs to be designed for the additional induced axial forces acquired at that specific story level. In other configurations the braced frame beams will also have to transfer the accumulated brace forces from above to the brace below in addition to the additional forces acquired at that specific story level. Because of the high axial loads, braced frame beam sections should be compact to prevent premature local...
instability. Another consideration for chevron brace configurations is designing the beam for the unbalanced
tension-yielding compression-buckling load as previously noted.

**Analytical Studies and Modeling**

Performance-based earthquake engineering relies heavily on past performance, experimental studies, and robust
analytical models to estimate global performance of structures. Many analytical models have become available and
are being improved upon to generate an understanding of expected demands. Unfortunately, the expected demands
can be large, pushing the limits of current reliable analytical models.

**Analytical studies.** Typical “stiff” systems, such as braced frames, are traditionally expected to have much
smaller drift demands compared to their flexible structure counterparts. However, due to the degrading behavior of
the systems, analytical displacements tend to be large. Analytical studies for a variety of conventional bracing
members have shown that typical drift demands for design level shaking in California can be on the order of 1.5% -
4%, depending on the building height, configuration and analytical model used, and that scatter in the results can be
quite large (Sabelli, 2001, Tremblay and Poncet 2004, Uriz 2005) More severe shaking (i.e. 2% in 50 year level)
indicates median drift values ranging as high as 6%, further complicated by the fact that some of the models predict
“collapse” of the structure. The scatter in response data is significantly greater at the larger intensities.

**Modeling.** Non-linear dynamic analysis using appropriate models, including the effects of fracture due to fatigue
(for example see: Uriz 2005, Sabelli 2001, Tremblay and Poncet 2004) is paramount to understanding global
response of structures to dynamic loading. Unfortunately, these models can be cumbersome to build and can become
computationally unstable. The analytical behavior may also be fairly sensitive to the earthquake record used in the
analysis. A popular and simplified method to estimate inelastic response of structures is the non-linear static
procedure. The procedure simplifies cyclic response by implementing “backbone” curves to represent inelastic
response. It is strongly cautioned that this method can lead to an erroneous understanding of the actual collapse
mechanism.

Figure 8 shows force-displacement relationships from cyclic testing of a two-story chevron-configuration SCBF
(Uriz 2005) along with a representative non-linear model utilizing zero-length elements. Force-deformation
relationships for the analytical model are taken from accepted standards (FEMA 356). Cyclic testing of the structure
weakened the structure fairly significantly in early cycles, leading to brace failure in the first story. Once brace
failure occurred, there was no distribution of damage to the adjacent story, and all force resistance was taken by the
lower story columns until they failed near the beam-column connection. No yielding was observed in either of the
beams spanning between columns.

The analytical model, however, predicts a rather uniform distribution of damage in both floors, resulting in large
flexural demands in both upper and lower floor beams, which was not observed in the tests. The large loss of
strength in the analytical model in Figure 8 is due to the loss of beam flexural capacity of the upper beam. During
physical testing, the beam remained elastic at the midspan. Neither tension brace in the model reached critical
strains. Both braces on the bottom floor failed in the experimentation.
Figure 8. Comparison of (a) “pushover” analysis, using values from FEMA 356 with (b) experimental studies.

Keywords
braced frames
concentrically braced frames

References
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**How To Cite This Publication**

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Introduction
The objective of this Blue Book article is to provide SEAOC recommendations on seismic issues related to reinforced concrete elements and systems, and to identify design issues in need of further study, clarification, and research. The January, 2008 edition of ACI 318, ACI 318-08, (ACI 2008) is used as a basis for the recommendations.

Special Moment Resisting Frames (SMF)

General. The differences in detailing requirements between beams and columns in Special Moment Frames (SMF) are meant to account for the following issues. (SMRF, the abbreviation often used for "special moment-resisting frames," is equivalent to SMF).

a) The construction procedures used to assemble moment frames;
b) The expectation or intent that plastic hinging will occur primarily in the beams;
c) The larger variation in axial load in columns compared to beams, resulting from gravity compressive loads and seismic forces.

The SEAOC Seismology Committee position is that the requirements of ACI 318-08 Section 21.5 apply to beams, and the requirements of 21.6 apply to columns. For beams with axial compressive forces exceeding $A_{g} f_c/10$, as is likely in many collectors and occasionally in chords of diaphragms, a greater quantity of transverse reinforcement is required. As the axial load in a member increases, the required confinement needs to be increased to provide ductility and toughness. This is consistent with the reinforcement requirements for columns.

ACI provisions allowing members with axial load less than $A_{g} f_c/10$ to be designed per 21.5 (beam requirements) have been used in the past to exempt columns from strong-column/weak-beam requirements. SEAOC Seismology Committee does not recommend this approach, and instead it recommends that only top-story columns be exempted from strong-column/weak-beam requirements, as indicated in section I.E below.

Geometrical Considerations for SMF Beams. Based on experimental evidence (Hirosawa 1977) and engineering intuition, stocky frame members with length-to-depth ratios of less than four, under reversals of displacement into the nonlinear range, respond significantly differently to relatively slender members. Design rules derived from experience with relatively slender members, therefore, do not apply directly to members with length-to-depth ratios less than four, especially with respect to shear strength and ductility. As a result ACI 318 requires the depth of the beams to be less than four times the clear span. ACI 318 also limits the width of the beam relative to the depth of the beam and to the supporting column size. These provisions were derived from practice and research on reinforced concrete frames resisting earthquake-induced forces (SEAOC, 1999). The limit of the width-to-depth ratio is intended to provide compact cross-sections that have a low risk of lateral instability in the nonlinear range of response. ACI 318 recognizes that the maximum effective beam width depends principally on the column dimensions rather than on the depth of the beam, as was suggested in the 2005 and earlier versions of ACI 318. The maximum-width limitation is intended to allow efficient transfer of moment from beam to column.

Longitudinal Reinforcement for SMF Beams. ACI allows a maximum upper-bound reinforcement ratio of 0.025 for the beams of SMF. Generally beams with higher reinforcement ratios are less ductile. As a result, SEAOC Seismology Committee recommends that the amount of reinforcement in the beam due to earthquake loading be limited to about 0.015 for typical conditions. The upper limit ratio of 0.025 as prescribed by ACI should only be used in isolated frame beams where other alternatives such as increasing beam depth or adding additional bays of SMF is not possible. The importance of providing nearly equal top and bottom reinforcement (i.e., positive and negative reinforcement) from a ductility perspective is also emphasized by SEAOC Seismology Committee,
particularly if upper limits of reinforcement ratio are used. Without nearly equal positive and negative reinforcement, the hysteretic behavior is prone to unidirectional post-elastic strain accumulation, which may result in residual deformation. (Fenwick, Dely, and Davidson 1999)

**Transverse Reinforcement for SMF Beams.** ACI 318 section 21.5.3 prescribes the use of hoops in the potential hinge regions of frame beams. The beam hinge region is considered to be a distance equal to twice the beam depth from the face of the column. ACI 318 requires that the hoops in this region provide lateral support for the longitudinal rebar in the same manner as that prescribed for columns per section 7.10.5.3. This protects the longitudinal reinforcement in the beam hinge region from buckling due to cyclical response during an earthquake. The longitudinal reinforcement in SMF beams is assumed to yield during a severe seismic event. As the beam cycles back and forth during the earthquake, the top and bottom layer of bars will yield and compress. The yielded reinforcement will stretch under tension and is then compressed back towards its original length when the earthquake reverses direction. This cyclical stretching and compression of rebar results in the buckling of the longitudinal bars if not properly braced.

The spacing and configuration of the hoops in the beam hinge region is critical in preventing the longitudinal reinforcement buckling. ACI prescribes a spacing of d/4, 8 times the longitudinal bar diameter, 24 times the diameter of the hoop bar, or 12 inches for these hoops. SEAOC Seismology recommends that the maximum spacing of the hoops be limited to 6” to be effective in preventing the buckling of the longitudinal bars. Additionally, when frame beams are upturned (which is commonly used in parking garage type structures) closed stirrup ties should be used in lieu of the two piece stirrups ties, inasmuch as open 90 degree end of the seismic hooks need positive encasement within the slab. Similarly, L-shaped beams must have the 90 degree end of seismic hooks on the slab side of the beam.

The Committee also believes the compression zone of the frame beams should be confined with an appropriate level of confinement reinforcement. The compression zone is limited to the portion of the beam where the compression stress block occurs and defines the neutral axis location. The depth of compression zone is typically small compared to overall depth. The compression zone occurs both at the top and bottom of the beam and alternates back and forth with earthquake cycles. Vertical legs of stirrups with seismic hooks at each end, arranged to engage at least every other longitudinal beam reinforcing, will typically be sufficient for confinement purposes. However in those rare instances where the depth of the compression block is deep (e.g. larger than 6") the addition of a horizontal tie should be considered. Such a tie is required in ACI 318 7.11.1. This additional horizontal tie should have seismic hooks at each end and not be spaced further than 8” inside both the top and bottom horizontal legs of the outer hoop. The confinement will enhance beam performance in light of the large inelastic rotational demands imposed on it in a large seismic event. Analyses have indicated that large building drifts cause rotational demands on the order of 4% in frame beams. Testing is required to determine the appropriate level of confinement pressure required in the compression zone of frame beams.

The transverse reinforcement for beams should also be designed to satisfy the entire shear demand without considering any contribution from concrete shear strength. SEAOC Seismology Committee believes that the concrete shear strength in the hinge region of the beams will be substantially compromised due to the hinging of the beam regardless of the level of earthquake shear demand in the beam.

**Strong-Column/Weak-Beam.** ACI 318 requires that the column strengths be compared to the beam strengths at each joint, which can be inadequate in preventing story mechanisms. Nonlinear analyses of frame structures have shown that current strong-column/weak-beam provisions can be inadequate in preventing a story mechanism (Bracci and Dooley 2001, Kuntz and Browning 2003). Inelastic analysis of frames has demonstrated that moments in the columns can increase well beyond the beam moment strength at the joint, resulting in plastic hinging of the columns. This phenomenon, commonly referred to as column moment magnification, is characteristic of moment frames in general including structural steel frames. Desirable and undesirable plastic mechanisms are shown in Figure 1. The SEAOC 1999 Blue Book commentary, Section C402.5, (SEAOC Seismology Committee 1999), recommended a
fundamental modification of code provisions for strong-column/weak-beam requirements in which column and beam moment strengths are summed over an entire story. The SEAOC Seismology Committee continues to support this position. This recommendation approximately equates to $\sum M_c / \sum M_b$ of 2 for the story at a joint, thus affording a higher confidence level in preventing a weak-story mechanism. However, this approach may result in larger columns with increased reinforcement.

The story approach referenced above can be effective in preventing the formation of undesirable story mechanisms, and may be used for higher performance objectives. Some SEAOC Seismology Committee members believe that the margin of 2 can be reduced when considering factors such as:

1. Reduction in plastic moments in the beams adjacent to frame columns in tension. Thus the total story moment demand calculated based on the beam strengths is actually less than calculated.
2. Column over-strength and limited yielding may be considered in reducing the margin.
3. Accurate assessment of column moment magnification through inelastic analysis may prove margins less than 2.

The story approach allows individual columns to be weaker than beams, provided that there is enough strength in other columns to prevent a story mechanism. Further, the column permitted to be weaker than the beams shall be demonstrated to afford sufficient ductility, and the designer is reminded that the ductility significantly reduces as the axial load increases. The designer should not rely on the code equations for confinement where axial load exceeds 0.3 $f'_c A_c$; rather a curvature analysis should be used. (Mander, Priestley, and Park 1988)

![Figure 1. Desirable and undesirable plastic mechanisms.](image-url)
**Transverse Reinforcement for SMF columns.** ACI 318 provides several equations for determining the transverse reinforcement in frame columns. Eq. 21-4 is based on maintaining the axial load capacity of the column given the spalling of the column concrete cover. This equation seldom governs the design except for small column cross sections. Eq. 21-3 and 21-5 of ACI and govern in most common conditions where column sections are larger, as is the case with most frame columns. The intent of these equations is to ensure adequate flexural curvature capacity in yielding regions. These equations increase the required amount of confining reinforcement in the columns in proportion to increase in concrete strength.


The SEAOC Seismology Committee recommends that the amount of transverse reinforcement in frame columns be a function of axial load level and that an increase in confining reinforcement above that currently recommended by ACI for higher axial loads is warranted. Additionally, testing has indicated that a reduction in the amount of confining reinforcement currently recommended by ACI is possible for lower levels of axial loads. Accordingly, the Committee recommends Eq. 1 for determining column confining reinforcement which is based on the research of Paultre and Legeron (2008) and is similar to the equation which appears in the Canadian building code (2004).

The total cross sectional area of rectangular hoop reinforcement, $A_{sh}$, should be taken as:

$$A_{sh} = 0.2k_p k_n (A_e/A_{ch})(f'_c/f_{yt}) sb_c$$  \hspace{1cm} (Eq. 1)

Where

- $k_p = P_u/A_g f'_c$ and shall not be taken less than 0.2
- $k_n = [0.6+0.4(n/n_{ls})][(h_x+12)/20]$, and the term $(h_x+12)/20$ shall not be taken less than 1.0
- $f_{yt} = \text{(per ACI 318) Specified strength of hoops and cross ties, and shall not exceed 100 ksi}$
- $n = \text{total number of perimeter longitudinal bars in the column cross-section}$
- $n_{ls} = \text{number of perimeter longitudinal bars in the column cross-section that are laterally supported by the corner of hoops or by seismic hooks of crossties that are} \geq 135^\circ$
- $h_x = \text{(per ACI 318) center-to-center horizontal spacing of crossties or hoop legs, in inches}$
- $P_u = \text{maximum factored axial force per current code load combinations}$

It is the intention of this committee that $P_u$ will typically be governed by the factored load combination which results in the maximum compressive load. The special seismic load combinations, which include the Omega factor, will typically not apply.

The committee recommends the adoption of Eq. 1 or a similar one by ACI. Various characteristics of Eq. 1 and pertinent background information as discussed in the following sections:

**Axial Load.** The effect of axial load on the required transverse reinforcing ratios has been incorporated into Eq. 1 in the form $k_p = P_u/A_g f'_c$. $P_u$ is defined as the maximum factored axial force per current code load combinations and is conventionally determined through an elastic analyses using code prescribed seismic forces. As discussed below, axial loads calculated in this manner may underestimate the actual axial load.
level in SMF columns. However, this is the prevailing method of analysis amongst practicing engineers and as such, the definition of $P_u$ provides for a more practical application of Eq. 1.

A minimum amount of confining reinforcement is required to ensure flexural curvature capacity in yielding regions. The lower limit on $k_p$ in Eq. 1 ensures this minimum level in cases where low axial load levels may otherwise imply lower confining reinforcement.

**Story Drift.** Drift demand is an influential factor in the determination of confining reinforcement. The anticipated drift level for the structure dictates the level of ductility capacity required and thus the amount of confining reinforcement. The anticipated drift demand for structures designed to the current code is at the heart of this issue.

ASCE7-05 specifies a drift limit of 0.02 for most structures 4 stories and higher. The drift is conventionally calculated based on an elastic analysis using the code prescribed seismic forces. The drifts are then amplified by $C_d$, the deflection factor, to estimate the inelastic drifts in the structure. The $C_d$ factor is on the order of 0.7 times $R$. Recent analytical studies in the BSSC Technical Committees and other nonlinear response history analyses, have indicated drifts higher than those estimated using the current $C_d$ factors. Specifically, these studies indicate a $C_d$ value equal to the $R$ factor would more appropriately estimate the inelastic drifts in the structures. Thus, structures designed to the 0.02 drift limit per ASCE 7-05 would actually experience a drift on order of 0.029 given the Design Basis Earthquake (DBE) ground motion. The DBE is approximately 2/3 of the Maximum Considered Earthquake (MCE) ground motion – the hazard level at which we accept heavy damage but intend to not have collapse. Under the MCE ground motion in Seismic Design Categories D, E and F the actual story drift demand will therefore be on the order of 0.04 for most code based designs (approximately 0.029/(2/3)).

Thus the committee believes a minimum drift limit of 0.04 at the near-collapse limit state would be appropriate as a basis for the determination of column ductility capacity.

Saatcioglu and Razvi (2002) propose an equation for determination of column confining reinforcement considering axial load, efficiency of the tie configuration, and drift as input parameters. Because it is cited in many building codes, they propose a drift level of 0.02 for SMF columns. Interestingly, when 0.04 is used in their equation, the resulting confinement quantities are similar to those resulting from the use of proposed Eq. 1.

**Confinement Efficiency.** Equation 1 considers the efficiency of the confining reinforcement through the $k_n$ factor. The configuration of the transverse reinforcement, whether all longitudinal bars are tied, the presence of a 90-degree bend at the end of cross ties, and the spacing of longitudinal bars can all impact the performance of the columns. Cyclic testing of columns (Xiao and Yun 1992) has indicated that the 90-degree hook ends of cross ties are prone to opening, leading to the buckling of the longitudinal rebar and sudden loss of strength. Pourzanjani and Englekirk (2000) recommend 135-degree hooks at both ends of cross ties in regions of the column susceptible to inelastic yielding. This is based on observing strength degradation due to loss of the cover concrete and the subsequent longitudinal bar buckling at the 90-degree hook end of the cross tie. Note that cross ties with a 90-degree hook on one end and a 135 degree hook on the other end can still be used under proposed Eq. 1, but a penalty is applied by increasing the $k_n$ factor. Even though Eq. 1 allows the use of 90/135-degree cross ties, the SEAOC Seismology Committee recommends and encourages the use of overlapping closed hoops or cross ties with 135-degree hooks at both ends within a region of column potentially subject to yielding. This may be assumed to be a distance equal to the 1.25 to 1.5 times the largest cross-sectional dimension of the column from each end.

Pourzanjani and Englekirk (2000) stress the importance of the spacing of the hoops in promoting ductility in the column, and recommend a maximum spacing of 5 inches for the ties. ACI section 21.6.4.3 allows for
spacing of transverse hoops up to 6 inches depending on the spacing of the cross ties in the cross section of the column.

Closer spacing of longitudinal reinforcement in conjunction with closer transverse spacing can be effective to optimize the confined core reinforcement (Saatcioglu and Razvi 2002). The factor $k_n$ in the proposed Eq. 1 accounts for this affect as does the equation proposed by Saatcioglu and Razvi. A smaller horizontal spacing of cross ties or hoop legs in a rectangular arrangement results in greater confinement efficiency and therefore smaller transverse reinforcing ratios are required as a result.

As captured by Eq. 1, increasing confining reinforcement and efficiently configured confinement enhances column ductility with increasing axial load. Ultimately, the most effective means of promoting ductility in SMF columns is to limit the level of axial stress on the columns. Pourzanjani & Englekirk (2000) noted an approximate attainable drift level of 3.5% for columns tested with an axial service load level of 0.34 $A_p f'_c$ (i.e. without load factor) versus a drift level of 6% for the same column with a lower axial load level of 0.2$A_p f'_c$. The proposed Eq. 1 above indirectly imposes a limitation on the axial load level on the column by producing impractical transverse reinforcement requirements. However, this deterrent may be insufficient in some cases and engineering judgment should be exercised.

The application of Eq 1 was studied by the committee in numerous examples with varying parameters. Figure 2 shows the relationship between ACI 318-08 Eq. 21-5 and Eq. 1 for one such example. Figure 2 shows that once the axial load factor $k_p$ exceeds approximately 0.35, Eq. 1 yields a greater amount of confining reinforcement than ACI. However, for lower levels of axial load, Eq. 1 relaxes the confining reinforcement.

![Figure 2. Comparison of proposed Eq. 1 to ACI Eq. 21-5. $k_p = 1$ is equal to 135 degree hooks on both cross tie ends, and $k_p = 1.25$ is equivalent to using alternating 90 and 135 degree cross ties.](image)

Although less experimental data is available for circular columns subjected to high axial loads, it has been suggested that the ACI confinement equation for spiral or circular tie reinforcement may not provide sufficient deformation capacity for columns with high axial loads. One possible approach, similar to the equation suggested by Elwood (Elwood, Maffei, Reiderer 2009), may be to modify Eq. 21-3 of ACI 318-08, as follows:
\[ \rho_s = 0.35 \frac{k_p}{k_p}(f'_c/f_{yt})(A_g/A_{ch}) \]  

(Eq. 2)

Where

- \( \rho_s \) = the volumetric ratio of transverse reinforcement and shall not be taken less than 0.12\((f'_c/f_{yt})\)
- \( k_p = P_u/A_gf'_c \), and shall not be taken less than 0.2

Note that a \( k_n \) term, used in Eq. 1 above, is not needed in this equation because spiral or circular tie reinforcement can provide effective confinement without additional crosstie or spacing limits for longitudinal bars.

**Design Axial Force for Columns of SMF.** Axial loading on frame columns is drastically impacted by inelastic dynamic response of frames and more specifically the beams, because beams are the major source of inelasticity in frames. An elastic analysis does not capture this axial loading on columns, whether for loading in one direction or concurrently in both directions. For a frame, the farther the column is from the center of a frame, the higher the axial loading on the column found by analysis. Thus, the end columns of the frame will experience the highest level of axial load. Where beams are present at each side of a frame column, the post-elastic loading imposed by the beams on the column is negated. Therefore the axial load levels from the elastic analysis will be sufficient for the design of the interior frame columns. However, the end columns are very susceptible to additional axial loading due to inelastic response of the frame. As the frame beam on one side of an end column yields, the additional shear generated by the yielded beam is delivered to the end column. The designer should exercise caution and provide an appropriate level of additional capacity for end columns.

An upper-bound loading assumes that all the beams above a level under consideration have yielded. However, this may be excessive, particularly as the number of stories of SMF in the structure increases. Consideration of this upper-bound level of inelastic axial loading should be balanced against other factors that lend columns additional capacity. These include increase in column concrete strength due to fast rate of loading, overstrength of column, an adjusted phi factor (\( \phi =1 \)) and expected material properties that reflect the upper-bound loading case. The amount of additional inelastic axial loading on the column should also be balanced against the degree of moment magnification on the column. Paulay and Priestley (1992 211 – 230) provide guidelines for increased axial loading for this purpose. Any adjustments in codes and standards should only be implemented with further research and compared with field observations from earthquakes. In light of the current state-of-the-art and need for more research, SEAOC Seismology Committee encourages exercise of judgment on the part of designers.

**\( \phi \) Factor for SMF Columns.** The ACI currently prescribes a different \( \phi \) factor for compression –controlled members (i.e. most columns) for spiral and rectangular transverse reinforcement: \( \phi \) is 0.75 for spirally reinforced members and 0.65 for members with rectangular tie reinforcement. This difference in the recommended phi factors is rooted in the recognition of the enhanced post-elastic performance of the confined concrete afforded by spiral reinforcement. It is believed that current design methods for rectangular hoops in SMF columns achieve equivalent confinement effectiveness. Accordingly, SEAOC Seismology Committee recommends that the \( \phi \) factor for columns with spiral reinforcement and with rectangular hoop reinforcement shall be identical, i.e. \( \phi = 0.75 \).

**Shear Walls**

Shear walls, by virtue of their geometrical configuration, respond differently to earthquake loading than moment-resisting frames, which requires careful consideration of different types of shear walls and how they dissipate energy. High aspect ratio shear walls \((h/l \geq 2)\) tend to be flexurally governed and can be designed to yield in flexure. Low aspect ratio or squat shear walls, on the other hand, tend to be shear-dominated and thus prone to shear failure or sliding shear. Tall shear walls with coupling beams respond by yielding of the coupling beams and flexural hinging of the coupled walls.

The most ideal ductile design for shear walls is to promote a flexural response in the wall. However, this is not practical or efficient in all conditions and in particular when considering low or squat shear walls. Shear-yielding
can provide sufficient post yield stiffness and ductility (ASCE 2007). A sliding shear mechanism, however, should almost always be avoided if possible. This can be accomplished by well distributed vertical reinforcement and/or the addition of vertical dowels. A practical way to achieve this strength hierarchy is to use diagonally reinforced shear walls. These walls resist shear and overturning by the diagonal bars and mitigate the risk of sliding shear (Paulay, Priestley, and Synge 1982). A combination of flexural-shear response may be appropriate for a mid-rise building. In the case of coupled shear walls, it is important to ensure the ductility of the coupling beams and limit rotation of the coupling beams to avoid excessive strength degradation.

**Design Shear Forces.** Flexural response requires that walls be designed to develop a hinge at the base and yield in flexure. For design of tall walls with higher mode effects, additional flexural hinging may occur and be desirable to prevent shear yielding in the wall. The shear capacity of the wall must be sufficient to develop the flexural capacity of the wall. For special reinforced flexural walls (See ASCE 7-05 recommendations), the design shear force, \( V_E \), can be taken as the shear associated with the development of the probable moment strength, \( M_{pr} \), at the potential plastic-hinge location of the wall, including inelastic dynamic amplification effects. \( V_E \) may be calculated per Equation 3 when an elastic analysis is undertaken using the prescribed static earthquake forces in the code, or by using Equation 4 when modal dynamic analysis is used for design. \( V_E \) in Equation 3 is increased to account for inelastic dynamic amplification of shear. The shear amplification factor, \( \omega_v \), in Equation 3 accounts for inelastic dynamic effects that can cause the vertical distribution of lateral seismic forces to differ from the inverted triangular pattern assumed in the analysis, resulting in greater shear corresponding to the same wall flexural strength. The formulas for \( \omega_v \) are taken from Paulay and Priestly (1992). Note that ASCE 7-05 prescribes a parabolic distribution of earthquake forces depending on the height of the structure. However Equation 3 can still be used as an approximation. Given that the shear amplification factor is intended to more realistically capture upper-bound design forces, a higher \( \phi \) of about 0.85 may be warranted for the calculation of \( \phi V_n \) with the static procedure. Equation 3 captures the elastic dynamic response of the structure and uses the relationship of story shear and moment obtained from the analysis to obtain demand \( V_E \) corresponding to \( M_{pr} \). Given that this procedure is essentially based on an elastic analysis and thus has a level of uncertainty with respect to the inelastic response, a \( \phi = 0.75 \) is recommended for the calculation of shear strength, \( \phi V_n \).

For analysis using code prescribed static EQ forces:

\[
V_E = (V_u M_{pr} / M_u)(\omega_v). \quad \text{(Eq. 3)}
\]

Where

- \( V_u \) = the shear demand from the static lateral force analysis occurring simultaneously with the Mu. Vu is in accordance with the factored load combinations
- \( M_{pr} \) = shall be calculated for the axial load case that results in the largest value of \( M_{pr} \).
- \( M_u \) = the flexural demand from the static lateral force analysis in accordance with the factored load combinations
- \( \omega_v \) = is the shear amplification factor when starting from a static lateral force analysis and shall be taken as \((0.9+N/10)\), for buildings up to 6 stories, and \((1.3+N/30)\), for buildings over 6 stories.

For analysis using linear dynamic analysis:

\[
V_E = (V_u M_{pr} / M_u)(\omega_d). \quad \text{(Eq. 4)}
\]

Where
\( V_u = \) the shear demand from the dynamic lateral force analysis occurring simultaneously with the \( M_u \). 
\( V_u \) is in accordance with the factored load combinations.

\( M_{pr} = \) shall be calculated for the axial load case that results in the largest value of \( M_{pr} \).

\( M_u = \) the flexural demand from the dynamic lateral force analysis in accordance with the factored load combinations.

\( \omega_d = \) is the shear amplification factor when starting from a dynamic lateral force analysis and shall be taken as (1.2+N/50).

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**Figure 3.** Relationship of the shear amplification factor over the number of stories.

**R Factors for Shear Walls.** In the determination of the \( R \) value for a shear wall, the historic distinction between a bearing wall versus a building frame wall is inconsistent with concrete shear wall design and behavior. Originally, in the first Blue Book (SEAOC Seismology Committee 1959), five \( K \) factors were defined without regard to material: moment-resisting space frame, space frame plus shear walls or braced frames, box (shear wall or braced frame), and an Other category for other building framing systems. In the box system category, there was no differentiation between wood framed shear walls, steel braced frames, or concrete shear walls. The \( K \) values, and the related \( R \) values incorporated into codes after ATC-3 (ATC 1978), were derived in part based on the goal of preserving vertical load-carrying capabilities. Today, however, the \( R \) values for structural systems correlate to specific material and detailing provisions that are related to system strength and ductility. ASCE 7-05 (Table 12.1-1) lists lower \( R \) values for Special or Ordinary reinforced concrete shear walls in the Bearing Wall System category than for the same types of walls in the Building Frame System category. It is the SEAOC Seismology Committee opinion that the concrete shear wall \( R \) value for bearing walls and building frame walls should not differ, and that shear walls should be identified under one category, regardless of whether the wall is load-bearing. The \( R \) value should not be influenced by the presence or absence of gravity axial load if the shear wall is properly detailed to maintain gravity axial load support under damaging cyclic loading. Seismology Committee cautions that the
The factored axial load on a shear wall should be limited to less than 0.35 \( f'c A_g \), unless a ductility assessment using a moment-curvature analysis is performed to justify a higher axial load level.

The practice of placing gravity-load frame reinforcement within shear walls in order to artificially create a “building frame” wall, as was suggested in the 1999 Blue Book, and which has become popular with building officials and insurance underwriters, may not be necessary to maintain gravity load support, and may adversely affect the seismic performance of the wall. Where confined boundary elements are adequately designed within the shear wall, the addition of gravity frame reinforcement within the walls should not be required. Where shear walls in high seismic regions are designed without boundary reinforcement or with only a single curtain of reinforcement, then the addition of hoops and ties around the vertical bars near the ends of walls could be beneficial, so long as the addition does not significantly alter the flexural vs. shear behavior of the wall. The engineer should consider the effect of heavy concentrated vertical loads or reactions within the wall length, for example from beam, column, or intersecting walls, where additional tied vertical longitudinal reinforcement may be provided. The detailing provisions of a shear wall system should directly address the distribution and confinement requirements that relate to the aspect ratio, gravity axial load, and distribution of walls. This would enable a consistent \( R \) value to establish systems design forces and displacement parameters. The SEAOC Seismology Committee understands that the state of practice has been to use the \( R \) value associated with the “Building Frame” system for virtually all reinforced concrete shear wall buildings. In the absence of further research and/or evaluation, the Seismology Committee recommends the use of the “Building Frame” Seismic Design Factors for reinforced concrete shear wall systems without incorporating an imaginary gravity frame within the wall, and without adding gravity load-bearing columns next to the wall.

Shear wall response to earthquakes is greatly influenced by geometric configuration, as discussed above. Each different type of wall configuration has a different degree of inherent ductility, thus warranting different detailing provisions. Walls designed for flexural yielding afford higher levels of ductility than shear-dominated walls, provided that the flexural compression zone is well confined. The type of shear response in shear walls will also yield different levels of ductility. Shear behavior of shear walls is best represented by diagonal strut-and-tie models. Shear strength is provided in a truss-like manner by reinforcing bars working in conjunction with diagonal compression zones in concrete. Shear failure is initiated when concrete fails in diagonal compression in a brittle manner, or by shear reinforcement yielding and eventually rupturing. The latter mode clearly provides a higher degree of ductility and is preferred when shear response in the wall cannot be avoided. Walls responding in a shear-yielding mode have more ductility than walls responding with diagonal compression failure. Diagonal compression in the wall is more likely when higher amounts of shear reinforcement are present in the wall. ACI 318 currently does not provide a limit on shear reinforcement in the walls to promote shear yielding, except for limits of shear strength under section 21.9.4.4. Further studies on this topic are required.

In any event, walls that yield in shear are generally considered to have a greater propensity than flexurally governed walls to exhibit strength degradation and low displacement capacity. As a result, the ductility demand on such walls may need to be limited. In multistory buildings, the post-elastic displacement of shear-yielding walls may concentrate at a single story or over a limited height where the shear yielding occurs, thus concentrating rather than distributing deformation demand, as compared to a flexurally dominated response. The local ductility capacity may be decreased for shear-dominated response. It is preferable to have a flexural response for high-rise construction. For low- and mid-rise buildings, where the total displacement demand is typically less than that of a high-rise, a shear-yielding mechanism may be an acceptable structural response.

As mentioned above, a sliding shear mechanism should be avoided, if possible. This has to be balanced with vertical reinforcement and its distribution. This is further complicated by the code requirement to decrease the actual presence of dead load used to resist overturning/flexure by the wall. The ASCE 7-05 requirement for load combination results in some cases with only 60% of the wall and tributary dead load being used to resist flexure. This leads to more vertical reinforcement, which in turn increases flexural strength, hence the direct and sliding shear demands increase. To offset this, concentrations of vertical reinforcement shall be avoided at wall ends and instead shall be distributed throughout the wall to increase sliding shear resistance by increasing the length of the
compression zone of the wall (Paulay and Priestley 1992). This is demonstrated in Figure 4. The effect of vertical accelerations is beyond this article but is cited here because of the significant detrimental effect it can have on the performance of shear walls.

![Diagram of vertical reinforcement]

**Figure 4.** Distribution of vertical reinforcement along the length of shear walls

SEAOC Seismology Committee recommends a study of the $R$ value and detailing provisions for the following conditions.

1) flexure-governed walls;
2) shear-governed walls without diagonal compression failure mode;
3) Shear-governed walls with diagonal compression failure mode versus shear yielding;
4) Shear walls with sliding shear mechanism.

Based on wall configuration, more than one type of wall behavior may be present in a building. SEAOC Seismology Committee recommends that a single $R$ value be used and that the detailing provisions be adjusted to account for the anticipated behavior.
Foundations of Shear Walls. SEAOC Seismology Committee recommends that designers compare wall flexural and shear strengths to the capacity of the wall's foundations, and to wall strength in sliding shear. This is particularly important for low-rise walls. Specific recommendations are given in Paulay and Priestley (1992) and ATC (1998).

Intersecting Wall Sections and Flanges. Connected or intersecting wall sections shall be considered as integral units. The strength of flanges, boundary members, and webs shall be evaluated on the basis of compatible interaction between these elements. The effect of wall openings shall also be considered.

For walls connected to each other by beams, slabs, or coupling beams, the coupling action shall be considered, unless the effects of coupling can be shown to be negligible.

For a wall L-shaped in plan, for example, the two legs of the L should not be designed or reinforced independently. The leg of the L parallel to the direction of earthquake forces being considered will act as the web of the section, while the other leg of the L will act as a flange, with an effective flange width as specified.

Walls with openings should be analyzed to identify the critical sections and behavior modes (e.g. flexure or shear) and the governing mechanism of inelastic response for each component of the wall. For instance, a wall with a tall opening may have a large compression force and shear on the short wall segment. In contrast, the long wall segment may be in tension and not until larger lateral deformation occurs will it be capable of developing significant shear capacity. Vertically aligned wall openings create a coupled wall system, which is typically analyzed as an assemblage of coupling beams and wall piers. Walls with irregular openings can be designed using strut and tie methods (Paulay and Priestley, 1992). Small, isolated openings that do not affect the location of critical sections or the governing behavior modes or mechanisms can be neglected. In any method used to track the shear demand, particularly where there is an interruption in the vertical shear (diagonal strut), the horizontal shear force will tend to be transferred through the compression zone, and an aggregate section capacity for shear strength may be inappropriate. Where this occurs, the strut and tie method and attention to ductility demand and details should be exercised, as discussed by Taylor, Cote, and Wallace (1998).

Diagonally Reinforced Coupling Beams. In large part due to the PEER Tall Building Initiative and research efforts on coupling beams (Wallace and Orakcal, 2002), a significant change and simplification of coupling beam shear and confinement reinforcement has occurred. This is reflected in ACI 318 -08. Under these provisions, the diagonal rebars in the coupling beams no longer require transverse reinforcement directly bracing them. Instead, transverse reinforcement is provided for the entire beam cross section along with longitudinal reinforcement with close spacing not exceeding 6 inches, and hoop legs or cross ties both vertically and horizontally, with spacing not exceeding 8 inches. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or larger diameter. In essence, the entire beam is confined and the diagonal bars are within that confinement.

Since the diagonal reinforcement resists flexure and shear in the coupling beam, conventional longitudinal and transverse reinforcement act principally as basketing to prevent spalled concrete from becoming a falling hazard and to provide confinement. SEAOC Seismology Committee recommends against fully developing the perimeter longitudinal reinforcement into the wall piers, as this would increase the flexural strength, and consequently the shear demands, on the coupling beam. Instead SEAOC Seismology Committee recommends continuing the additional longitudinal reinforcement 6 inches into the wall pier, which should be adequate to maintain the integrity of the coupling beam after cracking and spalling (Paulay and Priestley 1992).

As an alternative to the confined diagonal set of longitudinal reinforcing bars for coupling beams, the designer may consider use of an inclined rhombic layout of the longitudinal bars (Tegos and Penelis 1988, Galano and Vignoli 2000). Cyclic testing has shown that diagonal and rhombic reinforcing bar layouts behave in a similar manner. Testing has indicated that the inclined rhombic rebar layout can reduce cracking and damage across the length of the
Rotational Limits for Coupling Beams. Neither ACI nor IBC provide rotational limits for coupling beams. Wallace (ATC 72 draft-2008) reports rotations up to 8% with little strength degradations for coupling beams in the testing of four specimens designed to ACI318-08. Tests by Tegao and Penelis for coupling beams with rhombic reinforcement layout indicate rotations in the range of 2.5% to 3% without loss of strength and up to 4% with some degradation in strength. ASCE 41-06, Table 6-18, (ASCE 2007), indicates rotational capacity of 0.03 and 0.05 respectively for $a$ and $b$ (i.e. for peak and back bone portions respectively) for diagonally reinforced and confined coupling beams. SEAOC Seismology Committee recommends plastic hinge limitations of 0.03 to 0.05 for confined coupling beams unless higher values can be justified by testing specimens that have aspect ratios and reinforcement similar to those to be used in the design project. As additional testing data become available, it may justify a universally higher rotational limit for coupling beams of certain reinforcement configurations.

Compatibility Requirements for Gravity Load-Resisting Elements

The lateral load-resisting systems in high seismic zones are designed and detailed to provide bracing and required ductility for anticipated earthquake-induced deformations. However, the elements not part of the lateral load-resisting system, i.e. the gravity load-resisting system of the structure, also experience the seismic deformations and need to have proper ductility for that role. Concrete structures in particular will experience unintended frame-like behavior in the gravity load-resisting system when subjected to lateral loading. For example in buildings with flat slab construction, the slab and column combination will attempt to act as a frame during an earthquake event. As a result, slabs will experience additional bending and punching shear while the columns will be subject to additional axial load, moment, and shear imposed by the frame action of the slab. ACI 318 provides recommendations for the compatibility requirements but further recommendations are required. SEAOC Seismology Committee recommends that additional axial loading due to frame action be considered for gravity columns in concrete structures. This is particularly critical as the height of the structure increases. A possible approach may be to subject an elastic model of the building to the anticipated deformation to determine the axial loads on the columns. The amount of axial loading imposed may prove to be limited by the punching shear limit state of the slab. Alternatively, a subassembly representing the gravity frame of the structure may be subjected to the maximum anticipated drift, and the axial load obtained may be projected to the height of the building. A ductility factor of $R = 3$ consistent with that for an Ordinary reinforced concrete moment frame may be used for either type of analysis. The additional axial loading obtained in this manner may be treated as earthquake loading for the purpose of determining factored loads when using loading combinations prescribed by the code. SEAOC Seismology Committee recommends that transverse reinforcement for gravity columns conform to ACI 318 21.6.4.4 for the full height of the column with spacing not exceeding section 21.6.4.3. This will enhance the performance in light of these unanticipated seismic loadings.

High-Strength Concrete

The use of High-Strength Concrete (HSC) in regions of high seismicity is increasing with the construction of high-rise structures, particularly for residential and hospitality purposes. However, in the absence of a specified upper-bound concrete strength in ACI 318, the applicability of the prescribed seismic provisions in ACI 318 for design of HSC is unclear. Pourzanjani and Englekirk (2000) reported on the result of a testing program developed in southern California for use of HSC in SMF in regions of high seismicity. A related publication produced by the Carpenters Contractors Cooperation Council (C4) is referred to here as the C4 report (Carpenters Contractors Cooperation Council 2000). The testing program included large- and small-scale HSC columns and beam-column sub-assemblies. Recommendations for the design of SMF columns, joints, and beams for concrete strengths greater than 6000 psi are provided in the report. More recently, the “Report on Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications,” ACI (2007), referred to as the ITG report, was published. The ITG references many of the available research results on design of HSC and provides design recommendations for HSC special moment frames and shear walls, including development lengths of reinforcing
bars in HSC. The ITG defines HSC as concrete with strength of 6000 psi and higher in accordance with ACI 363R-92.

**HSC Concrete Mix.** The C4 report stresses the importance of HSC mix characteristics in providing ductility. Typically, the higher the concrete strength the more brittleness is evident in the stress-strain diagram for the mix. As a result, the C4 report recommends that the HSC mix shall have an extended stress–strain diagram reaching a strain of 0.004 on the descending side of the curve. The SEAOC Seismology Committee recognizes the importance of an extended stress-strain curve in yielding ductility for concrete mix in general and HSC mix in particular.

**HSC Special Moment Frames.** The ITG, in Chapters 5 and 8, provides design provisions for HSC SMF. A number of these recommendations are discussed and compared below with those provided by the C4 report.

ITG reports that the ACI 318 equation for column confining reinforcement is conservative for lower levels of axial load while high levels of axial load may result in columns with limited ductility. ITG cites a number of researchers supporting this statement. In recognition of this, ITG proposes an equation based on the work of Saatcioglu and Razvi (2002). However, the SEAOC Seismology Committee believes the proposed ITG formula, as presented, produces results lower than the current ACI 318 requirements for most levels of axial loading. This may be, in part, due to Saatcioglu intending to use capacity or mechanism level axial loads in the formula, while ITG’s adaptation implies the use of the code level factored axial loads. The Committee is in agreement with ITG, that axial loads should be a consideration in determination of the confining reinforcement, however, the ITG formulation appears to be unconservative and potentially unsafe.

In response to the uncertainty associated with proposed ITG confinement equation, the SEAOC Seismology Committee convened an Ad Hoc committee. The Ad Hoc committee recommended Eq. 1 after reviewing a number of documents, and compiling numerous examples. Based, in part, on the recommendation of the Ad Hoc Committee, the SEAOC Seismology Committee recommends Eq. 1 for determination of transverse reinforcement in both normal and HSC columns and recommends the adoption of Eq. 1 or a similar one by ACI and ITG.

ITG reports that the current recommendations contained in ACI 318 would be applicable to HSC beams. Because research on HSC frame beams is limited, the SEAOC Seismology Committee recommends that the above SMF beam recommendations for normal strength SMF beams also be used for HSC SMF beams.

ITG reports that tests confirm that the provisions contained in ACI 318 for shear strength of interior joints are adequate for concrete strengths of up to 15 ksi. C4 suggests an upper bound of 1500 psi for joint shear in HSC columns, based on the testing indicating a plateau at this approximately this level. This plateau corresponds to a maximum concrete strength of 10 ksi using the ACI 318 limit of $15\sqrt{f'c}$ for the shear strength of the interior joints.

ITG also concludes that the current ACI 318 recommendations are adequate for exterior joints of HSC. This is partly based on a study by Saqan and Keger (1998) of testing done in Japan for concrete strength of 6 ksi to 15.5 ksi. The maximum joint shear in this test was calculated based on drift ratios of 2%. While this is a drift limit set by the codes, the inelastic deformation in the members will likely exceed this limit as a result of a large seismic event, as is discussed in ITG section 5.4. Thus the performance of the joints beyond the 2% drift is in need of further research.

C4 reports that increased confining reinforcement in the joint does not yield increased shear capacity of the joint but does enhance the post-elastic deformation capability of joints. ITG reports similar results based on testing by Noguchi et al. (1998). SEAOC Seismology believes that given these studies it would be prudent to well confine joints to enhance joint performance, particularly in light of increased axial loading on the columns.

C4 reports that on lightly loaded joints with $P \leq 0.1f'cA_p$ it may be advantageous to balance deformations between the joint and the beam to distribute the potential concentration of damage in the hinge region of the beam. C4 suggests that an overstrength factor of 1.1f_y may be used for the beam longitudinal reinforcement for this
purpose as opposed to the traditional overstrength factor 1.25f\textsubscript{y}. SEAOC Seismology recommends that the joint overstrength factor be based on the 1.25f\textsubscript{y} factor. ITG also discusses the development of the reinforcement in the joints and suggests that the current provisions in ACI 318 will not prevent bond slip. ITG indicates that slippage in the earliest stages of the cyclical loading is possible even under more stringent requirements than those in ACI 318. Noguchi et al. (1998) as reported by ITG concluded that specimens with HSC and high-strength reinforcement demonstrated a significantly reduced ability to dissipate energy compared with beam-column joints of normal-strength concrete. ACI 352R-02 presents a discussion of beam column joints.

ITG provides a recommendation for determination of shear strength in HSC members. It reports that the minimum shear reinforcement by equation (11-13) of ACI 318 is appropriate for HSC. ITG emphasizes the importance of confinement in preventing rapid shear strength degradation in members subject to inelastic range of response. ITG also cautions that in some instances the compression in a load reversal may be insufficient in closing cracks formed when concrete and reinforcement are tensioned. This reduces the strut in the concrete or renders it ineffective. As a result, it is suggested that the strut factors developed on the basis of monotonic loading need to be adjusted for seismic design. ITG stresses use of confinement and closely spaced hoops to limit the width of the cracks under tension and to provide confinement to concrete in the struts.

By comparison, C4 suggests a shear strength equal to the provided confinement pressure in columns or specifically, 

\[ V_y = 0.09 f_c A_{core} \leq 0.9 A_{core} \]

This is based on the idea that the pressure induced on the sides of a column section due to shear and axial load must remain within the available confinement pressure for the column. C4 suggests the adoption of ACI 318 Eq. (21-5) for determining confining reinforcement in columns. Eq. (21-5) of ACI 318 in essence provides for a confinement pressure of 0.09f\textsubscript{c} for the core of the column confined by transverse reinforcement.

Development of Bars in HSC. ITG discusses the development of reinforcement in HSC and provides design recommendations for that purpose. According to ITG, the development lengths per ACI 318 for concrete strengths greater than 10 ksi may be un-conservative. Two ways are cited in ITG for addressing this condition: increasing the development length or adding transverse reinforcement to enhance splicing so development lengths obtained from ACI 318 may be used. ITG suggests the latter approach and provides design recommendations accordingly on the basis that the addition of transverse reinforcement improves the behavior of the spliced or developed bars. This will require the addition of transverse reinforcement over the splice length of bars located outside the compression zone of boundary elements in shear walls. ITG also indicates that the length of hook bars per ACI 318 may be unconservative for concrete strengths larger than 10 ksi. ITG reports that experimental data for development of hooked bars in HSC is limited. The specimens tested all indicated splitting of the side cover as the failure mode. As a result ITG provides design equations as the failure mode. As a result ITG provides design equations for determining the development of hook bars in HSC in section 7.2.

HSC Shear Walls. ITG discusses HSC shear walls in Chapter 9. Many of the recommendations contained in ITG are based on the study by Wallace (1998) and on research performed in Japan. ITG suggests that the design provisions for the detailing of the boundary elements for slender walls in ACI 318 are adequate for HSC shear walls. However, ITG highlights the significance of the minimum amount of longitudinal reinforcement in HSC shear walls in preventing failure due to rupture. This is particularly of importance given that the depth of the neutral axis decreases with higher concrete strength, and the strain demand in the reinforcement also increases. Referencing Wallace (1998), ITG also indicates that shear strength equations in ACI 318 become less conservative with an increase in the transverse reinforcement in walls with low aspect ratios. For high amounts of transverse reinforcement, the equations in ACI 318 were found to be unconservative. In particular, the Wallace study reportedly indicated that for HSC walls with \( \rho / f_c \geq 0.08 \), the current ACI 318 equations are unconservative and reported that in HSC shear walls the shear strength was not sensitive to the amount of transverse reinforcement. Wallace also determined that equations suggested by Wood for determining shear strength provide a uniform ratio of measured to calculated strength. ITG also references the work of Kabeyasawa and Hiraishi (1998), which reportedly indicates that walls designed to fail in flexure were capable of deformations past the yield point of the reinforcement but that energy dissipation in walls was low, as indicated by hysteretic loops. Reportedly, the equivalent damping coefficient in the HSC shear walls was in the range of 5-8% in comparison to about 20% for
normal-strength walls. They also reported that in walls designed to fail in shear, those with lower levels of transverse reinforcement failed after yielding of transverse reinforcement. The strength of these was safely estimated by the Japanese provisions. Walls with high amounts of transverse reinforcement failed due to the crushing of concrete, and their strength was overestimated by the Japanese provisions. The shear strength provisions in the Japanese code are based on a strut-and-tie model. Given the above discussion, SEAOC Seismology Committee recommends that for calculating the nominal shear strength limit, $8 \sqrt[4]{f'_c}$ (ACI 318 Section 21.9.4.4), the specified compressive strength of concrete shear walls in regions of high seismicity be limited to approximately $f'_c = 10$ ksi until further research and study provides more insight and recommendations for the design of HSC walls.

Pending further research and investigation, the committee recommends that transverse tie requirements in the boundary elements of shear walls be calculated based on current ACI 318 code, Ch. 21 requirements.

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**Keywords**
- concrete
- reinforced concrete

**How To Cite This Publication**
In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Introduction

The reinforced concrete tilt-up building is the most popular form of light industrial and low-rise commercial construction in the Western United States and is a significant portion of new construction nationwide. This popularity is primarily driven by the construction speed and economical nature of tilt-up construction. Architectural acceptance has become more widespread as tilt-up construction has adapted to the demands of taller buildings with better aesthetics involving irregular plan shapes, more glass, and accent treatments. Tilt-up construction is becoming more common for office buildings, assembly occupancies, and even schools. Originally a one-story form of construction, tilt-up buildings are now commonly two and three stories.

Unfortunately, the uniqueness and rapid evolution of tilt-up construction have made it a challenge for seismic provisions in building codes to keep pace. Poor performance in past earthquakes has been responsible for significant revisions to building codes and in some jurisdictions mandated seismic retrofit requirements of older tilt-up buildings. This article describes unique problems associated with tilt-up seismic design, how the past has shaped current recommended practice, and insights from recent research on tilt-up building behavior.

Tilt-up buildings consist of reinforced concrete wall panels that are formed, cast, and cured on the building floor slab or adjacent “waste” slab and then tilted up into a vertical position with a lifting crane. Surrounding the perimeter of the building, these concrete wall panels are typically between six and twelve inches thick, and are both gravity load-bearing and seismic force-resisting. Once the walls are in place and temporarily braced, a roof structure is erected consisting of either metal decking or panelized wood sheathing (structural-use panels) over steel or wood framing members. The most popular framing system in use today is the “hybrid” roof system consisting of a panelized wood roof of OSB (oriented strand board) and 2x4 framing, supported on factory installed wood nailers attached to the top chord of open-web steel joists (trusses). The roof structure is connected to the interior wall face, allowing the walls to extend above the roof as a parapet. See Figure 1.

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<tr>
<th>ASCE 7-05 reference section(s)</th>
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Figure 1. Typical structural features of a tilt-up building
The most critical component of tilt-up building seismic performance is the anchorage of the wall to the roof structure. This anchorage is accomplished with embedded concrete anchors or straps attached to the roof framing system. Finally, the wall panels are connected to the slab-on-grade through lapped reinforcing within a pour strip or fill-in strip of concrete floor. It is also strongly recommended that the wall panels be connected to the foundations in high seismic zones to provide a positive load path.

**Historical Background**

Many seismic code provisions that have evolved over the years envision either classic frame structures with rigid diaphragms or light frame buildings with small lightweight diaphragms. Tilt-up construction is unique with its many different material combinations and has unique performance issues that have not generally entered the code until evidence of a problem from a failure. Tilt-up construction has experienced many earthquake-induced failures in the past (EERI 1988, SEAONC 2001). The tilt-up provisions that do exist are scattered throughout the code, primarily because tilt-up does not fit neatly into one material type. IBC Chapter 16 (Structural Design), Chapter 19 (Concrete), Chapter 22 (Steel), and Chapter 23 (Wood) all contain important provisions for the design of the various materials associated with diaphragms, collectors, and wall anchorage, and with the main lateral force-resisting system.

Even though the code has become more prescriptive, there are many aspects of design that vary significantly between engineers. There still is no clear direction provided on some important engineering issues such as the following:

- How to properly tie into a re-entrant wall;
- How shear loads are distributed along a line of perforated shear walls;
- Whether wall ties should be designed for compression or tension only.

**Walls Loaded Out-of-Plane.** Prior to the development of slender wall code provisions in the 1980s, concrete walls were limited to arbitrary height/thickness (h/t) ratios creating walls much thicker than typically seen today. It was believed that very slender walls could buckle prematurely or deflect so much as to not be serviceable after earthquakes.

To justify thinner walls, panels were joined together with cast-in-place pilaster stiffeners formed between adjacent panels. The concrete wall panels spanned horizontally out-of-plane between the pilasters, and the pilasters spanned vertically between the floor slab and a supporting roof beam. Horizontal concrete panel reinforcing extended into the cast-in-place joint, effectively making all the panels behave monolithically.

In the 1970s, engineers began experimenting with wall panel designs spanning vertically between the floor slab and roof, leaving the panel joints dry and caulked with waterproof sealants. Pilasters cast with the panels were still provided (usually on just one panel edge), but only for the purpose of providing roof beam gravity load reactions. The panels were now themselves spanning vertically without the reliance on the pilasters, behaving as tall, slender walls. ACI 318 limited bearing wall slenderness to a height-to-thickness (h/t) ratio of 25, and these tall, slender walls did not conform to the typical UBC or ACI code provisions for concrete walls. The slenderness restriction prevented economical use of vertically spanning tilt-up panels.

Instead, engineers began experimenting with new analysis techniques to include second order effects, or $P\Delta$ moments, as a means to circumvent the arbitrary h/t limits prescribed by ACI. Inadvertently, many engineers used a moment magnification method in ACI 318-71 to account for these second order effects, but this method was not developed for flexural members with only a central layer of steel. Another approach used was to conduct rigorous strain compatibility reviews or to use published papers such as the 1974 report, *Tilt-Up Load-Bearing Walls—A Design Aid*, by the Portland Cement Association (Kripinarayanan 1984). Even though very thin wall panels were being erected successfully in the 1970s, there was growing concern over the engineering fundamentals behind the analysis of these walls.

In response to the explosive growth of tilt-up construction that was based on potentially misapplied code provisions, the Structural Engineers Association of Southern California (SEAOSC) published in 1979 their *Recommended Tilt-
Up Wall Design, also known as the “Yellow Book” (SEAOSC 1979), and was quickly followed in 1982 by the “Green Book” titled Test Report on Slender Walls (ACI-SEAOSC 1982). Based on the work of SEAOSC and the Southern California Chapter of ACI, this important publication contained the results of thirty full-scale tests of slender walls under out-of-plane loading (twelve slender concrete walls with the others being masonry), and it confirmed the recommended design provisions of the “Yellow Book” but without arbitrary height-to-thickness (h/t) limits.

These provisions recognized that with light axial loads, tilt-up wall panels behave more like flexural members than compression wall sections, thus eliminating the need for arbitrary height-to-thickness limits. Both P-delta effects and eccentric loading were deemed very important considerations, because slender panels are capable of undergoing large out-of-plane deflections, and the provisions provided equations that estimated the non-linear deflection characteristics out-of-plane. Also, the flexural reinforcement ratio and axial loading were restricted to ensure ductile flexural yielding while mitigating sudden buckling collapse. One of the inherent advantages of using what became known as slender wall design is the reasonably thinner wall panels, which in turn reduce the seismic force at the roof diaphragm. Even though the SEAOSC provisions were not incorporated into the UBC until 1988, engineers were quick to embrace these guidelines in their designs.

The very large deflections observed in the testing program raised serviceability concerns with the SEAOSC/SCCACI task committee. Slender walls designed to strength requirements alone, free of h/t ratios, could be overly flexible, possibly resulting in permanent deformation. On several of the full-scale test specimens, rebound studies were conducted and it was found that some permanent deformation was possible in wall panels prior to reaching theoretical yield. Quoting the Green Book, “The tests demonstrated that there was no validity for fixed height-to-thickness limits, but they did reveal the need for deflection limits to control potential residual deflection in panels after service loads experience.” Based on their limited rebound study and much discussion, the SEAOSC/SCCACI task committee recommended an L/100 deflection limit.

The recommended provisions of the Yellow Book and Green Book were the basis of the first building code requirements in the 1988 UBC. The one important difference between the Green Book and the UBC was the deflection limit at service loads was more restrictively set at L/150. This was set by consensus opinion during the 1984-1986 UBC code development process, but it is not clear what rational basis there was behind the revision.

Another important aspect of both the Green Book and UBC equations was defining the concrete cracking moment, $M_{cr}$, based on the modulus of rupture, $f_r = 5\sqrt{f'_c}$, of concrete. This is two-thirds of the traditional ACI equation $f_r = 7.5\sqrt{f'_c}$, but it matched the full-scale test data far better. By uniquely defining $M_{cr}$ and applying a bilinear curve equation, the UBC load-deflection curve matched well with the test results. See Figure 2. These equations continued to be incorporated up through the 1997 UBC.

Historically, the out-of-plane seismic performance of slender tilt-up wall panels has been relatively good, with failures of a panel itself very rare. Often the worst out-of-plane loading occurs during crane lifting in the construction process. In the 1987 Whittier Narrows, California earthquake, some out-of-plane damage was observed at the sides of large wall openings, but the damage was associated with post-construction saw cut openings installed without strengthening or engineering (EERI 1988).

Today, out-of-plane tilt-up wall panel design is incorporated into ACI 318-05 Section 14.8, and is still largely based on the original SEAOSC testing and recommendations.

The lateral force coefficient for out-of-plane structural wall forces for Seismic Design Category B and above is provided in ASCE 7-05 Section 12.11.1 as $F_p=0.40S_p\delta/W_n$. Using the lateral force coefficient to determine the out-of-plane wall forces, an engineer normally selects vertical design strips to determine moment and reinforcing requirements. At wall piers between wall openings and panel joints, the design strip usually is the entire pier width, with loads accounting for the increased seismic tributary loading associated with the panel portions above and below.
the openings. This approach typically neglects any additional strength or stiffness provided by the portions of the wall above and below the openings, using a simple strip of uniform width as an analytical tool.

Cantilever parapets that are part of a continuous wall element are checked separately for higher seismic forces per ASCE 7-05 Section 13.3.1. Unlike the structural wall panels, the parapet forces are based on nonstructural component equations. It is not appropriate to use the higher cantilever parapet forces to offset the wall positive bending moment below the roof.

Whereas the UBC equations checking strength remained essentially in concert with the ACI 318 adoption, the service level deflection equations were significantly altered. The most significant difference was use of Branson’s equation for the effective moment of inertia, $I_e$, in ACI in place of the UBC bilinear load-deflection equation. In addition, the value for $M_{cr}$ within Branson’s equation was set at the traditional ACI value.

\[
I_e = \left( \frac{M_{cr}}{M_u} \right)^3 I_g + \left[ 1 - \left( \frac{M_{cr}}{M_u} \right)^3 \right] I_{cr} \leq I_g
\]

Where,

\[
M_{cr} = S f_r = S \left( 7.5 \sqrt{f_c'} \right)
\]
There was concern within SEAOSC that the fundamental equations developed as a result of their landmark testing program in the early 1980s had been significantly revised by ACI. In addition, the ACI 318 Commentary continued to reference the SEAOSC experimental research partially as the basis for these new equations. In response, SEAOSC formed a Slender Wall Task Group in 2005 to conduct a comprehensive review of the original 1981 test data and determine the validity of the UBC and ACI approaches.

The SEAOSC Slender Wall Task Group reported their findings in its UBC 97 and ACI 318-02 Code Comparison Summary Report (SEAOSC 2006). The Task Group concluded that the 1997 UBC equations match the test data well, yet the ACI 318-02 equations do not correlate well with the test data and typically underestimate the service load deflections. Figure 3 provides a typical comparison of the UBC and ACI equations with the original test data of one slender wall specimen. The other wall specimens had similar comparisons. The ACI equations for service level deflection fail to properly estimate the cracking moment and the bilinear nature of the load-deflection curve.

The current ACI 318 slender wall provisions have been revised for ACI 318-08 resulting in service level deflection equations more in line with that of the 1997 UBC.

The reason why concrete slender walls behave so differently than predicted by traditional ACI equations has not been clear until recently. Neither the Yellow Book, the Green Book, nor the SEAOSC Slender Wall Task Group discuss any theory behind the lower cracking moment $M_{cr}$ or the bilinear moment-deflection equation unique to slender walls. Research conducted in Australia and Canada has provided good explanations to the disparity between traditional ACI deflection theory and the full-scale slender wall tests (Lawson 2007b).

The conflict between $M_{cr}$ observations and the ACI $M_{cr}$ equation is associated with internal concrete shrinkage stresses first investigated in deflecting flat slabs (Scanlon and Murray 1982). Normally, beam specimens tested for modulus of rupture are unreinforced and have no restraint, allowing free shrinkage. Once reinforcing is added, shrinkage is partially restrained as the reinforcing goes into compression, causing surface tension stresses to develop.
in the concrete. These pre-existing tension stresses cause reinforced members to crack earlier than expected in plain concrete. Flexural members with low reinforcement ratios such as tilt-up slender walls are especially sensitive to shrinkage restraint stress, thus significantly reducing the effective cracking moment (Scanlon and Bischoff 2007, Gilbert 2001).

The stiffness of members with only a central layer of reinforcing or that are lightly reinforced depends greatly on the tensioning effects of the plain concrete near the surface. Once this concrete cracks, there is usually an abrupt change in stiffness. Without consideration of the shrinkage effects, stiffness can be significantly underestimated and thus unexpected deflections can occur under service level loads. Other building codes around the world already have similar provisions for a reduced effective cracking moment or modulus of rupture due to shrinkage restraint, including the Australian Standard for Concrete Structures AS3600, the Canadian Code CSA A23.3 (for slabs), and the Eurocode. A comparison of these codes in conjunction with available testing data indicates that use of 0.67MKr or use of fcr = 5f'c in typical slender wall design is appropriate instead of the current ACI 318 (Scanlon and Bischoff 2007, Lawson 2007b).

Bischoff identified significant limitations with Branson’s equation for Ie when applied to thin concrete members with a central layer of steel, and he has proposed a solution. Branson’s Equation, first published in 1963, was based on larger test beams where the ratio of gross/cracked moment of inertia (Ig/Icr) was typically 2.2. Bischoff found that Branson’s equation became a poor predictor of deflection when this ratio exceeded three (Ig/Icr > 3). Slender concrete walls are far above this, with common values ranging from 15 to 25 for single layer reinforced walls and 6 to 12 for double-layer reinforced walls. Thus deflection is under predicted.

Bischoff has proposed a new equation as a replacement to Branson’s for effective moment of inertia. This equation matches well with both larger flexural beams and thin slender walls, effectively transitioning seamlessly to a near bilinear load-deflection curve at high Ig/Icr ratios.

\[ I_e = \frac{I_{cr}}{1 - \eta \left( \frac{M_{cr}}{M_u} \right)} \leq I_g \]

where \[ \eta = 1 - \frac{I_{cr}}{I_g} \]

Use of Bischoff’s equation with a reduced 0.67MKr provides a single approach to computing reasonably accurate deflections in lightly reinforced slender tilt-up walls (Scanlon and Bischoff 2007, Lawson 2007a). Figure 4 compares the load-deflection curve using ACI 318-05, replacing Branson’s Ie with Bischoff’s Ie, and replacing the ACI Mcr with 0.67MKr. This provides a much better fit than the current ACI 318-05 deflection equation. While this proposed curve is nearly parallel to the test data, it appears to consistently overestimate the service level deflections. This discrepancy maybe attributed to the panels being tested at 160 days instead of reaching ultimate drying shrinkage and thus a resulting lower Mcr.

Reveals, recesses, and form-liner surface treatments must all be considered as they reduce the wall section net thickness and flexural depth of the reinforcing. Narrow, reasonably shallow reveals that only occur occasionally across the panel height often can be justified as not significantly affecting the panel stiffness for determination of out-of-plane deflections, but these still must be included in the moment strength analysis if they exist at critical design sections.
The equations in ACI 318-05 Section 14.8 are very straightforward and can easily be written in spreadsheet form for the engineer’s own use. Several commercially available software packages are available on the market for designing or analyzing these walls. The widely available programs make basic assumptions associated with wall openings and are to be considered only estimates, but are appropriate practice. The true out-of-plane behavior of a tilt-up wall panel with openings is more like a two dimensional flat slab with penetrations requiring use of general-purpose finite element programs to obtain more accurate results. However, the inherent changes in concrete stiffness associated with flexural cracking due to the various seismic stresses and original lifting stresses makes the likelihood of a very accurate finite element analysis remote and therefore not warranted.

![Figure 4. Moment-Deflection Curve Comparison with Modified Ic and Mcr Values](image)

Engineers using finite element programs to design panels are cautioned to research the underlying assumptions and equations. Much of what is known about slender wall behavior is based on empirically derived equations from test data. Computer results from programs based entirely on theoretical formulas may be in error.

**Wall Anchorage, Subdiaphragms and Continuity Ties.** In the mid 1960s, engineers who worked closely with contractors began using wood ledgers as the default wall anchorage with anchor bolts clustered around wood purlins. This resulted in no positive direct tie anchoring the perimeter concrete wall panels to the supporting roof structure. Instead, the roof plywood sheathing was simply nailed a wood ledger which was bolted to the inside face of the wall panels (SEAONC 2001, chapter 3). 2x subpurlins and 4x purlins were supported from the ledger by metal hangers. The glulam beams and occasionally tapered steel girders were supported on top of the pilasters and had seat connections with minimal tie capacity. This indirect tie arrangement relied upon the wood ledger in cross-grain bending, a very weak material property of wood.
In the 1971 San Fernando Earthquake, tilt-up buildings performed poorly. Many wood ledgers split length-wise due to cross-grain bending loads, and plywood edge nailing pulled through plywood panel edges as the result of wall anchorage tension loads. Partial roof collapses and wall collapses were common in the areas of strong ground motion (NOAA 1971, SEAONC, 2001). It was clear more restrictive code requirements would be needed.

Beginning with the 1973 UBC, the requirement for positive direct wall ties was introduced and wall anchorage using wood cross-grain bending was expressly prohibited. In addition, continuous cross-ties were introduced for concrete and masonry walls supported by roof systems. In order to transfer seismic forces from the heavy perimeter walls into the main roof diaphragm, continuous ties or cross-ties are necessary to drag the load uniformly across the diaphragm depth.

In the 1976, UBC, the concept of subdiaphragms was introduced as an analytical device for transferring forces from the individual wall ties to the continuous cross-ties (Sheedy and Sheedy, 1992). Instead of creating a continuous tie at every wall anchorage location, continuous cross-ties can be placed at wider spacing using subdiaphragms. Subdiaphragms are portions of the main diaphragm that span between the continuous cross-ties and gather the wall anchorage loads and transfer this load to the cross-ties. Once the load is collected into the continuous cross-ties it is distributed across the main diaphragm for further distribution to shear walls and frames of the building.

In the 1979 UBC, the wall anchorage design force was increased 50% from 0.2Wp to 0.3Wp (where Wp is the tributary wall weight) for seismic zone 4. The code improvements of the 1970s were not tested until the 1984 Morgan Hill, California Earthquake. Even though only a few tilt-up buildings saw moderate levels of shaking, it provided the first performance comparison of tilt-up buildings built before and after the 1973 and 1976 code changes. Several 1960s era tilt-up buildings suffered wall anchorage or continuity tie failures. On the other hand, a tilt-up building built to more modern codes saw no structural damage despite the estimated 0.3g-0.4g ground accelerations (EERI 1985).

The Whittier Narrows Earthquake in 1987 tested significantly more tilt-up buildings than the Morgan Hill Earthquake, but forces were still only moderate. Pre-1973 UBC buildings saw failures at the wall in ledger cross-grain bending, cross-grain tension splits at interior diaphragm areas, and diaphragm nailing pulling through the plywood edges, similar to what was observed in the San Fernando Earthquake. In contrast, the tilt-up buildings designed under more modern codes performed much better. Despite the better performance of the more modern codes, there was some concern whether the code provisions were adequate to meet performance objectives under very strong levels of shaking (EERI 1988).

Research on instrumented rigid wall buildings with flexible diaphragms following the 1984 Morgan Hill (Çelebi et al. 1989) and the 1989 Loma Prieta Earthquakes (Bouwkamp, Hamburger, and Gillengarten 1991) indicated that the dynamic response of structures with predominantly solid walls is dominated by the diaphragms, amplifying roof accelerations and corresponding wall anchorage significantly more than previously thought. The recorded amplification of ground accelerations to roof accelerations resulted in the basis of a 1991 UBC revision increasing wall tie forces another 50% to 0.45Wp, for seismic zone 4 (Sheedy and Sheedy 1992). This higher wall tie force was only required at the center half of the diaphragm span.

The 1994 Northridge, California Earthquake was the first test of modern post-1979 building code tilt-up provisions under very strong shaking. Hundreds of tilt-up buildings were severely damaged with partial roof collapses (CSSC 1995, Brooks 2000). Damage in pre-1973 buildings was not necessarily a surprise, but the numerous wall anchorage failures in post-1973 code buildings were troubling.

The unexpected wall anchorage damage to newer buildings was primarily attributed to two main reasons: inadequate connection overstrength for the roof accelerations and excessive deformation of the wall anchor system (APA 1994, EERI 1996). Light-gauge steel twist straps were especially a problem due to their geometry and limited overstrength (Harris et al. 1998). Research and post-earthquake investigations have shown that rooftop accelerations may be three to four times the ground acceleration, and insufficient overstrength or ductility has been provided in past connection practices (Çelebi et al. 1989, Harris et al. 1998). Based on observations of Northridge Earthquake damage, it was
deemed best to resist brittle failure through the use of significantly higher design forces in conjunction with anticipated material overstrength instead of reliance on ductility. As a result, wall anchorage forces at the roof were increased from $0.45W_p$ to $0.88W_p/1.4=0.63W_p$ (ASD) and the load was applied over the entire span of the diaphragm in the 1997 UBC. Material-specific load factors (1.4 for steel, 0.85 for wood, 1.0 concrete/masonry) were specified to obtain uniform overstrength within the connection to resist the demand from maximum expected roof accelerations of $4C_a$. This approach is well documented in the 1999 SEAOC Blue Book, *Recommended Lateral Force Requirements and Commentary* (SEAOC Seismology Committee 1999, Section C108.2.8.1).

As further evidence of the intent of these wall anchorage provisions, SEAOC stated that the reduced $R_p$ value for nonductile and shallow anchorage in the 1997 UBC does not apply to wall anchorage designed using this overstrength approach (SEAOC Seismology Committee 1999).

No prescriptive deformation limits of the wall tie system have been introduced into the Code, however the compatibility of anchorage system flexibility and diaphragm shear nailing must be considered. Wall anchorage systems with too much flexibility will inadvertently load the wood sheathing edge nailing and either pull the nails through the edge or place the ledgers in cross-grain bending or tension. Pre-manufactured strap-type wall ties are designed to limit the maximum deformation to 1/8 in at their rated allowable load based on legacy ICBO Acceptance Criteria 13 (ICBO 2002), and pre-manufactured hold-down devices using anchor rods could allow even greater deformation. Engineers should contact the device manufacturer for additional deformation information. The hold-down device flexibility is solely within the steel component itself and is additive to other sources of deformation. Additional deformation can be contributed by other anchorage components (e.g. bolts and nails) and installation practices (e.g. oversized holes).

In 2001, the City of Los Angeles adopted a more stringent requirement that limits the deformation of the wall anchorage to 3/8" under $3F_p$ loading, with no more than 1/8 in occurring in the steel connector itself. The 3/8 in deformation limit includes contributions from slip in nails, bolts, or screws; wood shrinkage; deformation of steel, concrete, and wood components; and inelastic deformation in the anchor connector between the wall and the attached framing member (LADBS 2002). The intent is to rationally limit the deformation under maximum expected wall tie forces at the roof level ($3F_p$) to a dimension equal to the minimum nail distance to the sheathing edge (3/8 in). It is recommended that new editions of the building code limit seismic wall anchorage deformation using a similar approach.

**Current Wall Anchorage Provisions**

ASCE 7-05 Section 12.11.2.1 governs wall anchorage design for most of the tilt-up buildings in California, where Seismic Design Category C and higher applies for bearing walls. In the development of ASCE 7-05, the intent was to maintain the same wall anchorage forces as in the 1997 UBC for flexible diaphragms in high seismic zones. Substituting $C_a = 0.4S_{ds}$ (BSSC 2004b), it can be confirmed that ASCE 7-05 Eq. 12.11-1 is generally equivalent to the 1997 UBC.

The wall tie forces of $F_p=0.8S_{ds}W_p$ for flexible diaphragms are double the normal wall design force in Section 12.11.1 and four times the typical tilt-up building base shear to account for the expected rooftop amplification associated with flexible diaphragms. A steel material load factor of 1.4 is applied to the wall tie forces to reach necessary levels of material overstrength as in the past 1997 UBC Section 1633.2.8.1. This steel component factor applies to the wall anchorage system, and because the wall anchorage system is defined to include the wall ties, subdiaphragm, and continuity ties, the 1.4 factor applies to all steel components and members in these subsystems. Steel components governed by wood capacity, such as nails and bolts in shear, are not subject to the 1.4 multiplier because of the greater material overstrength available from wood (Harris et al. 1998). The 0.85 material load factor for wood components in the 1997 UBC Section 1633.2.8.1 is neither a part of the ASCE 7-05 nor the 2006 IBC provisions. Thus ASCE 7-05 will result in a more conservative design of the wood portion of the wall anchor system than the 1997 UBC.
To summarize, the ASCE 7-05 implements very high wall anchorage force levels to achieve uniform protection against brittle failure without reliance upon ductility. This was achieved using a rational approach considering inherent overstrength of various materials. Through an unrelated parallel effort, ACI 318-05 has inadvertently combined the elevated force levels of the overstrength approach in ASCE 7-05 with new ductility requirements for seismic anchorage in ACI 318-05 Appendix D Sections D.3.3.4 and D.3.3.5. These ductility requirements were inadvertently added on top of the elevated anchorage force levels in conflict with the original intent of the ASCE provisions.

Furthermore, 2006 IBC Section 1908.1.16 allows an additional 2.5 load factor on top of already elevated anchorage forces in ASCE 7-05 in lieu of the ACI ductility requirement. The IBC 2.5 load factor or ACI ductility requirement used on top of elevated wall anchorage provisions of ASCE 7-05 Section 12.11.2.1 is not appropriate and may be difficult to achieve in tilt-up wall anchorage design.

At non-flexible diaphragms, wall anchorage provisions are provided under ASCE 7-05 Section 12.11.1 resulting in $F_p=0.40S_{ds}IW_p$. This value for $F_p$ is significantly lower than that in flexible diaphragms due to the lesser amplification associated with that construction. Seldom are tilt-up wall panels defined as nonstructural walls (walls that have less than 200 plf superimposed gravity load and are also not lateral force-resisting walls). Anchorage of nonstructural walls to non-flexible diaphragms is designed under ASCE 7-05 Section 13.3 with additional provisions of Section 13.5.3. At nonstructural walls supported by flexible diaphragms, a footnote at the bottom of Table 13.5 references Section 12.11.2 in lieu of the provisions of Section 13.3, making anchorage of structural and nonstructural walls to flexible diaphragms the same.

Wall anchorage forces act in compression as well as tension. Panelized wood roof systems by their very nature are not erected tight against the perimeter wall ledger, leaving a small gap to potentially close during seismic compression forces. This gap is properly a result of casting and erection tolerances of construction. Strap-type wall anchors that have yielded and stretched under tensile forces are vulnerable to buckling and low-cycle fatigue as the gaps close. Cast-in-place anchor rods used in connectors can be checked for compression, but it is important to provide an additional nut against the interior wall surface to prevent the anchor punching through the wall. At steel ledger conditions, often wall anchorage is achieved with steel angle straps that are bolted to the roof structure and capable of resisting compressive forces. Although there have been no failures of wall panels collapsing into buildings, consideration of compressive forces will better maintain the integrity of the wall anchorage tie for tension forces.

Connections that are loaded eccentrically or are not perpendicular to the wall are required to be investigated for any bending and all resulting force components induced by the configuration. The bending induced by single-sided connections combined with the wall tie axial load may overstress the attached wood roof member and be a source of potential failure (Hamburger and Nelson 1999). It is recommended that wall tie connectors be applied symmetrically where possible.

Failures of beam seats or spalling of pilasters have occurred in recent past earthquakes (EERI 1996). Where pilasters occur, ASCE 7-05 Section 12.11.2.2.7 requires consideration of the force concentration due to the pilaster stiffening effect on the wall out-of-plane. The anchorage force at the top of the pilaster is determined by considering two-way bending action of the wall panel. This concentrated force is applied directly to any framing member anchored to the top of the pilaster. Reduction of the typical wall anchorage force elsewhere along the wall is not permitted.

**Anchorage to Wood Diaphragms.** As in previous codes since the 1970s, wall anchorage to wood roof systems is not allowed to depend upon cross-grain bending, nailing in withdrawal, or diaphragm sheathing in tension. Wall anchorage loads are transferred into the main diaphragm with subdiaphragms and continuous crossties. These provisions are a direct result of the poor performance during the 1971 San Fernando Earthquake.

Subdiaphragms are provided under ASCE 7-05 Section 12.14.7.5.1 as an analytical device to provide a rational load path for wall anchorage. Subdiaphragm aspect ratios are limited to $2\frac{1}{2}$ to 1, and this provides sufficient stiffness that the independent deflection between the subdiaphragm and the main diaphragm may be ignored. Tilt-up warehouse
buildings today often have large column spacing of 50, 60, even 70 feet, resulting in very large subdiaphragm spans and corresponding subdiaphragm depths. Pursuant to investigations of damages to wood diaphragms after the 1994 Northridge Earthquake, the joint task force recommended continuous ties at specified spacing to control cross grain tension in the interior of diaphragm and subdiaphragm shear limited to control combined orthogonal stressed within the subdiaphragm. As a result, Los Angeles City and Los Angeles County jurisdictions have taken a conservative approach by limiting spacing of continuous ties to 25 ft and subdiaphragm shears to 300 plf. In a recent update of the Los Angeles Regional Uniform Code Program (LARUCP) as part of adoption process of the 2007 California Building Code, the continuous tie spacing limit is increased to 40 ft and shear value increased to 75% of the code-allowable diaphragm shear in the determination of subdiaphragm depth. The revisions were based on improved performance and standards for diaphragm construction today. (2007 LARUCP.)

The benefit of the 300 plf upper bound on subdiaphragm shear strength is the reserve capacity available for orthogonal effects from seismic loading. Because subdiaphragms are a part of the main diaphragm, they are theoretically subject to shear loads from both orthogonal directions. Consideration of orthogonal loading effects in diaphragm shears is not normal practice today, however this approach may be more rational in lieu of arbitrary shear capacity limits. One such approach worth considering is to limit subdiaphragm shears to 1.0/(1.0+0.3) = 77% of their allowable diaphragm shear value, reserving the remaining 23% for orthogonal effects as discussed in the 2003 NEHRP Commentary Section 4.2.2 (NEHRP 2003).

Research indicates that the dynamic amplification associated with flexible diaphragms amplifies the wall anchorage forces, but this increase is limited by yielding of the roof diaphragm. Under low levels of ground motion, roof diaphragms remain elastic and amplify ground forces 3 to 3½ times, but under strong ground motion levels the amplification is reduced to approximately 2½ times due to nonlinear behavior. This reduction in amplification is beneficial to the wall anchorage system, because system failure is now initiating in the more ductile diaphragm instead of the wall anchorage components. However, this assumes the diaphragm is not excessively conservative. Because tilt-up buildings are often long and narrow, diaphragm designs are more governed by forces in the transverse direction, resulting in conservative overstrength in the longitudinal direction. This results in more elastic diaphragm behavior in the longitudinal direction, and thus larger wall anchorage force amplifications at the narrow ends of the building, with forces possibly exceeding the level of 1.2g anticipated in the current code provisions (Harris et al. 1998).

**Anchorage of Walls to Metal Deck Diaphragms.** Although less common in California than panelized wood sheathing, flexible metal deck diaphragms (without fill) are becoming more common in tilt-up construction in seismically active areas. When designed properly, metal decking can assist in providing wall anchorage and eliminate the need for subdiaphragms by acting itself as the continuous crossties. Important detailing issues must be carefully considered.

Metal deck can only provide continuous crossties parallel to the deck span direction. ASCE 7-05 Section 12.14.7.5.3 specifically prohibits use of metal deck perpendicular to the direction of span for continuity, because the deck flutes will stretch out and flatten. Where the decking is spliced, a common structural member is necessary to receive the attachment from both deck panels. In common steel joist (truss) systems with double top chords, it is necessary that both deck panels be attached to the same individual top chord half, otherwise crosstie loads will be inadvertently transferred through the steel joist (truss) top chord separation plate or web welding, depending on joist web configuration. Another concern at the deck panel splice and direct ledger attachment is the weld tear-out through the metal deck. Proper deck gauge and puddle weld edge distance must be maintained for adequate wall anchorage strength. A better approach is to provide steel angles perpendicular to the wall to transfer wall anchorage into the diaphragm, similar to a wood roof system approach.

Another challenge with metal deck diaphragms is the need for thermal expansion joints. Metal deck roof diaphragms are much more vulnerable to temperature swings than wood diaphragm systems, and with the trend towards larger roof dimensions, thermal expansion joints become more likely. However, these expansion joints interrupt the continuity of the wall anchorage system (crossties) and thus create several independent structural units to be analyzed separately. The wall anchorage forces must be fully developed into the main diaphragm and transferred to
the applicable shear walls before reaching the expansion joint. This results in larger diaphragm shears when compared with wood diaphragms without expansion joints.

If the metal deck is expected to carry wall anchorage forces, it must be investigated for tension and compression axial loads in conjunction with acting gravity loads. The axial compression loads are associated with inward wall forces and require a special axial/bending analysis of the decking. The North American Specification for Design of Cold-Formed Steel Structural Members (AISI 2001) provides design criteria for the decking, and the Structural Steel Education Council (Mayo 2001) illustrates one approach for this wall anchorage.

**Anchorage to Roof Framing.** Whether using a panelized wood sheathed roof or a metal deck roof, steel trusses or joists are now the most common roof framing members in tilt-up buildings in California. This trend began in the early 1990s when rising timber prices increased the cost of traditional wood roof systems. In terms of wall anchorage, the use of steel truss framing is advantageous because steel joists are attached by direct welding at the steel wall ledger and at interior crosstie splice locations. This eliminates the connection deformation or “stretch” problems that contributed to cross-grain bending and plywood edge nailing pulling out of sheathing edges during past earthquakes. Steel joist systems are typically designed by specialty engineers in association with the manufacturer, and the building design engineer is responsible for providing axial wall tie and continuity tie loads to the manufacturer along with information stating which load factors if any have already been applied (IBC Section 2206.2). In conditions where axial loads are transferred through the joist seat, it must be made clear to the manufacturer so that the seat strength can be checked also. There are limits to the amount of load that manufacturers can transfer through these joist seats. The explosive growth of the steel joist system in tilt-up buildings has occurred since the 1994 Northridge Earthquake, and it remains to be seen what level of performance may occur in these buildings when subject to severe shaking.

An additional result of the 1994 Northridge Earthquake was the passage by the City of Los Angeles of a new tilt-up retrofit ordinance. After estimating that one third of the nearly 1200 tilt-up buildings in the San Fernando Valley suffered significant damage in that earthquake, Los Angeles revealed an ordinance that was developed in conjunction with SEAOSC to require wall anchorage and continuity ties in existing pre-1976 tilt-up buildings. A similar ordinance was adopted in Los Angeles County and in other jurisdictions not long after. This ordinance does not attempt to force older tilt-up buildings to comply with current code requirements, but instead aims to obtain levels of performance consistent with acceptable minimum life safety. Additional earthquake hazard reduction information is also available from other publications (LA City 2002; LA County 2002, ICBO 2001, SEAONC 2001).

**Diaphragms**

The most common roof system used in tilt-up construction today in California is the hybrid roof. This consists of wood structural-use panels such as plywood or oriented strand board (OSB) nailed to wood nailers factory installed to the top chord of open-web steel joists. Current tilt-up development trends include larger and taller buildings with more clear-space and clear-height to facilitate warehousing and distribution. These trends are placing more demand on the wood roof diaphragms to span farther horizontally with higher shear stresses. From the 1967 UBC and through the current 2006 CBC, the maximum allowable shear in horizontal wood diaphragms is 820 plf per IBC Table 2306.3.1. The 820 plf table maximum may be exceeded with a special high-load wood diaphragm with multiple rows of fasteners per IBC Table 2306.3.2. In large tilt-up buildings, engineers have relied upon these high-load diaphragm values with shear capacities up to 1800 plf. First introduced as an ICBO Evaluation Service ER document, these high-capacity wood diaphragms are now incorporated into the IBC. Special inspection is required, however many special inspectors do not have experience or certification with this type of inspection. If some doubt exists as to the qualifications of the Special Inspector, it is recommended that a preconstruction meeting be held to clarify the inspection issues.

Less common in California tilt-up buildings is the use of metal deck as the roof diaphragm. These diaphragms are capable of higher shears than wood diaphragms, but concerns of thermal expansion often limit the diaphragm width...
by introducing expansion joints, as noted above. Metal deck diaphragms are capable of reaching over 3000 plf allowable design values with heavy gauge material and special attachments.

The performance of tilt-up building diaphragms will be more critical now as the problems associated with wall anchorage are overcome. With wall anchorage and collector design forces factored up to maximum expected levels, and with perimeter shear walls often consisting of solid wall panels providing excessive lateral strength, ductile yielding of the roof diaphragm is thought to be the likely failure mode in new structures. Unfortunately, damage to these diaphragms is not easily observed until significant separations or partial collapses occur.

**Diaphragm Chords.** Diaphragm behavior is similar to a flat beam, with diaphragm chords acting as beam flanges. With larger roof diaphragms, higher chord forces also occur. In the past, structural chords typically consisted of special reinforcing bars embedded in the wall panels near the roofline connected together with welding across the panel joints. With the disappearance of cast-in-place pilasters or stitch columns in the 1970s, the new dry panel joints resulted in chord connections that were more vulnerable to settlement and concrete shrinkage strains. Distress at chord connections across panel joints was taking on the form of concrete spalling and weld breaks even without earthquakes.

Prior to the late 1980s, reinforcing chord bars were generally of ASTM A615 Grade 60 material, which was vulnerable to weld embrittlement failures (in the welded splices) due to improper preheating. Broken chord connections at these welds were observed in the 1987 Whittier Narrows Earthquake (EERI 1988). Beginning with the 1985 UBC, ASTM A706 reinforcing was introduced for welded locations and seismic frame reinforcing because of the tight controls on carbon equivalency and yield limits for that material. ASTM A706 requires less preheating and results in fewer weld embrittlement problems, and it is thus recommended for welded chord reinforcing. IBC Section 1908.1.5 discourages welding of ASTM A615 reinforcing by requiring a report of material properties from the producer to establish proper preheating. ASTM A706 does not have this requirement.

Steel ledger systems began to appear in the mid 1980s, and these steel channels or angles were welded together across the panel joints to function as diaphragm chords. This circumvented the weld embrittlement problem with reinforcing bar connections, and steel ledgers were observed to have performed very well in the Whittier Narrows earthquake (EERI 1988). Steel ledgers are commonly used today for supporting the roof system and acting as diaphragm chord reinforcing.

Another source of earthquake damage in chord connections is the condition of skewed or angled wall connections. With tilt-up buildings now frequently being placed on irregularly shaped lots, building plans are no longer simply L-shaped or rectangular. Often corners are clipped or angled to accommodate adjacent property lines or easements. Tensile chord forces in flexible diaphragms must resolve themselves around the skewed wall connection, and these forces result in an inward or outward thrust component. This component is seldom resolved properly with a connection into the main diaphragm, and it results in chord reinforcing or steel ledgers being damaged (EERI 1988, EQE 1989).

In large diaphragm systems, the numerous continuity ties required for the wall anchorage system will behave inadvertently as a collection of chord elements across the diaphragm. Based on strain compatibility, the continuity ties all assist in providing a collective chord that distributes chord forces over a larger portion of the roof structure (Lawson 2007c). This approach is closer to the actual behavior of the diaphragm and can result in substantially lower forces in each chord member. For simplicity, engineers typically model chords as a single element at the perimeter of the building.

**Diaphragm collectors.** Significant collector forces are often found in tilt-up construction due to the large open interior spaces, with collectors dragging forces to re-entrant corner shear elements or interior braced frame elements. Transferring the loads from roof framing collectors into the concrete wall panels can be challenging, especially at re-entrant corners that may have two intersecting collectors. Typically, the collector is connected to an embedded steel plate with drag reinforcing used to distribute the load into the shear wall line. The collector loads are normally
dragged across panel joints in order to distribute the collected diaphragm load into sufficient wall length. The collector load in the roof structure is subject to the special load combinations referenced in ASCE 7-05 Section 12.4.3.2.

**In-plane Diaphragm Deflection.** In tilt-up shear wall buildings, diaphragm deflection is computed for the design of building separations and deformation compatibility. The in-plane shear wall drift is typically insignificant compared with the diaphragm deflection and is usually ignored. Also ignored is wall panel out-of-plane deflection when considering building separations. The estimated deflection of plywood diaphragms may be obtained from analysis per IBC Section 2305.2.2. Research indicates that under seismic loading, plywood diaphragms are stiffer than indicated by the conventional diaphragm displacement formulas and subject to less displacement amplification compared to force amplification (Harris et al. 1998). In larger roof systems, many of the continuity ties act inadvertently as collective chords reducing the actual diaphragm deflection (Lawson 2007c). Use of a single chord design model will typically be conservative; however, dynamic diaphragm modeling being used in research could overestimate the diaphragm period and obtain unconservative results. Metal deck diaphragm deflections are computed using formulas furnished in a particular product ICC Evaluation Report.

In tilt-up buildings, diaphragm deflection results in the columns and perpendicular walls rotating about their bases due to diaphragm translation at the top. Assuming the columns and walls were modeled with pinned bases during their individual design, this base rotation is permitted to occur even if some unintentional fixity exists. Unintentional fixity may be the result of standard column base plate anchorage or wall-to-slab anchorages combined with any wall-to-footing anchorages. ACI 318-05 Section 14.8 requires slender tilt-up walls to be tension-controlled and limits the factored vertical concrete stress to 0.06$f'_c$. This allows panels to better accommodate any localize yielding at the base while continuing to carry the vertical loads.

Diaphragm deflection is not normally included in the story drift limits of ASCE 7-05 Section 12.12.1. Story drift limits were developed with the intent to limit the deformation of the basic vertically-oriented elements of the seismic force-resisting system. In tilt-up buildings, these vertical elements deflect very little in-plane, with all of the translation occurring at other elements. Story drift limits do not apply to diaphragm deflection.

Over the past decade, most practitioners have calculated diaphragm deflections only for the purposes of building setbacks from property lines or other adjacent buildings, such as that required in ASCE 7-05 Section 12.12.3. With the larger and more flexible diaphragms being built today, IBC Section 2305.2.2 is becoming more important to consider:

> Permissible deflection shall be that deflection up to which the diaphragm and any attached load distributing or resisting element will maintain its structural integrity under design load conditions, such that the resisting element will continue to support design loads without danger to occupants of the structure.

By intention, this language is not clearly defined, with the approach left much to the engineer’s own judgment. In low-rise concrete or masonry buildings, excessive deflections in horizontal diaphragms can cause overall instability in walls and columns from the $P$-$delta$ effect. Gravity load-bearing walls and columns, when subjected to horizontal translation at the top, will begin to induce a horizontal thrust into the diaphragm, further exacerbating the deflection. Although it was not originally intended to be used to evaluate diaphragm deformations, ASCE 7-05 Section 12.8.7 can be used as to investigate roof stability under $P$-$delta$ effects.

**Diaphragm Re-entrant Corners.** Unfortunately, tilt-up standard practice has often been to ignore the stiffness at short re-entrant corners, with the false belief that the short re-entrant walls were not designated shear walls and would somehow accommodate the diaphragm movement. This mistaken assumption reflects the over-simplification of design made by engineers when analyzing these very flexible diaphragms. Even though deformation compatibility provisions have been in the code since the 1976 UBC, designers have often ignored this provision in tilt-up buildings. Deformation compatibility problems have been documented in the 1984 Morgan Hill Earthquake...
(EERI 1985), the 1987 Whittier Narrows Earthquake (EERI 1988), and the 1989 Loma Prieta Earthquake (EERI 1990), where diaphragm drift tore apart roofs at unintended stiff wall elements, such as at re-entrant corners and where fin walls were located, and racked interior partition walls in contact with either the roof or tilt-up walls.

Today, tilt-up buildings are receiving more attention in terms of architectural design, with city planners and architects articulating the formerly large flat wall surfaces in an effort to make industrial parks and warehouses more aesthetically pleasing. Often this results in numerous re-entrant corners, buttresses, fin walls, or other stiff elements inadvertently resisting the diaphragm drift. The most direct approach to address re-entrant corners is to design shear walls along each line of the corner, thus eliminating any deformation incompatibility issue. This solution unfortunately requires long drag strut or collector lines, and the configuration of the re-entrant corner may not be suitable for introduction of a shear wall at that location. Another approach could be to install an interior shear wall or braced frame element near the re-entrant wall to minimize the diaphragm deflection at the re-entrant wall.

In situations where the re-entrant corners or other stiff elements occur at locations of large diaphragm drift, several other options may be available. Where the re-entrant corner or stiff element depth is very small, an engineer can investigate the option of allowing the element to rock or begin to overturn in a controlled manner. The engineer can calculate the amount of force necessary to rock the stiff element and conservatively attach a diaphragm strut to develop that rocking force into the main diaphragm. This solution prevents roof separations due to stiffness incompatibility, but the designer must pay careful attention to detailing to ensure rocking panels are not inadvertently joined together or overly anchored to the foundation unless the resulting larger overturning force is accounted for. Supplemental vertical supports for major members framing into such re-entrant corners should be considered.

When investigating the rocking of a panel, it is possible that the panel itself is perforated to the point that the wall piers or frame-like members will fail in bending before the rocking relief takes place. In this situation, the wall piers
must maintain their integrity and are detailed under ACI 318-05 Section 21.11 for frame members not proportioned to resist forces induced by earthquake motions. The re-entrant panel is still connected to the roof structure to develop the load into the diaphragm necessary to achieve the pushover capacity of the piers.

Other solutions may involve the use of seismic isolation or expansion joints in the roof structure. Depending upon the configuration, the offending re-entrant elements can be isolated from the main diaphragm movement, however this requires the isolated portions of the building to remain stable on their own.

In ASCE 7-05, there are no provisions that indicate the amount of plan offset or re-entrant depth that these re-entrant corners can reach before requiring investigation. In 2000 IBC Section 1617.4.4.2, shear wall line offsets up to 5% of the building dimension are considered small enough in flexible diaphragms to allow the designer to neglect the offset and design it as one single shear wall line when distributing horizontal diaphragm shears. In the 2003 IBC, this language was removed when ASCE 7-02 became the reference document for this section. ASCE 7-05 does not have this language, other than identifying plan structural irregularities as 15% of the building dimension regardless of diaphragm flexibility (ASCE 7-05 Table 12.3-1). Because of the more critical nature of re-entrant corners in flexible diaphragms, it is unconservative to ignore re-entrant corners measuring up to 15% of the building dimension. It is recommended that engineers address the deformation compatibility issue associated with re-entrant corners where the out-of-plane wall offsets are more than 5% of the diaphragm span dimension perpendicular to the direction of lateral load, as shown in Figure 6.

![Figure 6. Plan of Typical Tilt-up Building Illustrating Guidelines for Re-entrant Corner Considerations in Flexible Diaphragms](image)

**Multiple-Story Diaphragm Compatibility.** Originally, tilt-up buildings were one-story warehouses with an occasional woodframe mezzanine. As these buildings gained greater acceptance for office environments, the mezzanines grew to full second floors and in some instances third floors. Higher-end office buildings are now using
Tilt-up Buildings

concrete floor systems over metal deck, while the roof system remains a flexible panelized wood roof or metal deck roof.

With concrete panels extending full height continuously past the upper floor, diaphragm deflection incompatibilities between the roof and floor diaphragms can lead to panel damage or anchorage failure. This type of damage was first observed in the 1989 Loma Prieta Earthquake where a full-height wall panel was anchored to both the roof and second floor. The panel experienced cracking along the second floor level, indicating the initiation of a horizontal hinge (SEAOC 1991).

For analytical purposes, engineers normally ignore wall continuity and assume the panel hinges at the intermediate floor lines, thus anchoring the out-of-plane wall load based on simple tributary wall areas to the floor and roof levels. In this situation, some cracking and hinging of the panel at the intermediate floor line is anticipated under strong shaking levels, and this is acceptable as long as the axial gravity loads are still characteristically small (limited by ACI 318-05 Section 14.8.2.6) and the wall panel is sufficiently flexible out-of-plane. A worse scenario is if the wall is excessively stiff out-of-plane and the relative roof and floor movements pry the wall anchorage loose causing a localized collapse. In conditions where diaphragm drift significantly varies from floor-to-floor or floor-to-roof, the designer should investigate the wall anchorage capacity for this additional effect by analyzing deformation compatibilities or designing the stability of the floor to be independent of the concrete wall system.

Is Tilt-up Construction Precast or Cast-in-Place?

In the past, tilt-up engineering has generally followed design and detailing provisions as for monolithic concrete. Precast concrete buildings were traditionally considered as structures comprised of numerous small concrete members cast at an off-site plant and transported to the jobsite for assemblage. Individual beam, column, plank, and narrow wall members are traditional precast elements, and ACI 318 precast concrete provisions were developed with that construction type primarily in mind. Generally, the joining of beams to columns for frame resistance or the coupling of narrow wall elements together for composite shear wall resistance represents traditional precast concrete resistance to seismic forces. In traditional precast construction, the individual element has little or no lateral resistance on its own, but relies upon the assemblage to achieve lateral resistance.

In contrast to traditional precast, concrete tilt-up construction consists of panels that are individually stable wall elements that seldom require coupling devices for composite action. Penetrations within walls are surrounded by deep beams above which in turn are monolithically cast to wall pier elements at the sides. The only discontinuities are at the vertical panel joints, often twenty or thirty feet apart, and at the panel-to-footing interface. Historically, the in-plane performance and ductility of this lateral force resisting system is closer to cast-in-place construction than traditional precast.

The ACI 318-05 defines precast concrete in Section 2.2 as a "Structural concrete element cast elsewhere than its final position in the structure." Under this broad definition, tilt-up concrete construction could be considered as site-cast precast. In fact, the tilt-up concrete construction method was first discussed in ACI 318-95 in the Commentary for precast concrete stating, "Tilt-up concrete construction is a form of precast concrete."

In ASCE 7-05 Table 12.2-1, "Design Coefficients and Factors for Seismic Force-Resisting Systems," there are now three categories under the heading for Bearing Wall Systems and for Building Frame Systems that potentially apply to tilt-up buildings as a form of precast concrete construction. These are Ordinary Precast Shear Walls, Intermediate Precast Shear Walls, and Special Reinforced Concrete Shear Walls. The category of Intermediate Precast Shear Walls is new and represents a transition in detailing and expected performance between ordinary and special systems. ACI 318-05 Commentary 21.1 indicates that the Intermediate Precast Shear Wall system is equivalent to a Cast-in-Place Ordinary Reinforced Concrete Shear Wall. However, ASCE 7-05 places height limits on the Intermediate Precast system in Seismic Design Categories D, E, and F, whereas Ordinary Reinforced Concrete Shear Walls are not permitted in Seismic Design Categories D, E, and F per ASCE 7-05: Intermediate Precast Shear Walls are permitted recognizing their additional wall pier detailing requirements and height limits. 2006 IBC
1908.1.8 gives detailing requirements for a special wall pier when design is based on ACI 318 Sec. 21.7, while IBC 1908.1.13 gives requirements for wall pier detailing when design is based on ACI 318 Sec. 21.13. As discussed later under Wall Pier and Shear Wall Classifications, the provision prescribed under IBC 1908.1.8 was introduced by SEAOC in legacy code for high seismic regions (SDC D, E or F), while the later provision prescribed under IBC 1908.1.13 has been introduced by others for lower seismic region (SDC C).

With the lack of the word “precast,” the applicability of the Special Reinforced Concrete Shear Wall category to tilt-up buildings may not be immediately obvious, but a quick reference to ACI 318-05 will clarify this application. The intent of ACI 318-05 toward the proper classification of Structural Walls can be obtained by reviewing the provisions in conjunction with the Commentary. Chapter 21 of ACI 318-05, Special Provisions for Seismic Design, Section 21.1 - Definitions, provides the following guidance. Structural Walls are defined as "Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall. Structural walls shall be categorized as follows.” A Special Precast Structural Wall is defined as "A precast wall complying with the requirements of 21.8. In addition, the requirements for ordinary reinforced concrete structural walls and the requirements of 21.2.2.3, 21.2.3 through 21.2.7, and 21.7 shall be satisfied.” The commentary for this definition states, "The provisions of 21.8 are intended to result in a special precast structural wall having minimum strength and toughness equivalent to that for a special reinforced concrete structural wall of cast-in-place concrete.” Therefore this establishes that a Special Precast Structural Wall is equivalent to a Special Reinforced Concrete Structural Wall.

Development of precast concrete seismic design provisions has been based on extensive research which resulted in development of acceptance criteria described in 2003 in FEMA 450-2, Commentary section 9.6 (BSSC 2004a). Hawkins and Ghosh (2004) provide information on testing research on this topic. Much of the research work has been directed toward investigating the seismic performance of traditional precast concrete structures with improved connections. The landmark research associated with the PRESSS (Precast Seismic Structural Systems) Research Program has greatly influenced the specific ACI 318 seismic provisions for precast concrete systems. ACI 318-05 Sec. 21.8 provisions on special structural walls constructed using precast concrete, however, was not written for tilt-up wall construction (Ghosh and Hawkins 2006).

The development of the special precast concrete system was separate from the over fifty-year development of the concrete tilt-up system. The current code language is ambiguous, because ACI 318 Chapter 21 encourages ductile detailing, including confinement reinforcement and development of tensile reinforcement in high-seismic regions. SEAOC affirms a new system approach for tilt-up system is needed. During the interim, SEAOC affirms proper load path and ductile detailing practice be followed in the design and detailing of tilt-up panels.

**Shear Distribution in Walls Loaded In-Plane**

Complex panel configurations and the wall panels with extensive perforations are relatively recent developments. Originally, designers had ample amounts of solid wall panels to use or “designate” as acting as shear walls. Often, there was enough overstrength along a wall line that little attention was paid to exact force distribution among the wall panels. It was common to simply divide the total wall line shear equally into each panel or proportional to panel length, accounting somewhat for the openings. Even though past codes required that forces be distributed in proportion to element stiffnesses, engineers justified designs by demonstrating adequacy of the collective wall line as a whole. A series of individual panels modeled together with multiple opening configurations and numerous panel joints made modeling too complex without computers for the average engineer.

Today, with computer usage common and essential in the office, engineers have enough computing power to better distribute shear loads along complicated shear wall lines in proportion to individual panel rigidities as described by ASCE 7-05 Section 12.8.4, considering both shear and flexural stiffnesses. Accurate distribution of in-plane shear forces along designated shear wall lines has become quite complex as buildings use more complex panel configurations with numerous openings. The classic conventional rigidity analysis often used for walls in poured-in-place concrete or masonry construction is not necessarily accurate for tilt-up walls unless each panel joint is also considered. The combination of repeating panel joints, various opening arrangements, thickened wall sections,
sloping roof heights and partially cracked concrete properties result in a very difficult analysis if distribution accuracy is paramount. Normally, the distribution may be simplified by assuming average roof heights, average panel thicknesses, and uncracked concrete properties.

Recognizing that stiff shear wall elements could become overloaded, crack, and redistribute their loads to other more flexible wall pier elements (see wall pier discussion below), wall piers are required to be detailed to ensure flexural yield failure unless the sum of the shear wall stiffnesses is at least six times greater than the sum of wall pier stiffnesses (IBC 1908.1.8).

Another possible approach used to further simplify the distribution of in-plane shears is to designate solid panels along a wall line as the primary shear walls for the total wall line shear. This simplifies the distribution by ignoring the more flexible panels and transferring the entire seismic load to a few stiffer and stronger panels through a collector. By ignoring portions of the concrete wall line, these ignored elements may be subject to the provisions of ACI 318-05 Section 21.11 for concrete members not designated as part of the lateral force-resisting system. The special detailing of these “ignored” members will provide more ductile behavior in the event that the solid walls crack or rock under overload and redistribute the seismic forces.

A method used by some engineers can be described as the “flexible-link” approach. This method assumes that the chord connections between panels can be designed to contain enough stretch or flexibility to effectively isolate or buffer the individual panels from each other (Eddington 1990, Brooks 2000). Truly isolated panels would theoretically only see seismic forces from the tributary diaphragm length in contact with the panel and forces from the panel seismic self-weight. The flexible link is a panel-to-panel chord connection that has significantly more flexibility than the differences in flexibilities between panels, so that the chord stretch deformation dominates any relative drift differences between panels. It is common in tilt-up construction to prevent the bonding of the chord reinforcing (or slot the continuous steel ledger bolting) in the vicinity of panel joints to help relieve thermal and shrinkage strains that develop. With this approach, the unbonded chord length that crosses a standard ½ in or ¾ in panel joint is considered the flexible-link buffer. Of course this flexible link must still have sufficient stiffness to be effective against chord forces without excessive diaphragm deflections, and analysis complications occur when collectors at re-entrant corners apply large seismic drag loads to the end panel of a wall line.

As just discussed, there are several available methods to distributing seismic shears within the shear wall system. ASCE 7-05 Section 12.8.4 states that seismic story shears shall be distributed to the various vertical elements of the seismic force-resisting system based on the relative lateral stiffness of the resisting elements and diaphragm. The 2003 NEHRP Commentary (Section 5.2.4) states, “Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm.” Regardless of which assumptions are used to distribute in-plane shears, adequate life-safety protection is expected under the actual distribution as long as a rational distribution approach is used and the detailing requirements associated with wall piers and frame elements are implemented. This is an area that could benefit from further research.

**Wall Pier and Shear Wall Classifications**

Traditionally, tilt-up buildings contained many solid wall panels that easily fit the definition of shear walls. However, as greater architectural demands on tilt-up buildings pushed doors and window closer together and closer to panel joints, the remaining wall piers became narrower and more frame-like. Building codes gave little guidance in classifying whether narrow wall segments were better judged as frames or shear walls, and building departments began to see tilt-up wall-frame like structures being designed under the more relaxed shear wall requirements.

In defining the different types of lateral force-resisting systems, concrete wall-framed type systems such as tilt-up buildings with deep spandrels above repeating sizeable openings have not been recognized and have no assigned seismic response R-factor. Codes prior to 1991 did not include lower-bound limits on shear wall lengths, and did not adequately cover the design and detailing of slender and narrow shear wall segments. Observed earthquake damage in cast-in-place and precast shear walls repeatedly showed distress due to short-column effects in narrow wall piers.
and showed a need for adequate transverse reinforcement. SEAOC Seismology Committee introduced a code change that was adopted into the 1991 UBC. Most of the original provisions sustained transition from UBC to 2006 IBC in defining regions that need special transverse reinforcement in wall piers in order to confine longitudinal wall reinforcement when subjected to in-plane seismic shear force. While IBC Section 1908.1.8 includes the wall pier provisions from the 1997 UBC, similar language has not been incorporated into ACI 318-05.

Wall panels supporting deep spandrels are similar to columns supporting discontinuous shear walls. When a wall segment with the length-to-thickness aspect ratio exceeds 2.5 and the compressive axial force including earthquake is less than $A_{p}f_{c}'/20$, the wall pier design provisions should be met. The design shear force, $V_{e}$, should be determined by considering the probable flexural strength, $M_{p}$, of the wall pier, based on the reinforcement tensile stress of $1.25f_{y}$ and a capacity reduction factor equal to one, and dividing by the clear height of the opening.

The minimum length-to-thickness aspect ratio was chosen to ensure flexural behavior of the wall segment. A wall segment that does not meet the prescribed aspect ratio may be designed as flexural frame members under ACI 318-05 Section 21.3 if lightly loaded axially and if it meets certain geometric limitations, otherwise the more restrictive column requirements of ACI 318-05 Section 21.4 are necessary. In wall lines where only a few elements are classified as columns, engineers have the option to ignore these members and classify them as “frame elements not proportioned to resist forces induced by earthquake motions” as long as they are detailed in accordance with ACI 318-05 Section 21.11.

The special reinforcing provisions for wall piers are not required when those piers under consideration are adequately braced by other wall segments of substantially larger stiffness. Such lateral bracing elements must have lateral stiffness to resist the lateral deflections of the story such that any resulting deflection will not affect the wall pier strength substantially. The ratio for the sum of relative stiffnesses between wall bracing elements to those of wall piers needs to be greater than or equal to six for the wall piers to be considered adequately braced. This was originally proposed in the Blue Book and has been in use in California and in the Uniform Building Code for a considerable amount of time, without incident. Some have questioned this relative stiffness and have suggested using a value of 12 based on a proposed code change in ACI 318 section 10.10.1 on the slenderness effect of compression members. At the present time, however, there are neither tests nor documented data known to the Seismology Committee to justify an increase from the original value of six.

Research done on thin tilt-up frame panels (similar to wall piers) has shown the benefits of close tie spacing in the hinge zone where flexural yielding initiates. Cyclic loading tests of full-scale tilt-up specimens provide insight into the behavior of wall piers that have various tie spacings. Four-inch tie spacing was effective in achieving ductility whereas eight-inch and twelve-inch spacings were not effective, allowing primary flexural steel to buckle (Dew, Sexsmith, and Weiler 2001).

Panel-to-Panel Connections
From its beginnings in the 1940s and into the 1970s, tilt-up wall panels were joined together with cast-in-place pilasters formed between adjacent panels. Horizontal panel reinforcing extended into the cast-in-place joint effectively making all the panels behave almost as monolithic concrete. Since the 1970s, panels have been left separated with dry panel joints, but building designs still require connections across these joints for diaphragm chords and drag struts. In addition, some engineers provide connections across the joints for overturning resistance.

Intermittent connections across the panel joints are vulnerable to force concentrations from concrete shrinkage strains and differential foundation settlement. The concrete panels are often lifted into place after only a week of curing with a substantial amount of concrete shrinkage potential remaining. Panels set on pad footings instead of continuous footings are more subject to differential settlement. This potential for relative movement between individual panels requires connections across these joints to be strong and well developed into the concrete to prevent brittle failure.
Chord connections and drag connections typically involve the use of reinforcing steel or rolled steel shape ledgers directly spliced at the panel joints. To accommodate the shrinkage strains that can occur between panels, the continuous chord members are often detached from the panel in the vicinity of the joint to allow for horizontal movement. In the case of reinforcing chord bars, the reinforcing is often wrapped or otherwise unbonded for a short distance from the panel joint. Where steel angle or channel ledgers are used, horizontally slotted bolt holes near the panel joints are provided. These methods of chord detachment allow the horizontal strains to better relieve themselves elastically across the unbonded chord distance and reduce the likelihood of brittle weld failure or concrete spalling at the connection.

Occasionally, overturning resistance of shear wall panels is supplemented with panel-to-panel embedment connections, effectively grouping adjacent panels together to behave as a whole. Often these connections are non-ductile with insufficient overstrength to prevent brittle concrete cone failure or weld rupture, and are not suitable in high seismic zones (Hofheins, Reaveley, and Pantelides 2002; Lemieux, Sexsmith, and Weiler 1998). Alternatively for overturning resistance, panels can be attached exclusively to the foundation for uplift forces allowing the panels to more freely move at their joints. It is recommended that any panel-to-panel connections for uplift resistance be used sparingly and only if they are well developed into the concrete with reinforcing welds designed to fully develop the reinforcing bars. ACI 318-05 Section 21.8 for “Special structural walls constructed using precast concrete” references Sections 21.13.2 and 21.13.3 which require panel-to-panel connections to yield within the steel elements or reinforcement, or otherwise be designed to develop at least 1½ times the connection yield strength. The SEAOC Seismology Committee recommends that panel-to-panel connections be designed with the Special Seismic Load Combinations per 2006 IBC 1605.4 in lieu of developing only 1½ times the connection yield strength.

Wall Connections to the Foundation
In concrete tilt-up construction, many regional areas have seen tilt-up panels not attached to the spread footing foundation that provides bearing support for the panels themselves. In these buildings, wall panels are vertically supported on the foundation, but lateral forces are resisted only by panel connections to the slab-on-grade. Typically these connections consist of reinforcing dowels from the wall panel and from slab-on-grade that lap within a narrow concrete pour-strip in the floor parallel to the wall panel.

ACI 318-05 Sections 15.8.3.2 and 16.5.1.3 allow precast concrete wall panels to forego the traditional footing connection requirements that cast-in-place walls have in Section 15.8.2.2. The exclusive connection to the slab is theoretically permissible provided that a rational load path is established to transfer the in-plane and out-of-plane forces through the slab-on-grade and to the supporting soil.

However, slab-sliding resistance is difficult to predict, especially where a plastic moisture/vapor retarder is provided below the slab. Also, it is desirable to mobilize the lateral sliding strength of the foundations. For these reasons, SEAOC Seismology Committee issued a “Position Statement” strongly recommending that designs in high seismically active areas include either a direct or indirect connection to the foundation (SEAOC 2000). An example of an indirect connection would be panel dowels tied into the slab-on-grade and additional footing dowels also tied into the slab-on-grade. It is very common to see tilt-up panels with no foundation connection, yet there has not been any reported damage associated with this in past earthquakes.

At heavily loaded shear walls, overturning may require an uplift connection from the wall panel directly to the foundation. ACI 318-05 Section 21.8 for “Special structural walls constructed using precast concrete” references Sections 21.13.2 and 21.13.3 which require panel-to-foundation connections to yield within the steel element or reinforcement, or otherwise be designed to develop at least 1½ times the connection yield strength.

New Thinking
Past performance of tilt-up buildings has been instrumental in making code revisions to wall anchorage and subdiaphragm requirements, but the process has been a series of incremental reactionary changes that have simply increased the anchorage force or added prescriptive detailing language. There is a belief among some engineers and
researchers that an entirely new approach may be needed in the seismic design of low-rise buildings with relatively rigid vertical elements and with flexible diaphragms, such as is common with tilt-up and masonry buildings today.

Much of the current building code seismic provisions were developed with classic lumped masses at the floor and roof assumed. The seismic response of the traditional lumped mass model is dominated by the stiffness of the vertically oriented lateral force-resisting system, and this assumption applies well to frame structures or tall shear wall structures with rigid heavy diaphragms.

Buildings with rigid shear wall systems supporting flexible diaphragms have several unique properties that make them behave substantially different than classic framing types. The short concrete or masonry shear walls have very little flexibility compared with the diaphragm, and thus the diaphragm period dominates the overall building seismic response (Fonseca, Wood, and Hawkins 1996). Analytical modeling and research has shown that this seismic response can be accurately modeled and predicted (Fonseca, Hawkins, and Wood 1999).

In contrast, the equivalent lateral force analysis in ASCE 7-05, as well as the 2006 IBC, estimates building seismic response as a direct function of building height and the lateral force-resisting system stiffness of vertical structural elements. In addition, current codes determine diaphragm and wall anchorage forces as a direct function of the ground motion only, or sometimes in consideration of the vertical lateral force-resisting system stiffness, without any consideration of the dynamic response of the diaphragm. This inaccuracy in current codes has led some to recommend entirely new code provisions for use in rigid shear wall / flexible diaphragm buildings. Freeman, Searer, and Gilmartin (2002) provide a rational approach that computes seismic response more accurately, based on diaphragm and wall out-of-plane periods. An approach similar to Freeman’s has merit and should be considered as the basis for future code provisions. ASCE 7-05 Section 12.7.3 does provide general language for an alternative dynamic approach, however such a complex approach is not mandatory for typical tilt-up buildings and is outside of normal engineering practices.

Another unique aspect associated with rigid shear wall / flexible diaphragm buildings is that lack of diaphragm stiffness effectively isolates adjacent, parallel lines of shear resistance from each other. Unlike the case with non-flexible diaphragms, any softening or degradation of an overloaded shear wall that is beginning to fail cannot have its loads effectively transferred across the diaphragm to adjacent lines of shear resistance until significant wall translation has taken place. This amount of wall translation is likely to result in complete failure of the shear wall before any significant sharing of loads takes place among the other walls.

The implication is that the redundancy coefficient, ρ, governed by a redundancy weakness in only one line of resistance, is very critical to that line, but need not impact the design of other parallel lines of resistance due to the diaphragm isolation effect. In a theoretical sense, ρ should apply only to specific lines of resistance judged independently, but the value of ρ should be higher to reflect the extra critical nature of the isolated problem being unable to share the load through the flexible diaphragm to other parallel lines of resistance. However, some engineers argue that the purpose of ρ is not to apply a rational approach to strengthening non-redundant elements, but to penalize the entire building system as a discouragement to engineers.

Also related to the isolation effects of flexible diaphragms is the critical nature of struts or collectors in rigid shear wall / flexible diaphragm buildings. Again, because a failure in a collector line has limited ability to transfer loads through the flexible diaphragm to other parallel collector lines, these collectors are non-redundant. ASCE 7-05 Sections 12.10.2.1 and 12.4.3.2 have special load combination provisions for collectors that increase the design loads to address their critical nature. Rigid and semi-rigid diaphragms are more able to redistribute loads through the diaphragm should a collector failure occur, but current codes do not differentiate between the two in this regard.

Further research is justified for rigid shear wall / flexible diaphragm buildings, including tilt-up construction. A new approach to code provisions governing this building type would allow a more rational consideration of building response, including appropriate provisions for redundancy coefficients, collector designs, deformation compatibility issues, and diaphragm deflection limits.
In 2005, the Tilt-Up Concrete Association (TCA) formed a seismic design task group as a proactive measure to advance research on the unique dynamic behavior of this form of construction. Although the work of TCA is in its infancy, its collection of tilt-up construction experts and researchers has great potential in guiding future code revisions. Additionally, in 2007 the University of British Columbia began a research program partially funded by the Portland Cement Association to study the dynamic behavior of tilt-up buildings and their connections as influenced with flexible diaphragms.

As tilt-up construction continues to evolve architecturally and structurally, new questions arise for engineers to consider. The state of the art in tilt-up design has made much progress over the last several decades, but important questions continue to be at hand for additional research:

- Can a parallel set of seismic code provisions be developed for rigid shear wall / flexible diaphragm buildings that considers more accurately their seismic response?
- Is the selected method of distributing in-plane shears critical to shear wall performance?
- Can a simplified method of shear distribution achieve acceptable results?
- Are deformation limits for wall anchorage systems necessary, and if so, how should they be set?
- As wall anchorage is eliminated as the weak element of tilt-up structures, will the mode of failure simply transfer to another vulnerable portion of the system?

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Keywords
concrete shear walls
flexible diaphragm
low-rise construction
tilt-up
wood diaphragm
steel deck diaphragm

How To Cite This Publication
In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Background

In the Northridge Earthquake (January 17, 1994) eight major parking structures suffered partial or total collapse (NISTIR) and at least twenty others were heavily damaged. Most of these structures were relatively modern, having been constructed in the past 25 years prior to the Northridge earthquake. They ranged in height between one and eight stories, and were constructed of cast-in-place, precast concrete, or a combination of the two systems. The lateral force resisting systems included both cast-in-place or topped precast diaphragms with either concrete or masonry shear walls or concrete moment frames.

No other modern concrete building type, with the exception of tilt-up construction performed as poorly relative to the primary code objective, protection of life safety. Observations from Northridge revealed that collapse of the gravity systems sometimes occurred while perimeter walls and frames were undamaged. Other observations included failure of diaphragm collector and chords, large diaphragm deflections, and distress at precast connections due to lateral movements (Holmes and Somers). On the other hand, many parking structures in the area of strong shaking received little or no damage, suggesting that some design and construction practices used in these structures were inherently better than others.

Based on observations from the 1994 Northridge Earthquake, the following code changes were subsequently added for concrete structures in regions of high seismicity:

| Table 1. Building code changes since 1994 intended to affect design of concrete parking structures |
|-------------------------------------------------|---------------------------------|------------------|------------------|------------------|
| Structural Element | Intent of Code Change | ASCE 7-05 | ACI 318-05 |
| Diaphragm and Collectors | Specified the minimum thickness of topping slabs. Limited the spacing and bar size at lap splices for force transfer, | | | |
| Collector Design Forces | Increased the collector design forces | ASCE 12.10.2 | |
| Prestress Tendons | Excluded the use of prestressing tendons to be used in boundary and collector elements, except for the precompression from unbonded tendons. | | ACI 21.9.5.2 |
| Strength Factor, $\Phi$ | Reduced $\Phi$ from 0.85 to 0.60 for the design of reinforcement used for diaphragm chords and collectors placed in topping slabs over precast. | | ACI 9.3.4 |
| Beam to Column Connection | Added requirements for precast concrete gravity frames for improved beam to column connections | | ACI 21.11.4 |
| Transverse Reinforcement of Frame Members | Prescriptive requirements for transverse reinf. for frame members not proportioned to resist seismic induced forces. | | ACI 21.11.2 ACI 21.11.3 |
Parking Structure Characteristics

Parking structures have a number of unique characteristics, compared with conventional concrete buildings, which affect their seismic performance. Some of the key characteristics which may make this category of structure more susceptible to seismic damage than other concrete buildings are summarized below.

Typical parking structures differ from office buildings in that they do not have discrete story levels. Instead the stories are connected with long, sloping ramps, or shorter speed ramps. These ramps can be detrimental to the intended seismic response of the building by acting as unintended diagonal braces. Additionally, sloping ramps create interior short columns which are likely to be governed by shear action rather than bending.

Parking structures are usually very large in plan area, with relatively thin post-tensioned or precast concrete diaphragms compared to a typical office building. Architectural, traffic, security, and economical demands push for long spans and large open areas. Prestressed concrete is a system that is suitable because long spans are economical with smaller member sizes. The long-span floor systems tend to vibrate, but the resulting vibrations are acceptable to uninhabited spaces such as parking garages. As a result, the structural long-span framing systems often used in parking structures are not found in other types of building occupancies. Additionally, the open nature of parking structures has resulted in less redundant structures with fewer shear walls.

Parking structures usually have a few interior non-structural elements, such as partitions, ceilings, and mechanical systems. This inherently leads to lower damping than could be expected from a typical office building. Damping ratios ranging from 3% to 4% were observed in an instrumented parking structure during the Northridge earthquake (Hilmy et al).

Ramps

All ramps that interconnect the primary story levels in a parking structure can significantly affect the seismic behavior of the building. All ramps can change the stiffness and deflection of the building and change the distribution of loads to the designated seismic resisting elements, in some cases attracting a significant percentage of the force (Lyons, et al.). The consequences of these effects will vary among different parking structures depending on the selected seismic resisting system, the plan configuration, the structure height, and other factors.

The current building codes do not provide specific guidelines suitable for analyzing the complex story interactions that can occur in parking structures nor provisions for detailing seismic capacity in the ramps. In some cases, assuming discrete story levels may be too simplified an approach and could cause the designer to overlook unintended structural shortcomings. The level of analytical sophistication required to identify and to address these concerns may vary based on the complexity of the project.

Shortcomings of Current Code Provisions for Ramps. There are several reasons why the code-prescribed definitions of seismic vs. non-seismic elements and discrete story levels are unsuitable for some parking structure designs. First, ramps do not constitute officially recognized seismic resisting elements, like shear walls and moment frames. Yet, ramps can be stiff and massive enough to interact with the designated seismic resisting systems. A literal interpretation of the 2007 CBC might place ramps in the “other components” category like gravity columns and non-frame beams, which are often excluded in seismic analysis models. When ramps are categorized as non-seismic elements, their effect on the seismic behavior of the structure could be inadvertently overlooked. Consequently, force distribution to the shear walls and/or frames might not accurately reflect the likely behavior of the building.

If a stiff ramp attracts seismic forces, presumably these forces are unintended and can cause undesirable performance. The 2007 CBC does not provide detailing guidelines suitable for slabs to function as seismic resisting elements. Interconnected ramps are not held to the ductility detailing provisions prescribed for the shear walls and frames. The diaphragm collector and shear reinforcement is not intended to yield, and thus boundary member
confinement would not be required. Similar concerns regarding the greater force demands have been raised pertaining to the discussion of highly flexible diaphragms with perimeter-only lateral restraint systems (Fleischman).

Parking structure buildings often have a spiral or split level configuration that is not clearly represented by discrete story levels. For example, one segment of the deck could connect from level 3 to level 4. Ramps that connect directly to shear walls or moment frames further deviate from the idealized distinct story levels used in the current codes.

The actual performance of an integrated ramp structure may not match the ductile behavior upon which seismic factors, such as the R-factor, were based. For example, the R-factor for a special moment resisting frame (SMRF) is a high number (8) which results in a low design base shear. For comparison, the R-factor (2007 CBC) for a special shear wall is 6 in a building frame system. In other words, the base shear for a SMRF building is permitted to be 75% of a shear wall building because of the relative implied ductility (6/8) for the two systems by the code. If a ramp in a SMRF parking structure is permitted to stiffen up the building, reducing the true flexibility and altering the hinge formation mechanism, then the use of R=8 in this case is non-conservative.

As a consequence of these shortcomings, seismic forces in ramped parking structures are often improperly distributed to the building components. When ramps are rigidly connected, particularly in frame buildings, then the following additional issues are a consideration:

- It is common practice to release ramps at grade, but to rigidly connect them at the elevated parking decks. When only the upper levels are connected, a significant difference in relative story rigidity and/or strength could be created at the ground level. This may result in soft and/or weak story performance in areas of high seismicity.
- The shift from connected to disconnected levels can cause a local redistribution of the shear forces, causing the second story diaphragm to act like a transfer slab with substantial load demands. This is more critical for moment frame structures.
- The top level floor often has shear resisting elements on three sides only, and thus relies on cantilever diaphragm rotation to distribute seismic forces at that level. The horizontal irregularity types as noted in the building code lack guidelines to limit cantilever diaphragm distance.
- Stiff ramps also can alter the balance of lateral resisting components, causing secondary torsion effects that redistribute the story forces, potentially increasing loads to specific seismic resisting elements.

The 2007 CBC does not specifically address the need to evaluate these potential deficiencies that can occur in buildings with ramps or sloping floors.

Current Practice for Evaluating Ramps. Currently, the challenge for the structural designer of a parking structure is to overcome the disparity between the actual building configuration and the available code procedures. It is common in the industry to neglect the interconnectivity of the story levels in the analysis stage of design. Geometric simplifications of the building system are made so that the code prescribed seismic parameters and detailing provisions can be implemented during the analysis and design of the building. This methodology risks the unintended structural shortcomings noted above.

A less common approach due largely to its impracticality is to design the ramp with a physical release at each level, using expansion joints. Expansion joints at the ramps are intended to change the building type to match the code. While analytically possible, this approach is impractical as the lateral seismic loads imposed by the sloped ramps, which are connected to the horizontal diaphragms on one side only, contribute to undesirable torsional effects, unless additional lateral bracing systems are provided to address this. Additionally, the added initial cost, ongoing maintenance, and the added aesthetic drawbacks of the expansion joints further render this approach impractical.

Some practitioners believe that interconnecting sloped floors provide for structural “toughness,” judging that a well tied together building is inherently more robust. While it is valid to assert that connected ramps provide reserve stiffness or redundancy to a building, it also is true that concurrent load paths are inherently unpredictable. Secondary systems can inadvertently absorb a disproportionate share of the load, even functioning as primary load...
paths. For example, stiff non-ductile ramps can dominate a moment-frame system, short circuiting the ductile members that are designed to dissipate the energy (SEAOSC/LA City Task Force). If this were to happen, (this type of failure has not yet been witnessed) the ramp might fail before the ductile frames are engaged. Dissipating energy in a slab system is not a rational choice because the diaphragm does not posses the same ductility as the frame. However, this effect is not as prevalent in shear wall buildings, given the increased lateral stiffness of these structures, but can occur where the shear walls are not significantly stiffer than the ramp. While ramp connectivity remains a valid design choice for the design of parking structures, particularly in shear wall buildings, the limitations of this approach should be recognized.

Many practitioners prefer to include shear walls in the direction of the ramps, while maintaining more flexible moment resisting frames in the orthogonal direction. This practice allows less seismic deformation along the sloped ramps and reduces seismic loads imposed on short columns.

In any event, the designers should always be aware that ramps have two different characteristics: orthogonal and longitudinal. In the longitudinal direction ramps act as trust elements transmitting axial forces. The predominant concern however is in the orthogonal direction where the main deformation of the diaphragm occurs. In typical parking structures with long ramps, where intermediate lateral bracing is absent, the lateral deformation of the diaphragms could be significant and reach up to \( \frac{1}{2} \) of the total drift. The computer modeling of such ramps shall be carefully considered with appropriate effective moment of inertia. An effective moment of inertia on the order of 50\% \( I_g \) of diaphragms are often used in computer modeling for this purpose. Additionally a ductile behavior should be promoted for the diaphragms in such cases.

In all instances, the engineer must demonstrate a rational representation of the seismic load paths through the structure. In the absence of published guidelines, the best approach currently being used to study these effects is project-specific computer analysis, with each unique building being modeled to evaluate the effects of the particular ramping configuration. The corresponding interpretation of the building code and applicable detailing is at the discretion of the design engineer. Previous analysis tools assumed rigid diaphragms at each discrete level, which is consistent with the 2007 CBC force distribution equations. Today's computational tools permit more complex analysis, including flexible diaphragms and more complex definitions of deck levels including sloped ones. Therefore, the current computer output is even more difficult to correlate with the prescribed design approach specified in the 2007 CBC because seismic loads are resisted by other members of the structure such as the sloped ramps, not just the lateral force resisting system. However, the computer modeling of sloped ramps is a more realistic representation of the structural.

**Ramps in Moment Frame Buildings.** Moment frames are an inherently flexible seismic resisting system which permits large story drifts. In parking structures, it appears that inter-story ramps could not go through these movements without significant distress. Correspondingly, analytical studies show that the ramp interactions may be severe (Lyons, et al.). Under lateral loading, the stiff ramp may limit the building movement and inhibit the movement of the frames as intended. The stiff ramp may short-circuit the frame system may contribute towards the creation of a weak story condition.

Given these concerns, some considerations for the design of moment frame parking structures include:

- Generally, expansion joints at the base of each ramp level can alleviate the interaction problems in moment-frame buildings. Once the stories are isolated, the building behaves like a “typical” structure, and the code provisions remain valid for detailing ductile moment frames. However this solution is impractical as indicated above.

- Short columns between the sloped ramp and adjacent deck are susceptible to concentrated shear demands in moment frame buildings. Supporting the ramp and deck with separated column pairs is a means of alleviating this problem.
Additionally, the impact to the first elevated deck diaphragm could be significant for cast-in-place concrete structures with moment frames. In these structures, the gravity beams may be stiff enough to attract significant amounts of lateral force. In some cases, 25 percent or more of the lateral load may be resisted by the gravity frames. Where “pinned-type” foundations are used under the gravity columns, such as spread footings, these loads may shift at the first elevated deck from the gravity beams to the moment frames due to the reduced stiffness of the gravity frames. Damage in the field has not been identified with this characteristic and should be more closely monitored for future consideration.

**Ramps in Shear Wall Buildings.** In a shear wall building, continuous ramps are not likely to have a significant detrimental effect on the design of the walls. The ramps acting in the strut and/or wall direction typically will not compete with the stiffer shear walls. In the computer analysis the ramps will attract a portion of the shear stresses, and these stresses should be reviewed. In some cases, it is possible that the ramp could shift the balance of the building and cause a redistribution of the story forces among walls. Some guidelines that have been useful in evaluating the stresses in shear wall buildings are as follows:

- If a building is going to be designed with a continuous inter-story ramp, this ramp should be included in the lateral analysis models. Including the ramp provides a convenient way to rationally evaluate incidental loading and the ramp’s effect on the distribution of story forces.
- If the net stress in the plane of the ramp is negligible (less than 0.1f’c using the compression member analogy per ACI 21.3.1.1), the axial loading may be considered acceptable within the current slab detailing provisions. Compare the incidental axial load in the ramp to the tensile capacity of the slab reinforcement, and increase the ramp steel if needed.
- In the current code, no special detailing requirements are provided for seismic forces in ramps. Therefore, any force inadvertently transmitted by the ramp in the computational analysis should be re-assigned to the primary seismic resisting elements. This adjustment will ensure that the designated seismic resisting elements are detailed with the capacity to resist the full shear force of the building.
- If the computer analysis indicates that the net stress in the ramp would be greater than 0.1f’c, then consider increasing the lateral stiffness of the building. A heavily loaded ramp under earthquake loads undermines the role of the shear walls as the primary lateral resisting elements.

**Columns**

Many columns intended to carry only gravity loads failed during the Northridge earthquake because they did not possess sufficient strength or ductility to safely undergo the same seismic excursion as the rest of the structure. Even gravity columns that would normally be considered unrestrained failed, which could be attributed to inadvertent foundation restraint and vertical accelerations. Also, typical geometric configurations of parking structures tend to create short, brittle columns at split-level ramp conditions and at exterior columns where spandrel rails are not sufficiently separated from the columns. Excessive diaphragm deflections in structures with large plan areas contributed significantly to the amount of deformation that gravity columns were required to withstand.

More recent code provisions (ACI 318-05 section 21.11) now include prescriptive requirements for transverse reinforcement for gravity columns in areas of high seismicity. These requirements will significantly improve the expected seismic performance of columns in parking structures, as it was found after Northridge that deformation compatibility checks required by code were routinely ignored, and elements crucial to the integrity of the structure were inadequately detailed to sustain gravity loads under inelastic lateral deformations.

The shear forces in short columns may exceed their design capacity. Based on ACI section 21.11.3, if induced deformation moments are not calculated, it is necessary to provide confinement ties conforming to the requirements of special moment resisting frames. Due to the abrupt curvatures that may occur, especially in a moment frame structure where large inter-story drifts occur, the addition of confinement ties may not be enough to ensure ductile behavior. It is recommended that the design engineer perform deformation calculations to anticipate the forces that may be generated in short columns.
A recommended detail for ramp columns is to separate them from the on-grade ramp allowing them to rotate over the full height of the first level.

**Diaphragms and Load Paths**

Significant damage was observed in Northridge parking structures as a result of poor detailing of diaphragm load paths (Iverson and Hawkins). Parking structures usually have massive floors that generate large seismic inertial forces. These forces are often resisted by very few lines of support, creating exceptionally large concentrations of force in the collecting elements. Many examples of failed collectors and insufficient load paths strongly suggested inadequacy of prior building codes and construction practices.

Since the diaphragms function as both parking surfaces and ramps, they are generally split in the longitudinal direction and contain both flat and sloped surfaces that tend to be relatively long and narrow. Lateral deformations in these long-span diaphragms with high aspect ratios tend to significantly contribute to the seismic response of the structure in the transverse direction. Deformations resulting from this response may put large ductility demands on columns and other non-seismic elements when the diaphragm deflections are added to those of the lateral force resisting elements. Diaphragms that span long distances between seismic resisting elements may not perform in a rigid manner. ASCE 7-05 section 12.3.1.3 considers diaphragms that have diaphragm deflections exceeding two times the drift of the adjoining vertical lateral resisting elements to be flexible. When checking the diaphragm deflection, cracked section properties must be accounted for when stress levels exceed the elastic gross section properties. Adding interior lateral resisting elements with sufficient stiffness to reduce the effective length of the diaphragm is one of the most cost-effective ways to control excessive diaphragm deflections.

Fully developed recommendations to limit span lengths and aspect ratios of diaphragms and diaphragm segments are not available at this time. Research on long span diaphragms (Fleishman) indicates that diaphragms spanning more than 180-feet with aspect ratios of 1:3 do not have adequate response in areas of high seismicity at any design strength. This study recommends a performance-based design procedure for flexible diaphragm wall structures. This study also recommends that a constant diaphragm force pattern be used for flexible diaphragms using the top-level force for all levels.

Code modifications for collectors and boundary elements were based on damage observed in Northridge. This damage included fractured collector reinforcement (Holmes) and diaphragm bars buckling out of the slab, especially in thin topping slabs over precast systems. As noted in the 1999 Blue Book, Appendix F and Commentary C407.6, there were also partial collapses due to the breakdown in the load path of diaphragm loads to shear walls that remained relatively intact. ACI 318-05 Commentary for section 21.9.8 briefly comments on the restrictions added to section 21.9.8.3 at splices and anchorage zones at chord and collector reinforcement.

In response to the lessons learned from the Northridge earthquake, overstrength factors were introduced in the code. These factors must be applied to critical structural elements where elastic behavior is essential for satisfactory performance. Diaphragm collectors, and their connections, are among these critical elements. When applied to parking structures with few shear walls, the collector design forces become exceedingly large. Often it is not possible to install the great quantities of collector reinforcement directly into the shear walls. Instead, the bars are placed in the slab alongside the walls. Since ACI 318-05 section 21.9.8.3 requires a minimum concrete cover of two and one-half bar diameters, but not less than 2 inches, it usually takes a large number of small bars within the thin slabs to satisfy the reinforcing requirements. When these bars are placed alongside the shear walls with adequate spacing between bars, some of the collector bars may be a great distance (10 feet or more) away from the shear wall. Orthogonal reinforcement that is proportioned using the overstrength factors must be added to complete the load path between the collector bars and the shear walls. These orthogonal bars need to be fully anchored into the wall and extend a development length beyond the most remote collector bar. Concrete shear stress using the overstrength factors must be checked in the slab along the length of the shear wall. Sometimes it is necessary to increase the slab thickness, add a monolithic curb or beam, or increase the length of the shear walls to satisfy these requirements.
Satisfying these collector requirements in topped precast systems is especially difficult because the topping slabs are thinner and less substantial than cast-in-place slabs. Even though ACI 318-05 section 21.9.4 specified the minimum topping slab thickness as 2 inches for composite slabs and 2-1/2 inches for non-composite slabs over precast floor systems, there is not much room for reinforcement. Increasing the thickness of the topping slab may be necessary to accommodate reinforcement in structures with long diaphragm spans. The Seismology Committee recommends a minimum of 4” topping slab. Adding more lines of lateral resistance to reduce diaphragm demands is an alternative solution.

Although the topic of providing adequate connection between shear walls and slabs is not unique to parking structures, the thin slabs commonly used in parking structures create situations where normal detailing practices are not adequate. A common condition occurs at the roof, where vertical shear wall bars are detailed to terminate into the roof slab with a standard hook. Unless the slabs are thickened at these locations, the thin slab sections do not provide adequate hook embedment to fully develop the bars. Hence, strength based on shear friction is not fully developed. Vertical wall bars terminating into slabs with less than the required hooked bar embedment should be limited to values used for anchor bolts. Continuing the shear walls above the roof to provide a straight bar embedment above the slab is another option that allows full shear friction values.

ACI 318-05 section 21.9.5.2 also recognizes the beneficial role post-tensioning has in reducing collector and boundary stresses. In order for the conventional reinforcement to elongate, it is first necessary to overcome the axial elastic shortening in the concrete due to the post-tensioning, which is a small value. Prior to the code addition of utilizing the post-tensioning to reduce the quantity of conventional reinforcement for collector and boundary reinforcement, there was a concern for the integrity of the anchoring system for the unbonded post-tensioned system, either from a partial collapse or failure at the anchorage itself. There has not been any significant system failures observed due to the loss of anchorage of unbonded tendons. ASCE 7-05, section 14.2.2.7, Material Specific Seismic Design and Detailing Requirements for Concrete, adds section 21.2.9 requiring anchorages for tendons to demonstrate minimum loading cycle requirements. The loading cycle requirements are intended for seismic force-resisting frames with prestressing tendons, but acceptable behavior for these tests helps qualify the integrity of gravity system prestressing to be used to resist chord and collector forces.

In the past, high strength tendons without the sheathing, (i.e., “bare” or “bonded” tendons) have been used for collector and boundary reinforcement. In these conditions, the bare tendons were placed in the slab but not post-tensioned. The intent of the design provision was to minimize the volume of steel in the diaphragm by taking advantage of the high strength steel and to minimize the need to splice tendons. Observed damage in Northridge included buckled chord and collector reinforcement that ruptured through the diaphragm slabs. This buckling was caused by the compression cycle of the stress reversals once the reinforcement had been significantly elongated. The code limits the reinforcement characteristics for frame members and wall boundary elements, as noted in ACI 318-05 section 21.2.5, but does not have similar limitations for diaphragm elements. ACI 318-05 section 21.9.5.2 specifically allows the use of bonded tendons as primary reinforcement in diaphragm chords or collectors, provided the stress due to design seismic forces does not exceed 60,000 psi.

**Precast Concrete**

Historically, precast concrete systems in parking structures include precast long-span double-tees supported by short span precast girders. Precast columns are generally used to support the girders. Due to inherent long-term shrinkage in prestressed precast units, precast beams and girders are seldom connected to the supporting corbels at each end, and rely mainly on the thin concrete topping slab to tie the structure together. This practice has led to visible damage of corbels or similar bearing supports caused by diaphragm displacement during earthquakes.

Several precast parking structures performed poorly during the Northridge earthquake, and as a result, several improvements were made to the code. Precast elements are inherently more difficult to tie together than monolithic construction (NISTIR), and special attention to detailing is required in order to achieve continuity. This is sometimes accomplished through welded embedded plates and sometimes through relatively thin topping slabs containing substantial amounts of reinforcing. These systems often have limited ductility. The code changes (ACI
318-05 section 21.11.4) included provisions for deformation compatibility of precast concrete frame members. Perimeter gravity columns must have positive connection to the diaphragm, complete calculations for the deformation compatibility of the gravity system must be made, and bearing length is required to be 2 inches more than required for bearing strength (PCA, 1997). The requirement for the additional 2 inches of bearing is to prevent slippage of the horizontal framing members off their supports (Ghosh, Nakaki, et.al.). This increase was based on a 4 percent angular rotation over a 50 inch girder depth.

There were also some changes made to the UBC-based version of the California Building Code. Those modifications were not carried over into the IBC-based version of the CBC. Therefore, these improvements, which were developed and put into practice in UBC-based jurisdictions since the Northridge Earthquake, are no longer code requirements. The UBC-based requirements that were not adopted by IBC are described below, and designers are encouraged to consider them in their parking structures designs.

The flexibility of thin cast-in-place topping slabs that form the horizontal floor and roof diaphragms was observed to be an area of weakness. Large lateral drift caused the unseating and collapse of double tees, buckling of chord reinforcement, and significant compression in diaphragms. (PCA, 1994). In response to these observations, the code adopted a provision that seeks to raise the performance of these diaphragms to a level equivalent to monolithic cast-in-place construction. This provision provides two options aimed at increasing redundancy to achieve this goal. The first option limits the diaphragm aspect ratios for topping slabs to no more than three times the width of the diaphragm or diaphragm segment, thereby requiring well distributed lateral force resisting elements. Where moment frames are used, this section requires that a minimum number of moment frame bays be provided along the frame lines. The second option allows the increased redundancy to be provided through “partially restrained” gravity frame connections where diaphragm aspect ratios cannot be economically met (Ghosh, Nakaki, et.al.).

Special requirements for design of thin cast-in-place topping slab diaphragms over precast concrete members should also be considered. Joints in precast concrete members below result in natural crack lines in the topping slab. Diaphragm failures along these lines were observed in the Northridge Earthquake. Cracking patterns in thin cast-in-place topping slab diaphragms were observed to differ from those in monolithic reinforced concrete slab diaphragms (Wood, Stanton, & Hawkins). Recommendations to mitigate this type of failure, in the cast-in-place topping slab diaphragms, include: 1) the use of shear-friction calculation methods to determine the reinforcement necessary to resist the diaphragm shear along these lines, and 2) the use of more widely spaced and larger gage wire reinforcement.

Other recommended seismic design provisions for the use of thin cast-in-place topping slab diaphragms over precast concrete members are given by the 2003 NEHRP Recommended Provisions (FEMA, 2003) in the Appendix to Chapter 9 and discussed by Hawkins and Ghosh. These provisions recommend that the diaphragm be designed using the lesser of $\rho_\Omega$ times the diaphragm force, or a shear force corresponding to 1.25 times that corresponding to yielding of the seismic force resisting system using $\phi = 1.0$.

Effect of vertical acceleration should be considered in the design of pre-stressed gravity beams and slabs. For example, the increased shear demands caused by vertical acceleration should be considered in the design of the pre-tensioned precast double-tee beams.

References


**How To Cite This Publication**

In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2009)

In the writer’s reference list, the reference should be listed as:

**Introduction**

The objective of this Blue Book article is to discuss current seismic design provisions for the seismic design of reinforced masonry structures and present specific SEAOC Seismology Committee recommendations interpreting and implementing those provisions.

**Relevant Codes and Standards**

The standard cited in the 2007 California Building Code (CBC) or the 2006 International Building Code (IBC) for masonry construction is ACI 530-05/ASCE 5-05/TMS 402-05, compiled in the MSJC (Masonry Standards Joint Committee) Standard (MSJC 2008). This Standard is modified in Chapter 21 of the 2006 IBC. The classes of reinforced masonry shear walls that can be used for resisting seismic loads are:

- **Ordinary reinforced masonry**
  - MSJC Section 1.14.2.2.3
- **Intermediate reinforced masonry**
  - MSJC Section 1.14.2.2.4
- **Special reinforced masonry**
  - MSJC Section 1.14.2.2.5, 1.14.6.3 and 1.14.6.4
- **Intermediate prestressed masonry**
  - IBC Section 2106.1.1.2
- **Special prestressed masonry**
  - IBC Section 2106.1.1.3

Masonry walls, other than those reinforced in accordance with the MSJC Standard and the 2006 IBC, are not permitted as a part of the seismic force-resisting system in Seismic Design Categories C, D, E and F. Table 12.2-1 of ASCE 7-05 (ASCE 2006) further restricts the use of ordinary reinforced masonry and intermediate reinforced masonry as a part of the seismic force-resisting system to SDC C. Maximum height limits, specific for each type of reinforced masonry shear wall when used for seismic force resistance, are stated in Table 12.2-1.

Although plain (unreinforced) masonry shear walls are permitted in SDC A and B, that type of construction is not discussed in this article. Autoclaved aerated concrete (AAC) masonry shear walls are only permitted in SDC A in accordance with 2006 IBC Section 2101.2.2. In 2006, 2007 Supplement Section 1613.6.4 permits plain AAC shear walls in SDC A or B and ordinary reinforced AAC shear walls in SDC A, B, and C, with the MSJC providing prescriptive requirements for AAC masonry shear walls in even the higher seismic design categories. These will not be discussed in this article, other than to state that it is the SEAOC Seismology Committee position that plain AAC shear walls should be limited to Seismic Design Category (SDC) A and ordinary reinforced AAC shear walls to SDC A or B (SEAOC 2006). The reader is directed to the MSJC Commentary for discussion of out-of-plane loads and slender walls, as those are not directly addressed in this Blue Book article.

The provisions for reinforced masonry shear wall systems in the MSJC, ASCE 7-05, and 2006 IBC are similar, with the exception of prestressed masonry shear wall systems. Prestressed masonry shear wall systems are adopted as one system in Table 12.2-1 of ASCE 7-05 permitting an $R = 1 \frac{1}{2}$ and only to be used in Seismic Design Categories A...
and B. There are no special detailing requirements for prestressed masonry shear wall systems in ASCE 7-05, but it adopts those seismic detailing requirements for masonry shear walls through the MSJC provisions. However, it is not clear what class of masonry shear wall detailing is required by ASCE 7-05 for prestressed masonry shear walls listed in Table 12.2-1, therefore it may have to be assumed as unreinforced masonry. The 2006 IBC also adopts the seismic detailing requirements for masonry shear walls through the MSJC provisions for the prestressed masonry wall systems in Sections 2106.1.1.1 through 2106.1.1.3 (for each class of system – for reinforced systems they are Intermediate and Special). These provisions also include special seismic design and detailing requirements for each prestressed masonry shear wall system. Unfortunately, it is not clear what design coefficients ($R$, $\Omega_s$, $C_d$) and system limitations apply to the prestressed masonry shear wall systems in the 2006 IBC. Under the 2006 IBC, it is recommended that design coefficients ($R$, $\Omega_s$, $C_d$) and system limitations for prestressed masonry wall systems follow that of the conventionally reinforced masonry systems in Table 12.2-1 of ASCE 7-05 for like seismic detailing.

Selection of Design Procedure

The 2006 IBC permits the use of Allowable Stress Design (ASD) procedures (also called Working Stress Design) and Strength Design procedures. The MSJC Standard permits a one-third increase in stress (MSJC Section 2.1.2.3) when considering ASD load combinations that include wind (W) or earthquake (E). The load combinations in the MSJC Standard are superseded by the load combinations prescribed in IBC Section 1605.3. A one-third stress increase is not permitted when using the basic load combination for allowable stress design in Section 1605.3.1. However, if the alternate basic load combinations of Section 1605.3.2 are used, the one-third increase is permitted. The MSJC Standard permits the use of Strength Design procedures for design of concrete and clay masonry and requires the use of Strength Design procedures for design of prestressed masonry shear walls (MSJC Section 4.4.3).

It is recommended that designers use strength design procedures for all facets of design of seismic resistance of reinforced masonry shear walls (Paulay and Priestley 1992, 535). Experimental research conducted by the Joint US-Japan research program, Technical Coordinating Council for Masonry Research (TCCMaR) found that strength design procedures closely predict experimental results when the yield stress of the reinforcement used in the experiment is used in the calculations of nominal strength.

Ordinary reinforced masonry and intermediate reinforced masonry shear walls have traditionally been partially grouted masonry. MSJC Section 3.1.3, Design Strength, will discourage use of partially grouted masonry in higher seismic regions as this Section requires the design factored nominal shear strength to exceed the shear strength corresponding to the development of 125% of nominal flexural strength or 2.5 times the calculated loads. Since it would be difficult to provide sufficient shear strength in partially grouted walls, it is recommended that walls be grouted solid. The nominal flexural strength of an element includes the development of yield stress in the end reinforcement and the development of the yield tensile stress or less than yield tensile stresses in the uniformly distributed reinforcement. Calculation of the position of the neutral axis, $c$, allows the designer to equate the stress in the distributed reinforcement to a strain that varies from zero to yield strain and expedites this calculation of nominal flexural strength. There is no similar simplification in ASD design procedure for computation of an equivalent "nominal" flexural strength (Paulay and Priestley 1992, 535), therefore designing to reduce the probability of brittle shear failures is not possible. ASD design procedures do require that the calculated shear be increased by 1.5 times, but this does not assure that shear failure will not occur prior to the development of the shear wall flexural strength.

Basis for seismic design of reinforced masonry Structures

The Joint US-Japan research program on reinforced masonry (TCCMaR) established coordinated nominal values for shear and flexural strengths. The design of lines of reinforced masonry seismic load resistance involves the following phases:

- Determination of relative rigidity of types of walls, such as cantilever, coupled, and shear walls with openings.
- Determination of rigidity of lines of seismic resistance composed of types of walls.
- Distribution of seismic loading to lines of resistance or a type of wall in a single line of seismic resistance.
- Consideration of the limit of elastic behavior of elements in each type of shear wall; i.e. nominal strength.
- Estimation of locations of plastic hinges and sequence of formation of plastic hinges in the system.
Before selecting a system configuration for a masonry walled structure, MSJC Section 3.1.3.1 should be reviewed. This section requires that the structure be designed with an \( R \) factor of 1.5 if the seismic force-resisting system is non-conforming with Section 3.1.3.1. This penalty is equally applicable for all types of masonry shear walls used as a part of the seismic force-resisting system. The TCCMaR research program found that seismic design factors related to ductility, \( \mu \) and element overstrength, \( \Omega_{\text{element}} \), must be related to flexural failure modes. The ductility, \( \mu \), associated with the shear failure mode is inadequate for consideration of nonlinear behavior (Paulay and Priestley 1992, 656-659). The flexural failure mode in the shear wall portion of the seismic load-resisting system should be caused by reinforcing bar tension strain limits due to moment related curvature at the base of the cantilever or coupled shear walls rather than a masonry compression strain failure.

All elements in a line of seismic resistance have a forced common displacement. Occurrence of individual flexural failure modes in load-resisting elements are related to their aspect ratio, applied flexural moment and axial loading. MSJC Section 3.1.3.1 does not prohibit use of shear walls with openings in lines of seismic load resistance; this section limits the stiffness (rigidity) of shear walls with openings to 20% of the total stiffness of a line of seismic resistance and to the sum of the lines of seismic resistance. Note that this limit is on stiffness, not in-plane strength. The base shear assigned to a line of seismic load resistance is determined by its relative stiffness and its resistance to torsional response. The portion of base shear assigned to an element in that line of resistance is determined by its stiffness relative to the stiffness of other elements in the line of seismic resistance.

A weakness of the MSJC standard is that it does not define what a shear wall in a line of seismic resistance is. Piers and columns are defined in MSJC Section 1.6 by an in-plane dimension. This Section also defines a "wall" by an in-plane dimension that overlaps with the definition of a pier. It is recommended that the definition of a shear wall be taken as that defined in textbooks such as (Paulay and Priestley 1992) and (Drysdale, Hamid, and Baker 1999) namely a non-coupled shear wall that cantilevers from the base of the building. A coupled shear wall is not a cantilever shear wall. The difference between a "coupled shear" wall and a cantilever shear wall is given in a footnote, \(^{6}\), to Table 12.12-1 of ASCE 7-05, which states that "moment transfer between shear walls (coupling) is minimal" for cantilever shear walls. The allowable interstory drift is increased for cantilever shear walls recognizing that reliance on a single nonlinear hinge at the base of the shear wall for ductility is desirable.

MSJC Section 3.1.3.1 will necessitate that the designer pre-plan the in-plane location of reinforced masonry shear walls. This precedes the strength determination of the parts of the seismic force-resisting system but is required as the stiffness analysis enables the designer to assign the base shear to elements of seismic resistance by a rational procedure. Section 3.1.3.1 in the 2005 MSJC Standard contains an error that is corrected in the 2008 Edition. The second sentence states "Along each column line": this should be corrected to "Along each line of seismic resistance" to conform to the 2008 MSJC Standard. The current edition implies that lines of gravity support by columns equals a line of seismic resistance.

MSJC Section 1.14.5.2.2 requires in Seismic Design Category C that "masonry elements that are not designed to resist vertical and lateral loads, other than those induced by their own mass, shall be isolated from the structure," and that the isolation joint accommodate the design story drift. MSJC Section 1.14.5.2.1 has an anachronism in its text: it implies that "load-bearing frames or columns" may be considered as not a part of the lateral force-resisting system. There is no definition of "load-bearing frame" in the MSJC Standard. The concept of a special reinforced wall-frame was deleted from the MSJC Standard several editions ago. It is recommended that MSJC Section 1.14.5.2.1 not be interpreted in conflict with Section 3.1.3.1. Section 3.1.3.1 explicitly uses "piers and columns" in its description of a "load-bearing frame." This description parallels that in masonry design textbooks as a shear wall with openings. It is recommended that load-bearing frames or columns mentioned in Section 1.14.5.2.1, not part of the seismic force-resisting system, be evaluated for deformation compatibility based upon 1) a moment-curvature analysis using parameters taken from cyclic tests of masonry beam or column components and 2) the ability to avoid a shear failure under the design story drift.

For all Seismic Design Categories, IBC Section 2106.1 requires masonry walls to be designated as part of the seismic force-resisting system unless isolated on three edges from the in-plane lateral deformation of the structural...
Other masonry elements in Seismic Design Category B not designed to resist vertical or lateral loads other than those induced by their own mass must be isolated from the structure in accordance with IBC Section 2106.3.1 (similar to MSJC Section 1.14.5.2.2).

MSJC Section 3.1.3.1 limits the acceptable elements in a line of seismic load resistance. Reinforced masonry walls may consist of the following:

- Solid cantilever shear walls
- Coupled cantilever shear walls
- Shear walls with openings composed of piers, columns, and spandrels

The first component is defined in footnote (d) of Table 12.12-1 of ASCE 7-05 as a cantilever shear wall. The Drysdale, Hamid, and Baker textbook (1999, 456) states: "If the area of the openings in a wall is only 5% to 10% of the total wall area and well distributed, the effect of the discontinuities in strain distribution over the length of the wall is usually ignored and the analysis of rigidity can be based on the stiffness of the gross cross-sectional area."

The second type of seismic force-resisting reinforced masonry elements are coupled shear walls. Detailing of the shear resistance of the coupling beams (spandrels) is critical to the dynamic response of this component. The designer is confronted with two possible behavioral modes of coupled shear walls. First; the nonlinear strength and stiffness of the spandrels may degrade and the system behavior will convert to a cantilever shear wall with a demand moment at its base equal to that of a cantilever shear wall (Paulay and Priestley 1992, 538). The second behavioral mode is that the total strength of coupling beams framing into the shear wall will modify the desirable single curvature of the shear wall into double curvature (fixed ends) in low-rise structures.

The MSJC Standard does not have any Commentary on coupled shear walls. In the absence of commentary it is recommended that the sum of flexural strengths of coupling beams be limited at any story level to 10% of the flexural strength at the base of the shear wall and to a total of 20% of the base flexural strength of the shear wall when distributed over the height of the shear wall. This recommendation is limited to calculation of relative rigidity of components in a line of seismic resistance.

The third type of seismic force-resisting system is the shear wall with openings, which is composed of piers, columns, and spandrels. The definitions of piers and columns in Section 1.16, Definitions, of the MSJC Standard is incompatible with the less restrictive definition of piers in design textbooks. The definition in textbooks is that the stiffness of spandrels that frame into a joint at the end of a pier restrains the end of the pier, placing the pier in double curvature. This is an undesirable condition for any nonlinear behavior as the ductility of the pier may have to be provided by shear displacement of the pier and nonlinear flexure in the vertical length of the pier. The seismic force-resisting system with uniform pier length and story height would develop a soft-story mechanism in the lowest story and the system would have very limited ductility (Paulay and Priestley 1992, 536-537). By implication, MSJC Section 3.1.3.1 limits the participation of shear walls with openings to providing a maximum of 20% of the total in-plane stiffness of the line of seismic resistance.

When the shear wall has coupling beams from adjacent shear walls or shear walls with openings, it is recommended that the rigidity be estimated by superposition of effects of coupling beams on the top displacement of the shear wall calculated as a cantilever from its base. The dimensions of this cantilever wall are the dimensions of the coupled shear wall. This calculation of the top displacement as a cantilever includes the deflection due to shear. The coupling beams have no effect on the shear displacement.

The flexural moments of the coupling beams at the edges of the coupled shear wall reduce the cantilever displacement, because these moments are in opposition to curvature due to the unit force at the top of the wall. The moments in the coupling beams are induced by the curvature of the "cantilever" wall. This recommended superposition is based on the assumption that the "curvature" is near-linear. Stiffness degradation at the base of the cantilever wall in lieu of smeared degradation over the height of the shear wall will cause this behavior.
Formulas for top displacements due to application of coupling beam moments, $M_s$, are rewritten to use $M_s$ as a percentage of $M_o$, where $M_o$ is the base moment due to a unit force applied at the top of the cantilever. This enables direct superposition of calculated top displacement for coupled shear walls. It is recommended that the maximum percentage of base moment used as coupling beam moments be 10% at any level and 20% total distributed over the height of the shear wall. An example is:

- Assume the structure is four stories in height.
- The recommended sum of coupling beam moments is 20% of $M_o$. One-quarter of 20% of $M_o$ is applied at the roof level and 2nd, 3rd and 4th floor levels.

The top displacements due to the coupling beams are deducted from the $\Delta_c$ of the cantilever shear wall. The inverse of the net top displacement is the rigidity of the coupled shear wall, $R_{coupled}$. Coupled shear walls are stiffer than cantilever shear walls of the same dimension, but their nonlinear behavior is limited by the ductility demand on their coupling beams. The ductility demand on coupled shear walls is partially controlled by limiting their allowable story drift to 0.007 $h_{sx}$. The allowable story drift for cantilever shear walls is 0.010 $h_{sx}$.

This section is to provide information to the designer on compliance with MSJC Section 3.1.3.1. This approximate procedure cannot be as accurate as the actual moment diagram and end conditions because strips, coupling beams, and vertical cantilever are not considered. However, since load distribution between elements and components is possible, the results of the approximate procedure are generally satisfactory. Rigidity of components of a seismic load-resisting system when estimated by these procedures will have unacceptable results for conditions such as the following:

- A solid shear wall has openings at lower-story levels and is supported at those levels by columns and/or piers.
- A shear wall with openings has randomly sized and distributed openings. These openings cannot be logically separated into vertical or horizontal strips.
- The flexural strength of coupling beams of coupled shear walls substantially exceeds the recommended strength limits.

The reason for unacceptable results for these and similar conditions is that the "stiffness" of a component of the seismic load-resisting system has to be a single value for computational simplicity. In reality it is not. A multistory solid shear wall over lower stories that contain openings in the wall provides resistance to interstory displacements in the upper stories. The resistance and redistribution of shear forces from this seismic loading is provided by "rigid" diaphragms at or above the level with openings. The path of resistance forces and reactions on this discontinuous shear wall now includes the diaphragms and all systems that control the diaphragms displacement relative to the base of the building.

These relative rigidities of the seismic load-resisting system having one or more of the above-stated conditions may be determined by an elastic finite element analysis. A frame analysis is not recommended; rigid links would have to be assumed at the ends of piers and spandrels in the frame model, and the frame analysis would have to consider shear deformation in joints and frame members (Drysdale, Hamid, and Baker 1999, 459).

**Requirements for Ductile Behavior of Reinforced Masonry Shear Walls**

The MSJC Standard has three Sections with special requirements. The intent of these Sections is for assurance of ductile behavior of shear walls functioning as lines of seismic load resistance. These are:

- MSJC Section 3.1.3.1: At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral force-resisting walls. Along each line of lateral resistance at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral force-resisting walls. (The wording here is revised to conform to the 2008 MSJC Standard). This MSJC Section has an exception: Where seismic loads
are determined based on a seismic response reduction factor, $R$, which is not greater than 1.5, piers and columns are permitted to be used to provide seismic resistance.

- MSJC Section 3.1.3: The design shear strength, $\phi V_u$, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, $M_u$, of the member, except that the nominal shear strength, $V_u$, need not exceed 2.5 times required shear strength demand.

- MSJC Section 3.3.5.5 and 3.3.6: The combined maximum axial load, as limited by Section 3.3.6.6 where special boundary elements are not used, and the quantity of flexural reinforcement, are limited by these MSJC Sections. Section 3.3.3.5.4 provides an exemption to 3.3.3.5 for masonry members where $M_u/V_u d_i \leq 1$ and when designed using $R \leq 1.5$. Where $M_u/V_u d_i \geq 1$ but designed with $R \geq 1.5$, Section 3.3.3.5 is applicable. Section 3.3.6 provides for boundary elements, except when a number of axial stress conditions, aspect ratios and shear stresses are not exceeded. When these conditions are exceeded, determination of boundary elements based on calculation of the elastic top deflection of the cantilever shear wall is required. It is recommended that Section 3.3.3.5 be used for determination of combined axial loading and maximum reinforcement quantities.

MSJC Section 3.1.3.1 requires the designer of the structure to rely on shear walls, coupled walls, or cantilever shear walls, that have the highest potential for ductility. This requirement is applicable to ordinary, intermediate, and special reinforced masonry seismic load-resisting systems. The penalty for non-compliance is minimal for ordinary reinforced masonry but significant for intermediate reinforced masonry systems and severe for special reinforced masonry systems. ASCE 7-05 limits the use of ordinary and intermediate reinforced masonry to SDC B and C. Special reinforced masonry is not limited by SDC, it only has height limits for SDC D, E and F. The benefit for the public of this requirement is reduction of possibly irreparable earthquake damage.

MSJC Section 3.1.3 requires that shear strength of a member exceeds the shear associated with flexural strength (Paulay and Priestley 1992, 661). If the member is a cantilever, the shear is that associated with the development of the base moment. If the member is deformed in double-curvature, it is the shear associated with development of the sum of flexural moments at the fixed ends. This requirement reduces the probability of a shear mode failure rather than a flexural failure mode. The ratio of factored shear strength to shear associated with development of nominal flexural moment is 1.25; the ratio of nominal shear strengths is 1.5; $\phi$ is 0.8. The factored shear associated with development of nominal flexural strength is: $\phi V_u / M_u \cdot d_i$. This can be modified to this form: $\phi V_u / V_u \cdot (h/d_i)$, where $h$ is height above the base of the wall associated with development of $M_u$. If the component is one story, $h$ is the height of the wall. If the structure is multistory, $h$ is approximately 70% of the height of the structure above the level under consideration. The design nominal shear strength need not exceed 2.5 times the required shear strength demand at that story level. This 2.5 factor is also applicable to squat shear walls that have an undesigned in-plane flexural strength in excess of the required flexural strength.

In accordance with IBC Section 2106.5.2, where it is known that a flexural plastic hinge at the base of the wall will form before a shear mechanism, the nominal wall shear strength must be calculated by neglecting the masonry shear stress contribution to the wall shear strength.

MSJC Section 3.3.3.5 describes a procedure for limiting the quantity of flexural reinforcement in elements that are a part of the seismic load-resisting system. The intent of this section is to provide an upper bound on axial loading and on the quantity of flexural reinforcement and to minimize the probability of a flexural failure mode due to excessive compression strain in the compression block.

The procedure is to assume a strain profile at the end of the flexural member or base of a shear wall. The strain varies from the maximum compressive strain (0.0025 in./in. for concrete units, 0.0035 in./in. for clay units) to a multiple of yield strain (0.0021 in./in.) in the reinforcement. The axial loading, $P_w$, and the sum of all nonlinear and linear forces in the flexural reinforcement in the tensile strain length, $d_i - c$, is in equilibrium with compressive forces in the compressive strain length, $c$, of the member or wall. The compressive capacity of the compression block is assumed to be $0.8 f'_c * 0.8 c * I$. The compression capacity of reinforcement within the compressive strain length can be added to the compressive capacity of the compression block. The compression force in this
reinforcement is $A_v f_y$ when the compression strain exceeds 0.0021 in./in. and is $\varepsilon_s E_s A_t$ when $\varepsilon$ (strain) is less than 0.0021 in./in.

MSJC Section 3.3.3.5.1 limits the general application of Section 3.3.3.5 to masonry members where $M_h/V_{h} > d_r \geq 1$. Section 3.3.3.5.4 modifies this limitation to: where $M_h/V_{h} > d_r \leq 1$ and the member or wall is designed using $R \leq 1.5$, there is no upper limit to the quantity of flexural reinforcement. It is recommended that designers ignore this exception” and consider effects of axial loading and quantity of flexural reinforcement to determine what is called a "tension controlled design" in strength design procedures. A “tension controlled design” is the state of stress and strain wherein the tension reinforcing steel yields before the limiting compression stress in the masonry occurs. Since yield strain in reinforcement commonly exceeds the minimum specified for a given grade of reinforcing steel, it is recommended that the strain gradient specified in Section 3.3.3.5.1(a) be used to determine the axial load, $P_w$, and the quantity of flexural reinforcement that constitutes a tension-controlled design.

Compliance with Section 3.3.3.5 requires data such as dimensions of members and walls, axial loading, required flexural strength, extent of grouting, seismic design coefficients, and similar information that is needed for the seismic design process. Compliance with Section 3.3.3.5 is a check on the tentative seismic design. The minimum quantity of uniformly distributed reinforcement is determined by required strength for out-of-plane loading, maximum spacing, and/or minimum area of vertical reinforcement. If the designer elects to provide the required flexural resistance with reinforcement concentrated at the edges of the member or wall, that quantity and location of this reinforcement can be used in the check for equilibrium. If this equilibrium check shows the capacity of the compression block is inadequate, the quantity of reinforcement proposed is excessive for compliance. As an alternative, the location of concentrated reinforcement can be estimated and the quantity of concentrated flexural reinforcement can be an unknown in the equilibrium equation. When determined, this quantity of reinforcement is compared to the quantity required for seismic resistance. Even if vertical reinforcement is concentrated at the wall ends, distributed steel is required along the wall to satisfy minimum reinforcement and out-of-plane loading requirements. All steel, concentrated and distributed, must be included in calculating the flexural strength of the member, except that only steel confined by lateral ties in special boundary elements can be relied upon to resist compression.

MSJC Section 3.3.3.5 is applicable to all members in the seismic load-resisting system. A short definition of these members is spandrels, piers, columns, and joints. These members should have the quantity of flexural reinforcement that does not significantly exceed that required by the ASCE 7-05 load combinations for in-plane and out-of-plane loading. Section 3.1.3 requires that the shear strength exceed the shear corresponding to the flexural overstrength of the members. Flexural overstrength of these members is not desirable. Shear overstrength as required by Section 3.1.3 is desirable for enhancing global ductility of the seismic load-resisting system.

Prestressed masonry shear wall provisions reference the conventionally reinforced masonry shear wall provisions for design and detailing, including minimum steel requirements. However, only the bonded tendons are permitted to be part of the minimum steel considered for seismic resistance. IBC Section 2106.1 requires the moment strength along the section to be not less than one-fourth of the maximum moment strength. IBC Section 2108.4 does permit an increase in the maximum tensile strain in prestressing steel to 5 times the yield strain for Special Prestressed Masonry Shear walls.

For Allowable Stress Design (ASD), ASCE 7-05 Section 14.4.6.2.3 and IBC Section 2107.8 sets a maximum steel ratio for Special Reinforced Masonry Shear Walls that are flexurally controlled and subject to high axial loads. This maximum steel ratio, $\rho_{\text{max}}$, is intended to eliminate the tendency for over-reinforcement in flexure when using ASD. Over-reinforcement in flexure results in non-ductile failure mechanisms, such as a shear failure, under seismic or other abnormal loading. It is recommended that $\rho_{\text{max}}$ be applied without limitation to all gravity members and all masonry seismic force-resisting systems when using ASD.

For Allowable Stress Design in Seismic Design Categories D, E and F, IBC Section 2106.5.1 requires seismic force demand be multiplied by 1.5 for in-plane shear or diagonal tension design. These IBC provisions are carried over from the 1997 UBC and are intended to avoid a shear-dominated response of the shear wall designed by the ASD.
procedure. However, this requirement does not always result in a significant reduction on the possibility of shear failure. As discussed earlier, Allowable Stress Design is not recommended for the seismic design of masonry structures.

**Special Detailing Required to Enhance Ductility**

The 2005 MSJC Standard contains detailing requirements in Chapters 1 through 3. In many of the following examples the detail requirements and design provisions are contradictory. It is recommended that the information in the MSJC Sections listed be ignored and the referenced Section in the MSJC Standard or IBC be substituted. Many of these are corrected in the 2008 MSJC, but are noted here as a caution to the user of the 2005 MSJC:

- 1.7.2: Substitute ASCE 7-05 for ASCE 7-93. ASCE 7-05 is referenced for loading in the 2006 IBC.
- 1.7.5.2: Contradicts Section 1.7.5, loading is distributed by member stiffness.
- 1.9.2: Contradicts Section 3.1.5.2 which requires use of cracked cross-section.
- 1.9.4.2.3: Contradicts experimental research (He and Priestley 1992) and (Priestley and Limin 1990). It is recommended that the conclusions of this research be considered by designer.
- 1.9.4.2.5: Contradicts Section 1.9.4.2.4, which requires design of intersection for shear.
- 1.13.2.1: Contradicts Section 3.3.3.1.
- 1.13.2.3: Joint reinforcement not accepted as reinforcement in Chapter 3, Strength Design.
- 1.13.3.4: Contradicts Section 3.3.3.6, bundling of reinforcement is prohibited.
- 1.14.4: Ignore all references to AAC masonry in SDC B and above. Section 2101.2.2 of the 2006 IBC prohibits use of AAC masonry in the seismic load-resisting system in SDC B, C, D, E or F.
- 1.14.2.1: Joint reinforcement is not acceptable for required reinforcement in Strength Design.
- 1.14.3.3: Contradicts IBC Section 1604.8.2. Loading of wall anchorage of masonry walls is specified in Section 1604.8.2.
- 2.1.3.4: Strength design of reinforced masonry is prescribed in Chapter 3 of the MSJC Standard.
- 3.1.2: Substitute ASCE 7-05 for ASCE 7-02. ASCE 7-05 is referenced in the 2006 IBC.
- 3.1.5: Substitute ASCE 7-05 for ASCE 7-02. ASCE 7-05 is referenced for deformation limits in the 2006 IBC.

Detailing requirements for reinforcement in Sections 3.3.3 and 3.3.4 should be used in lieu of the detailing requirements specified in Chapter 2 of the MSJC Standard. Strength design procedures, Chapter 3, be used for design of seismic load-resisting systems.

For special detailing provisions not in the MSJC, IBC Section 2106.4.1 requires additional detailing for masonry columns and pilasters (minimum transverse reinforcing ratio of 0.0015 and a maximum spacing of 0.25 times the member depth or width) that support reactions from discontinuous systems such as shear walls in Seismic Design Category C and higher. Discontinuous shear walls produce large overturning forces on supporting columns below commensurate with the overstrength of the wall (ASCE 7-05 Section 12.3.3.3). These provisions are intended to assure additional strength and ductility in the columns or pilasters to resist the remainder \( R/\Omega \) \( \times \) \( Q_e \) forces and the associated lateral deformation demands.

Welded and mechanical splice requirements for reinforcing bars given in IBC Section 2108.3 update MSJC Section 3.3.3.4 with further ductility enhancements, which are similar to the provisions of ACI 318-05. Welded splices are not permitted in plastic hinge regions of Intermediate and Special reinforced masonry wall systems. Type 1 and Type 2 connectors are now required for rebar mechanical splices. Type 2 connectors are more ductile and are permitted anywhere in the member length.

For Special Prestressed Masonry shear walls, IBC Section 2106.1.1.3 requires prestressing tendons to be limited to ASTM A722 bars and all cells of the wall to be grouted solid. These detailing provisions match those of wall test specimens successfully cyclically tested to appropriate lateral displacements (Laursen and Ingham 2001). These cyclic tests are the basis for the acceptance of this special system in the IBC 2006.
Recommended Design Checks Not Required by 2006 IBC

The above discussion on Special Detailing to Enhance Ductility recommended that the designer should consider the conclusions of experimental research regarding the effects of flanges on shear walls. Section 1.9.4.2.3 of the MSJC Standard limits the width of the effective flange to six times the flange thickness. The 2003 NEHRP (BSSC 2004) separates the recommendation into two parts: compression flange and tension flange. The recommendation for compression flange is identical to the MSJC Section. The recommendation for width of the tension flange is ¾ of the wall height, not story height. The recommendation of the TCCMaR research report (He and Priestley 1992) and the TCCMaR research report on the multistory specimen (Seible et al. 1994) are more generous on the effective width of compression flanges. Their effective compression width is based on the height of the wall as is the effective width of the tension flange.

Neglecting the effects of the effective compression flange is minor. Neglecting the effects of the effective tension flange may be critical in that the effective tensile reinforcement in the shear wall is understated. This understatement in reinforcement quantities in the shear wall causes two critical understatements and anti-symmetrical response to earthquake shaking. The anti-symmetrical response may be in non-compliance with the ratio of factored shear strength, the shear related to development of nominal flexural strength, and non-compliance with the maximum quantity of flexural reinforcement. This underestimation of the tension flange width in “T” and “L” shaped walls, and therefore the amount of tension reinforcement, will result in large compression strains in the opposite non-flanged end of the wall causing a premature compression failure under the design ground motions. Compliance with these MSJC requirements is essential to assurance of displacement ductility that corresponds to the R factor prescribed by ASCE 7-05.

Compliance with the code is required for calculation of nominal strengths, but the designer should consider using the width of flange recommended by NEHRP 2003 for checking compliance with MSJC Sections 3.1.3 and 3.3.3.5. Using greater quantities of flexural reinforcement in compliance checks is not a code violation.

The seismic design of coupled shear walls is based on expected coupling beam behavior. In coupling beams, the end moments and shears related to development of these end moments, are induced by gravity loading and by curvature of the shear wall. The seismic design moments at the base of the shear wall and at the ends of the coupling beams are those related to inelastic curvature of the shear wall but are made to emulate elastic (nominal) design demands by a seismic design reduction factor, $R$. The elastic curvature of the shear wall is caused by story level loading equal to the base shear. This curvature is modified by the effects of end moments of the story level coupling beams. The seismic end moments in the coupling beams are due to the curvature in the shear wall at the floor level under consideration. The theoretical solution of the elastic deformations using the specified loading, and the moments and shears induced by the elastic deformations, can be made by a validated nonlinear analysis programs for concrete or masonry, such as NFEAP (Hsieh et al. 1980) or IDARC (Valles et al. 2006). The programs must consider the effects of the location and quantity of reinforcement (both flexural and shear), a rotating crack mode, a bond slip model, and reduction of stiffness in the masonry due to flexural and shear cracking. The programs should be validated by replication of the hysteretic behavior of experimental specimens. This procedure requires an iterative design of the coupled shear wall system.

A theoretical elastic analysis cannot predict the nonlinear behavior of a coupled wall system. The elastic analysis limits the strain in the reinforcement to elastic strains. The nonlinear coupled shear wall system develops a nonlinear hinge at the base of the shear wall. The nonlinear displacements of the shear wall above the center of the hinge are the sum of elastic top displacement and rigid body rotation of the shear wall (Drysdale, Hamid, and Baker 1999, 477), above the center of the hinge. The linear displacements, expressed as "curvature" effects $\varphi_y$, are increased by the effects of the nonlinear curvature, $\varphi_p$, at the base of the shear wall. The displacement at any story level can be estimated as sum of $\Delta_1 + \Delta_p$ and proportional to the height, $h$, above the center of the hinge. The seismic loading used for calculation of elastic displacements is distributed base shear divided by $R$. The top displacement, $\delta_{xw}$, due to this loading, is not amplified by $C_d$. 
The ductility of the shear wall is affected by the quantity of flexural reinforcement, the level of axial load, and the aspect ratio, \( h/l \), of the shear wall (Drysdale, Hamid, and Baker 1999, 478-479). The displacement ductility expected for Special reinforced masonry is about 3.5. This estimate is based on an expected element overstrength, \( \Omega_{\text{element}} \), of 1.5. The \( R \) of 5 is the product of \( \mu \) * \( \Omega_{\text{element}} \) (NEHRP 2003). The displacement ductility of a shear wall is enhanced by using uniformly distributed reinforcement for all or part of required flexural reinforcement (Paulay and Priestley 1992 534-535). The yield strength and yield displacement of a shear wall with uniformly distributed reinforcement is about \( \frac{1}{2} \) less than the yield strength if \( \frac{1}{2} \) of the total reinforcement is concentrated at the ends of the shear wall. The strength limit state (ultimate strength) for uniformly distributed and concentrated is almost equal and the displacements at the strength limit state are identical. Use of uniformly distributed reinforcement increases \( \mu \), the ratio of displacements at yield and strength limit state and \( \Omega_{\text{element}} \) the ratio of strength limit to yield strength. The curvature ductility of the nonlinear hinge can be related to quantity of flexural reinforcement, \( \rho, f'_{m} \), axial force, \( P_{n} \), and aspect ratio, \( h/l \) (Drysdale, Hamid, and Baker 1992, 478-479). The relationship between displacement ductility, \( \mu_{d} \), and curvature ductility, \( \mu_{p} \), is given in the equation:

\[
\mu_{d} = 1 + (\mu_{p} - 1) \times \frac{L}{h} \times [1 - (0.5 \times \frac{L}{h})]
\]

The maximum quantity of reinforcement in the nonlinear hinge zone using strength design is limited by MSJC Section 3.3.3.5. It is implied that the maximum nonlinear curvature in the plastic hinge will be limited by the allowable story drift, \( \Delta_{p} \). However, the actual maximum nonlinear curvature in the plastic hinge will be much larger for short columns or wall piers where the story drift must be accommodated within the shorter column or pier height. The displacement ductility, \( \mu_{d} \), of the shear wall is increased by nonlinear curvature ductility, \( \mu_{p} \). The curvature ductility of the hinge increases as the quantity of reinforcement decreases, or if longitudinal rebar confinement is provided at wall ends and the axial load is reduced.

The inelastic curvature of the shear wall has a high probability of causing inelastic behavior in the coupling beams and deterioration of the shear strength of the coupling beams. Paulay and Priestley (1992, 659) provide an equation for estimating the shear ductility of a squat cantilever shear wall. This equation can be modified to one-half of a fixed-end coupling beam in double curvature. The referenced equation is:

\[
\mu_{sf} = 0.5 \left( \left( 3 \times A_{c} \right) + 1 \right) \leq 5
\]

For the cantilever wall, \( A_{c} = h_{c} \times l_{c} \). The definition of \( A_{c} \) would be reversed for the one-half length of the coupling beam to 0.5 * \( h_{c} \times l_{c} \). If the depth of the coupling beam is 3 feet, for example, and the length of the coupling beam is 12 feet, \( \mu_{sf} = 0.5 \left( \left( 3 \times 6 / 3 \right) + 1 \right) = 3.5 \) when expected member ductility is shear ductility. In this example, the design shear demand in the coupling beam that was reduced by \( R \) should be increased by \( R \times 3.5 \geq 1.0 \). This procedure has assumed that the expected ductility is shear ductility and only the design shear strength should be increased. The design shear strength in the coupling beam to be increased is the nominal shear strength determined by MSJC Section 3.1.3. This procedure is recommended for the shear design of coupling beams in a coupled beam seismic load-resisting system. The rationale for this procedure is that if the strength of the coupling beam deteriorates during earthquake shaking, the loss of coupling beam strength increases the required flexural strength demand at the base of the shear wall, but decreases the overturning axial force on the vertical wall segments.

The MSJC Standard ignores the effects of joint dimensions and joint shear on the behavior of shear walls with openings that are a part of the seismic force-resisting system. The joint is defined as that wall area that is bounded by beams and piers. The following design and detail standards are recommended:

- The minimum horizontal or vertical dimension of joint is the development length of the horizontal or vertical reinforcement used in the spandrel or pier.
- That all joints in the shear wall with openings that is a part of the seismic force-resisting system be grouted solid.
- The above two points are applicable to ordinary, intermediate, and special reinforced masonry shear walls.
- Joint horizontal shear shall be calculated for joints in special reinforced masonry walls.
The joint shear, $V_{jh}$, is calculated as:

$$V_{jh} = (M_T + M_B - (V \cdot h_c)) \div h$$

The horizontal dimension of the pier is $h_c$. The vertical dimension of the beam is $h_b$.

The nominal shear strength of the joint may be conservatively taken from MSJC Section 3.3.4.1.2(b). If horizontal shear reinforcement is required, the quantity required is $A_{jh} = [(0.5 \cdot V_{jh}) \div f_y]$. This reinforcement shall be distributed uniformly, crossing the corner-to-corner diagonal joint shear crack, and anchored with a 180° bend around the continuous vertical reinforcement in the pier.

**Concerns with the IBC 2006 Provisions**

Section 2106.5.2 of the 2006 IBC modifies Section 3.3.4.1.2 of the MSJC Standard. This modification of the MSJC Standard for nominal shear strength deletes any shear strength due to masonry and requires that the entire nominal shear strength be based on the quantity of shear reinforcement. This modification of the equation for calculation of nominal shear strength is applicable in the "region defined by the base of the shear wall and a plane at the distance $L_w$ above the base of the shear wall." This modification of the equation developed by the principal investigators on the U.S. side of the Joint US-Japan Coordinated Research Program for Reinforced Masonry (TCCMaR) appears to be taken from the 1997 UBC. A modification of the current MSJC equation is in the 1994 NEHRP (BSSC 1994). This 1994 modification reduces the participation of masonry in the calculation of nominal shear strength in a plastic hinge zone but retains the 50% efficiency factor on the contribution of shear reinforcement to the nominal shear strength. This modification of the current MSJC equation does not appear in the 1997 or later NEHRP Provisions.

The current requirement in the MSJC Standard that the factored nominal shear strength equal or exceed that corresponding to the development of nominal flexural strength appeared in the 2000 edition of the NEHRP Provisions (BSSC 2001) and the 2002 MSJC Standard. These documents do not include any modification of the current equation used for determination of nominal shear strength in a plastic hinge zone. The only possible rationale for the 2006 IBC modification of the MSJC equation is in Paulay and Priestley (1992, 600), but that is related to an entirely different design procedure. The cited experimental specimen did not require the shear strength to exceed the shear associated with development of flexural strength. The ideal flexural strength of the walls was 28% in excess of its ideal shear strength. The specimen was a squat wall that had a limited displacement ductility. The hysteretic behavior presented in the Paulay and Priestley reference shows that as the forced displacement on the specimen was increased, shear failure occurred. MSJC Section 3.1.3 adopts the recommendation in Paulay and Priestley (1992, 661) that the shear strength exceed flexural strength. This recommendation supports the current criteria in the MSJC and, as a result, neglecting the masonry shear capacity may be overly conservative.

The reason for the 0.5 efficiency factor on the quantity of horizontal shear reinforcement as given in MSJC equation 3-22 appears to be the difficulty in developing anchorage that develops yield stress in the reinforcement. The MSJC Standard in Section 3.3.3.2.1 specifies that anchorage of horizontal reinforcement be a 180° hook around the edge vertical bar. If the end of the wall is an intersection, Section 3.3.3.2.2 specifies a 90° bend into the intersecting wall and development of the reinforcing in that wall as a complying anchorage of horizontal reinforcement. Specimens using a 90° bend down into the end cell contributed to splitting (compression failure) of the compression block and, therefore, such 90° bends are not recommended.

**References**


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**Keywords**

masonry
prestressed masonry
reinforced masonry
shear walls

**How To Cite This Publication**

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Background

Wood framed shear walls with openings are commonly encountered in both commercial and residential projects. Consequently, design challenges arise where windows and shear walls compete for space along the building perimeter. This Blue Book article discusses applicable code requirements and provides guidance so that a sensible shear wall design with a clear rationale may be produced.

The critical nature of shear wall designs has increased in recent years as perimeter walls have trended toward larger and more numerous openings. Accordingly, engineers of commercial and especially residential projects of light wood frame construction are incorporating many narrow shear wall segments because architectural requirements emphasize higher dwelling densities and rooms with more daylight and better views. At the same time, the magnitude of design stresses increased in many regions with the introduction of the Near-Source Factor, $N_a$, and other force-related features required since the 1997 UBC (ICBO 1997). Also, the use of new high-strength continuous tie-down devices has eliminated a holddown design limitation that often created a practical limit of allowable stresses. With the high-strength tiedowns common on wood framed buildings three to five stories in height, shear stresses at the first level are often pushed to the code limit. Collectively, these construction and seismic code trends have resulted in higher densities, taller buildings, shorter shear wall segments, and greater forces, all of which have intensified the demands on the structural system. This intensity spotlights the necessity of good engineering practice with regard to woodframe shear walls and the treatment of openings.

A prudent shear wall scheme is commonly considered essential for good seismic performance of light wood framed construction (Breyer et al 2006). It is recognized that shear wall placement will compete with nonstructural or architectural requirements, particularly at the perimeter of the building. Therefore it is useful for designers to appreciate the nonstructural requirements, such as for large or numerous windows, in order to improve early coordination and location of shear wall segments. Designers are encouraged to address these issues early in the design process when flexibility exists. The proper distribution of shear walls is considered by many practitioners to be more of an art-form than a science, but in practice a prudent engineer will tend to minimize long drag elements and will consider the load sharing of the elements. A good design is often described as elegant in that it is simple and not wasteful of resource materials. Most importantly, a prudent design acknowledges the essential issues, rather than ignoring them. It should be emphasized to those gaining experience that no amount of additional calculations or detailing of openings will make up for a poorly planned shear wall scheme.

Opportunity for Improvement

There are several practical opportunities for improving shear wall design and performance. One example is to consider the way contractors approach wood framed exterior wall construction. Prior to the 1997 UBC, engineers in the Western United States often made a simplifying assumption of only considering the full-height shear wall segments and did not include any contribution from spandrel sheathing. This assumption, which has come to be known as “segmented design,” often differs with field conditions because:

1) Contractors may continue the sheathing above and below openings to provide a backing of even thickness for finish materials, and
2) the additional sheathing is sometimes required as a good engineering practice when calculated stresses are high.

As a result, with segmented wall design, woodframe construction with repetitive small rooms, such as low-rise hotels or apartment buildings, have a high percentage of exterior sheathing material ignored in the calculations. Thus the opportunity exists to take advantage of the repetitive stiffening and coupling action created by the sheathing below (and perhaps above) windows. The coupling of shear walls provides the efficiency of greatly reducing tiedown forces. As is exemplified in the SEAOC Seismic Design Manual (SEAOC 2002, p. 71) many western engineers add tiedowns at the window jambs as a good design practice even when not required by the calculations. Thus efficiency may be realized with a method of accounting for these tiedowns.

**Review of Code Provisions, Intent and Application**

The following pertinent IBC code sections that are related to the design of shear walls with openings are highlighted for the designer:

1) Shear wall/pier requirements, (IBC 2305.3.4 and Figure 2305.3.5)
   a) Shear wall/pier height (IBC 2305.3.5)
   b) Shear wall/pier width (IBC 2305.3.6)
2) Design for force transfer around openings (IBC 2305.3.8.1)
3) “Perforated Shear Wall” Design, (i.e. no design for force transfer around openings). (IBC 2305.3.8.2)

**Shear Wall/Pier Requirements**

Both the UBC and IBC have similar limitations regarding wood shear wall aspect ratios. Aspect ratios have become more restrictive in recent years since the 1994 Northridge Earthquake, which helped draw attention to researchers who identified problems with drift control and stiffness degradation in shear walls of short length. Some initial testing was conducted by University of California at Irvine (ATC 1995) and Tissel and Rose (1994). The 1997 UBC reduced the maximum aspect ratio, $h/w$, from 3.5:1 to 2:1. This UBC 2:1 limitation applied to Seismic Zone 4, and Seismic Zones 0, 1, 2, and 3 were allowed the traditional aspect ratio of 3.5:1 per UBC Table 23-II-G. More recently, the IBC incorporated a variation of this aspect requirement. The IBC (and NEHRP Provisions) state that all full-stress shear wall designs have a maximum aspect ratio of 2.0 but allow a stress-reduction exception that is applicable to all Seismic Design Categories. Specifically, IBC Table 2305.3.4 allows the 2:1 limitation on a wall segment or pier to be 3.5:1 if sheathing shear resistance values are multiplied by a $2w/h$ reduction factor. This factor yields a reduction from 0% to 43% as the 3.5:1 aspect ratio is approached. In contrast, the only relief provided in the UBC for the Zone 4 aspect ratio of 2:1 was at the sides of garage openings not over 10 feet in height. Note that the 5th edition of the 1997 UBC contains corrected language on exception 3 of Table 23-II-G.

To apply aspect ratio limitations to a typical geometry, consider a shear wall with top plate height of 9 feet subject to the full stress 2:1 aspect ratio. In this most common situation, the minimum full-height shear wall segments must have a width of 4 ft 6 in. Similarly, the common case of a window 5-ft tall designed for force transfer requires a pier 2 ft 6 in wide. If these aspect ratios are not met, the shear wall segment is not deemed to qualify in the lateral force-resisting system and can be ignored with respect to direct shear transfer.

Practitioners have been faced with many interpretations as to the intent of the code and how to best apply the code requirements. For example:

1. To determine shear wall height, $h$, the codes specifies “the bottom of diaphragm framing.” Current practice generally takes this as the top plate height rather than the height of the diaphragm sheathing in platform framing.
2. Regarding non-rectangular shear elements: Where a raked (sloped) top or bottom plate occurs the designer should consider taking the height, $h$, as the average top plate height in each segment. In the case of a stepped shear wall, (for example, at a stepped foundation or an area with a raised top plate elevation) the tallest height should be conservatively used and the segment formed by the
step should also meet the aspect ratio requirements. In all cases the codes indicate the width is the out-to-out dimension of the sheathing on the segment considered.

3. Regarding the minimum panel dimension of 24 in (UBC 2315.5.3, IBC 2305.2.4): The importance of this provision is that it promotes good performance of horizontal and vertical diaphragms by precluding an excessive number of panel joints, and to keep the panel joints a sufficient distance away from the boundary elements. The 1997 UBC and 2006 IBC allow undersized segments provided the edges are supported by blocking. However the triangular pieces formed in hipped roofs are rarely blocked in practice.

4. Regarding the use and approval of narrow panels with aspect ratios greater than 3.5:1, it is not clear under which conditions these walls may be site-built, and what special design and detailing provisions are mandated for mixed systems utilizing steel columns or frames.

5. Regarding perforated shear walls, the definition of element length, $l_w$, is not clear when calculating the redundancy factor, $\rho$.

As can be inferred from the preceding text there are many instances where the engineer-of-record must use experience and judgment. Engineers should consider the importance and repetitiveness of the design and generally be aware of research and empirical data including current testing, case histories, or manufacturer’s data.

For the routine design of shear walls, the aspect ratio limitations mandated by code must be satisfied. The ratios apply to the overall shear wall as well as for the piers formed by the opening (Ref. UBC 2315.1, IBC 2305.3.8.1, and NEHRP 12.4.2.9.) Once the shear segments are qualified, the designer must determine forces and design the segments.

**Shear Wall Design with Force Transfer Method (FTM)**
For lateral forces, the UBC and IBC stipulate that openings be designed by “rational analysis.” Similarly, ASCE 7-02 Sec. 9.5.2.6.2.2 requires reinforcement at the edge of openings be developed a “sufficient distance.” These statements require the designer to select a method that satisfies the basic principles of mechanics, but they cannot require perfection since all commonly used methods available today have limitations. Thus the designer and jurisdiction need to determine which assumptions to use based upon judgment, the intensity of the seismic design criteria, and local practice. For many designs, the following assumptions are used:

1. Shear forces are resisted by the sheathing
2. Bending resistance is resisted by the boundary framing
3. Shear forces are uniformly distributed along the length of the sheathing
4. The stiffness of shear wall segments is calculated as the inverse of the shear wall deflection formula provided in the code
5. The stiffness of a shear wall with an opening may be modeled with an approximate method, such as the unit-strip method used in masonry design or with the application of the Shear Reduction Factor, $C_o$. The $2w/h$ reduction factor need not be applied within the stiffness calculations.
6. Stiffness of individual wall piers may be assumed linearly proportional to wall length
7. Free-body analysis of panel areas is sufficient for determining stresses around the opening
8. Average shear stresses may be used for sizing sheathing, perimeter members, and horizontal strap reinforcement
9. Chords and collectors are idealized to accumulate forces at the perimeter; see discussion in NEHRP Commentary 12.3 (BSSC 2004b)
10. Only structural-use panels designated as a part of the lateral force resisting system are considered in the calculations.

One acceptable method of providing force transfer around window openings may be found in the SEAOC Seismic Design Manual (SEAOC 2002, p. 69). This method is analogous to a portal frame analysis, which calculates shear stresses by taking moments about each frame joint. In the Seismic Design Manual, a residential shear wall with window opening is divided into 8 panels: 3 above the window, 2 piers about the opening, and 3 panels below the
window. Average edge stresses on each free body panel are then calculated to determine the nailing pattern for the sheathing. It can be seen from the relative stresses in the 8 panels that the designer is encouraged to center openings in the shear wall both horizontally and vertically. Yet, in residential construction, a common height for the exterior window head is only about 12 in. Since the 12-in depth is much less than the typical qualifying pier width, the calculated stresses would be several times higher above the window than in the piers. Therefore this method is often inefficient for commonly encountered situations such as 5-ft tall windows within a wall 9 ft tall, unless the stresses above the window are ignored.

To overcome the portal frame limitation that occurs with the typical limited sheathing thickness above the window, other creative methods can be mentioned. For example, in multi-story construction, some engineers can ignore the sheathing over the window and provide fixity above the piers with a continuous rim member tied to the jamb posts. In this scenario, the continuous rim member and/or sheathing below the window sill elevation resists overturning of the piers. Another method of providing fixity at the top of the window piers may occur by extending the sheathing over the top plate onto the floor or roof system. This system should be avoided with conventional framing where shrinkage of joists could buckle the sheathing, and appropriate gaps should be provided in any case. In other instances, stiffening elements such as the foundation or rim member below the window opening may be designed and detailed to help couple the shear piers together. The net uplift at the window jamb may then be resisted by a combination of the sheathing below the window and a continuous rim member tied with an adequately stiff holddown device. Note that where the sheathing below the window is used to justify simultaneously occurring horizontal and vertical stresses, the resultant forces in the sheathing must be checked.

A word of caution is in order when evaluating seismic designs for low-period structures where some wall lines are dominated by short shear walls and others by long shear walls. An example of this unbalanced geometry may be a hillside home with an open front and a partial height concrete masonry unit (CMU) retaining wall with a wood cripple-wall at the back. Another common example is an R-1 occupancy apartment or motel with a centrally loaded corridor. As the 1994 Northridge Earthquake illustrated, long stiff walls become critical after initial seismic cycles soften up the short wall elements. Another significant effect is that the center of rigidity may change in subsequent earthquake cycles. Accordingly engineers should consider that earthquake performance of woodframe structures tends to ultimately depend upon the longer shear walls.

**Shear Wall Design Without Force Transfer Around Openings: The Perforated Shear Wall Method (PSW)**

The Perforated Shear Wall (PSW) Design Method is a method specially approved for designing shear walls without force transfer at the openings. Recognition of the PSW method has developed from work published by Sugiyama in 1981. These efforts were expanded by Sugiyama and Matsumoto (1993a, 1993b); the Engineered Wood Association’s APA 157, by Rose and Keith (1996); research at the University of Virginia by J. Daniel Dolan, C. Heine (1997), and Johnson (1997); the National Association of Home Builders; CUREE (Cobeen et al. 2003); Douglas (1994); Ge et al. (1991); Patton-Mallory et al. (1985); Rose and Keith (1997); Thurston (1994). An example of early structural testing research on this topic is that by Yasamura (1986). The PSW method was first codified into the 1997 SBCCI Standard Building Code and has extended into the provisions given in NEHRP 12.2, AF&PA SDPWS 4.3.3.4, and IBC 2305.3.8.2.

The PSW Design Method is based on empirical data derived from the racking behavior of shear walls in tests. Only one pair of tie-downs is used at the far ends of the PSW. On long walls with multiple openings no additional tie-downs at the window or door jambs are required. The new PSW method found in the IBC shows a table of adjustment factors representing the loss of capacity caused by the un-reinforced openings (IBC Table 2305.3.8.2). Accordingly the Shear Resistance Adjustment Factor, \(C_{o}\), is selected from entering the table given 1) the percentage of qualifying full height segments and 2) the ratio of maximum opening vertical dimension to the wall height.

The IBC also specifies that the following limitations must be satisfied:

1. Full-height segments are required at each end of the PSW
2. Maximum allowable shear of 490 plf on the full-height segments
3. No out-of-plane offsets within the PSW
4. Continuous collector elements
5. Uniform height required

In addition, boundary nailing is required on all edges of the openings, and height-to-width ratios are applied to each recognized segment as well as to the wall as a whole. With these limitations the PSW method is readily applied to structures with a low-to-moderate percentage of openings and low-to-moderate stresses at the shear walls. The PSW method is very useful in many parts of the United States, especially where it is common to use wood structural-use panels along the entire exterior of the building. Often, these panels act as backing for finish materials such as proprietary one-coat stucco systems or for brick veneer. In these cases the PSW method provides a sensible way to recognize the benefit of the redundant sheathing to the lateral strength and stiffness of the structure, without adding tiedowns at every opening that may occur.

For designs that face more intense seismic challenges, the PSW method faces some limitations. In high seismic areas the reductions of capacity are significant enough to limit application to one- and two-story structures, because shear demands greater than 490 plf (ASD) are typical in light frame construction over two stories. Note that although ASCE 07-05 suggests a higher value be permitted, this code change was disapproved in the 2006 IBC. Another issue is that it is difficult to meet the PSW limitations commonly occurring in residential architecture. Consider a new residential structure with a typical exterior bedroom about 12 feet wide, with a 9-ft-high top plate. As with much new construction, the bedroom shear wall is also part of a small out-of-plane jog in plan at the exterior wall. For a window centered in the wall, the unreduced PSW method requires 4 ft 6 in shear segments on each end, since full-height segments are required with \( h/w \) limited to 2.0. Subtracting the shear walls from the room length yields only a 3-ft wide window, which is substantially less than the typical architectural requirement. If the end segments are shortened from 4 ft 6 in to 2 ft 7 in (i.e. 3.5:1 aspect ratio) an additional 43% loss of capacity occurs from the 2w/h factor per IBC Table 2305.3.4. In PSW designs, the aspect ratio reduction of 43% is cumulative with the Shear Resistance Adjustment Factor of 0.63, which yields a net capacity of only 36%. On the other hand, if the wall is designed for force transfer using FTM, the 2:1 aspect ratio may be applied to the window pier, and therefore with a minimum pier length of 2 ft 7 in no reduction is required. Furthermore, the FTM method is not bound by the 490 plf allowable shear limitation, so double-sided sheathing with capacities up to 1740 plf could be specified. Consequently, the PSW method has some severe stress geometric and limitations.

For situations where the PSW method is chosen the designer must be aware of several other items:

1. The PSW methodology may not be accurate with openings having extreme aspect ratios. For example, application of the PSW method to an 8 ft by 8 ft shear wall with an extreme 1 ft by 6 ft vertical hole yields no appreciable reduction. Likewise, a PSW 8 ft by 18 ft with a 3-ft tall by 10-ft wide window yields only a 6% reduction. Engineering judgment suggests that a smaller, well-centered opening should have less effect on performance.

2. Some engineers have indicated concern that the PSW method discourages holddowns at the edges of full-height openings, which has been regarded as a good practice. Consider a wall with two full-height segments separated by a large full-height opening. By inspection this geometry would substantially benefit from having a pair of holddowns to stabilize each full height shear wall. Additionally, where full-height openings exist, the results of the PSW method are particularly sensitive to the distance between the shear segments. Arguably, once the walls are separated, the distance between them should have little affect on the capacity.

3. The PSW method does not penalize openings unless they exceed \( h/3 \). The SEAOC Seismology Committee cautions designers to consider the effect of any notch-like openings located near the top plate or sill plate. Also, note that if the equations are used in lieu of the table values, Co values will exceed 1.0 at opening heights less than \( h/3 \) and therefore values over 1.0 must be truncated as shown in the tables. In other words, cutting a hole in a shear wall should never produce an increase in capacity.
4. The empirical values listed in the IBC and NEHRP do not list the common top plate height of 9 ft. The codes state that intermediate sheathing ratios may be interpolated but do not indicate if intermediate maximum opening height ratios are to be interpolated. For production design, engineers may utilize the underlying empirical formulas. In doing so, they should be aware that typographical errors exist with the empirical PSW formulas shown in the 2000 NEHRP commentary (BSSC 2001) (C12.4.3.3a1 and C12.4.3.3b). Readers are referred to 2003 NEHRP (BSSC 2004a, 2000b), APA 157 (Rose and Keith 1996), or the 2001 National Design Specification (AF & PA, 2001) that is adopted as ANSI standard NDS-2001, because they provide the correct formulas that are not shown in the IBC. Note that to develop the Co values shown in the tables of the IBC and NEHRP, all openings are conservatively assumed to share the same maximum clear height. Another difference to note with APA 157 is that it is a reduction applied to the entire wall rather than to the qualifying segments. Therefore the full-height sheathing ratio must be applied to APA 157 formulas to match the tabular values of the IBC and NEHRP.

5. When calculating overturning forces it is important to recognize that tension and compression are not equal as some could presume with 2000 NEHRP Sec. 12.4.3.4.4 and IBC Equation 23-3. Compression forces, especially on multi-story construction, are usually greater due to the presence of a resisting moment and differing basic load combinations. Note also that if a header or beam frames into the ends of the perforated shear wall, the greater load (including live load) must be considered in compression rather than the lighter dead load couple that is assumed for resisting tension. Further, when sizing the compression member note that the buckling capacity will usually be critical, rather than the capacity of the selected tie-down device.

6. Where stacked openings occur it is not clear how the Co values given in IBC Table 2305.3.7.2 are applied. Stacked openings are common on walls between 10-ft and 20-ft tall where the PSW method is permitted.

Future Research
Wood shear walls with openings are very complicated compared to the mathematical models used in most design offices. Some researchers have developed sophisticated modeling techniques. White and Dolan (1995), for example, refined a finite element program that calculated forces and stresses of shear wall elements. However, like many mathematical tools, typical field variations such as overnailing, issues with panel placement, and the presence of random splits or misalignments can challenge the uniform assumptions. Additionally there can be great significance to the effect of finish materials and walls that are not integrated into the model. On larger buildings with many interior walls there can be a significant capacity that is not considered as a part of the lateral force-resisting system, although it is also acknowledged these redundancies are typically brittle and may not be reliable lateral force-resisting elements. Future methods should improve the accuracy of our analytical tools. In addition, the industry can benefit from design methods that reinforce window openings rather than merely estimating the residual strength in the shear wall.

Keywords
force transfer method
holddowns
tiedowns
openings in wood shear walls
perforated shear wall
shear walls
wood shear walls
woodframe construction
wood framed construction

References


How To Cite This Publication
In the writer’s text, this article should be cited as:

(SEAOC Seismology Committee 2007)

In the writer’s reference list, the reference should be listed as:

The Purpose and Definition of Hold-downs

Hold-downs (or tie-downs) are load path elements that resist uplift and overturning effects in wall panels with wood or metal light framing. They function by connecting the boundary element of a wall panel to supporting structural elements such as foundations, grade beams, slabs, other frame or wall elements, or to connect with hold-downs in an adjacent story. According to ASCE/SEI 7-02 (ASCE 2002) section 9.2.1, hold-downs “are intended to resist load without significant slip between the device and the shear wall boundary element or be shown with cyclic testing to not reduce the wall capacity or ductility.” This definition has been omitted from ASCE 7-05, but AF&PA Special Provisions for Wind and Seismic (AF&PA 2005) defines a Tie-Down (Hold Down) as “a device used to resist uplift of chords of shear walls.” Note that the language used in the 2004 AISI Standard for Lateral Design (AISI 2004) also defines boundary elements as shear wall chords.

For the purposes of this discussion, the hold-down assembly consists of a steel body and its connections via nails, screws, or bolts to adjacent structural elements, to the wall edge member or post, and an anchor rod to the supporting structure. Another type of hold-down assembly, continuous rods with couplers, are increasingly used in multi-story structures to create a continuous hold-down over several stories of stacked shear walls (Nelson and Patel, 2003).

History

Hold-downs first came into use with the introduction of plywood-sheathed shear walls around 1957. At that time, earthquake design loads calculated on an equivalent lateral force basis for box or shear wall buildings in California used a factor of 0.133g, and the 1955 Uniform Building Code (UBC) limited panel height-to-width ratios to a maximum of 3.5 to 1. The hold-downs were intended to resist any net overturning or uplift at the ends of the panel. Engineers initially designed hold-downs as standard details using long-legged, rolled steel angles, with UBC allowable shear values for bolts in wood posts and allowable anchor bolt tension values in concrete. The thickness of the angle was based on bending stress due to the moment of the anchor bolt tension value times the distance from the anchor to the bearing face of the post. Plate washers were typically installed on the far side of the post because the calculated tension load on the bolt (due to the eccentricity of the center of the hold-down bolt and the center of the post) exceeded the bearing capacity of cut washers.

Soon, manufacturers began producing bolted hold-downs using 3- to 7-gauge bent plate channels with tapered flanges and welded bases up to 3/8 inch thick. Allowable tension loads were based on the lesser of UBC total bolt values or the maximum tension value from tests of the hold-down in a steel jig, divided by 2.5. No requirement was listed in the manufacturers’ tables requiring plate washers.

A similar test program was later used to develop and market 7- to 14-gauge bent plate hold-downs with shorter tapered flanges and bent plate bases. The observed successive failures of manufactured hold-downs, installed without plate washers and exhibiting excessive vertical deformations, created some revisions to installation instructions.

Testing of narrow (h/d = 3.5) shear walls (Shepherd and Allred, 1995) also indicates some concern with holddown connections at the ends of the walls. Observed failures included splitting of the wood post at the bolt holes, and substantial rotation and elongation of the holddowns. Sliding of sill plates after the sill bolts split the wood was also observed. It is possible that splitting of the sill plate is related to cross-grain bending due to deformation of the hold-down. The American Plywood Association (APA) is also studying this issue. Early results of the APA tests were reported by Commins and Gregg (1994), and indicate that plywood nailing and holddowns distorted significantly, the National Design Specification (NDS) bolt spacing values may be inadequate, and code deflections were...
exceeded when code forces were applied. Unfortunately, though the testing was cyclic, displacements were not taken to the levels that might be experienced during an actual earthquake, but rather to levels near the much lower values that might be used in code design (California Seismic Safety Commission, 1994).

Design Loads and Capacities

The design loads for hold-downs in wood construction are based on the load combinations of ASCE 7-05. The design loads for hold-downs in cold-formed steel construction are based on ASCE 7-05 and 2004 AISI sections B4 and C5, which requires boundary members of shear walls and the uplift anchorage thereto to have the nominal strength to resist the induced forces including the amplified seismic loads. Therefore, hold-downs in wood construction can be designed for the basic load combinations of ASCE 7-05 section 12.4.2.3 while hold-downs in cold-formed steel construction must be designed for the load combinations including the over-strength factor of ASCE 7-05 section 12.4.3.

2006 IBC section 2305.1 references 2005 AF&PA NDS (AF&PA 2005a) as its standard for wood design and the 2005 AF&PA National Design Specification for Wood, Special Design Provisions for Wind and Seismic (AF&PA 2005b), or SDPWS, as its standard for wind and seismic provisions for wood design. However, the design of hold-downs is not addressed in the 2006 International Building Code (IB) or these AF&PA documents, other than 2006 IBC section 2305.3.7, which requires anchoring devices where the dead load is not sufficient to resist uplift due to overturning moments based on allowable stress design load combinations found in IBC chapter 16.

In light-framed shear wall buildings both the hold-down and its anchor should be designed to be as ductile as the shear wall, or the less ductile elements should be designed for the nominal strength of the structural panel sheathing. This is not specifically required by the building code, but it is good design practice. Nominal strengths of shear panels are divided by a safety factor, Ω for ASD design, or multiplied by a resistance factor φ for LRFD design. For wood construction, 2006 IBC Table 2306.4.1 lists ASD values based on a safety factor of 2.0, and 2005 AF&PA SDPWS Table 4.3A lists the nominal shear values of panel sheathing, which can then be adjusted per section 4.3.3 for ASD design (Ω = 2.0) or LRFD design (φ = 0.80). For shear panels on cold-formed steel, 2004 AISI Standard for Lateral Design Section C2.1 lists safety factors and phi factors as Ω = 2.0 for ASD design and φ = 0.80 for LRFD design. For shear panels on cold-formed steel not listed in the tables, 2001 AISI Section F gives guidelines for determining ASD safety factors and LRFD phi factors based on how the panels were tested. These values vary significantly depending upon the number of tests run, standard deviations, and other parameters.

Over-strength requirements. 2004 AISI Standard for Lateral Design Section C5.3 specifies that for shear walls in steel construction “boundary members and anchorage thereto shall have the nominal strength to resist the amplified seismic loads, but need not be greater than the loads that the system can deliver.” This section essentially requires the shear wall boundary member and hold-down to be designed for the amplified seismic loads similar to design of columns of frames so that shear yielding of the panel is the predominant yielding mode. For these elements, the nominal strength does not need to be reduced by a safety factor or phi factor. This over-strength approach is not required for the design of wood shear walls, but it might be appropriate given the more predictable yielding behavior of wood shear panels compared to their boundary elements and hold-down anchorage.

Discontinuous walls. ASCE 7-05 section 12.3.3.3 requires the special load combinations with Ω₀ for elements supporting discontinuous walls when specific irregularities exist. In these cases, it is the SEAOC Seismology Committee position that the special load combinations need not be applied to the design of the shear wall nailing, the boundary member, or the hold-down but should be applied to any non-ductile anchorage of the hold-down. Also, note that the UBC provision specifically exempted concrete slabs supporting wood shear walls. Although 2003 IBC section 1620.1.2 modifies ASCE 7-02 essentially to match the UBC, neither ASCE 7-05 nor 2006 IBC makes this exception. In lieu of using Ω₀ times the design seismic load, it is also acceptable to limit the amplified seismic force to the maximum the system can deliver. For example, the yield strength of the hold-down rod, including the expected material strength and strain hardening, could be used as the “maximum force the system can deliver” to the element supporting the discontinuous wall.
Capacity. In *NEHRP Provisions* Seismic Design Categories D, E, and F, it is the position of the SEAOC Seismology Committee that hold-down connectors should be designed using either values from approved cyclic tests or 75 percent of the earthquake design values from non-cyclic tests.

Furthermore, it is the Seismology Committee position that through-bolts connecting the hold-down to the wood end post or other wood framing should have steel plate washers as shown in Table 1. This requirement was developed in response to split posts and failed bolts observed after the 1994 Northridge Earthquake. Increased contact area between the washer and the post, as well as re-tightening of the bolts, is expected to improve performance.

The Committee has proposed adding these requirements as part of the California Building Code.

**Table 1.** Recommended minimum washer size for use with hold-down connectors to wood end posts

<table>
<thead>
<tr>
<th>Through-bolt diameter (in)</th>
<th>Minimum steel plate washer (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>3/16 x 2 x 2</td>
</tr>
<tr>
<td>5/8</td>
<td>1/4 x 2-1/2 x 2-1/2</td>
</tr>
<tr>
<td>3/4</td>
<td>5/16 x 2-3/4 x 2-3/4</td>
</tr>
<tr>
<td>7/8</td>
<td>5/16 x 3 x 3</td>
</tr>
<tr>
<td>1</td>
<td>3/8 x 3-1/2 x 3-1/2</td>
</tr>
</tbody>
</table>

Hold-down Deformation

ASCE 7-05 section 12.12 and Table 12.12-1 set deflection limits for all building structures based on the type of lateral load-resisting system. Section 2305.3.2 of the 2006 IBC provides a general deflection limit based on all elements maintaining their structural integrity as well as an equation that incorporates hold-down deformation for estimating the deflection of a blocked wood structural panel with uniformly distributed fasteners. Section 4.3.2 of (AF&PA 2005b) has a deflection equation, though it has only three terms rather than four (it incorporates the previous second and third term into one term). Originally, 1997 UBC section 2315.1 set deflection limits for wood shear walls and referred to UBC Standard 23-2 (section 23.223) for a method of estimating elastic deflections that accounts for deformation in the hold-down. The equation in Standard 23-2 was also included in 2000 NEHRP Commentary section 12.4 (BSSC 2001), the APA diaphragm and shear wall guide (APA 2001), and 2003 IBC section 2305.3.2. (In the IBC, the final term of the equation is defined differently but represents the same thing as in the other versions.) The derivation and history of this equation are described in APA (2000).

The equation in Standard 23-2 defines the wall deflection as the sum of four contributions: flexural deformation of the wood elements, shear deformation of the wood elements, deformation due to nail slip, and rigid body rotation due to hold-down deformation or slippage. The total hold-down deformation, $d_{a}$, can include contributions from:

- slippage of the hold-down relative to framing members
- oversized holes at through bolts
- distortion or elongation of the hold-down body or connectors (e.g. bolts, screws, nails)
- anchor rod elongation
- additional slippage or looseness due to wood shrinkage
- additional slippage or looseness due to shrinkage compensating devices and other load transferring devices (e.g. bearing plates, rod couplers, plate washers, etc.)
- additional slippage or looseness due to localized crushing of the sill plate (from gravity or earthquake loads).

Since displacement associated with some of these items can be significant, neglecting their contribution to the overall wall lateral displacement results in substantially lower story calculated drifts than would actually occur. This is especially the case for narrow shear walls (e.g. shear walls with a height-to-width ratio more than 1:1) since the variable $d_{a}$ plays a larger role in the lateral displacement of such walls based upon wall geometry. In addition to including these effects in the design, steps should be taken to minimize the deformation. For example, 2000 NEHRP
Design examples using the IBC deflection equation are given in *Structural/Seismic Design Manual (SEAOC 2006)* examples 1B, 2, and 3. The last example is for cold-formed steel studs, and it should be noted that for the last term in the displacement formula in part 2a of the example should be $d_\text{u}h/b$, not $d_\text{u}$ alone. Examples are also given in the 2003 *NEHRP Recommended Provisions: Design Examples* (FEMA 451, Section 10.1.4.4.), though the references in that example are based on the 2000 NEHRP rather than the 2003 NEHRP. For specific design values, APA (2000, 2001) refers users to information provided by the hold-down manufacturer. Nelson and Patel (2003) discuss briefly the relative magnitude of these deflection contributions. In general, however, comprehensive load-deflection data to support estimates of $d_\text{u}$ are not yet available. The ICC’s relatively recent Acceptance Criteria for Hold-downs (Tie-downs) Attached to Wood Members (AC 155) (ICC ES 2005) provides a testing standard for hold-downs. This standard includes acceptable design load value determination based on several criterions, including a tested displacement criterion of the hold-down. The displacement criterion provides two different testing conditions: one for the isolated hold-down device itself on a steel jig, and one for the hold-down as part of a wood assembly as would be found in its typical wood post installation. For the latter, no wood sheathing is present, because the testing is for the hold-down device alone. At the time of writing this article, the ICC’s AC 155 is not a required standard for ICC Evaluation Service reports listing acceptable design load values for hold-downs, and study by the Structural Engineers Association of San Diego indicates that there are some popular large capacity hold-downs that do not meet that displacement criterion. Some hold-down manufacturers have tested sub-assemblies of the bare post, hold-down, and sill, but these were based on testing prior to the advent of the AC-155 standard for such tests.

Since the 1994 Northridge Earthquake, manufacturers have incorporated features to reduce hold-down deformation. The hold-down is now commonly connected to the post with 1/4 inch diameter lag screws instead of through-bolts, which were prone to slip within oversized bolt holes. Also, so-called “pre-deflected” hold-downs are configured so as to minimize distortion and elongation in the body of the device. Using bent plate steel assemblies attached to the wood post with multiple 1/4 inch lag screws significantly reduces the flexing and distortion of light-gauge hold-downs. Alternatively, hold-down devices using a short steel tube section sandwiched between and through-bolted to wood posts can also reduce the flexing and distortion of the hold-downs.

For multi-story continuous tension rod hold-down applications, shrinkage-compensating devices are now frequently utilized to minimize the effects of slippage or looseness of the tie-down system associated with floor joist shrinkage occurring in the depth of the member. Such devices help maintain continuous tension rod engagement with the tie-down system, even as the floor joists shrink vertically, which would normally cause disengagement of the rod from the relevant tie-down system components. In other words, because the rod would not shrink as would wood floor joists, there would be a resultant gap between the load transfer components and the continuous tension rod, which must first be closed before the tie-down system would resist load. Such behavior can lead to potentially excessive shear wall lateral displacements and, therefore, excessive story drifts. In fact, shrinkage-compensating devices are required by some building departments (City of San Diego, 2003) when such multi-story tie-down systems are used, which is consistent with recommendations noted by Nelson and Patel (2003). For multi-story hold-down systems not using a continuous tension rod, other factors commonly associated with typical single story hold-downs already discussed must be considered in the displacement associated with the hold-down, in addition to that of wood shrinkage.

Previous to innovation such as pre-deflected hold-downs, a team led by Ben Schmid tested 4 ft wide by 8 ft tall panels with 1/2 in plywood sheathing and doubled 2x4 edge studs. They found that the proprietary light-gauge hold-down with two 3/4 in diameter bolts to the edge member accounted for 50% of the top horizontal displacement.

In Seismic Design Categories D, E, and F, the SEAOC Seismology Committee has the following recommendations to minimize displacement associated with hold-downs:
- use pre-deflected type hold-downs or similar devices to reduce hold-down device deformation
- use hold-downs with lag screws as opposed to through bolts
• if through bolts are used, provide bearing plate washers (this was a code provision in the 2001 CBC, Section 2315.5.6, added as part of the November 7, 2003 Emergency Supplement)
• hold-downs should be re-tightened just prior to covering of the wall framing
• for multi-story continuous tension rod applications, use shrinkage compensating devices and/or require lumber moisture content less than 12% at time of hold-down installation The 12% criterion is based on second draft versions of the ICC proposed AC 348 for continuous tie-down systems, item 1.2 (ICC ES 2006).
• for large compressive load conditions (e.g. where end posts already take substantial gravity loads), have the post bear directly on a steel base plate.

Drift limits and Performance. Code requirements generally allow a story drift up to 0.025 times the story height. Such deformations can be accommodated by plywood or oriented strand board (OSB), but nonstructural partitions of gypsum wallboard have been shown to crack at much smaller drifts, around 0.01/h (CUREE, 1999; Schmid, 2002). While nonstructural damage in buildings that experience design earthquakes is frequently acceptable, it can also require expensive repair. Using hold-down assemblies that deform and deflect minimally can help control damage and thus achieve higher performance.

Load-slip curves for nails and wood sheathing indicate initial yielding of nails occurs approximately at 0.1 in, according to test data from APA Report No. T2004-14. Shear wall tests indicate yielding of the shear wall between approximately 0.2 in and 0.5 in of top-of-wall lateral displacement (i.e. drift), with the variance sometimes associated with nail spacing and/or wall length. This is based on comparisons of March, 2002 CUREE I.4.4 testing and December, 2001 City of Los Angeles (CoLA) testing at the University of California Irvine (SEAOSC-UCI December 2001). Splitting of the foundation sill plate is frequently the observed damage in shear walls based on tests and post-earthquake observations (ATC, 2007, BSSC 2004-2003 NEHRP Commentary for Section 12.2.3.11 and 12.2.3.12, Nelson 2001). This problem of sill splitting is more pronounced for narrow shear walls of which large hold-downs are often associated in light wood-framed structures. In fact, in static load tests of narrow plywood shear walls, the sill plate splits when the sheathing panel corner had a vertical displacement of approximately 0.2 in (ATC 2007). Excessive hold-down deformation has also been cited for causing failure of the nails connecting the sheathing to the sill plate (2003 NEHRP Commentary for Section 12.2.3.11 and 12.2.3.12). For both dynamic and static load tests of narrow walls (ATC 2007) slippage and/or deformation of the hold-downs was cited as the initiating event for eventual shear wall failure.

As light-frame design progresses toward the use of strength design and deflection control of light-framed residential structures, hold-down manufacturers should be encouraged to publish load versus uplift data on the hold-down alone up to strength limit cyclic forces of the sheathing.

Design of Adjacent Members
Posts connected to hold-downs must be designed for the worst case of combined stresses experienced during cyclic loading. The designer should consider combined tension from uplift (accounting for the net section at hold-down connector holes) and bending due to anchor eccentricity relative to the hold-down post. The effects of anchor eccentricity are difficult to quantify with typical analytical models. Component testing has reproduced combined tension and flexure failure of hold-down posts similar to the brittle failures observed after the 1994 Northridge Earthquake (Nelson and Hamburger, 1999). However, the poor-performing test specimens did not use plywood sheathing. Later tests with sheathing have suggested that moments due to anchor eccentricity are largely resisted by the stiff sheathing and its nails. This indicates that edge-nailed hold-down posts within the limits of shearwalls are effectively protected from the worst combined stresses (EQE International, 1999). However, if a shear panel fails by tearing of its sheathing or withdrawal of its nails at the lower corners of the wall, or if a hold-down post is not located within a shear wall, then the post would be subjected to some eccentricity effects during cyclic loading. To resolve this issue, ICC Acceptance Criteria AC-155 now requires hold-down manufacturers to state clearly in evaluation reports that wood members must be checked for any eccentricity effects. Discussion of the potential mitigating effects of sheathing is left to the discretion of each manufacturer (Stochlia, 1999), and accounting for any expected eccentricity effects is ultimately the designer’s responsibility. Hold-down posts should also be designed for...
compression stress parallel to grain, which becomes more significant in multi-story stacked shear walls and with larger floor-to-floor heights (Nelson and Patel, 2003).

Sill plates under the ends of hold-down posts must be checked for compression stresses perpendicular to grain. This may govern the size of the hold-down post, especially in cases where the hold-down post also serves as support for a beam or header. An alternative to increasing the size of hold-down posts, where sill plate crushing and its associated deflection governs, is to stop the sill plate short of the post and allow the post to bear directly on a steel base plate or connector plate to the foundation. This bearing plate should be designed to effectively transfer the compressive load to the foundation.

The foundation or structural base should be designed to resist the hold-down anchor force and provide as much ductility as the shear wall develops; or it should be designed for increased forces proportional to the ductility ratio.

**Cold-formed steel framing.**

For cold-formed steel stud framing using structural panels, the design loads are similar to wood framing. Sheathing values for ASD and strength design are provided in 2004 AISI Standard for Lateral Design. The hold-downs are typically vertical light-gauge straps welded to a base angle or bent to form a base, and the fasteners are typically multiple self-tapping screws. The studs at the anchor are typically back-to-back C-shaped, which should be suitably fastened together above the hold-down strap to transfer the sheathing forces at the panel edge to the anchor. The hold-down studs should be checked for buckling as well as tension forces.

Buckling of studs in the UC Irvine tests (SEAOSC-UCI 2001) can be detected on cyclic load-displacement plots as a sudden loss of panel strength. Increasing stud gauge size and/or horizontal blocking might be required to maintain the panel strength through the design ductility level. Stud buckling is generally not related to the hold-downs.

Since deformation of the hold-down base can be a major contributor to panel deflection, manufacturer’s listed hold-down capacities and deformations should be reviewed on a strength-loading basis.

**Recommended Research**

Improved design and reliable performance of hold-downs will require a standard for sub-assembly testing that still does not exist. While such a standard must resolve a number of issues, it is the SEAOC Seismology Committee position that specimens for hold-down testing and qualification should use the manufacturer-recommended connectors and the minimum-size end posts for which the hold-down is intended. Also, manufacturers’ design tables should give the minimum post size required to obtain the listed hold-down capacity.

As recommended by 2003 NEHRP Commentary section 12.2.3.11 (p.237), design values should be based on cyclic tests, using a pattern similar to the “sequential phased displacement” tests developed at Virginia Polytechnic Institute by Daniel Dolan. Loading protocols should also be studied and standardized for this type of testing.

Owners are typically unaware of the extent of costly damage that can occur in moderately intense seismic events. Well-designed and properly installed hold-downs can play a major role in limiting drift and therefore damage.

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**Keywords**

hold-down
tie-down
light-frame wood
woodframe
light-frame metal

**How To Cite This Publication**

In the writer’s text, the article should be cited as:

(SEAOC Seismology Committee 2008)

In the writer’s reference list, the reference should be listed as:

Introduction

The subject of this article is the anchorage of wood-frame sill plates on structural walls in light-frame construction. Wood-frame shear walls have traditionally been connected to concrete foundations with cast-in anchor bolts or post-installed anchors in accordance with varied local practices. At shear wall locations, substantial loads are assumed to load the anchor bolts in pure shear parallel to the concrete edge. For the purposes of this article the following are assumed:

- Typical cast-in place “L-bolt,” minimum 7-inch embedment
- Bolt diameter of nominal ½ inch through ¾ inch
- Standard or 3-inch square plate washer with standard nut
- Bolts assumed to act in pure shear, loaded parallel to free edge of concrete
- Bolt corner distance minimum 8 inches
- Preservative-treated wood sill plate (2x4, 2x6, 3x4, 3x6, etc.)
- Typical wood-frame construction with redundant anchor bolts
- Foundation minimum f’c=2500 psi, conventional or pre-stressed concrete.

The change of model codes in California in January 2007 from the 1997 UBC to the 2006 IBC required a number of fundamental changes to the accepted design practices of wood-frame sill plate anchorage in light-frame structures. A significant change to design practice was also necessary to apply the IBC provisions for the seismic design of anchor bolt connections occurring near a concrete edge. These changes have been a source of much discussion and frustration for code users in high seismic areas subject to the IBC and ACI codes.

Two sensitive assumptions that affect the ACI Appendix D calculation are the ductility parameter and the cracked concrete parameter. The ductility parameter of IBC 1908.1.16 [D3.3.5] alone requires a 60 percent reduction to the connection capacity in concrete if the attachment to concrete is not ductile at the concrete design strength. (ACI 318-08 has reduced the reduction to 50 percent in light-frame construction.) The resultant low concrete capacity values indicate that a failure of the connection is expected to occur in the concrete long before it occurs in the anchor bolt or the wood sill plate, which is counter-intuitive. The SEAOC Seismology Committee performed a literature search of anchor bolt testing for wood sill plates with small concrete edges distances and discovered very limited research was available. The SEAOC Seismology Committee then decided to embark on an anchor bolt testing program. Using the Tyrell Gilb facility in Stockton, California, members of the SEAOC Seismology Light-frame Subcommittee conducted the first test program of its kind where the behavior of light-frame wood sill plate anchorage at small edge distances was targeted. Additionally, the test program included non-destructive impact-echo readings to continuously monitor the progression of any delaminations in the concrete. The results of this testing program are published in the document “Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances,” dated March 29, 2009. This report is available for download from the SEAONC website: www.SEAONC.org/member.

The SEAOC test data show that the yield strength of the wood sill plate connection governs over the strength of the concrete in the subject connections. This component testing was necessary to determine the specifics of the connection behavior, particularly the large amount of yielding the bolts achieve above the concrete surface and the beneficial clamping effect due to the square plate washer.
In this article we present additional commentary to the test report findings and review the underlying assumptions that may be appropriately considered by the designer. The recommendation is presented that the subject anchors may be conservatively designed assuming a wood yield mode as predicted by the yield limit equations associated with Mode III, and Mode IV behavior in the ANSI/AF&PA NDS-2005 National Design Specification® (NDS) for Wood Construction. These values are subject to the same limitations as NDS Table 11E and are included at the end of this article for reference. These values do not apply to anchorage in light-weight concrete, post-installed anchors, or anchorage of cold-formed steel track. Finally, recommendations for further testing are discussed.

**Background**

In California, the design procedure and code-prescribed capacity of the subject bolts had not changed since the values were first tabulated and introduced in the 1979 UBC. In the IBC jurisdictions outside California, new ACI strength-based provisions for the design of seismically loaded cast-in anchors have been a part of the IBC since the 2000 edition. Regarding the provisions of 2006 IBC, which are currently applicable in many states, anchor bolt design is covered in IBC sections 1911 (Allowable Stress Design) and 1912 (Strength Design). IBC 1911 requires that with any seismic loading, anchor bolt capacities must use a strength-based design procedure. Per IBC 1912, the subject L-bolt is specifically required to be designed to the requirements of ACI 318 Appendix D provided its application “is within the scope of the appendix.” The strength design of anchors that are not within the scope of Appendix D shall be designed by an “approved procedure.” Therefore the subject anchor bolts are required to use strength-based design for seismic loads, but for wind loads the anchor bolt capacities may be taken from IBC Table 1911.2, which still contain the historical values used prior to the IBC.

The scope and provisions of ACI 318 Appendix D resulted from many years of testing and substantial effort directed at providing designers more transparency into the limit states associated with various classes of concrete anchorage. Wood sill plate anchorage forms a small subset of possible anchorage conditions covered by Appendix D. This connection is of greater regional importance than international importance, and there was a gap in the literature addressing this condition prior to the SEAOC testing. As a result, the present code provisions did not fully anticipate this narrow but important condition, and the generalized provisions produced design results inconsistent with the needs of light-frame design.

The problems light-frame designers have faced with the ACI Appendix D provisions are rooted in the very low capacity values that seemed to be required relative to past practice. As described herein, proper application of the ductility and cracked concrete parameters provide a rational, usable set of bolt values. Such a rational anchor bolt value should embody the following characteristics:

1. The capacity is internally consistent with other material chapters (e.g. shear capacity due to embedment in concrete should be proportionately stronger than masonry or wood).
2. The seismic capacity versus wind capacity is internally consistent with that required for other code-approved components and assemblies.
3. The design capacity is not overly sensitive to any particular assumption. (For assumptions that are highly sensitive by nature, it is appropriate to use a continuous function or finely divided steps).

Light-frame designers have derived bolt values through Appendix D on the order of one-quarter to as little as one-fifth of the traditional value when assuming a non-ductile connection and cracked concrete. Such a result is very low and leads to a design solution that would be inappropriate for the wood sill attachment of many code-listed shear wall systems. For example, some designers have derived a capacity of approximately 300 pounds (ASD) for an anchor that traditionally carried approximately 1200 pounds (ASD). Accordingly, a fairly heavily loaded shear wall that would have traditionally required two anchors per stud bay would now require eight anchors per stud bay, which do not physically fit.

A final complication has been the inconsistency of design capacities determined by different designers. The traditional practice of using table values for anchor capacities was replaced by a design procedure with over a dozen variables. Amid the added complexity, practitioners have questioned the marginal benefit in implementing dramatic
changes to the anchor bolt design methodology. Since issues with the old values were not apparent, the need for substantial change was puzzling.

Testing
The primary goals of the SEAOC Anchor Bolt Test program were to:

1) Determine whether the wood connection yielding controls the connection capacity when loaded parallel to an edge and if the equations found in each material standard are good predictors of behavior.
2) Determine whether the connection exhibits ductile behavior.
3) Propose rational design capacities for the connection.

It was decided to test the 5/8-inch diameter bolts since they are representative of most medium and heavy duty shear wall applications. While much residential concrete construction is specified at f’c=2500 psi, in-service concrete is expected to experience some strength gain over time. For this reason, a range of 2500 to 3000 psi was specified for the test concrete compressive test. In actuality, the highest compressive test cylinder result was 2710 psi. As also detailed in the SEAOC “Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances,” the tests included two unique features. First, the effect of friction was isolated on half of the tests by providing a lubricated polyethylene membrane at the wood-concrete interface. This allowed the contribution of friction to be better understood from the test data. Second, impact-echo testing was conducted during the test to continuously monitor the status of delamination that developed in the concrete that may not have been visibly apparent. Aside from these unique features, every effort was made to test materials representative of the most common shear wall connections.

The independent variables tested were:

<table>
<thead>
<tr>
<th>Item</th>
<th>Configuration Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sill plate size</td>
<td>2x4, 3x4, 2x6 and 3x6</td>
</tr>
<tr>
<td>Anchor bolt edge distance</td>
<td>1.75 inches or 2.75 inches, dependent upon sill plate</td>
</tr>
<tr>
<td>Testing protocol</td>
<td>monotonic versus pseudo-cyclic</td>
</tr>
<tr>
<td>Wood-concrete interface condition</td>
<td>friction versus “frictionless” membrane</td>
</tr>
</tbody>
</table>

To properly generate test data for the purpose of assessing behavior, a new displacement based loading protocol was developed. Using data from an initial set of monotonic pull tests, cyclic tests were calibrated so that damage produced by the test would best represent actual in-service failure modes. For the new protocol, the SEAOC Seismology Committee used a hybrid approach essentially taking the CUREE protocol with additional cycles added at low load levels. Independently, the SEAOSC sequential phased displacement (SPD) loading was used on several tests to compare results.

Findings
The first result to note was that the monotonic tests were an accurate predictor of the elastic performance characteristics exhibited in the cyclic tests. Once the anchors were loaded to approximately 5000 pounds, the anchors slowly started to exhibit some plastic behavior as further displacement occurred. The frictionless membrane applied under the length of sill plate had a minor effect at small displacements within the elastic range. For loads in the range of design values, which were well within the elastic range, there was little difference between the pseudo-cyclic, monotonic, and sequential phased displacement test results.

Second, the test showed that fastener fatigue was not a limit state influenced by any of the various loading protocols. This is an important observation since it limits the area of concern to the strength of wood and concrete elements tested.
Third, the class of anchorage tested was ductile, and concrete side-breakout was not detected until the resistance force was significantly beyond the elastic range, specifically not until the peak value was achieved. In addition to the observation of significant bolt bending, peak strengths from cyclic tests of the 1¾-inch edge distance case (e.g. 2x4 and 3x4 sill plate) ranged from 2.3 to 2.9 times the NDS calculated yield values for the wood sill plate connection, which indicates substantial loading beyond the yield limit state of the connection. The peak value was generally accompanied by a complete, but shallow concrete delamination. Use of the impact-echo measurements often signaled internal concrete delamination prior to any visual evidence, although no evidence of any sort was noted in the elastic range or below 6000 pounds in any test. After the initial shallow delamination occurred, the anchors were in tension, and a secondary peak was recorded—often with a higher ultimate value than the initial peak (see Figure 1). Significant ductile mechanisms were observed in the form of large deflections of the sill plate and bending of the anchor bolt. The failure mechanics of concrete predict that delaminations form initially from a series of micro-cracks. These internal micro-cracks propagate and interconnect along an eventual failure surface that corresponds roughly to the path of least energy. Although the study of fracture mechanics has not progressed to the point to have accurately predicted the information obtained from the SEAOC tests, it does predict significant energy can be absorbed by the concrete after the onset of inelastic behavior. The tests showed that in the post-elastic range, strength gain is slowed as micro-cracks grow to the point where the peak strength value occurs. The peak strength was noted to coincide with the point where initial delamination occurred.

Fourth, the ACI Appendix D concrete break-out strength taken from the estimated mean appears overly conservative for the 1¾-inch edge distance case (e.g. 2x4 and 3x4 wood sill plates). From cyclic test results, the tested peak strengths ranged from 1.7 to 2.2 times the ACI Appendix D calculated values adjusted to represent mean-based concrete break-out strengths. Taken on the whole (i.e. with and without the friction-reducing membrane) the 2x4 and 3x4 cyclic tests averaged 1.9 times the ACI concrete break-out calculated value adjusted for the mean strength. Similarly the 2x6 and 3x6 cyclic tests achieved 1.4 times the ACI equation. If the equations were to accurately reflect the test results, the comparison would be expected to be on the order of 1:1.

Since the ACI 318 Appendix D break-out equation approximates the 5 percent fractile strength, the SEAOC test report adjusts the Appendix D break-out strength to the mean-based estimate in order to provide an appropriate

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**Figure 1:** Typical inelastic behavior showing secondary peak.
comparison to the mean of the test data. There is some degree of assumption regarding the variance of the data, and
details are given for the specifics of each concrete test specimen in the SEAOC Anchor Bolt Test Report. However,
whatever adjustment is made, the aggregate of testing has shown the connection to exhibit good capacity and
ductility that was previously unaccounted for.

Finally, since the ultimate values corresponded to large drifts, the data reduction used in the test report was
conservatively modified from the ASTM E2126 standard. In particular, the first peak was used rather than the
ultimate load specified by the standard. This peak value was defined by the SEAOC Seismology Committee as the
highest load prior to any drop of 5 percent in capacity.

Assumptions Applicable to Anchor Bolt Design

Scope. As indicated above, ACI appendix D is utilized for cast-in L-bolts “provided they are within the scope of
Appendix D.” The ACI scoping provisions of D2.1 and D2.2 indicate the Appendix applies to “cast-in anchors” and
“connected structural elements” of which the subject anchor bolts are clearly included. However, the ACI-05
commentary states that the scope envisions anchorages where a single anchor failure could result in a loss of
stability of the structure. Generally speaking, sill plate anchorage is not a low redundancy application. There are
typically at least four connections present in the sill plate (two hold downs and two anchor bolts), there are often
other interior walls present, and there is also the likelihood of substantial friction at the sill plate connections. Thus
multiple load paths exist. Therefore, some engineers have suggested that the subject anchor bolts may not fall
within the scope of Appendix D based upon the commentary. While this point may have certain merits, the IBC
provides that if anchors are not to be regulated by Appendix D, another “approved method” is necessary. Such an
approved method should incorporate a similar level of sophistication as Appendix D. The IBC Table 1911.2 does
not incorporate the various failure mechanisms that are addressed by Appendix D.

Supplementary Reinforcement. ACI 318-05 section D.4.4 provides for the use a strength reduction factor of
Φ=0.75 (rather than Φ=0.70), if “reinforcement is proportioned to tie a potential concrete failure prism to the
structural member.” ACI 318-08 section D.4.4 and related commentary further clarifies that supplementary
reinforcement need only be present, and explicit design is not required in order to utilize the higher factor. Most
light-frame foundations have a continuous #4 or #5 reinforcement bar (or a post-tension tendon) near the top and
along the edge of the slab or curb, and it has been suggested that this bar may allow for an assumption of the higher
factor. The Committee cautions designers who may be tempted to categorize this bar as supplementary
reinforcement since in our experience the bar location is not sufficiently controlled in the field in a manner that
would allow for relatively shallow embedments.

Cracked Concrete Assumption. The first UBC code reference regarding cracked concrete appeared in 1997
UBC section 1923.2, which referred to anchorage embedment in “tension zones.” At the time, overhead anchorage
of structural members and equipment were a primary concern, and these regulations applied to anchorage occurring
below the neutral axis on bending members such as beams or elevated concrete decks. IBC has also incorporated a
cracked-concrete anchor reduction since the 2000 IBC [1319.5.2.7]. In the current code, ACI 318-05 section D6.2.7
stipulates “where analysis indicates cracking at service load levels,” Ψc,V shall be taken as 1.0 for anchors “with no
supplementary reinforcement or edge reinforcement smaller than a No. 4 bar.” (For testing, a crack width of up to
0.12 inches is produced.) Thus, in strength design, when the uncracked concrete is justified, cast-in anchors are
allowed a 40 percent capacity increase, since Ψc,V can be taken as 1.4.

The uncracked assumption is generally justified in light-frame construction as can be seen from the review of
original testing in cracks. A good review of available test information was recently published by Eligehausen,
Mallé, and Silva in the publication Anchorage in Concrete Construction (2006). In this publication the authors
explain that cracked concrete is a concern with anchors in tension since diminished values have been obtained with
testing and over time the fastening can loosen. However for shear loading they report that where “a shear load acts
perpendicular to the crack, then the load-displacement behavior does not differ significantly from the behavior in
non-cracked concrete. . . . [E]ven anchors that exhibit inferior performance when loaded in tension in cracks are
usually adequate to resist shear loads in cracked concrete” (p. 157). It should be expected that the subject sill plate
anchors will not be compromised by any significant degree, since it would require cracks intersecting the anchor and running parallel to the concrete edge, which are highly unlikely in typical light-frame applications. Any cracks occurring in the concrete substrate would be expected to be more or less perpendicular to the concrete edge and thus perpendicular to the applied load and not affecting groups of anchors.

The code requires the determination of cracked versus uncracked to be made at service level loads and that the crack reduction applies to a full-depth crack along the axis of the anchor. In the practical sense, it is possible that in combination with the effects of restraint, expansive soils, or frost heave, limited areas of a conventional foundation, deck, or post–tension slab-on-grade could experience curvatures in excess of the cracking modulus as redistribution occurs. However, given the inherent redundancy of anchors in light-frame construction coupled with the low probability of coincidence between qualifying cracks and typical anchor placement, it is not reasonable to assume a cracked substrate unless specific conditions clearly indicate otherwise.

**Conclusion and Recommendations**

Based upon the SEAOC Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances, the connection will yield at the wood sill plate prior to the formation of a concrete limit state when loaded parallel to a concrete edge. In other words, the concrete exceeds the strength of the wood. In the non-linear range of performance, an initial and secondary peak load was recorded that indicated the connection showed excellent ductility.

The test data and examination of assumptions detailed above indicate that it is rational to use the values obtained from ACI Appendix D assuming uncracked concrete and a ductile attachment. Also based upon the test results that indicate concrete will not govern for the anchorage of the subject 2x and 3x sill plates, it is conservative to use the NDS design values for bolts up to ¾ inch in diameter that meet the requirements shown at the beginning of this article. While ¾-inch diameter bolts were not specifically tested, they may be used with 6-inch nominal width sill plates due to increased cover. Additionally, the NDS predicts the same type Mode III, failure for the ¾-inch anchors. Table 1 shows representative anchor bolt shear values based upon the NDS-05.

### Table 1. Anchor Bolt Shear Values Based on the NDS 05 (C_D=1.6)

<table>
<thead>
<tr>
<th>Sill Plate</th>
<th>Bolt Diameter</th>
<th>1/2”</th>
<th>5/8”</th>
<th>3/4”</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x</td>
<td>1/2”</td>
<td>1040</td>
<td>1488</td>
<td>2032</td>
</tr>
<tr>
<td></td>
<td>5/8”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3/4”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3x</td>
<td>1/2”</td>
<td>1232</td>
<td>1888</td>
<td>2426</td>
</tr>
<tr>
<td></td>
<td>5/8”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3/4”</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 ¾" anchor bolt limited to 6-inch nominal width sill plates
2 Values are shown in lbs. (ASD basis)

Another benefit of the testing was isolating the effect of friction under the sill plate. The testing data indicates that a portion of the shear load can be transferred through friction between the bottom of the sill plate and the concrete. The amount of load that is transferred by friction is significant for monotonic testing and less so for cyclical testing. This supports the notion that friction is significantly increased due to bending of the anchors and the clamping action of the plate washers. In a wall assembly, the studs and boundary elements in compression may play a more significant role than previously assumed and present the opportunity for further study.

Finally, the reader is cautioned that any damage occurring to this connection may not be readily apparent. Therefore, post-event observers should review the photos contained in the test result and be aware that severe damage can be masked by the top of the sill plate.

Through much effort coordinated by the Light-Frame System’s subcommittee, new testing specifications and loading protocols were developed to ensure that the data would be properly generated and assessed. In addition to the efforts of the 2008-2009 SEAOC Seismology Committee, a number of firms donated time, materials and/or
effort, including Scientific Construction Laboratories, Inc., Structural Solutions, Inc., Certus Consulting, Inc., and VanDorpe Chou Associates, Inc. In addition, Phil Line of the American Forest & Paper Association provided valuable effort and input. The Committee was also very fortunate to be able to conduct the tests at the Tyrell Gilb Research Laboratory owned by Simpson Manufacturing Company, in Stockton, California. This facility is accredited to comply with ANS/ISO/IEC Standard 17025:2005.

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**Keywords**

light-frame  
wood  
anchor bolt  
edge distance

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In the writer’s reference list, the reference should be listed as:


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