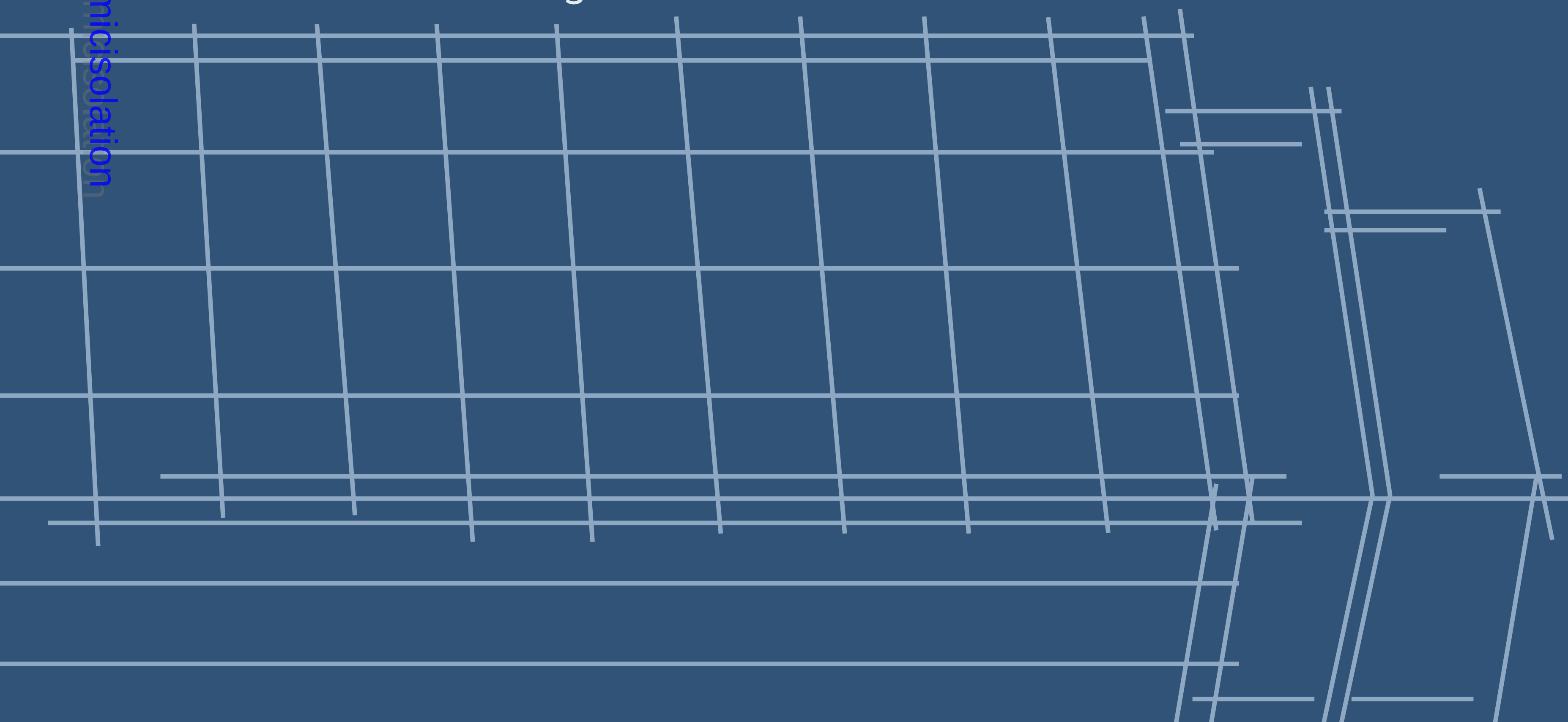
SEAOC

Blue Book

Seismic Design Recommendations 2019



SEAOC Blue Book

Seismic Design Recommendations 2019

Seismology Committee
Structural Engineers Association of California

Table of Contents

Concrete

Masonry

Wood and Light Frame

Base Isolation

About this Compilation

Welcome to the 2019 Edition of the *SEAOC Blue Book: Seismic Design Recommendations*.

This edition is SEAOC's ninth since 1959 – and a lot has changed in 60 years. In its first decades, this collection of interpretations and recommendations from SEAOC's Seismology Committee served as the basis for the seismic provisions of the Uniform Building Code (UBC) used in the Western US. Since the move in the 1990s to a national code-development process, the Blue Book's tenor has shifted from establishing the code to interpreting the code for practicing structural engineers and putting it context. Importantly, the 2019 edition captures how that context is evolving – from a historic focus on "life-safety" to the modern pursuits of performance-based design and resiliency.

So while the Blue Book tracks the history both of an important discipline and a proud professional organization, it is more than a story of the past. You will find the 2019 edition a useful reference for your current work, and rich in hints about the future.

This edition represents a multi-year volunteer effort by the SEAOC Seismology Committee. It builds on the work of generations of Seismology Committees before them. We thank all the authors, editors, reviewers, the SEAOC Foundation, and the donating individuals and firms who devoted hours and resources to this Blue Book.

Enjoy!

Ben Mohr, SE
Project Manager

Fred Turner, SE
Principal Technical Editor

Joyce Fuss, SE
SEAOC President, 2018-2019
on behalf of the SEAOC Board of Directors

Richard Dreyer, SE
SEAOC Foundation Chair
on behalf of the SEAOC Foundation Board of Directors

Don Schinske
Executive Director
SEAOC and SEAOC Foundation

Acknowledgments

The 2019 SEAOC Blue Book, *Seismic Design Recommendations*, reflects the work of the 2015 through 2019 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 builds 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, is provided below.

The technical editors for this edition of the Blue Book were Fred Turner and Benjamin Mohr. This edition was also reviewed by the SEAOC Structural Standards Committee, chaired by Kevin Moore. Don Schinske coordinated printing and publishing, and the SEAOC Foundation coordinated fundraising.

The Seismology Committee would like to thank all of the authors and reviewers who contributed to this edition of the Blue Book:

Gabriel Acero
Sam Adiputra
Federico Aldana
James Baty
Karen Beebe
Cairo Briceno
Katy Briggs
Jeremy Callister
Joseph Collins
Steve Earl
Ali Fattah
Victor Garcia-Delgado
Andrew Fennell
David Gonzalez
Tom Hale
Meaghan Halligan
Martin Hudson
Jesse Karns
Nelson Kawamura
Philip Kopf
James Lai
John Lawson
Jason Leung
Francis Lo
Roy Lobo
Gary Mochizuki
Jorge Meneses
Benjamin Mohr
Naeem Nemati-Raad
David Palmer
Maryann Phipps
Alan Robinson
Jacqueline Rock
Andrew Salber
Phil Samblanet
Arturo Schultz
Bill Seckler
Rahul Sharma
David Sommer
Ali Sumer
Fred Turner
Andreanna Tzortzis
Tom Van Dorpe

Past Seismology Committees

2018 – 2019 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Katy Briggs	Cairo Briceno	David Palmer (Chair)	
Benjamin Mohr	Jesse Karns (Past Chair)	Fred Turner	Victor Garcia
Meaghan Halligan			

2017 – 2018 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Katy Briggs	Josh Gebelein	David Palmer	
Benjamin Mohr	Cairo Briceno	Fred Turner	Victor Garcia
Meaghan Halligan	Jesse Karns (Chair)		Jeremy Callister (Past Chair)

2016 – 2017 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Katy Briggs	Josh Gebelein	Roy Lobo	
Benjamin Mohr (Past Chair)	Mehran Pourzanjani	Ali Sumer	Victor Garcia
Silvia Mazzoni	Jesse Karns		Jeremy Callister (Chair)
Chris Tung			

2015 – 2016 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Katy Briggs	Josh Gebelein	Tom Hale	
Benjamin Mohr (Chair)	Mehran Pourzanjani	Ali Sumer	Victor Garcia
Silvia Mazzoni	Jesse Karns	Roy Lobo (Past Chair)	Jeremy Callister (Vice Chair)
Chris Tung			

2014 – 2015 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Silvia Mazzoni	Bahram Zarin-afsar (Past Chair)	Roy Lobo (Chair)	Victor Garcia
Russell Berkowitz	Mechran Pourzanjani	Ali Sumer	Jeremy Callister
Chris Pettys	Gabriel Acero	Amir Gilani	

2013 – 2014 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Russell Berkowitz	Josh Gebelein	Roy Lobo (Vice-Chair)	
Silvia Mazzoni	Mehran Pourzanjani	Amir Gilani	Victor Garcia
Chris Petteys	Mikhail Gershfeld		Jeremy Callister
Geoff Bomba (Past Chair)	Bahram Zarin-afsar (Chair)		

2012 – 2013 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Bryan Cortnik	Matt Skokan	Hussasin Bhatia	Manny Sinha
Kevin Morton	Ricardo Hernandez	Ryan Kersting (Past Chair)	Jeremy Callister
Russell Berkowitz	Mehran Pourzanjani		
Chris Petteys	Bahram Zarin-afsar (Vice Chair)		
Geoff Bomba (Chair)			

2011 – 2012 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Geoff Bomba (Vice Chair)	Mehran Pourzanjani (Past Chair)	Ryan Kersting (Chair)	
Bryan Cortnik	Arash Altoontash	Hussain Bhatia	Amir Miraftab
Kevin Morton	Martin Hudson	Tom Hale	Manny Sinha
	Matt Skokan		

2010 – 2011 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Geoff Bomba (Vice Chair)	Mehran Pourzanjani (Past Chair)	Ryan Kersting (Chair)	
Bryan Cortnik	Arash Altoontash	Hussain Bhatia	Amir Miraftab
Kevin Morton	Martin Hudson	Tom Hale	Manny Sinha
	Matt Skokan		

2009 – 2010 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Kevin S. Moore (Past Chair)	Mehran Pourzanjani (Chair)	Ryan Kersting (Vice-Chair)	
W. Andrew Fennell	James S. Lai	Tom Hale	Doug Magee
Geoff Bomba	Tom Van Dorpe		Manny Sinha
Nicolas Rodrigues	Martin Hudson		

2008 – 2009 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Kevin S. Moore (Chair)	Mehran Pourzanjani (Vice-Chair)		
W. Andrew Fennell	James S. Lai	Ryan Kersting	Doug Magee
Geoff Bomba	Tom Van Dorpe	Tom Hale	Christopher Kamp
Nicolas Rodrigues			

2007 – 2008 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Kevin S. Moore (Chair)	Mehran Pourzanjan (Vice-Chair)		John Diebold (Past Chair)
Mark Moore	Doc Nghiem	Tom Hale	Doug Magee
W. Andrew Fennell	Tom Van Dorpe	Ryan Kersting	Christopher Kamp
Nicolas Rodrigues			Geoff Warcholik

2006 – 2007 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Rafael Sabelli (Past Chair)	Mehran Pourzanjani		John Diebold (Chair)
Mark Moore	Doc Nghiem	Tom Hale	Gary Lau
Kevin S. Moore	Tom Van Dorpe	Ryan Kersting	Christopher Kamp
W. Andrew Fennell			Doug Magee

2005 – 2006 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Rafael Sabelli (Past Chair)	Mehran Pourzanjani		John Diebold (Chair)
W. Andrew Fennell	Doc Nghiem	Tom Hale	Gary Lau
Kevin S. Moore	Tom Van Dorpe	Ryan Kersting	Christopher Kamp
Mark Moore			Doug Magee

2004 – 2005 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Rafael Sabelli (Chair)		Chris Tokas (Past Chair)	
Stacy Gaddini	Y. Henry Huang	Tom Hale	John Diebold
Gary Mochizuki	Martin Johnson	John Tehaney	Christopher Kamp
Kevin S. Moore	Bob Lyons		
Mark Moore			

2003 – 2004 SEAOC Seismology Committee

SEAONC	SEAOSC	SEAOCC	SEAOSD
Doug Hohbach (Past Chair)		Chris Tokas (Chair)	
Gary Mochizuki	Y. Henry Huang	John Tehaney	John Diebold
Kevin S. Moore	Martin Johnson	Tom Hale	Christopher Kamp
Rafael Sabelli (Vice Chair)	Bob Lyons		

1958 – 2003 SEAOC Seismology Committee

SEAONC

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
Merle A. Ewing 1957-1959
Nicholas Forell 1980-1982
Paul F. Fratessa 1981-1985
Sigmund A. Freeman 1975-1987
Ron Gallagher 1989,1991-1993
Mark Gilligan 1985-1987
Leslie W. Graham 1968
Phillip G. Griffin 1984-1986
Ronald Hamburger 1991-1993
Harold B. Hammill 1957-1959
Stephen K. Harris 1996-1999
Doug Hohbach 1998-1999,2001-2002*
William T. Holmes 1977-1979
Mark Jokerst 1993-1996
H.S. Kellam 1957-1959,1963-1970
Charles Kircher 1992-1994
Marvin A. Larson 1968,1969
RB (Fritz) Mathiesen 1979-1981
Joseph G. Matuting 1975,1976
Frank E. McClure 1957-1959
Andrew Merovich 1990-1992
David L. Messinger 1982-1984
Roumen Mladjov 1995,1997
Gary Mochizuki 2002-2005
Simin Naaseh 1997-2000
Joseph P. Nicoletti 1970-1973
Henry C. Powers 1957-1959,1961
Robert F. Preece 1960,1983-1985
John E. Rinne 1957-1967
Rafael Sabelli 2001-2004
Mark C. Saunders 1987-1989
G. A. Sedgewick 1957-1959
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
Frederick J. Willsea 1988-1990
Thomas D. Wosser 1971-1974
Loring A. Wylie 1975-1977
Edwin G. Zacher 1988,1990-1995
Theodore Zsutty 1978 -1981

SEAOSC

Robert Bachman 1989-1995,1994*
Stephenson B. Barnes 1957-1959
Reuben W. Binder 1957-1963
Gregg Brandow 1985-1987
Walter A. Brugger 1957-1959, 1963,1965
Robert C. Byrum 1968
Ted Christensen 1982-1984
Leroy L. Crandall 1957-1959
Schaefer J. Dixon 1975,1976
John J. Driskell 1967-1968
Murray Erick 1957,1958
Ullrich A. Foth 1975,1976,1987
William R. Gates 1979-1981
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
Martin W. Johnson 1997-1998,2002-2005
Roy G. Johnston 1957-1959,
1961,1963,1969-1971
Norman Jones 1967
Tom T. Kamei 1971-1973
John C. Kariotis 1974-1977,1979
James J. Kesler 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
Richard Phillips 1991-1993,1996-1997
Clarkson W. Pinkham 1962-1970*-1975
Phillips J. Richter 1980-1984,1987
John W. Robb 1974-1980
Thomas A. Sabol 1994-1996
Hassan Sassi 1998-1999
MJ Sidnner 1957-1959
John M. Seinbrugge 1957-1959
Hans G. Steinman 1971,1972
Donald R. Strand 1971-1975*
Edward J. Teal 1970-1982
James H. Thompson 1970
William L. Travis 1969,1972,1982,1983
Earl H. Vossenkemper 1969,1970
William T. Wheeler 1957-1960
Robert Whitelaw 1985-1987
Robert Wilder 1957-1959

SEAOCC

Jack S. Barrish 1957-1959,1974-
1976,1978*,1979
R.W. Bradley 1957-1959
Allen H. Brownfield 1957-1959
Walter D. Buehler 1968
David B. Campbell 1985
Robert Chittenden 1990-1995,1997-1999
Eugene E. Cole 1979-1984,1989,1990*
Joyce Copelan 1997-1998
David A. Crane 1969-1971,1977
Bruce Doig 1987,1988
Herman F. Finch 1957-1963
Charles Greenlaw 1985
Tom H. Hale 1994-2009
Gordon M. Hart 1957-1959
M.T. Knox 1957-1959
Kenneth Luttrell 1983-1987
John F. Meehan 1959,1969-1974
John T. Ragsdale 1977,1978
William D. Rumberger 1963-
1967,1975,1976,1980-1982
Arthur A. Sauer 1966
John Tehaney 2002-2003
Robert C. Thacker 1986-1990
Chris Tokas 1999-2002
Kenneth V. Venolia 1962-1967
Ajit S. Virdee 1972 -1976,1988,1996
Hoi W. Wong 1978-1979

SEAOSD

Raymond Flores 1981-1983
Edward A. Hanlon, Jr. 1988,1993
Robert Hendershot 1991,1992
Edwin H. Johnson 1972-1977,1983-1986
Christopher Kamp 2003-2008
John H. Khadivi 1999-2002
Jay Kinhal 1998-2002
Eric T. Lehmkuhl 1996-1999
James R. Libby 1978-1982
Kevin P. Lyons 1988
Doug Magee 1984-1986, 2005-2009
Melvin H. Mark 1970,1972-1978
Daniel L. McNaughton 1977-1981,1993
Vilas S. Mujumdar 1991-1993
Alfred L. Parme 1984
Ali M. Sadre 1994*-1998
Arthur K. Serata 1995,1997
Carl Schulze 1994-1998

*Chair

Blue Book Donors

The SEAOC and SEAOC Foundation boards of directors are proud to acknowledge the firms and individuals who have contributed to the production and printing of this publication. Their contributions ensure that all SEAOC members can own a complementary Blue Book – and that the long tradition of the Blue Book continues on. We are profoundly grateful.

Firms and Organizations

California Code Check
Coffman Engineers
Degenkolb Engineers
Raj & Helen Desai
Gilsanz Murray Steficek Engineers and Architects
Gouvis Engineering Consulting Group
International Code Council
JCE Structural Engineering Group, Inc.
James & Yim Fong Lai
Patel Burica & Associates, Inc.
SSG Structural Engineers

Individual Gold Sponsors

Kelly Cobeen
John Coil
Josh Gebelein
Darrel Harris
Tom Harris
Tom Harris, Sr.
Franklin Lew
Charles Mendenhall
Daniel McNaughton
Mike & Kelsey Parolini
Ali Sadre
Bryan Spain
James Tai

Individual Silver Sponsors

CK Allen, Jr.
Mehri Ansari
David Bonowitz
Victor Cuevas
James Day
Linda Force
Joyce Fuss
Thomas Heausler
Ephraim Hirsch
Jack Howard
Bo Jaquess
Pierson Jones
Morgan Prescott Jones
Christopher Kamp
Alan Kropp
Joseph Laquian
Terry Lundeen
Steven McCabe
Edward Mei
Richard Niewiarowski
Steve Pestell
Ryan Pintar
Steven Provenghi
Theodor Rosantsson
St. Onge & Associates
Guillermo Santana
Don Schinske
Simon Solorio
Chiawang Yeh
Zeltmacher Structural Design Engineering
Theodore Zsutty

Donation levels

Firms and Organizational Sponsors $1,000 • Individual Gold $500 • Individual Silver $100

@Seismicisolation

Preface

Since 1959, the Recommended Lateral Force Requirements "Blue Book" published by the SEAOC Seismology Committee has heavily influenced the seismic design of structures. The intent of the Blue Book up until the seventh edition (1999) was to act as a vehicle for writing recommended minimum code requirements and providing commentary regarding best practice. The Blue Book no longer contains building code recommendations, which now fall under the purview of the Building Seismic Safety Council's Provisions Update Committee that maintains FEMA's NEHRP Provisions (FEMA 2015) as well as committees that maintain national standards.

Recently, there has been momentum for building more earthquake-resilient structures, and growing public awareness regarding the expected performance of minimum code designed buildings and their potential economic impact on a community in a major earthquake. A significant barrier to implementing such change is that code provisions are characteristically minimum requirements, and affecting major improvements within the code amendment framework is difficult, and takes a significant length of time before adoption. There is clearly a need for the industry to have a reputable set of consensus guidelines the practicing engineer can use to meet the challenge of "better than code" seismic design for improved performance, which has historically been the role of the SEAOC Blue Book commentary.

This edition, launched in 2019, expands and updates the eighth edition, originally published in 2009. Similar to the eighth edition, it focuses 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 ninth overall, looks to extend that tradition.

While much is new, the Blue Book still offers background and commentary on the building code, and is intended to be a companion document useful to practicing structural engineers. The Blue Book is the primary 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 and consensus development.

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 are the unofficial resource documents for ASCE 7 (which itself has a 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.
- Engineering journals carry more papers about earthquake-resistant design than ever.
- National and regional research centers such as EERC and NCEER regularly produce reports, as do leading university labs.
- There are 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, supplement 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 now embraces a new role, concerned less with writing code provisions and more with improving actual engineering practice. 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 standards; the NEHRP Provisions and the national standards and commentaries do that. Rather, the Blue Book's job is to bridge between standards and practice. This emphasis will show itself in subtle ways. For one thing, much of the technical background can now be left to others. The Blue Book's role is no longer to justify provisions in standards but to assess their impacts on practice and to confirm whether they are supported by relevant analysis, research, 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—both historically and technically – that examines the standards in the context of performance, and explains their requirements in ways that foster good design.

Seismology Committee Positions

Despite its reduced role in the standards 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 Committee positions can influence standards change proposals, interpretations, recommended practices, accepted alternatives, or voting positions on proposals put forward by other organizations.

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 future standards. Even with the shift to national codes and standards, however, the Seismology Committee is still routinely asked to resolve questions of regulatory interpretation. Indeed, with the new national standards process (and the attendant national markets), Seismology Committee positions can provide significant insights for engineers and regulatory officials in California, the western United States and other regions of high or moderate seismicity.

A Seismology Committee position is:

- The consensus of a 12-person SEAOC committee, as opposed to the view of an individual engineer or researcher that might be found in a journal, handbook, seminar, or trade publication
- The recommendation of practitioners, as opposed to a proprietary opinion that 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 and standards, Degenkolb and others recognized the inevitable watering down of California practice for purposes of achieving national consensus (p. 147 ff.).

Today, the national standards resolve regional concerns by exempting low regions of seismicity from most detailing requirements. However, there is still room for the Blue Book to improve the practice for local conditions. California practice can still mean prudent measures beyond the letter of the code and standards, such as bracketed analysis, site-specific geotechnical studies, or tighter quality control. It can also mean attention to those principles that the code and standards still have difficulty quantifying, like regularity, reliability, capacity design and resilience.

The New Blue Book

The principal changes relative to the previous edition are:

- References to the latest base code and standards. This edition is based on the ASCE 7-16, the 2019 CBC, and the associated material standards.
- Significant revisions to most articles.
- New articles on Seismic Isolation, Performance-Based Seismic Engineering, and Seismic Design of Stairs.
- New medium. The previous version of the Blue Book was intended to be primarily an online resource with evolving content. This version returns to the concept of a single document.

Production

Work on the Blue Book's ninth edition began in 2015 with a review of existing articles and discussion of articles to be added. The Committee then selected individual experts to serve as lead writers and reviewers, as well as technical editors 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 requires a vote of the full Seismology Committee.

Approved for publication by the Seismology Committee, 2019

References

ATC (1978). *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC 3-06), Applied Technology Council, Redwood City, CA. (A second printing, in 1984, includes 1982 amendments.)

BSSC (2015). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 1—Provisions* (FEMA P-1050). Building Seismic Safety Council, Washington, D.C.

BSSC (2001b). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 2—Commentary* (FEMA 369). Building Seismic Safety Council, Washington, D.C., March.

EERI (1994). *Henry J. Degenkolb* (OHS-1), in *Connections: The EERI Oral History Series*, Earthquake Engineering Research Institute, Oakland.

Holmes, W. T. (1998). "The history of U.S. seismic code development," *Past, Present and Future Issues in Earthquake Engineering: The E.E.R.I. Golden Anniversary Volume, 1948-1998*, Earthquake Engineering Research Institute, p. 13-18.

Porush, A. R. and Zacher, E. G. (1987). "SEAOC, the Blue Book and seismic codes: past, present, and future," *Proceedings: 56th Annual Convention*, Structural Engineers Association of California.

Seismology (1999). *Recommended Lateral Force Requirements and Commentary, Seventh Edition*, Seismology Committee, Structural Engineers Association of California, September.

Strand, D. R. (1984). "Code development between 1927 and 1980," *Proceedings: 1984 Convention*, Structural Engineers Association of California.

Keywords

preface
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 2019)

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

SEAOC Seismology Committee (2019). "Preface" August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Glossary of Acronyms

ANSI American National Standards Institute.

ASCE American Society of Civil Engineers. Producer of the standard ASCE 7, *Minimum Design Loads for Buildings and Other Structures*.

ATC Applied Technology Council. Producer of ATC 3-06 (ATC, 1978), precursor of the NEHRP Provisions.

BOCA Building Officials and Code Administrators International, Inc. One of three statutory members of the ICC. Producer of the National Building Code (NBC).

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

CBC California Building Code. The 2001 CBC is based on the 1997 UBC model code.

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.

FEMA Federal Emergency Management Agency. Funding source for NEHRP Provisions and Commentary.

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).

NFPA National Fire Protection Association. Producer of NFPA 5000, *Building Construction and Safety Code*, published as a model code since 2002.

SBCCI Southern Building Code Congress International. One of three statutory members of the ICC. Producer of the Standard Building Code (SBC).

STC Seismic Task Committee. A technical task committee of the ASCE 7 standards committee and developer of earthquake design provisions for ASCE 7.

UBC 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.

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 2019)

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

SEAOC Seismology Committee (2019). "Glossary of Acronyms," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Codes and Standards Referenced by the 2018 International Building Code and the 2019 California Building Code

ACI (2014). ACI 318-14, *Building Code Requirements for Structural Concrete with Commentary*, 2014 edition. American Concrete Institute, Farmington Hills, MI.

AF & PA (2001). *National Design Specification for Wood Construction.* American Forest and Paper Association, Washington, DC.

AISC (2016). ANSI/AISC 360-16, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL.

AISC (2016). ANSI/AISC 341-16, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL.

ANSI/AWC (2017). NDS-2018, *National Design Specification for Wood Construction*, American Wood Council, Leesburg, VA.

ANSI/AWC (2015). SDPWS-2015, *Special Design Provisions for Wind and Seismic*, American Wood Council, Leesburg, VA.

ASCE (2017). ASCE/SEI 7-16, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.

TMS (2016). TMS 402, *Building Code Requirements and Specification for Masonry Structures.* Masonry Society, Longmont, CO.

Development of Earthquake Design Provisions for Building Codes

Summary

Earthquake design provisions have changed rapidly and substantially in the past century, and so has the process by which they are developed and codified. Up until 1998, earthquake-prone cities and states (mostly in the west coast) adopted the UBC, which had more stringent earthquake design requirements, while the rest of the country adopted the NBC or the SBC. With the new millennium, a big push was made to unify the different codes into the IBC as the primary nationally-recognized building code. The result is a building code that has become more stable over time and more uniform between jurisdictions, but also one that has less adaptability to local conditions and customs. The nationalization of standards also meant changes in the role and impact of its participants. In this new environment, national industry associations and academics play more significant roles than in the past, while the roles of code officials and statewide professional associations like SEAOC have been reduced. The development of earthquake design provisions, once the franchise of SEAOC and, in particular, the Seismology Committee, is now a national effort with more rigorous procedures for consensus building and balloting. While individual practicing structural engineers are still essential participants, the codes and standards development process has become more complex and bureaucratic. Design codes have also become more complex, extensive, and difficult to apply.

This article describes the historical earthquake design codes and standards development process (prior to the nationalization of codes in 2000) and the current process, as well as the organizations and publications that motivated the transition. An evaluation of how the current nationalized code has been accepted, and the challenges ahead, will also be mentioned.

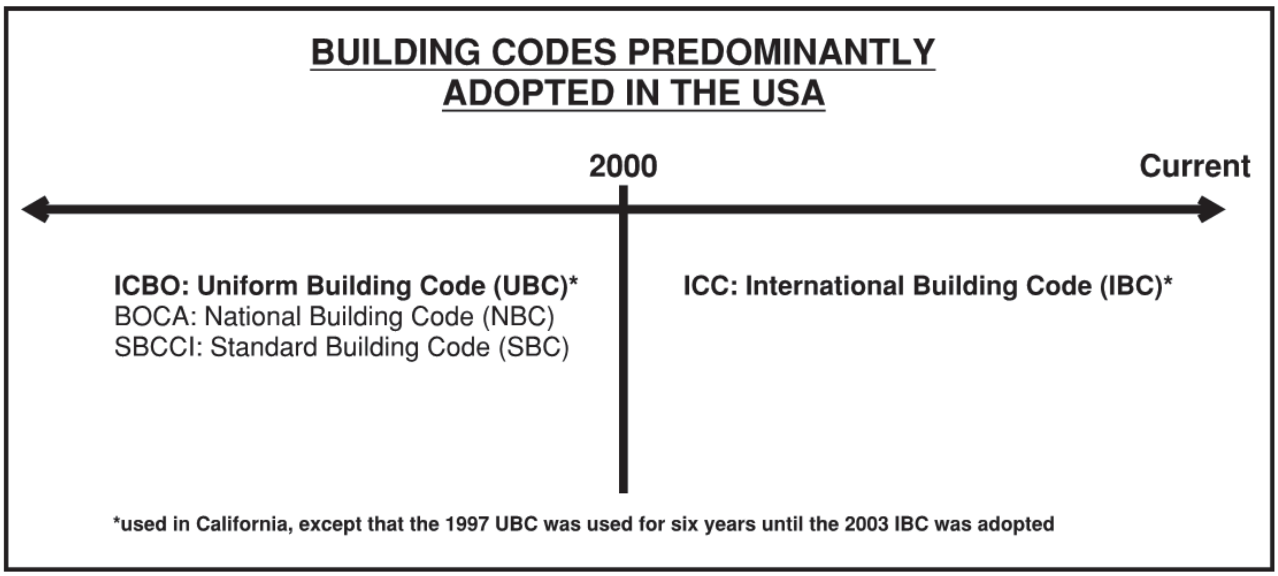


Figure 1. Evolution of Building Codes in the US

Pre-1959: The UBC and early earthquake design provisions

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 were already in use in many countries by the early 1920s. These are examples of modern design provisions, which are 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 (a forerunner of the International Conference of Building Officials and, more recently, the International Code Council) 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 Appendix recommended that structures designed for seismic resistance should have the strength to resist a total lateral force proportional to the building weight (3% of W). 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 ensure that it acted 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).

1959 - 2000: The UBC and the role of SEAOC Seismology Committee

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 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 SEAOC's Blue Book recommendations.

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. 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.

Table 1. Recent earthquakes and building codes and standards provisions they motivated

Earthquake	UBC/IBC Edition	Enhancement
1971 San Fernando	1973	Direct positive anchorage of masonry and concrete walls to diaphragms
	1976	Seismic Zone 4, with increased base shear requirements
		Base shear dependence on site conditions through coefficient *S*
		Occupancy Importance Factor *I* for certain buildings
		Interconnection of individual column foundations
		Special Inspection requirements
1979 Imperial Valley	1985	Diaphragm continuity ties
1985 Mexico City	1988	Requirements for columns supporting discontinuous walls
		Separation of buildings to avoid pounding
		Design of steel columns for maximum axial forces
		Restrictions for irregular structures
		Ductile detailing of perimeter frames
1987 Whittier Narrows	1991	Revisions to site coefficients
		Revision to spectral shape
		Increased wall anchorage forces for flexible diaphragm buildings
1989 Loma Prieta	1991	Increased restrictions on chevron-braced frames
		Limitations on *b/t* ratios for braced frames
	1994	Ductile detailing of piles
1994 Northridge	1997	Restrictions on use of battered piles
		Requirements to consider liquefaction
		Near-fault zones and corresponding base shear requirements
		Revised base shear equations using 1/*T* spectral shape
		Redundancy requirements
		Design of collectors for overstrength
		Increase in wall anchorage requirements
		More realistic evaluation of design drift
		Steel moment connection verification by test
2010 Chile	2018	Concrete Shear Walls
2011 Christchurch	2018	Stairs, ramps and their connections to Seismic Force Resisting Systems

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 had a dominant influence. 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 Seismology 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 seismic portions of the UBC.

Despite their wide acceptance, the Blue Book's recommended provisions were developed in a relatively closed process, with relatively few participants. 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 publicly accessible reviews, comments, and responses with respect to Seismology Committee positions.

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.

ATC 3-06 and NEHRP Recommended Provisions

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 and development 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 in 1979 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 and standards nationwide. The modified document became the first National Earthquake Hazards Reduction Program (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 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).

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 research, 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).

2000 – Current: The International Building Code

In 1995, BOCA, ICBO, and SBCCI agreed to coalesce into a single model building code development organization (the International Code Council), with the intent of publishing a single nationally applicable building code. 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. BSSC introduced into the 1997 NEHRP provisions that West Coast engineers felt were imperative in the seismically active Western US, as well as lessons from the 1994 Northridge and 1995 Kobe earthquakes. At the same time, SEAOC's Seismology and Code Committees revised the Blue Book and proposed 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.)

The IBC Model Code

Most U.S. states and the jurisdictions therein enact regulations by adopting (and sometimes modifying) the International Building Code (IBC). Large cities, including Los Angeles and San Francisco, are historical exceptions that maintain and update numerous local amendments, but even they today rely on the IBC for their basic provisions.

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, NCSEA, 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 IBC code change proposals primarily with or through NCSEA in order to gain wider support.

Standards

Model building codes no longer print most of their own technical provisions; instead, they adopt their provisions by reference to approved consensus standards developed and maintained by other organizations. Standards are documents produced in accordance with American National Standards Institute (ANSI) or American Society for Testing Materials (ASTM) rules for committee representation, balloting, consensus, etc. (Past model codes and guidelines, including the SEAOC Blue Book, the UBC, the NEHRP Provisions, and the IBC, did not meet these criteria.) Model codes adopt standards developed by other organizations. For example, the 2018 IBC adopts structural loading criteria (including earthquake design provisions) by reference to ASCE 7-10. Similarly, material design provisions are adopted by reference to standards produced by ACI and AISC.

This contrasts with past model codes that were self-contained documents. Even when past model codes would adopt provisions authored by others (such as those of ACI or 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.

There is no law that requires the adoption 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 organizations.

Today's Codes and Standards Development Process

Development of earthquake design provisions for building codes and standards has evolved substantially since the first UBC and the IBC. Nationalization and standardization between 2000 and 2010 intended to minimize differences between model codes and standards across jurisdictions, have led to a complex process of linked and sometimes conflicting provisions developed `through near-constant and sometimes overlapping revision cycles.

Figure 2 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.) and testing.

Figure 2 also shows how the Blue Book is no longer the primary 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, and the standard-writing organizations.

Figure 2 represents in a rough way the development of the 2015 edition of the IBC. This process is roughly 15 years old, and a few observations can be made.

- Competition between model codes has become much less of an issue due to the predominance of the IBC as the model building code. Adoption of national standards by reference has simplified practice for engineers working in multiple jurisdictions, reducing the likelihood of oversight or omission due to unfamiliar local codes. When a state adopts the IBC, it is also expected to make modifications to reflect established practices (especially for special occupancies). Local jurisdictions at times make further modifications to the state's building code, but these are typically few in number.

- In the past, the new code development process had 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. However, the development of FEMA P695 paved the way for quantifying performance and acceptance criteria for new seismic systems. The Seismology Committee recommends the use of FEMA P695 to quantify performance and acceptance criteria for new and undefined seismic force resisting system.

- Nationalization of the code has generally reduced the number of large changes from one edition to the next. This is primarily a function of the small window for substantive standards modification work within the three-year cycles of the IBC (and, therefore, the CBC).

- Emergency revisions and amendments may still be promulgated by local jurisdictions.

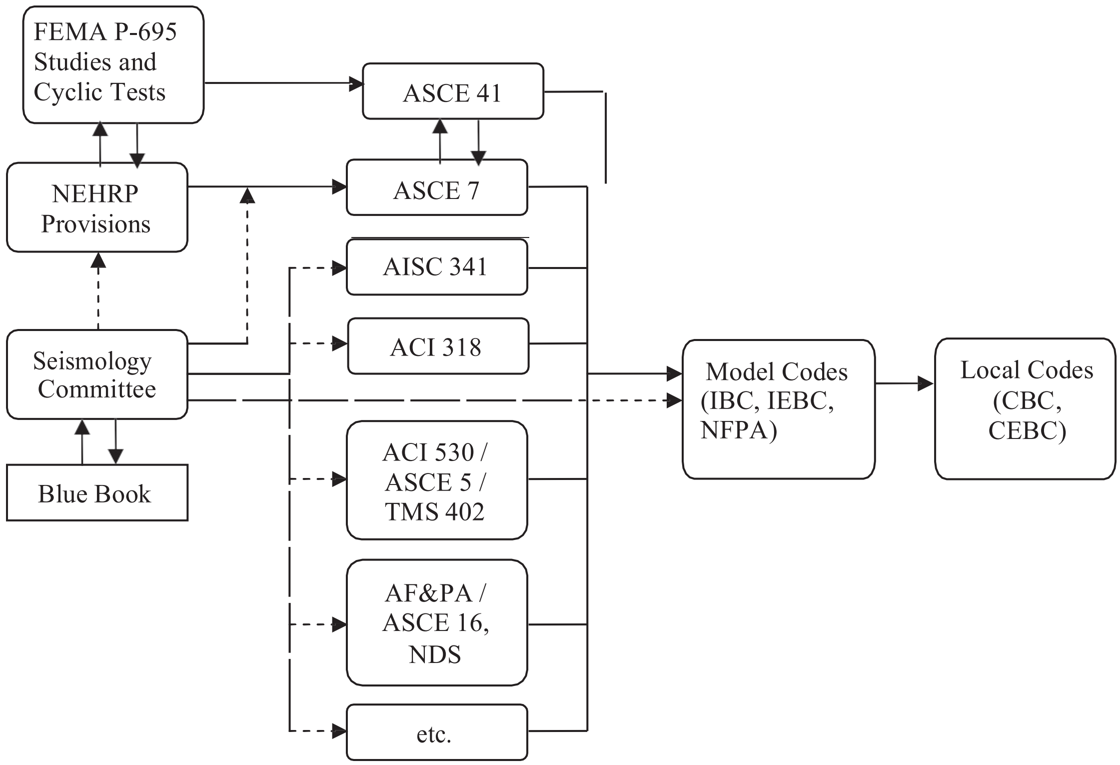


Figure 2. The Blue Book, the Seismology Committee, and the new code development process

- There exists a long lag between a change proposal and an eventual new code or standards provision. The implications of such a lag on design quality are difficult to quantify. 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 new idea and its enforcement can be six or even eight years. For example, ASCE 7-16 was based on the 2015 NEHRP Provisions. New proposals for technically significant modifications were due in 2013. Changes balloted in 2011-2013 will come into effect on January 1, 2020 (via the 2018 IBC and 2019 CBC). The lag is complicated further by the long balloting schedule required for consensus standards. Oftentimes, voting parties (such as Seismology) find themselves voting on standards before referenced or source documents are completed.

- The lag between new thinking and an enforceable code also has potential implications for engineer liability. It is often 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 soon-to-be-replaced 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 codes and standards development process.

- ANSI rules for developing consensus 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 specific materials (concrete, masonry, steel, wood, dampers and isolators, etc.) tend to support criteria that would promote the use of the materials or technology. 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. Indeed, it is simply human nature that would compel these individuals to act in their own self-interest, even unconsciously. With less influence on standard-writing committees, it has become more difficult for independent parties, including the SEAOC Seismology Committee and NCSEA, to effectively serve their watchdog roles. With separate standards for each structural material, it is also more difficult for the Seismology Committee and others to ensure consistency of design philosophy among different types of construction.

Adoption of Model Codes in California

The California Building Code sets minimum requirements for all California jurisdictions. Prior to 2003, the California Building Code used the UBC as its model code. In 2003, California announced its intent to adopt NFPA 5000 as the baseline for the future California Building Code. However, upon election of new government officials, California rescinded the directive to use NFPA 5000, and adopted the IBC. The 2016 edition, which is in use as of the writing of this document in mid-2017, uses the 2015 International Building Code (IBC) as its model code. The IBC is updated every three years, and the CBC adopts the most recent version of the IBC when it is published with state amendments the following year. Local jurisdictions typically adopt the CBC without local amendments. However, larger jurisdictions such as San Francisco and Los Angeles and surrounding jurisdictions routinely modify the CBC with local amendments based on recommendations from regional ICC Chapters.

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.

ASCE American Society of Civil Engineers. Producer of the standard ASCE 7, *Minimum Design Loads for Buildings and Other Structures*.

ATC Applied Technology Council. Producer of ATC 3-06 (ATC, 1978), precursor of the NEHRP Provisions.

BOCA Building Officials and Code Administrators International, Inc. One of three statutory members of the ICC. Producer of the National Building Code (NBC).

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

CBC California Building Code. The 2001 CBC is based on the 1997 UBC model code.

CRSC Code Resource Support Committee.

FEMA Federal Emergency Management Agency. Funding source for NEHRP Provisions and Commentary.

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).

NFPA National Fire Protection Association. Producer of NFPA 5000, *Building Construction and Safety Code*, published as a model code since 2003.

SBCCI Southern Building Code Congress International. One of three statutory members of the ICC. Producer of the Standard Building Code (SBC).

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.

References

ATC (1978). *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC 3-06), Applied Technology Council, Redwood City, CA. (A second printing, in 1984, includes 1982 amendments.)

Bachman, R. E. (1995). "Briefing on the 1995 Blue Book, approved seismic code provisions for the 1996 UBC Supplement and proposed 1997 UBC seismic provisions for strength design," *Proceedings: 64th Annual Convention*, Structural Engineers Association of California.

Blume, J.A., Newmark, N. M., and Corning, L. H. (1961). *Design of Multi-story Concrete Buildings for Earthquake Motions*, Portland Cement Association.

BSSC (2001a). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 1—Provisions* (FEMA 368). Building Seismic Safety Council, Washington, D.C., March.

BSSC (2001b). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 2—Commentary* (FEMA 369). Building Seismic Safety Council, Washington, D.C., March.

Bush, G. H. W. (1990). *Executive Order No. 12699: Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction*, available from the Library of Congress, Washington, D.C.

CBSC (2005). "Building Standards Commission March 16, 2005 Meeting." (See http://www.bsc.ca.gov/.)

Cobeen, K. (1995). "Nationalizing the design and materials chapters of the 1997 UBC: SEAOC Code Committee proposal for the Uniform Building Code," *Proceedings: 64th Annual Convention*, Structural Engineers Association of California.

Degenkolb, H. J. (1986). "Code provisions resulting from earthquake observations," in *Learning from Earthquakes*, Structural Engineers Association of Northern California, San Francisco, March.

EERI (1994). *Henry J. Degenkolb* (OHS-1), in *Connections: The EERI Oral History Series*, Earthquake Engineering Research Institute, Oakland.

Geschwind, C-H. (2001). *California Earthquakes: Science, Risk, & the Politics of Hazard Mitigation*, Johns Hopkins University Press.

Ghosh, S. K. and Chittenden, R. (2001). *2000 IBC Handbook: Structural Provisions*, International Conference of Building Officials, Whittier, CA.

Hamburger, R. O. and Kircher, C. A. (2000). "Preface: Recent developments in seismic design methodologies and codes," *Earthquake Spectra*, v. 16 n. 1, February.

Henry, J. R. and Johnson, M. A. (2003). "Keeping up with the changes: Referenced standards and the 2003 IBC," *Structural Engineer*, June.

Holmes, W. T. (1998). "The history of U.S. seismic code development," *Past, Present and Future Issues in Earthquake Engineering: The E.E.R.I. Golden Anniversary Volume, 1948 -1998,* Earthquake Engineering Research Institute, p. 13-18.

Mahoney, M. and Sheckler, T. (2001). "The world of building codes, standards and provisions: A view according to FEMA," *SEAOC 2001 70th Annual Convention Proceedings*, Structural Engineers Association of California

Melvyn Green & Associates (1995). *Comparison of the Seismic Provisions of Model Building Codes and Standards to the 1991 NEHRP Recommended Provisions* (NIST GCR 95-674), National Institute of Standards and Technology, Gaithersburg, MD, May.

National Bureau of Standards (1982). *Minimum Design Loads for Buildings* (ANSI A58.1), Washington, D.C.

Olson, R. A. (2003). "Legislative politics and seismic safety: California's early years and the 'Field Act,' 1925-1933," *Earthquake Spectra*, v.19 n.1, February.

OSHPD (2003). *Evaluation of the 2003 Model Codes: Final Report*, Office of Statewide Health Planning and Development, July 29. Available at www.oshpd.state.ca.us/fdd/About_Us/Organization/Regulations/EVALfinal-ALL.pdf.

PCBO (1928). *Uniform Building Code*, Whittier, CA.

Porush, A. R. and Zacher, E. G. (1987). "SEAOC, the Blue Book and seismic codes: past, present, and future," *Proceedings: 56th Annual Convention*, Structural Engineers Association of California.

Seismology (1959). *Recommended Lateral Force Requirements*, Seismology Committee, Structural Engineers Association of California, July

Seismology (1960). *Recommended Lateral Force Requirements and Commentary*, Seismology Committee, Structural Engineers Association of California.

Strand, D. R. (1984). "Code development between 1927 and 1980," *Proceedings: 1984 Convention*, Structural Engineers Association of California.

Tobriner, S. (1984). "The history of building codes to the 1920s," *Proceedings: 1984 Convention*, Structural Engineers Association of California.

Keywords

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

How to Cite this Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Development of Earthquake Design Provisions for Building Codes," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
11.1.1 C11 C11.1.1		

Introduction

For many years, code provisions for earthquake design of new structures have acknowledged three fundamental assumptions:

1. Nonlinear, inelastic behavior is expected, but prescriptive analysis procedures use linear, elastic models with response modification factors to account for building ductility.
2. Cyclic loading is expected, but linear elastic models do not capture this. This is indirectly addressed by material-specific detailing requirements based on the level of ductility assigned to the structure.
3. Real deformations will exceed those predicted by linear analysis with reduced seismic forces. The expected displacement response may be approximated as a multiple of a linear analysis (linear deformation multiplied by Cd/I_E).

Taken together, these simplifying assumptions have made it possible to regulate structural response in terms suitable for adoption in building codes. When shortcomings in these assumptions have been discovered, additional code provisions have been developed, or existing provisions have been updated. However, the underlying philosophy of designing for inelastic behavior using elastic analysis has not changed since it was first introduced into the building code in the 1960s.

Recent earthquakes in countries with building codes similar to the United States indicate that linear analysis, when combined with ductile detailing, generally meets the intent of the building code with regard to life safety and collapse prevention. However, such earthquakes have also shown that buildings designed to satisfy code-minimum requirements generally do not result in resilient buildings that can be reoccupied after a major earthquake. While this statement is well understood by most structural engineers, it is not widely understood by the general public.

In this article, code-based design refers to prescriptive design based on an elastic analysis of equivalent static lateral forces (ELF), or forces associated with an elastic modal response spectrum (MRS) analysis.

History of Building Code Design Methods

These assumptions –allowing designs based on linear, elastic analysis with reduced forces – came to characterize code-based earthquake design in the second half of the twentieth century. They remain the basis of most earthquake design in 2017, though they are largely the consequence of traditional engineering practice that was developed many years ago. These assumptions were until recently a technical necessity, and are still a practical alternative, as the basis for a rational approach to earthquake design that balances safety with economy.

As modern building codes developed, linear elastic design for 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 interpret as a consequence 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 this "ductility" was achieved by requiring the seismic lateral systems of these types of buildings to include moment resisting space frames. *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 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 standards. By comparison with today's standards, the design and detailing of many of these structures was very rudimentary.

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 most 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 7-16 still does not require nonlinear analysis for any structure type. Buildings that are designed outside the prescriptive limits of the building code, such as those that follow the Tall Building Initiative (PEER 2010 and LATBSDC 2014), require nonlinear analysis, however. The NEHRP Seismic Design Technical Brief "Nonlinear Structural Analysis for Seismic Design" (Deierlein et al 2010) gives a good background for practicing engineers on how to implement nonlinear analysis techniques.

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 response to code-level ground motions. 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. In theory, building codes could have kept structures 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.

The 1974 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. Code-based design (i.e., the elastic procedures allowed in building codes) retained the previous framework: reduced design forces, presumed significant inelastic behavior, and inelastic response via appropriate detailing. 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, following the lead of ATC 3-06 (ATC 1978), values of the new design parameters were selected to assure that customary design forces would not dramatically 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 as many code changes still are.

Factors other than Analysis that affects Seismic Performance

Structural analysis is only one aspect of ensuring that buildings will perform adequately during earthquakes. Detailing, proportioning to avoid irregularities and other less-predictable sequences of actions, thorough in-office quality assurance, regulatory plan reviews, inspections, special inspection and testing, and construction observation by design professionals are also key components for earthquake-resistant design. A 2016 study by Reinoso et al showed that many of a random sample of 150 buildings in Mexico City failed to meet the city's standards and some did not even have enough documentation to conduct a full assessment. "Building inspections have essentially been outsourced to a network of private engineers who are hired and paid for by developers, creating conflicts of interest that can undermine even the best standards" (Ahmed, Azam). For this reason, one news article quoted "luck" as the reason many buildings did not collapse in the 2017 Mexico City earthquake.

Two case studies will be used to illustrate the importance of some of the key factors other than analysis in the design and construction of earthquake resistant buildings. The first is of the seismic performance of the Hotel Grand Chancellor in Christchurch New Zealand. This building experienced the September 4, 2010 earthquake, the Boxing Day aftershock and the February 22, 2011 earthquake. After the February 22, 2011 earthquake, a wall failed on the ground floor of the hotel, causing a near-collapse and making the building unsafe to occupy. One possible reason for this is that this building was subjected to earthquake forces that were higher than design loads. However, the Canterbury Earthquakes Royal Commission found that this wall would still have failed even if it was subjected to its design loads because two major errors were made in its design. The first is that the axial load was severely underestimated, so the wall was inadequately reinforced and detailed. The report mentions that the building code did not properly require confinement of reinforcement in shear walls with high axial loads. This showed that while accurate analysis is critical, potentially catastrophic collapses can also occur without proper detailing such as confinement of reinforcement in concrete shear walls. Secondly, the building was highly irregular and thus, simplified analysis methods, such as the modal response spectrum method, were unable to accurately capture the forces on the seismic force resisting elements. There is a recommendation in the report by the commission that more attention should be placed on understanding and detailing highly irregular structures, and that the current code does not do enough. In addition, some stairs collapsed in the upper section of the hotel parking garage. This left several people stranded without a way to exit the building. These people had to make their way to the roof and be evacuated by crane. Even if the seismic system had performed satisfactorily, this building should not be considered to have performed satisfactorily because of this stair failure.

The Canterbury Royal Commission published recommendations following its evaluation of building performance in the Christchurch Central Business District during the 2010-2011 Christchurch Earthquakes. Complementary to the recommendations addressing analysis in the building code, the commission also made recommendations such as "Designers should define load paths to ensure that the details have sufficient strength and ductility to enable them to perform as required" (Canterbury Earthquakes Royal Commission). The commission hoped that these recommendations would be incorporated in continuing education programs so that practicing engineers would become aware of them.

The second case study is described in the verdict of a coroner's jury investigating the deaths of two individuals due to falling material from buildings during the 1933 Long Beach Earthquake. The jury concluded that these deaths were accidental and did not hold any person criminally accountable. This begged the question, "If nothing was done wrong, how did something like this happen and how can the building code change to prevent something like this from happening in the future?" (County of Los Angeles). While this question was not answered directly, the jury recognized that the responsibilities delegated to them were more than just determining the cause of death, so they made recommendations to improve building performance in future earthquakes. While most of the buildings within the vicinity of the earthquake survived without any serious damage, the damage that occurred resulted mainly from faulty construction. Shoddy workmanship, poor quality of materials, lack of structural inspection and inadequate building ordinances regulating design and construction were all contributing factors to the serious damage. Later in 1933, California passed the Riley Act in response to the Long Beach Earthquake. It requires all local governments to review plans, issue building permits, and conduct inspections to ensure earthquake-resistant construction statewide.

Parapet walls and architectural ornaments presented very serious falling risks that caused the deaths in the Long Beach Earthquake. "It was an outstanding fact that the greatest hazard from falling material was at the entrances of the buildings and along the sidewalks; and since it is an almost irresistible impulse with many people to run out of a building during an earthquake, it is a fact that the places where people were most likely to be or to go during the excitement were the most dangerous" (County of Los Angeles). This showed that public's responses during earthquakes are also important factors in ensuring earthquake safety. Since then, the nation's "Drop, Cover, and Hold On" advisories have become an annual exercise to, encourage safer human behavior during earthquakes.

These two case studies highlight different aspects of earthquake engineering that are important to a satisfactory performance of buildings or other structures in a seismic event. In the first case study, poor detailing, inability to address the structure's irregularities and inadequate design of non-structural components lead to a near collapse condition for the Hotel Grand Chancellor. In the second case study, a jury investigating the 1933 Long Beach Earthquake attributed shoddy construction to the cause of most of the damage in this earthquake.

Is Code-Based Earthquake Design Sufficient?

The real test of modern design assumptions is whether they lead to buildings that perform satisfactorily. The ASCE 7 Commentary (section 5.2) states: "This approach has substantial historical precedent. In past earthquakes, structures with appropriately ductile, regular, and continuous systems that were designed using reduced design forces have performed acceptably" (ASCE). 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 ASCE 7 Commentary's assessment relies, in part, on the theory of ductility reduction 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 definitively prove the adequacy of newer designs. Several structure types once considered code-compliant are now seen as deficient in some respects. 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 can have a limited shelf life.

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 configurations, 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 sometimes 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."

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). As discussed before, past earthquakes have uncovered deficiencies in code compliant buildings that have caused revisions in codes and standards. The great majority of buildings that collapse or suffer disproportionate damage lack proper, continuous load paths, poor quality in construction, or other fatal flaws.

Designing for Inelasticity

Because of the codes' 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- 16 section 11.1.1 (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 when load combinations that do not contain earthquake effects indicate larger demands than combinations including earthquake effects." 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 and displacements 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 *un*reduced earthquake effects. Other special conditions for which inelastic response should be avoided are now addressed by the Ω_0 or overstrength factor. In addition to limiting the inelastic response in certain elements, there is a limit on the inelastic drift (ASCE 7: Sec 12.12.1 and Table 12.12-1) as well as special detailing requirements for elements that are subject to significant local ductility demands (ASCE 7: Sec 12.2.5.1 to 12.2.5.8 and the materials standards with CBC amendments).

The actual amount of inelasticity presumed by code provisions is only roughly estimated by parameters such as R, C_d and Ω_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, 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, more familiar systems might warrant new testing. On the other hand, the current building code and its adopted standards have introduced many new seismic force resisting systems that can be designed using linear elastic methods.

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 design earthquake ground motions. To make up for this, ASCE 7 amplifies the calculated deflection by the factor C_d. The corresponding factor in the 1997 UBC was 0.7R. The amplified deflections are then compared to specified drift limits. Still, actual deformations cannot always be reliably predicted, as noted in the ASCE 7 Commentary: "Research over the past 30 years has illustrated that inelastic displacements may be significantly greater than Δ_E for many structures and less than Δ_E for others."

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 parameters for engineering decisions. 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 deformations 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.

Overview of ASCE 7 and FEMA P695

Per ASCE 7, "for Risk Category I and Risk Category II structures… acceptable Life Safety risk is defined by an 'absolute' collapse probability of 1% in 50 years and a 'conditional' probability of 10% given MCE_R ground motions." Based on this criteria, the US Geological Survey developed probabilistic MCE_R ground motions that are incoporated into the building code. Structures designed and detailed according to the ASCE 7 standard should satisfy the conditional probability of collapse of 10% in 50 years. This is ensured by significant research done for the development of FEMA P695, *Quantification of Building Seismic Performance Factors*. Similarly, structures are designed for a condtional collapse goal of 5% at the MCE_R for Risk Category III and 2.5% for Risk Category IV. This is achieved by using an importance factor of 1.25 for Risk Category III and 1.5 for Risk Category IV, as opposed to 1.0 for Risk Categories I or II. The importance factor is used in the seismic response coefficient, and increasing the importance factor has the effect of increasing the design base shear.

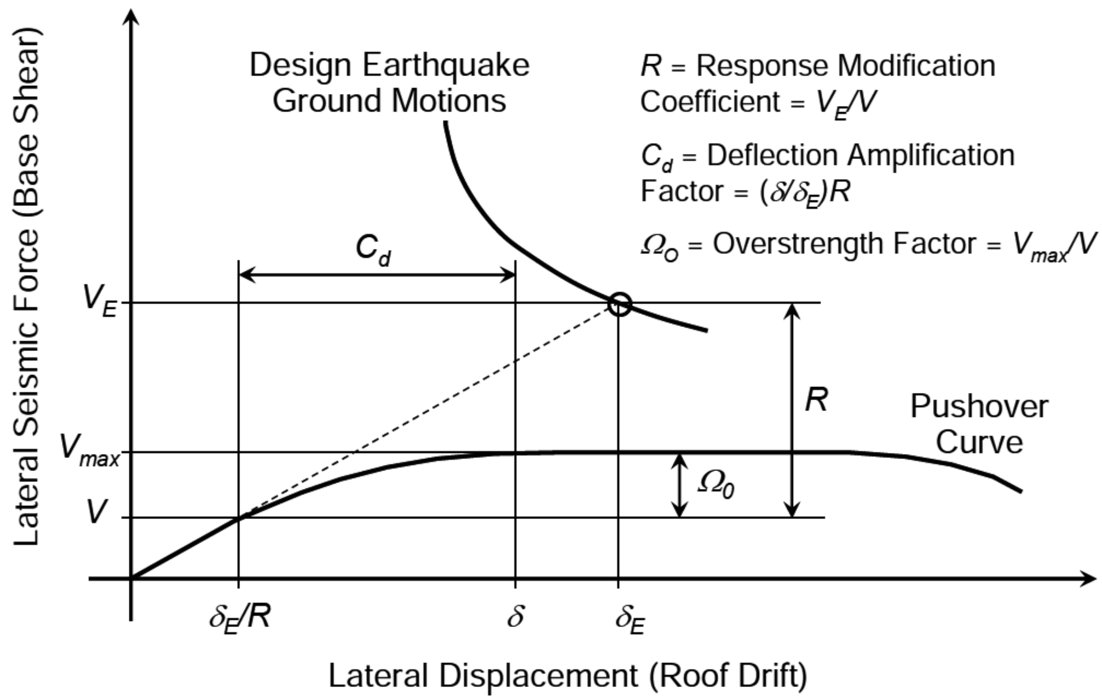


Figure 1: Idealized Pushover Curve showing Design Basis Earthquake Demands

Per ASCE 7, structures are designed for earthquakes considering overall system performance, instead of each member's individual performance. A system's seismic performance can be quantified through three parameters: the response modification factor (R), the overstrength factor (Ω_o) and the deflection amplification factor (C_d). Initially, these factor were determined based on the system's or similar system's performance in past earthquakes. Since many of the systems in the building code have not yet been tested by design earthquake ground motions, many of these values are primarily based on engineering judgment. In 2009, FEMA P695 was developed to link non-linear collapse simulations with these performance parameters. These parameters can be explained through an idealized pushover curve shown in Figure 1. If the structure were to remain completely elastic during the design earthquake ground motion, its base shear would be equivalent to its elastic base shear, V_E. The purpose of R is to reduce V_E to the structure's design base shear, V, which occurs at the structure's first significant yield. Ω_o is the ratio between the systems overstrength capacity to its first significant yield. The intent of Ω_o is to protect elements that do not have ductile behavior after yielding. C_d is a factor used to estimate the inelastic deformations of a structure from its elastic deformations. Typically C_d is less than R. For structures with high damping. C_d is significantly less than R.

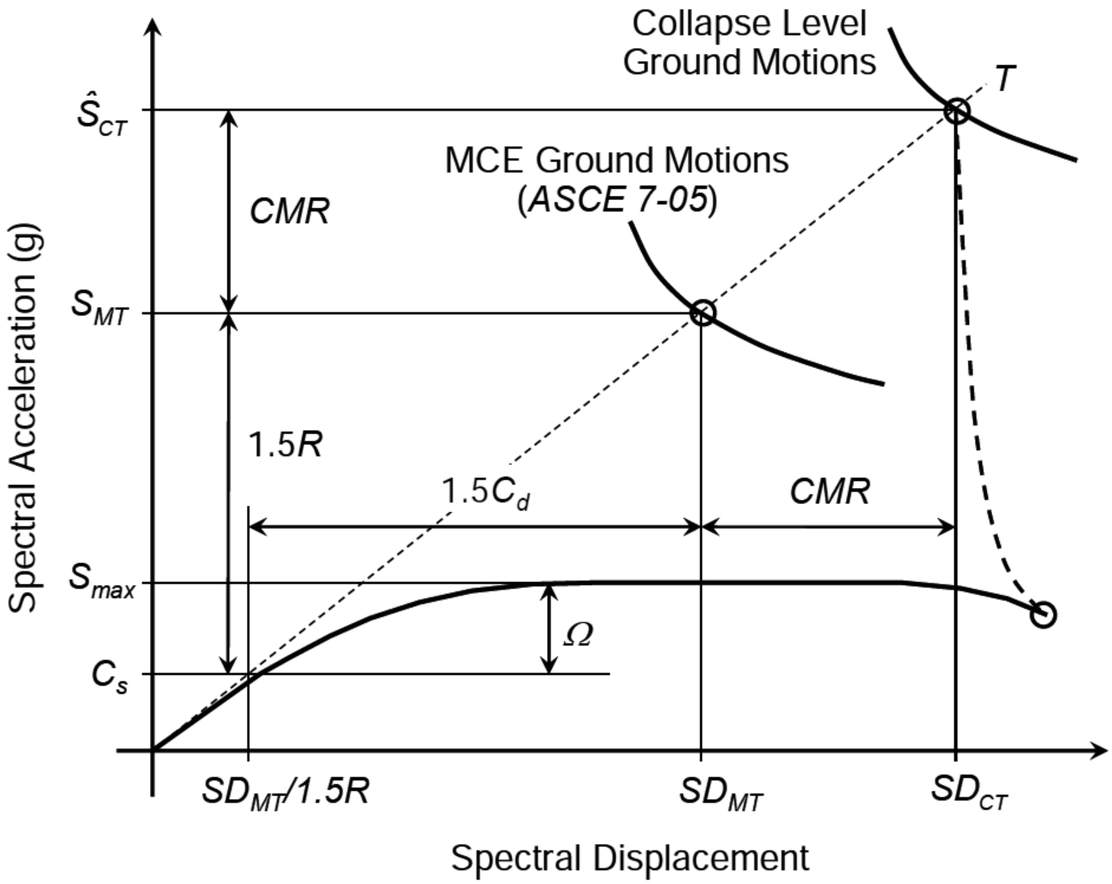


Figure 2: Idealized Pushover Curve showing Maximum Considered Earthquake Demands

R-factors were originally introduced to the building codes and standards through the ATC-3-06 report *Tentative Provisions for the Development of Seimic Regulations for Buildings* in 1978. They were based primarily on engineering judgment and performance comparisons to similar lateral resisting systems in past earhquakes. The 2003 edition of the National Earthquake Hazards Reduction Program (NEHRP) *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* contained more than 75 seismic systems, many of which had never been subjected to design earthquake ground motions. Therefore, the FEMA P695 methodology was developed to provide analytical validations for these systems' R-factors. This methodology uses collapse under maximum considered earthquake ground motions as a performance metric. This performance objective comes from the Commentary in the NEHRP Recommended Provisions that states "if a structure experiences a level of ground motion 1.5 times the design level (MCE), the structure should have a low likelihood of collapse." FEMA P695 develops seismic parameters consistent with the ones provided in ASCE 7. Figure 2 shows how these parameters are evaluated at the MCE level. The R factor is determined by the ratio of the spectral acceleration at the period of the system (S_{MT}) to 1.5 times the seismic response coefficient (C_s). The overstrength parameter (Ω) is determined by the ratio of the maximum strength of the fully yielded system normalized by its weight (S_{max}) to the seismic response coefficent (C_s). Different configurations of a seismic system will produce different Ω factors. The one that is deemed most appropriate is used for Ω_o, which is the factor used in design to protect nonductile elements. As shown in figure 2, C_d is defined as the inelastic displacement of the system at the MCE (SD_{MT}) divided by 1.5 times C_s. From the diagram, it can be seen that C_d is equal to R. In 2018, a BSSC proposal to make Cd equal to R is under consideration (BSSC). This is true for most common systems with an inherent damping of 5%. This methodology defines the collapse level ground motion when one half of the archetypical models used for this methodology become either fully or partially unstable. The collapse margin ratio (CMR) is the ratio of the spectral acceleration at the collapse level ground motion (S_{CT}) to S_{MT}, which is equivalent to the ratio of the spectral displacement at the collapse level ground motion (SD_{CT}) to SD_{MT}. The CMR term can be thought of as a safety factor against collapse and depends upon many factors, such as ground motion variability and uncertainty in design, analysis and construction of the structure.

The FEMA P695 methodology provides a rational basis for understanding and quantifying the seismic performance factors that are used in the design of new buildings. The method correlates these seismic performance factors for a given lateral system to its global probability of collapse. This methodology considers a "system approach" because "collapse failure modes are highly dependent on the configuration and interaction of elements within a seismic-force resisting system" (FEMA P695). This methodology can shed light on the adequancy of the expected performance of lateral systems in the building. It is likely that some systems will be found by future FEMA P695 studies to not meet the expected performance implied in ASCE 7.

Future of Code based Design

Performance-based design, and nonlinear response history analysis, will increasingly play a role in the design of building structures as computers and software continue to improve. However, the majority of buildings will be designed using linear elastic methods for the foreseeable future. Nonlinear analysis, and the FEMA P695 methodology in particular, will likely play a significant role in the development of new building code provisions.

References

Ahmed, Azam, Franco Marina and Fountain Henry (2017), "Luck, Not Tougher Building Standards, Spared Mexico in Quake", *The New York Times,* New York City, New York

ASCE (2016), *Minimum Design Loads for Buildings and Other Structures,* ASCE/SEI Standard 7-16, Reston Virginia

ATC (1978). *Tentative provisions for the development of seismic provisions for buildings*, ATC-3-06, Applied Technology Council, Redwood City, CA

Berg, Glen V. (1983). *Seismic design codes and procedures*, Earthquake Engineering Research Institute, Oakland, CA.

Blume, John A. (1960). "A reserve energy technique for the design and rating of structures in the inelastic range," *Proceedings of the 2nd World Conference on Earthquake Engineering*, Tokyo and Kyoto, Japan.

Blume, John A., Newmark, Nathan, and Corning, Leo (1961). *Design of multistory reinforced concrete buildings for earthquake motions,* Portland Cement Association, Skokie, IL.

Brooks, M. (1987). "Structural engineering software: a survey / status," in *Proceedings: 56th Annual Convention*, San Diego, October 8-10, 1987. Structural Engineers Association of California, Sacramento, CA. 2009*Seismic P 9* P-750

Bozorgnia, Y., and Bertero, V. (2004). *Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering*, CRC Press, Boca Raton, FL.

BSSC (2018), IT2-5 "Cd = R for Deformation Compatibility," Building Seismic Safety Council, Washington, D.C.

California Statutes of 1933. *Riley Act,* Section 19000 et seq. Health and Safety Code. (California)

Canterbury Earthquakes Royal Commission (), "Volume 2 The Performance of Christchurch CBD Buildings," Christchurch, New Zealand

County of Los Angeles (1933), “Verdict of Coroner’s Jury in Earthquake Inquisition”

Deierlein, G.D., Reinhorn, A,M., and Willford, M.R. (2010). “Nonlinear Structural Analysis for Seismic Design”. NEHRP Technical Brief No. 4, produced by the NEHRP Consultants Joint venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 10-917-5.

FEMA (2009). *Quantification of Building Seismic Performance Factors.* FEMA P695. Building Seismic Safety Council, Washington, D.C.

Goel, S.C. (1968). “Response of multistory steel frames to earthquake forces,” in *Steel research for construction* (Bulletin No. 12). American Iron and Steel Institute.

Habibullah, A. (1987). “Structural engineering software for personal computers,” in *Proceedings: 56th Annual Convention*, San Diego, October 8-10, 1987. Structural Engineers Association of California, Sacramento, CA.

Housner, George and Jennings, Paul (1982). *Earthquake design criteria*, Earthquake Engineering Research Institute, Oakland, CA.

LATBSDC (2014). “An alternate procedure for seismic analysis and design of tall buildings located in the Los Angeles region”, Los Angeles, CA.

Maley, R. P. and Cloud, W. K. (1973). "Strong-motion accelerograph records," *San Fernando, California, earthquake of February 9, 1971*, National Oceanic and Atmospheric Administration, Washington, DC.

Martini, K., Amin, N., Lee, P.L., and Bonowitz, D. (1990). “The potential role of non-linear analysis in the seismic design of building structures,” in *Proceedings of the 4th U.S. National Conference on Earthquake Engineering*. Earthquake Engineering Research Institute.

Nakashima, Masayoshi, Inoue, K., and Tada, M. (1998). “Classification of damage to steel buildings observed in the 1995 Hyogoken-Nanbu Earthquake," *Journal of Engineering Structures* 20 (4-6) 271-281.

Olson, R. (2003). “Legislative politics and seismic safety: California’s early years and the ‘Field Act,’ 1925-1933,” *Earthquake Spectra*, v.19 n.1, February.

PEER (2010), “*Tall Buildings Initiative: Guidelines for performance-based sesismic design of tall buildings*”. Report No. 2010/05, College of Engineering, University of California, Berkeley.

Penzien, Joseph (1960). "Elasto plastic response of idealized multi-story structures subjected to strong motion earthquake," *Proceedings of the 2nd World Conference on Earthquake Engineering*, Tokyo and Kyoto, Japan.
Porush, A. and Zacher, E. (1987). “SEAOC, the Blue Book and seismic codes past, present, and future,” in *Proceedings: 56th Annual Convention*, San Diego, October 8-10, 1987. Structural Engineers Association of California, Sacramento, CA.

Reis, E. and Bonowitz, D. (2000). “State of the art report on past performance of steel moment-frame buildings in earthquakes.” FEMA 355E, Federal Emergency Management Agency, Washington, DC.

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

SEAOC Seismology Committee (1960). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1966) *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1990). *Recommended lateral force requirements and commentary,* Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1996). *Recommended lateral force requirements and commentary*. Structural Engineers Association of California, Sacramento. CA.

SEAOC Seismology Committee (1999). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (2019). *Performance-Based Earthquake Design*. Structural Engineers Association of California, Sacramento. CA.

Uzarski, J., and Arnold, C., editors (2001). *Chi-Chi, Taiwan, Earthquake of September 21, 1999 reconnaissance report. Earthquake Spectra*, Supplement A to vol. 17, April.

Veletsos, Anestis and Newmark, Nathan (1960). "Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions," *Proceedings of the 2nd World Conference on Earthquake Engineering*, Tokyo and Kyoto, Japan.

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 2019)

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

SEAOC Seismology Committee (2019). "Code-based earthquake design," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
1.3.1.3		

Introduction

This article is intended to provide an overview of performance-based seismic design (PBSD) history and methodologies. The methodologies presented are not comprehensive, as new methods will continue to surface as practicing engineers and researchers advance the science and art of structural engineering.

The SEAOC Seismology Committee emphasizes that performance-based design is a developing field. In any analysis, there are significant uncertainties in hazard level, soil-structure interaction, construction quality, material properties, and component behavior. For these reasons, the performance implied by these methodologies may not match actual performance.

Contemporary building codes are highly prescriptive in nature. Although they are intended to ensure acceptable minimum levels of performance, there is little direct link between the intended performance and the requirements of the building code. The building codes specify how strong and stiff a structure must be, and the level of detailing that must be provided, but provide little insight into how these characteristics should be modified if better performance is desired. Performance-based design is an alternative to traditional prescriptive methods of design embodied in the building codes. Performance-based design has been under development in the structural community since the early 1990s. The basic premise of performance-based design is that, rather than restricting a design to conformance with a series of prescriptive criteria, the designer must demonstrate, using any of several methods, that a proposed design is capable of providing a specified performance objective. Recognized methods of demonstrating acceptable performance include the following:

- Prototype testing – in which a model or prototype of the design is constructed and subjected to the design loading and the resulting performance observed
- Simulation – in which a mathematical model of the design is developed and its response to design loading simulated so that performance can be estimated
- Conformance with deemed-to-comply, prescriptive criteria.

Testing is commonly used to demonstrate the performance capability of new structural systems; however, laboratory size limitations and testing costs generally restrict testing to small components of systems and substructures rather than full-scale tests of entire buildings. Therefore, for building design, testing is commonly used in conjunction with simulation. Simulation is the most common method of performance-based design for buildings.

Historical Overview

Interest in performance-based earthquake engineering initiated in the mid-to late 1980s as building owners in California considered seismic evaluations and retrofits of existing structures. This interest, fueled by various FEMA grants as a response to the 1971 San Fernando, 1989 Loma Prieta and 1994 Northridge earthquakes, resulted in numerous efforts by multiple organizations and researchers to develop PBSD criteria and methodology. Noteworthy events related to PBSD are illustrated in the table that follows:

Year	Publ.	Description/Summary
1979	TM-304	Army technical manual that took an early lead in developing formal PBSD criteria. Required simulation of inelastic response and prediction of performance using two methods. One utilized elastic analysis and calculation of Inelastic Demand Ratios (IDR's) as measures of element performance, while the second introduced the concept of nonlinear static (pushover) analysis and the capacity-spectrum technique to estimate inelastic structural performance

1987 - 1992	ATC-14, ATC-22, FEMA-178	Formalized the process of evaluating performance of existing buildings, and in particular, the ability of a structure to provide suitable levels of life safety protection
1993	Priestley, 1993	Introduced Direct Displacement-Based Design (DDBD) philosophy.
1995	SEAOC Vision 2000	Presented a conceptual framework for the development of a comprehensive set of performance-based criteria and procedures. Was successful in the popularization of performance-based design, generating world-wide interest in the subject and a demand for development of useable performance-based design criteria
1996	FEMA 283	Proposed an action plan and tasks to be implemented in order to develop the technical basis for PBSD.
1996	ATC-40	Developed performance-based evaluation and retrofit methodology for nonductile concrete frame buildings. Enhanced the capacity spectrum technique originally introduced in TM-304 and provided extensive acceptance criteria for evaluating the performance of existing nonductile concrete buildings.
1996 1997	ATC-33, FEMA 273/274	Major effort in the development of guidelines for the seismic rehabilitation and retrofit of buildings. Later adopted as a prestandard, FEMA-356 (2000), and as a standard, ASCE 41 (2006). This document represented the first comprehensive performance-based design methodology that included performance criteria, simulation procedures for predicting performance, and comprehensive acceptance criteria that could be used to convert the results of a simulation into a performance estimate.
1999	SEAOC Blue Book	Proposed a deemed-to-comply prescriptive approach and provided guidelines for a displacement-based engineering approach
2000	FEMA 356	Pre-standard based on FEMA 273/274.
2000	FEMA 349	Action plan built upon the efforts of the FEMA 283 publication, describing tasks that should be completed to further develop performance based seismic design criteria.
2003	ASCE 31	Based on FEMA 178, Standard for the Seismic Evaluation of Existing Buildings
2006	FEMA 445	Proposed a Program Plan for the development of the "next-generation" Performance-Based Seismic Design Guidelines. Refinement and extension of FEMA 283 and FEMA 349. Further development of the Program has been completed under the ATC-58 project.
2006, 2013, 2017	ASCE 41	Based on FEMA 273/274 and 356, Standard for the Seismic Rehabilitation of Existing Buildings. In 2013, it merged with ASCE 31 and was retitled Seismic Evaluation and Retrofit of Existing Buildings.
2008	San Francisco A.B. 083	City and County of San Francisco adopted this provision, clarifying the requirements and guidelines for tall buildings that do not conform to prescriptive code requirements.
2010	PEER TBI	*Tall Building Initiative*, presented guidelines for the implementation of alternative design procedures for tall buildings.
2010	ASCE 7-10	Introduced limited PBSD procedure language to this widely used Standard for new buildings.
2011	LATBSDC	*The Los Angeles Tall Buildings Structural Design Council* developed an *Alternative Analysis and Design Procedure* document. Provides a performance-based approach for seismic design and analysis of tall buildings.
2012	ATC-58, FEMA P-58	Work initially began in 2006 on this monumental Next-Generation Performance Based Seismic Design Procedures methodology. Developed procedure for seismic performance assessment of buildings attempting to summarize building performance in terms of losses measured in "death", "dollars", and "downtime".
2018	ATC 114	Development of updated hysteretic envelope models for use in seismic analysis. Intent was to support the development of updated building code criteria contained in ASCE 7 and ASCE 41.
2018	FEMA P-2006	Example Application Guide for ASCE/SEI 41-13 with Additional Commentary for ASCE/SEI 41-17 – an excellent resource for how to use ASCE 41.
2018	ATC 120	Seismic Analysis, Design, and Installation of Nonstructural Components and Systems with a focus on performance-based approaches.
2019	FEMA P-58-2	Project to enable and encourage use of FEMA P-58 methodology. Although not yet publicly available, it will provide improved fragility library; calibration, and benchmarking of results, simplified design aides, and tools to assist stakeholders in the decision-making process.

2023 (est.)	ATC 140	Update of Seismic Retrofitting Guidance will help generate case studies and calibrations as the basis for comprehensive standards update proposals for ASCE 41-23.

Displacement Based Methodologies

There are numerous displacement-based design methodologies that have been proposed and used over the years for both steel and concrete structures. Currently, these methods are sometimes used during the early design stages in order to select initial seismic force resisting member sizes.

Direct Displacement Based Design (DDBD)
First introduced in 1993 (Priestley, 1993), the Direct Displacement Based Design (DDBD) approach is based on the concept that the structure should be designed to achieve a specified performance level defined by strain or drift limits. The design process can be described as providing "uniform-risk" structures, and is somewhat analogous to the concept of "uniform-risk" spectra used in previous approaches.

The DDBD procedure includes the following steps:

1. The structure is conceptualized as a single degree of freedom and its displacement ductility is calculated.
2. The damping ratio is determined as a function of displacement ductility through hysteretic relationships.
3. The effective period is related to design displacement using a displacement response spectrum.
4. The effective stiffness is determined from the effective period.
5. The base shear is determined from the effective stiffness and the displacement.

The design process is relatively simple, and is summarized in the 2007 publication by Priestley et al, which also includes proposed model code language incorporating the Direct Displacement Based Design methodology.

Equal Displacement-Based Design (EBD) Procedure
As described in Appendix I of the 1999 SEAOC Blue Book, the EBD procedure is based on the equal displacement approximation (i.e. the displacement of an inelastic system is approximately equal to the displacement of an equivalent elastically responding system). The EBD is an adaptation of the traditional force-based code procedures, but it uses the displacement response spectrum rather than the acceleration response spectrum, and focuses on displacements rather than forces. The procedure includes the following:

1. Estimation of target drifts
2. Determination of required initial stiffness based on the design ductility limits and the acceleration displacement response spectrum
3. Determination of required strength based on the target displacement, initial stiffness, and system ductility
4. Design of members based on required stiffness, strength, and ductility.

It should be noted that the SEAOC performance-based design procedure represented a majority opinion of the ad hoc subcommittee. A minority of the subcommittee was of the opinion that the documents should be further evaluated and tested before being published. Consequently, it was recommended that the 1999 guidelines be used with caution.

ASCE 41

ASCE 41 *Seismic Evaluation and Retrofit of Existing Buildings* is a national standard that uses performance–based principles. The standard contains a three-tiered process for seismic evaluations, with each tier increasing in complexity and accuracy. The Tier 1 procedure, or "screening" procedure, is intended to be used as a quick first pass evaluation methodology, where checklists and "quick check" calculations are used. The Tier 2 procedure is intended to be used for either evaluation or retrofit, and is deficiency based in that only specific deficiencies deemed to be of concern for certain building types and heights need be retrofitted. The Tier 3 procedure is intended to be a systematic analysis of the building, and can also be used for evaluation or retrofit. Four analysis procedures are presented (linear static, linear dynamic, non-linear static, and nonlinear dynamic), with acceptance criteria and modeling requirements.

As part of an ASCE 41 evaluation or retrofit, the user selects a target performance level (such as life safety, collapse prevention, etc), and the seismic hazard level (such as a more frequent smaller earthquake or a larger more rare earthquake). In most cases, dual performance levels and hazard levels are analyzed. Building components are assigned as being “deformation-controlled” (i.e. some ductility) or “force-controlled” (i.e. little to no ductility). If linear analysis procedures are used, then deformation-controlled elements are assigned “m-factors”, which are component demand modification factors that account for expected ductility associated with a specific force action and structural performance level. These m-factors are similar to the R factors of ASCE 7, in essentially reducing the earthquake demands an element is analyzed against; however, they only apply to specific deformation-controlled elements, and not the entire lateral system as is the case with the R factors of ASCE 7. If non-linear procedures are used, then deformation-controlled elements are assigned idealized force-displacement properties, and component acceptance is based on displacements and/or rotations.

ASCE 41-17 provides numerous revisions to the standards’ Basic Performance Objectives, seismic hazard used in the Tier 1 and Tier 2 evaluations, nonlinear analysis provisions, nonstructural performance levels, out-of-plane wall demands, modeling parameters and acceptance criteria for steel and concrete columns, and anchor testing (Pekelnicky, et al. 2017). The revisions to the modeling and acceptance criteria for steel columns were informed by a series of studies conducted by NIST (NIST 2015a, 2015b, and 2015c). The intent of the studies was to “benchmark” ASCE 41 procedures with traditional building code procedures (i.e. ASCE 7). The results of the studies confirmed that the steel column provisions in ASCE 41-13 and previous editions resulted in more conservatism when compared to ASCE 7-10 based designs, as the buildings designed under the ASCE 7-10 and AISC 341-10 criteria did not meet the Basic Performance Objective for New Buildings (BPON). Therefore, ASCE 41-17 essentially calibrated both the m-factors and nonlinear modeling and acceptance parameters to be less conservative and should result in new building designs meeting the BPON when analyzed using ASCE 41-17 procedures.

Tall Building Procedures

City of San Francisco Administrative Bulletin 083

In 2008, the City and County of San Francisco adopted Administrative Bulletin 083, *Requirements and Guidelines for the Seismic Design of New Tall Buildings using Non-Prescriptive Seismic-Design Procedures*. The purpose of the document is to clarify the requirements and guidelines for buildings that do not conform to the prescriptive requirements of the building code. The intent of the document is to provide building designs with seismic performance that is at least equivalent to that of code-prescriptive seismic designs.

This document outlines requirements for structural design review, submittals and seismic design. The designer is required to evaluate the building for a Service-Level Evaluation, a Code-Level (Design Basis Earthquake) Evaluation, and a Maximum Considered Earthquake-Level (Nonlinear Response History) Evaluation. This document also specifies load combination criteria and maximum peak story drift ratios.

PEER Tall Buildings Initiative (TBI)

The *Guidelines for Performance-Based Seismic Design of Tall Buildings*, initially published in 2010, and revised in 2017, present an alternative design procedure to the prescriptive procedures for the seismic design of buildings contained in standards such as ASCE 7, and are intended to result in building designs that are capable of meeting the same performance objectives intended by ASCE 7. The Guidelines were developed considering the unique seismic response characteristics of tall buildings, including long natural periods of vibration, significant mass participation in higher modes, and relatively slender profiles.

The Guidelines require a dual seismic hazard approach, where a Service Level evaluation is performed using either linear response spectrum procedures, or if beneficial, nonlinear response history analysis. The Maximum Considered Earthquake evaluation is required to be performed via nonlinear response history analysis. Similar to ASCE 41, components are designated as either force-controlled or deformation-controlled, depending on their perceived ductility. Deformation capacities are allowed to be taken from ASCE 41, or may be based on analytical models that are supported by experimental evidence.

Los Angeles Tall Building Structural Design Council

The Los Angeles Tall Buildings Structural Design Council developed the *Alternative Analysis and Design Procedure* document in 2011. The most recent publication occurred in March of 2018. The intent of the document is to provide a performance-based approach for seismic design and analysis of tall buildings. Similar to the PEER TBI Guidelines, this document presents an alternative design procedure to the prescriptive building code requirements.

This procedure is based on capacity design principles and a series of performance-based design evaluations. A dual seismic hazard approach is used, wherein the building design is checked against two earthquake ground motion intensities. One hazard check can be described as the serviceability check at the frequent earthquake, and consists of building structural behavior remaining essentially elastic for the service level design earthquake ground motions having a 50 percent probability of being exceeded in 30 years (43 year return period). The second hazard check can be described as the low probability of collapse check at the extremely rare earthquake ground motions. At this hazard level, nonlinear behavior of the structural elements would be expected at the Risk Targeted Maximum Considered Earthquake (MCE_R) ground motions as defined by ASCE 7-16. The document requires that a three-dimensional mathematical model with nonlinear element properties be used at the MCE_R hazard check.

Probabilistic Procedures (e.g. FEMA P-58)

In 2006, the Applied Technology Council (ATC) began development on Next-Generation Performance Based Seismic Design Guidelines for New and Existing Buildings (ATC-58-1 project). The main purpose of this project was the development of a methodology for seismic performance assessment of individual buildings that accounts for uncertainty in our ability to accurately predict building and component response, and communicates performance more effectively to building owners and stakeholders. This procedure attempts to summarize building performance in terms of human loss (death), direct economic loss (dollars), and indirect losses (downtime). To allow for implementation of the methodology, an analysis tool titled Performance Assessment Calculation Tool (PACT) was developed that includes a collection of fragility and consequence data for most common structural systems and building occupancies, and performs probabilistic computations.

The methodology can be summarized as follows:

1. Create Building Performance Model – a model of the building including structural components, assemblies, nonstructural systems and components that can be damaged by the response of the building to earthquake shaking. Fragility data are associated with these individual elements, expressing the vulnerability of the components to damage, and the consequences of this damage in terms of risk to human life, repair methods, repair costs, and repair time.
2. Define Earthquake Hazards – ground shaking hazards can be considered in different ways, including intensity-based assessments, scenario-based assessments, or time-based assessments.
3. Analyze Building Response – structural analysis is used to predict the response of the building to the ground shaking, typically expressed in terms of story drift, floor accelerations, and residual drift.
4. Develop Collapse Fragility – define the probability of incurring structural collapse as a function of ground motion intensity.
5. Calculate Performance – Monte Carlo simulation is used to account for the many uncertainties associated with seismic performance. The PACT tool was developed to assist in performing the probabilistic computations and manage the data from the Monte Carlo calculations.

Although rigorous and powerful, the implementation of the FEMA-58 methodology and the PACT tool in the engineering community has been slow. This is due to the difficulty in use of these tools, as the methodology is complex and the PACT tool is somewhat onerous to use. In response to these difficulties, the Haselton Baker Risk Group has developed the Seismic Performance Prediction Program (SP3) to simplify the use of the FEMA P-58 methodology. This tool allows users to quickly obtain comprehensive prediction of building damage via a user-friendly interface, and quantifies building performance for losses (i.e. dollars), repair time (i.e. downtime), and fatalities/injuries (i.e. death).

Phase 2 of this project (ATC 58-2) is nearing completion, and its primary purpose is to enable and encourage use of the FEMA P-58 methodology in both new building design and retrofit (Hamburger 2017). An initial task involved using the methodology to quantity building performance of typical prescriptively-designed code-conforming buildings, thereby allowing the creation of "code-equivalent" performance objectives expressed in the FEMA P-58 terms of "dollars", "downtime", and "death". This phase also has developed the following: engineering design aids to assist the engineer in rapidly identifying performance characteristics based on system strengths and stiffnesses, educational aides and tools specifically designed for use by "decision-makers" (i.e. buildings owners and tenants), incorporation of an environmentally conscious model with metrics including CO2 and energy usage, and updated and enhanced existing fragility and consequence functions.

The FEMA P-58 methodology can be used indirectly to assist in building design and retrofitting; however, it does not include any direct design guidance or tools.

Performance Based Design and ASCE 7

ASCE 7-10
For many years, building codes have provided "alternative means and methods" language that allows for the use of materials, design, and construction that differ from the prescriptive code requirements, provided equivalent performance can be justified (see Section 104.11 of the IBC). ASCE 7-10 contains a brief section on performance-based procedures and indicates that structural and nonstructural components and their connections designed using performance-based procedures are to demonstrate by analysis, or analysis supplemented by testing, that their reliability is not less than that expected for similar components designed using the prescriptive sections of the Standard. The commentary of the Standard provides target reliabilities, in terms of conditional probability, of failure caused by MCE_R shaking based on Risk Category. The Commentary indicates that rigorous methods of reliability analysis can be used to demonstrate adequate reliability, but also recognizes that any method that evaluates the likelihood of failure considering the potential uncertainties to the satisfaction of peer reviewers and the authority having jurisdiction should be acceptable, and then specifically mentions ASCE 41 as a potentially acceptable method.

ASCE 7-16
With the publication of ASCE 7-16, much of the PBSD direction that had been part of the commentary has now been relocated to the Provision sections. The Provision language now indicates that PBSD may be allowed, if approved by the Authority Having Jurisdiction. The Standard indicates that when PBSD procedures are used, the structural and nonstructural components and their connections are to demonstrate a reliability that is "generally consistent" with the target reliabilities stipulated in the Standard. The Commentary essentially prequalifies ASCE 41 and the *Tall Buildings Initiative* (TBI) guidelines, and states that these documents "were either calibrated by structural performance level or were demonstrated in comparison with prescriptive design methods to provide reliabilities equal to or better than [the tables in the Provisions]".

Current Practice and Improvements

Performance Based Seismic Design is still being refined and developed by practicing engineers and researchers. There is not one "best" or "preferred" methodology; however, the effectiveness of one methodology or another may depend upon the building type, size, height, whether the building is new or existing, and the schedule and cost demands. The current state of PBSD practice is generally consistent with the following:

For evaluations/retrofits of existing structures: ASCE 41 methodologies are typically used. Basic safety performance objectives are most common for typical building and occupancy types, and Enhanced performance objectives are more commonly used for higher risk category structures, high-dollar manufacturing or distribution facilities, corporate headquarters, and culturally significant structures. The FEMA P-58 methodology is occasionally used (relying on criteria provided by an ASCE 41 analysis) to provide estimates for "deaths", "dollars", and "downtime" losses for current building conditions, and comparative retrofit scheme investigations as well.

For new design of tall buildings: If the building height or proposed system exceeds the code allowed height limits, then typically an analysis is performed in accordance with the TBI guidelines or the Los Angeles Tall Building Structural Design Council procedure. The selected performance objective is typically intended to meet code minimum performance levels; therefore, enhanced performance objectives are typically not explored or targeted for these types of buildings.

For new design of buildings: The only new buildings (besides tall buildings mentioned previously) that consistently utilize PBSD methodologies are essential facilities in California, with the most likely candidates being large hospitals and emergency operation centers, and base-isolated structures.

Calibration Efforts

Since the vast majority of building design and retrofit projects will continue to rely on prescriptive design procedures for many years to come, one of the best uses of performance-based seismic design is the calibration and adjustment of prescriptive practices to achieve more reliable and consistent performance.

Various NIST programs have been implemented to address the calibration of both traditional code designed buildings (i.e. based on ASCE 7) and buildings designed or analyzed using ASCE 41. NIST recently completed a study of three structural steel framing systems to evaluate the correlation between ASCE 7 and ASCE 41 (NIST 2015a, 2015b, and 2015c). One key conclusion is that the collapse likelihood of code-compliant (ASCE 7) buildings should be better understood.

The NIST program "Improved Assessment Criteria for Performance-Based Seismic Design" has the goal of establishing PBSD assessment criteria that accounts for a components action's dependence on the loading history. This research focuses on eccentrically braced frames; however, its methodology and findings will shed light on other systems as well.

ATC 140 began in 2017, and is scheduled to produce case studies and standards change proposals each year until 2023 in support of ASCE 41's 2023 edition. These case studies will likely provide insights into the validity and shortcomings of ASCE 41-17.

FEMA P-695 studies are being used to calibrate R and Cd factors for both new and existing building systems and proprietary subsystems.

Conclusion

Performance-based seismic design is more than just the sum of the various methodologies, research papers, test results, guidelines, and software tools available; it provides a language with which the structural engineer can communicate directly with the public regarding seismic risk and building performance. Unfortunately, building owners and stakeholders are typically unaware of the potential for PBSD techniques to be used for their particular projects, and engineers have not effectively communicated the benefits of using these available tools.

SEAOC hopes that funding and grants to NIST, ATC, and similar organizations continue so that additional testing and research can be implemented, calibration efforts can be extended, and current tools can be refined and improved. With these continued efforts, the implementation of PBSD will continue to grow, benefitting not only the engineering community, but the world at large.

References

ASCE (American Society of Civil Engineers) (2006). ASCE 7-05, Minimum Design Loads for Buildings and Other Structures, Including Supplement No. 1. American Society of Civil Engineers, Reston, VA
ASCE, 2010, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, Virginia.

ASCE, 2003, Seismic Evaluation of Existing Buildings, ASCE 31-03, American Society of Civil Engineers, Reston, Virginia.

ASCE, 2006, Seismic Rehabilitation of Existing Buildings, ASCE 41-06, American Society of Civil Engineers, Reston, Virginia.

ASCE, 2013, Seismic Evaluation and Retrofit of Existing Buildings, ASCE 41-13 and -17, American Society of Civil Engineers, Reston, Virginia.

ATC (1978). Tentative provisions for the development of seismic provisions for buildings, ATC-3-06, Applied Technology Council, Redwood City, CA.

ATC (2018). Seismic Analysis, Design, and Installation of Nonstructural Components and Systems, ATC-120, Applied Technology Council, Redwood City, CA.

ATC (2018 and annually thereafter).Update of Seismic Retrofitting Guidance, ATC-140, Applied Technology Council, Redwood City, CA.

Bozorgnia, Yousef, ed.; Bertero, Vitlelmo V. (2004) Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering, CRC Press, New York.

FEMA (Federal Emergency Management Agency), FEMA-178 NEHRP Handbook for Seismic Evaluation of Existing Buildings.

FEMA 273, 1997, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.

FEMA 274, 1997, NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, prepared by the Applied Technology Council, for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.

FEMA 283, 1996, Performance-Based Seismic Design of Buildings – An Action Plan for Future Studies, prepared by the Earthquake Engineering Research Center for the Federal Emergency Management Agency, Washington, D.C.

FEMA 349, 2000, Action Plan for Performance-based Seismic Design, prepared by the Earthquake Engineering Research Institute for the Federal Emergency Management Agency, Washington, D.C.

FEMA 350, 2000, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, prepared by the SAC Joint Venture, a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering, for the Federal Emergency Management Agency, Washington, D.C.

FEMA 356, 2000, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

FEMA 445, 2006, Next-Generation Performance-Based Seismic Design Guidelines, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA P-695, 2009b, Quantification of Building Seismic Performance Factors, prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

FEMA P-2006, 2018, Example Application Guide for ASCE/SEI 41-13 with Additional Commentary for ASCE/SEI 41-17, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

Hamburger, Ronald, et al. (2016) “ATC-114 Next-Generation Hysteretic Relationships for Performance-based Modeling and Analysis”, SEAOC Convention Proceedings, Structural Engineers Association of California, Sacramento, CA.

ICC, 2015, International Building Code, International Code Council, Washington, D.C.

NIST GCR 09-917-2, Research Required to Support Full Implementation of Performance-Based Seismic Design, National Institute of Standards and Technology, Gaithersburg, MD, 2013

NIST GCR 13-917-23, Development of NIST Measurements Science R&D Roadmap: Earthquake Risk Reduction in Buildings, National Institute of Standards and Technology, Gaithersburg, MD, 2013.

NIST Technical Note 1863-1 (2015a), Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 1: Special Moment Frames, National Institute of Standards and Technology, Gaithersburg, MD.

NIST Technical Note 1863-1 (2015b), Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 2: Special Concentrically Braced Frames, National Institute of Standards and Technology, Gaithersburg, MD.

NIST Technical Note 1863-1 (2015c), Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Volume 3: Special Concentrically Braced Frames, National Institute of Standards and Technology, Gaithersburg, MD.

Pekelnicky, R., Hagen, G., and Martin, D. (2017) “A Summary of Significant Updates in ASCE 41-17”, 2017 SEAOC Convention Proceedings, Structural Engineers Association of California, Sacramento, CA.

Priestley, M.J.N. (1993) “Myths and Fallacies in Earthquake Engineering – Conflicts Between Design and Reality” Bulletin, NZSEE 26(3)

Priestley, M.J.N. (2003) Myths and Fallacies in Earthquake Engineering, Revisited. The Mallet Milne Lecture, 2003, IUSS Press, Pavia.

Priestley M.J.N., Calvi, M.C., and Kowalsky, M.J. (2007) Displacement-Based Seismic Design of Structures IUSS Press, Pavia.

SEAOC Seismology Committee (1996). *Recommended lateral force requirements and commentary,* sixth edition, Structural Engineers Association of California, Sacramento, CA.

Keywords
Performance-based
PBSD
Seismic

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Performance-Based Seismic Design Methodologies," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 REFERENCE SECTION(S)	2019 CBC REFERENCE SECTION(S)	OTHER STANDARD REFERENCE SECTION(S)
	§104.11	FEMA P695 FEMA P795 ASCE/SEI 41—17 ISO/IEC Guide 17065:2012 ISO/IEC 17007:2009

Background

Evaluation Reports (ERs), sometimes referred to as Code Reports, Code Compliance Research Reports or Technical Evaluation Reports, are created to either evaluate products or methodologies that are not explicitly addressed by the building code, or to streamline the design and review processes of those that are defined by the code. ERs have been used for decades to provide engineers, building officials, building owners and others an efficient way to assess the adequacy of new or unique products for use in buildings and other structures. As a result, a single product does not need to be re- engineered, re-tested, and re-reviewed with each use.

ERs are generally authorized under the building code's Alternative Means of Compliance (AMC) provision (CBC Section 104.11) portions of which are shown below.

> *104.11 Alternative materials, design and methods of construction and equipment. ... An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, ... equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.*
>
> *104.11.1 Research reports: Supporting data, where necessary to assist in the approval… shall consist of valid research reports from approved sources.*
>
> *104.11.2 Tests. ... building official shall have the authority to require tests…*

Evaluation reports rely upon on a list of specific testing criteria referred to as Acceptance Criteria or Evaluation Criteria (herein referred to as AC). Once a product is tested and found to meet the requirements of an AC, an Evaluation Report can be created for that product. ERs can then be submitted as evidence that a product complies with the building code, often via the AMC provision.

In 2009, the SEAOC Evaluation Reports Committee was tasked with evaluating the current practices relating to the development and implementation of AC and ERs and providing recommendations for improvement. **The Committee's key finding is that better understanding of the AC and ER development process, as well as proper implementation by developers and users, will result in the most immediate and broad improvement to their appropriate use in the construction industry.** (For more detailed information including additional information and recommendations to AC and ER providers beyond this paper, see SEAOC (2011), *An Evaluation of Current Practices Related to Product Evaluation Reports and the Development of Acceptance Criteria*.)

One difficulty posed by the AMC process, and with evaluation and approval of ERs, is that products, codes, and standards have become more complex in recent years. Additionally, staffing and technical resources available to building officials have not grown at the same rate and, in many cases, have been reduced. Due to increasing pressure on design professionals and building officials from building owners, developers, and builders to use new products or systems (especially during economic downturns), difficulties arise when a building official does not have the expertise or resources to properly evaluate a product AMC submittal.

ERs were originally predominately used for new products, materials, design or construction methods, or systems that were not considered in the development of code provisions. We refer to this application of an ER as *Scenario 1, Alternate Means of Compliance*.

More recently, ERs have been used in two other scenarios. When a product is only partially addressed by code, ERs may be used to facilitate the use of the product. In this case, industry experts are tasked with interpreting or developing evaluation and design guidelines based on the available code provisions, product application, and an understanding of code intent. We refer to this as *Scenario 2, Assisting with Interpretation of Code Ambiguities*. In *Scenario 3, Expediting Product Acceptance*, ERs aid in the design procedure for a product that is prescribed by the code, but in which code provisions are too complex or time consuming to follow. In this case, creation of an ER helps to expedite the design and review process for products or systems for these otherwise complex or difficult products.

ERs improve the construction industry practice in several ways. Some of the benefits are identified below.

1. Structural engineers have more confidence that a product meets code intent and will be approved by the Authority having Jurisdiction (AHJ), and are therefore more willing to incorporate new and innovative technology into their designs.

2. Structural engineers can design more efficiently.

3. Building officials use ERs to facilitate and expedite the process of reviewing and approving complex products to meet increasingly elaborate codes and standards.

4. Inspectors have a clearly defined scope of field inspection requirements.

5. Structural engineers and contractors can use alternatives more readily.

6. Manufacturers with ERs gain industry acceptance of their products.

7. Manufacturers provide evidence to only the ER Provider, reducing risk of losing control of proprietary information.

8. Manufacturers differentiate their products from those of competitors who do not seek or obtain a report. This differentiation primarily consists of increased industry confidence that a product complies with the code or code intent. Of course, confidence may vary depending on the robustness of the ER Providers' AC and ER processes.

9. Building owners and users have higher confidence that the new products used in their buildings are code compliant, and therefore safe, and may offer more economical solutions.

A Brief History of Evaluation Services

Evaluation Reports were initially created by building code organizations to help building officials evaluate products, methods, and materials (referred to as "products") that were not addressed in the building code, or to evaluate product conformance to prescriptive code requirements.

Because jurisdictions have limited technical staff and budgets, a group of building officials formed a research committee to review technical data on products and issue research reports regarding product code compliance to the building officials. In California in the mid- to late-20th century, these were based on the provisions of the Uniform Building Code (UBC) published by the International Conference of Building Officials (ICBO). ICBO's subsidiary, ICBO Evaluation Services, published Evaluation Reports and Acceptance Criteria.

Over time, other regional model code writing organizations, such as Building Officials and Code Administrators (BOCA) and Southern Building Code Congress International (SBCCI), also became involved in evaluation of building products and sought to improve the process. These were later joined by National Evaluation Service (NES) which was a combination of ICBO, BOCA and SBCCI, and may be considered the predecessor entity to ICC. In 2003, all of these regional model code groups (ICBO, BOCA and SBCCI) merged into the International Code Council (ICC) that we know today. In order to continue the historical activity of product evaluation for code compliance, ICC created a subsidiary non-profit organization, ICC Evaluation Service (ICC-ES).

It should be noted that ICC-ES uses the term "Legacy Reports" to describe ERs originally issued by BOCA ES, ICBO ES, NES or SBCCI PST and ESI, which were still valid when those organizations combined to form ICC-ES. A Legacy Report also refers to any ER that was issued by ICC-ES before March 1, 2003. Legacy reports can only be editorially updated and can only be technically updated if converted to an ES Report complying with applicable current acceptance criteria.

Other organizations and firms (also referred herein as ER Providers) have begun providing product evaluation services in response to the increased market for ERs. These include the American Plywood Association (APA), Intertek, the International Association of Plumbing and Mechanical Officials Uniform Evaluation Service (IAPMO UES), Dr. J, and NTA, Inc.

Some of the ER Providers are accredited by the American National Standards Institute (ANSI) per ISO/IEC Guide 17065, *General Requirements for Bodies Operating Product Certification Systems*, including ICC-ES, IAPMO UES, NTA and ATI, while others have varying methods and levels of accreditation. Guide 17065 focuses on providing third party verification of procedural quality control of product tests.

SEAOC supports the position that ER Providers must be accredited to Guide 17065, possibly with additional requirements. To this end, ISO/IEC Guide 17065 has been adopted as the standard of care by ICC-ES, IAPMO UES and others. Note that IAPMO UES currently bases the content of an ER on ISO/IEC 17007:2009(en).

AC Development Process

Discussion of the AC development process below is based on policies of ICC-ES, which has developed ACs for longer than any other ER Provider, and is the largest developer of ACs. IAPMO UES, the second largest developer of ACs, is also a source for this discussion. The policies and processes of ICC-ES and IAPMO UES related to the development of AC differ in the following manner:

1. The ICC-ES Evaluation Committee convenes in formal public hearings three times a year. The IAPMO UES Evaluation Committee schedules public hearings on a case-by-case basis.

2. ICC-ES requests applicants to submit new AC proposals or proposed revisions to existing AC typically three and a half months prior to the Evaluation Committee meeting. ICC-ES then revises the draft, as they feel necessary, prior to posting. IAPMO UES requests that applicants submit new AC proposals or revisions to existing AC within a 20-business day public commenting period, and the process may be repeated as determined by staff review of the proposals.

3. ICC-ES posts the drafts of the AC to be discussed at the hearing on its website (in the public domain) 30 days prior to the hearing to solicit comments from all interested parties. They also post the public's comments, rebuttal comments by proponents, and ICC-ES's staff's responses to the public's comments. Therefore, there are two opportunities for the public to comment on AC drafts (on the website post and during the public hearing). IAPMO UES schedules an Evaluation Service Committee meeting and places proposed criteria on the agenda. If accepted by the committee, the completed Evaluation Criteria is posted on the IAPMO UES website. IAPMO UES does not post the public comments or their responses.

4. ICC-ES has a regimented, transparent, formal hearing process covering numerous new AC proposals and existing AC revision proposals. It involves the in-person participation of the Evaluation Committee as well as ICC-ES technical staff and interested industry members. The IAPMO UES endeavors to resolve public comments prior to a hearing and may involve multiple public notices. The public hearing is scheduled once the comment period is judged complete, and the hearing process involves the Evaluation Committee, which (at the judgment of staff) can vary from a conference call to a public hearing. The IAPMO UES technical staff, IAPMO UES consultants, and interested industry members attend the meeting either by conference call or electronic web-based meetings.

5. ICC-ES hearings involve formal, structured discussions between the public, ICC-ES staff and the Evaluation Committee. All public participants are given the opportunity to express their opinions, with an opportunity for the Committee and Staff to have discussions between themselves and the public participants. At the conclusion of these discussions, the Committee votes on the disposition of the criteria. The disposition may be "approval", "approval-as-amended at the meeting", or to "hold the criteria for further-study" pending resolution of issues that the Committee deems necessary. If the Committee holds an AC for further-study, the staff is requested to bring the criteria back to the public hearings when the issues have been resolved. The Committee never "rejects" an AC proposal, but gives the proponent an opportunity to revise the criteria and bring it back to the hearings.

   The IAPMO UES process regarding resolution of comments is considered in the revision of the AC that was posted to their website for public comments. IAPMO UES staff will revise the AC to reflect the public comments that they believe are relevant to the health and safety of the public. IAPMO UES will not revise an AC to reflect comments that they believe do not have implications to the health and safety of public. The IAPMO UES staff then presents the IAPMO UES Evaluation Committee with the AC or revised AC for the Evaluation Committee's approval or rejection or further study. IAPMO UES evaluation committee may "reject" an AC.

6. ICC-ES determines a compliance date by which products evaluated using an existing AC must comply with the newly adopted revisions. IAPMO UES typically lists an effective date for each AC, and only requires a mandatory compliance date where deemed necessary.

Acceptance Criteria developed for products or systems typically contain the following information:

1. Purpose: Why the AC is required (e.g., lack of requirements in applicable code).
2. Scope: Description of the limits or the scope of the AC.
3. Referenced Standards: Listing of applicable code and standard references.
4. Basic Information and Test Reports: Minimum submittal information requirements.
5. Test Performance Requirements: Description of tests required, test setups, acceptable boundary conditions, test methods, permitted test variance, load rating methodology (sometimes with displacement as well as strength considerations).
6. Analysis Details: Minimum analysis requirements.
7. Materials and Workmanship: Minimum requirements for materials and product fabrication.

8. Quality Control: Requirements for initial and/or ongoing inspection requirements as well as quality document submittals.
9. Evaluation Report Recognition: Requirements for specific language to be included in the ER.

Choosing which AC to use for product evaluation can be difficult. There are a variety of circumstances where inconsistencies arise with respect to the AC chosen for some products. A few scenarios are presented below.

1. Using the same AC to evaluate similar products. This can be helpful in maintaining consistency between evaluations but can also be detrimental if the two products are dissimilar in certain ways.

2. Similarly, using different AC to evaluate similar products can be problematic. For example, one AC may be a better way to tailor testing to the specific behaviors of a product. However, it may also generate inconsistencies and different results between two products where it may not be warranted.

3. The existence of similar ACs can bring about the issue of value judgments between ACs where industry members are forced to choose which AC is better than another.

4. The economics of AC development may also factor in to the decision to use a previously developed AC. Because significant time goes into developing AC, it may be more economical to use an existing AC rather than develop a new one, even if a new one is warranted.

5. Another economic factor is that of copyright challenges. Because ACs take time and money to develop, there are incentives for developers to copyright them. Organizations that develop ACs view the AC as their intellectual property and, unlike standards such as those developed by ASTM, they develop ACs for their internal use. ICC-ES is one such organization. This limits the use of AC within the industry as a whole, and may limit the selection of useful AC for a specific product.

Some ACs reference other ACs. An example would be a product AC that references an AC that covers the requirements for test reports as well as an AC that covers the requirements for quality control program documentation submittals.

There are several procedural challenges that to the development of ACs. If understood, these challenges can be managed effectively, which can promote positive changes in the development of AC:

1. **Short Review Periods:** The public comment periods for AC hearings adhere to a strict timetable so that AC approvals can be obtained in a timely manner. This presents difficulties for volunteer organizations, like SEAOC and NCSEA, to review and provide vetted, consensus comments. Similar to the code provision development process, floor amendments to AC can occur at the Evaluation Committee meetings, making it difficult for SEAOC to respond, as there is not time to reengage committee expertise.
2. **Coordination with Standards and Code**: Sometimes the process of AC creation conflicts with the development and evolution of standards and codes. For example, seismic system coefficients are prescribed in ASCE 7, Table 12.2-1. These seismic system coefficients have evolved through a qualitative and largely judgment-based vetting process, and there is an ongoing debate regarding the consistency and accuracy of these values. New products (both individual components and systems) are required to align with the code-prescribed systems for the system/component use in design. ER Providers do not develop system coefficients, and the standards writing bodies (i.e. ASCE 7) concur with this limitation of power. However, by the mere existence of a new product, one must consider the assignment of seismic design coefficients. Without prescribed seismic design coefficients, one must establish a context for assignment of equivalent seismic design coefficients without direct comparative data. As a result, gaps in the AC creation process

occur that no single organization can address.

3. **ER Provider Quality Controls**: As previously discussed, ISO/IEC Guide 17065 provides a process whereby the ER issuer certifies that the product is compliant with the associated AC, with the expectation that others will rely upon the certification and allow the use of the product. The current challenge with this method of quality control is that Guide 17065 is not required, nor has it necessarily been vetted regarding its appropriateness for all products and procedures. Since there is no mandated quality control measure for all ERs, it can be difficult to establish equivalency between ERs.

The most complicated technical challenge at this time is the incorporation of new components and systems used to provide resistance to seismic forces. While this draws much focus in the development of ACs, non-structural components may also have challenges of equal importance; the SEAOC ER Committee encourages all interested parties to participate in the AC development process. Below is a list of technical challenges present in the creation and implementation of AC.

1. **AC Source**: The issuer of an ER may have their own AC, but in some cases the issuer will use an AC developed by another ER Provider to perform a product evaluation. This scenario may present a "higher bar" for the new product using the non-specific AC. Also, ER Providers, such as ICC-ES and IAPMO UES, expend financial and technical effort to create ACs, and typically retain copyright ownership, as cited above. This can create technical difficulties in the acquisition and motivations for use of certain AC. Additionally, developers of ACs maintain that only they are qualified to interpret the technical language in ACs. SEAOC supports the notion that single, universal AC should be established to evaluate ER suitability for similar products.

2. **Determining Seismic Performance Equivalency**: When a new seismic-force-resisting system requires the formulation of an AC document, difficulty arises in determining system coefficients. There are several approaches, standards and existing ACs that serve as references in establishing seismic performance characteristics or assessing equivalency. These include FEMA P-695 and P-795, ASCE/SEI 41, and ACs for pre-fabricated lateral load resisting walls. While some methods may be outdated, and there may be debate on the best approach, they are summarized below for reference.

   Recently, for some seismic force-resisting products, ICC-ES added an evaluation procedure to determine equivalency to code prescribed products. In 2007, an ICC-ES Ad Hoc Task Group of various industry experts and representatives developed an equivalency approach to determine the equivalency of prefabricated shear panels evaluated to AC130 – Prefabricated Wood Shear Panels, or AC322 – Prefabricated, Cold-formed, Steel Lateral-Force-Resisting Vertical Assemblies, to code recognized site-built wood framed shear walls. ICC-ES then inserted this methodology as a requirement into AC130 and AC322. Then ICC-ES made the decision to add this methodology to several of its other existing AC such as AC04 – Sandwich Panels, AC230 – Power- driven Pins for Shear Wall Assemblies with Cold-form Steel Framing and Wood Structural Panels, and AC269 – Racking Shear Evaluation of Proprietary Sheathing Materials Attached to Light-Frame Walls with Proprietary Fasteners. The equivalency procedure developed by the Ad Hoc Task Group should be evaluated carefully prior to using it beyond the Task Group's original scope of AC130 and AC322, as different equivalency benchmarks may be deemed necessary.

   a. Products with ERs using AC130 and AC322 are intended for complete prefabricated shear panel assemblies, versus individual shear panel components.

   b. FEMA P695 is a probabilistic approach that utilizes analytical collapse assessment to determine seismic resistance factors for seismic force resisting systems (SFRS). System factors and coefficients are ultimately justified through nonlinear analysis of archetype models subjected to many seismic records. The SFRS must have clearly defined design requirements identifying construction materials, components, and system configuration, as well as any limits to system application. Testing results on the material, component, and system levels are used to define strength, stiffness, and ductility properties, as well as serve as a basis for inelastic behavior in the nonlinear analysis. Structural system archetypes are then developed based on anticipated representative applications. Nonlinear static and dynamic analyses are performed, with the static results determining the overstrength factor and trial R coefficient. The R value is validated through nonlinear dynamic analysis based on an acceptable collapse margin ratio, which is determined by the median collapse point through incremental dynamic analysis (IDA) and suites of MCE ground motions. This methodology requires an independent peer review throughout the process.

c. FEMA P795 seeks to provide a simplified application of the FEMA P695 methodology, adapted to evaluate proposed components by demonstrating equivalence with components in a reference seismic force resisting system (SFRS), which must be a code-recognized SFRS with sufficient design criteria and test data. The component methodology allows for the "mixing" of proposed components with reference components in the reference SFRS. Cyclic and monotonic tests are required to establish equivalency of the proposed and reference components by evaluating critical performance parameters identified in the methodology. Equivalency is demonstrated through probabilistic-based acceptance criteria based on a comparison of median deformation capacity of the components.

d. Current ICC-ES AC documents refer to the lack of referenced standards in the IBC, or other ordinances or regulations, for establishing code compliance for lateral systems not included in the code. As such, a number of AC (including AC04, AC130, AC230, AC269, and AC322) cite IBC Section 104.11 (AMC) to establish seismic coefficients and factors by demonstrating equivalence or compatibility with code-recognized light-frame walls with respect to ductility, drift, and overstrength.

e. ASCE/SEI 41 provides component demand modification factors (m factors) for linear analysis and deformation capacities for nonlinear analysis of existing buildings. These factors and capacities are provided for many different building components typical in existing buildings. The standard also outlines a method for determining these factors and capacities from laboratory test results; however, these have yet to be quantitatively benchmarked to model building codes.

3. **Loading Protocol**: For decades, this has been a challenging and controversial topic, as selecting suites of earthquake ground motions is an issue at the forefront of the development of AC. The research work and findings of the ATC62 document yield modest recommendations for improvement. Each loading protocol has unique characteristics that lend it to specific applications based on construction materials, geography, in-situ loading, or the engineer's opinion, among others. For instance, ASTM E2126 lists three approved loading protocols for determining performance of shear resistance of walls: Sequential Phased Displacement (SPD), ISO 16670 Protocol, and CUREE Basic Loading Protocol [Krawinkler, 2009]. This lack of consensus in defining the "best" loading protocol is typical throughout all loading protocols and materials. For further information on the subject, Krawinkler provides a good summary and discussion [Krawinkler, 2009]. In establishing equivalency, the selection of loading protocol should be the same as that of the benchmark or reference system or component to which equivalency is being demonstrated. Similarly, the reference system should have similar cyclic behavior characteristics (i.e. the comparison of an SCBF and a BRB may provide misleading equivalency measurements).

Because of the life safety implications of seismic force resisting systems and the wide disparity of views on testing protocols amongst standards, researchers, and practitioners, as well as the relative benefits of specific protocols for certain applications, an appropriate testing protocol should be selected through a public, transparent process on a case-by-case basis. The distinction between cyclic and monotonic load testing is pertinent and should be carefully considered. For example, monotonic test results can significantly overestimate strength and ductility of components compared to cyclic testing. However, monotonic test results may provide valuable information about the performance of the component as it is loaded to collapse. Additionally, the anticipated performance level of the component will guide the selection process. For instance, cyclic testing may not be warranted for components that are expected to remain essentially elastic.

4. **Boundary and Loading Conditions**: Defining boundary conditions is vital to creating an AC that is representative of the actual behavior of the system or component. Some of the standards define specific requirements for boundary conditions, such as ASTM E2126. Boundary conditions of a component may be tested or modeled explicitly, or included in the design through rational analysis or statics. Similarly, loading conditions should be identified and included in the testing procedures when determining performance. Concurrent seismic and gravity loading should be considered, as vertical loads may improve or diminish the performance of the system or component. Other considerations, such as bi-directional loading and slackness or tightness of hold-downs for a shear wall, for example, should also be evaluated. The methodology behind the definition of the component or system, including boundary and loading conditions, needs to be transparent to facilitate appropriate designs. That is to say, AC (and, subsequently, ERs) should include guidance on limitations or methodology assumptions that pertain to boundary conditions.

5. **Component versus System**: Development of AC for seismic force resisting systems (SFRS) is generally more straightforward than it is for components within a system. For instance, testing an entire system captures the behavior of each component in the context of overall system performance, whereas testing an individual component may not be indicative of the performance of the system as a whole. In cases where ACs are developed for components, appropriate measures should be included in the testing or analysis to consider compatibility with other components and overall system behavior through applicable boundary conditions or other techniques. For more information on these challenges, see [FEMA P795] and [Line et al, 2008].

6. **Laboratory Testing vs. Numerical Analysis**: While laboratory testing is widely accepted as the best measure of evaluating system and component performance, evaluation of an entire LFRS may be impractical in a laboratory setting. Numerical analysis may be substituted as an evaluation method in some cases. For example, this is clearly a practical solution for the design of tall buildings that seeks to employ alternate structural systems. However, some testing is necessary to determine the strength and stiffness parameters of new systems prior to or alongside numerical analysis.

   Further, the level of evaluation should be commensurate with the anticipated performance level, degree of non-linearity, and confidence of the system; all are subjective parameters. For an undefined system that is designed to remain essentially elastic, it may be acceptable to utilize an R of, say, 1.0 or 1.5 without performing substantial numerical or laboratory testing. Such an approach would require, however, a substantial amount of engineering judgment and understanding of the entire structure to ensure that all components and attachments are compatible, with no undesired strength hierarchies developed from using a low R value of 1.0 or 1.5. A system that is anticipated to undergo significant yielding and inelastic behavior would obviously require more evaluation and performance verification, possibly through physical testing or a hybrid of component testing and numerical assessment. The suitability of one method over the other should be determined on a case-by-case basis. For numerical, specifically nonlinear, analysis, refer to Deierlein et al [NEHRP, 2010] for analysis techniques.

7. **Code and Standard Update Cycles**: Code and standard updates vary greatly. The current model code (IBC) is made up of material and design standards (e.g., AISC, ACI, AISI, NDS, etc.), which in turn reference testing/material standards (e.g., ASTM) that may be several generations behind the most current edition. The use of outdated testing and material standards creates a lag in technology and can hinder the industry from meeting the code intent and providing the high quality, cost effective products the industry now demands.

8. Common sense allows product manufacturers to avoid repeating their tests using antiquated test standards. However, if the manufacturer uses the most current ASTM test standards, and they are less restrictive than the code referenced standard, this is procedurally unacceptable unless it can be shown that code intent can be met.

   Thus, developers and reviewers of AC (and ERs) should use their professional judgment in selecting test standards that may cause less conservative results, and have those vetted through public bodies and recognized committees. The fact that AC and ERs are provided as support for approval via an AMC, use of premature standards may be a prudent option.

Evaluation Report Development Process

ERs are issued for products after the requirements of an AC are deemed to be met through proper testing, documentation and inspection. Test reports by recognized laboratories are submitted to the ER Provider for review. Quality control documentation is also required to demonstrate that product manufacturing meets the quality control requirements of the code and the ER Provider (e.g., ISO/IEC17065, AC). Initial and ongoing unannounced regular inspections of manufacturing plants are typically required by an ER Provider. Following the approval of these items, the ER Provider will issue an ER, which will typically be publicly available on the ER Provider's website. ERs will be reviewed and reissued in the following years per the ER Provider's re-examination process.

Evaluation Reports typically contain the following information:

1. Report Holder: The applicant for the ER
2. Evaluation Subject: The specific product covered by the report
3. Evaluation Scope: The codes and standards that were used to evaluate the product
4. Properties Evaluated: The properties that were evaluated and whether the product can be used for structural purposes
5. Uses: The scope of the ER and relates the product evaluated to code provisions
6. Description: A general description of the product and its features
7. Design: Design and detailing requirements and limitations
8. Installation: Requirements to help inspectors ensure that the product is installed per code and AC requirements
9. Conditions of Use: Statement that the product complies with or is a suitable alternative to the code
10. Evidence Submitted: Data and evaluation criteria such as the Acceptance Criteria (ACs) that were used in evaluating the product
11. Identification: Information that is used to identify the product in the field through labeling or other means

The main challenge in the procedure for ER creation is the fact that there are no opportunities for public comment prior to the issuance of an ER. While the public has opportunities to comment on AC creation, ERs are created according to the procedure listed above, with no further opportunity for public comment or vetting prior to issuance. This leaves a critical stage of the ER process entirely out of the hands of selected industry experts. Once the ER is posted in the public domain, there are typically complaint procedures to address any further comments. Therein lays the concern of sole source providers and the importance of participation by groups such as SEAOC and NCSEA.

The main technical challenge for the proper creation of ERs is ensuring that the appropriate publicly vetted AC has been followed for the testing of the product in question. While ISO/IEC17065 endeavors to ensure that testing procedures are followed by the testing facilities, it does not check to see that the guidelines of any particular AC are necessarily being followed. And since building officials and engineers rarely reference the AC that is the basis for an ER being utilized, there are often no external checks to ensure that the findings listed in an ER are those that were sought after in the associated AC document.

The main technical challenge for the proper use of ERs is the lack of understanding on the part of the user (engineer, building official, etc.). Because the process of ER creation is not well understood by the industry, the findings in an ER are often taken for granted, not well-reviewed, and sometimes simply assumed to comply with code provisions. Not understanding what the product was tested for, and hence what applications it is appropriate for, can lead to the incorrect use of products or the premature/inappropriate approval of a product by an AHJ.

The use of ERs as an AMC submission to an AHJ can create its own technical challenges as well. For example, when a building official receives an AMC submittal (either in the form of an ER or in another accepted form), the AHJ can charge additional fees to offset the cost of reviewing the submission, and may engage a third party to afford a fair and thorough review. The cost of this is then borne by the proponent of the building permit application, which can place considerable burdens on some manufacturers and potentially sideline products that are code compliant.

Additionally, some ER Providers continue to post ERs that have been evaluated to outdated ACs and earlier editions of the building codes. This has confused users regarding the appropriateness of an ER evaluated to an older code, and may lead to the use of a product that does not comply with the current adopted code.

AC and ER Development and Implementation Recommendations

The SEAOC ER Committee received valuable input from a variety of industry representatives including building officials, ER Providers, and manufacturers. From the findings listed in this article, the Committee makes the following recommendations to different industry members in hopes of improving the process of AC and ER development and implementation.

General Recommendations for All Industry Members

1. Use the term "ER listed" or "product ER listed" rather than "product approval". A product approval by an evaluation service represents the professional position of the ER Provider. The final approval that the product "meets code" is provided by the building official based on his/her professional opinion after consideration of items provided to him/her in accordance with Section 104.11.

2. It is in the best interest of all industry members to carefully review and fully understand ERs before using, or, in the case of a building official, approving products. Engineers and building officials should make sure they understand and agree with the design and configuration limitations; installers of the products should be aware of the means and methods requirements; and, lastly, inspectors should be knowledgeable regarding proper use, limitations, installation requirements, and other important items to observe or verify. The interested parties are encouraged to contact ER providers with any questions or concerns that arise.

Recommendations for Structural Engineers Associations and Code and Standard Committees

1. Continue the SEAOC ER Committee as a venue for discussion of wider issues outside of the logistical challenges of discussing specific products, AC, etc. SEAOC and NCSEA have been more proactive in providing resources, but still more resources are required to have a long-term and effective influence on the process. Other organizations representing non-structural products and systems should become more involved.

2. Distribute this document and present its information at relevant conventions and industry gatherings, as a way to continually disseminate the findings of this committee, educate the industry about AC and ERs, and improve our understanding and use of AC and ERs.

3. Improve dialogue and coordination between ER Providers and code provision writers, to ensure references to documents are accurately maintained. The modification of an AC can unintentionally result in a code provision reference to a document that has been superseded or does not exist. This committee suggests that code writers provide a stand-alone log of code references that may be of interest to ER Providers, and ER Providers reach out to the code writers when they foresee changes to their documents.

4. Change Section 104.11 as follows:
   a. **104.11 Definitions**: Introduce AC and ERs and expand the various tracks and requirements to obtaining approval for an AMC. Include an ER definition, specific requirements regarding the development of AC and ERs, and public vetting and quality assurance requirements, which could include adoption of ISO/IEC17065. This could be done while maintaining the current individual engineer submittal and individual building official review AMC process on a project-by-project basis.

   b. **104.11.1 Research Reports**: Provide minimum requirements for research to be classified as a source for an ER. Solicit public comments and post these comments, as well as the ER Providers' response, during AC development. The balance between expediting the review to facilitate timely product use in the market place and performing a comprehensive review will be a case-by-case matter. To improve the quality and achieve a more consistent level of review and timeliness, this committee recommends a code change requiring some form of process or accreditation for an ER to be classified as such. Additionally, to prevent ERs evaluated to outdated codes from being used, it is recommended that the code use provisions such as "An approved ER may be used to assist the building official to ascertain the product's code compliance, but that the ER must be evaluated in accordance with this code."

   c. **104.11.2 Tests**: Provide minimum requirements for tests to be classified as a source for an ER. Perhaps identify ISO/IEC 17025 or IAS accreditation as a minimum. This section could be updated to reflect the AC, ER, standard, and code development processes. Perhaps it could even afford direction on expiration of testing standards.

5. The length of an updating cycle for these documents should be carefully considered, balancing improvements to products, timeliness, and costs to industry. Some criteria may affect hundreds of products that might require extensive testing after an AC has been revised.

Recommendations for Architects, Designers and Structural Engineers

1. Architects, designers, and structural engineers should take great care in reading and understanding ERs and in ensuring they fully understand the limitations and design assumptions forming the recommendations therein.

2. Permit documents should specifically identify products for which an alternate means of compliance are requested. The building official's prior acceptance of these products should be obtained.

Recommendations for Design Reviewers, Building Officials, Authorities Having Jurisdiction, and Inspectors

1. The AHJ should regularly request background information to assist in their review of ERs, particularly if they are to include the ER in code provisions, unless - based on the information provided and their knowledge of the AC and ER Process - they can confidently determine that the product meets code intent.

2. The building official is the final party responsible in determining if a product meets the code intent. As such, he/she should request the technical data used to develop an ER if he/she feels this is necessary to determine code compliance under the AMC provision.

3. Products are often approved for use in applications that are not explicitly contained in an ER for that product. Building officials should develop a policy for how to effectively evaluate the use of products in applications not covered by their ER.

Conclusion

While the issues discussed herein are challenging, they are not unlike other issues that building officials face on a regular basis. As with many complex issues, the specifics of each case must be studied carefully. A robust ER process can be of great benefit to the built environment and to public safety in this age of increasing code and product complexity, and declining building department resources. Certainly, there is no perfect process or document and the building industry must not get lost in the minutia of the many provisions of the building code and lose sight of its primary role, which is to "safeguard the public health, safety, and general welfare." Often, the bureaucracy of our processes, financial interests and market competition, as well as personal politics, can slow down the improvement of our codes and procedures. This can result in other consequences, such as increased inconsistency and decreased safety of the built environment, or creation of less economical solutions. However, the solution is certainly not to rush to use new products or systems without thorough evaluations of their code compliance by qualified individuals. Hence, we must strive to lead the industry to something in between, balancing the goals of promoting ingenuity and ensuring safety in our products and designs.

References

American Society of Testing and Materials, "*Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings*," ASTM E2126.

Applied Technology Council, 2005. *Advanced Seismic Analysis Methods*, ATC 62, Redwood City, California.

California Seismic Safety Commission, 1995, "Northridge Earthquake, Turning Loss to Gain," CSSC Report No. 95-01, Sacramento, California.

Deierlein, G., Reinhorn A. and Willford, M., 2010, "Nonlinear Structural Analysis for Seismic Design, A Guide for Practicing Engineers," NEHRP Seismic Design Technical Brief No. 4. NIST GCR 10-917-5.

FEMA P695, 2009. *Quantification of Building Seismic Performance Factors*, Federal Emergency Management Agency, Washington, D.C.

FEMA P795, 2011. *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology,* Federal Emergency Management Agency, Washington, D.C.

Hamburger, R., 2010, "Code Development, NCSEA Explains the Code Development Processes and Standards," Structural Connection.

ISO/IEC 17007, 2009. "*Guidance for Drafting Normative Documents Suitable for Use for Conformity Assessment*," International Standards Organization.

ISO/IEC 17065, 2012. "*Requirements for Bodies Certifying Products, Processes and Services*," International Standards Organization.

Krawinkler, H., 2009, "Loading histories for cyclic tests in support of performance assessment of structural components," 3rd International Conference on Advances in Experimental Structural Engineering, San Francisco, California.

Line, P., Waltz, N., and Skaggs, T., 2008. "Seismic equivalence parameters for engineered wood frame wood structural panel shear walls," Wood Design Focus, 18 (2).

Moore, Mark, Chair, SEAOC Seismic Evaluation Reports Committee, 2011. *"An Evaluation of Current Practices*

Related to the Development and Implementation of Acceptance Criteria and Product Evaluation Reports," SEAOC 2011 Convention Proceedings, Sacramento, California.

PEER, 2010, *Guidelines for Performance-Based Seismic Design of Tall Buildings*, PEER Report 2010/05, TBI Guidelines Working Group. Berkeley, California: Pacific Earthquake Engineering Research Center, University of California.

Keywords

Acceptance Criteria (AC)
Alternate Means of Compliance (AMC)
American National Standards Institute (ANSI)
Applied Technology Council (ATC) Authority
Having Jurisdiction (AHJ)
ER Producer/Provider (Organizations which develop ACs and issue ERs, such as ICC-ES, IAPMO UES, NTA, etc.)
Evaluation Criteria (same as AC),
Evaluation Service Report (ER)
Federal Emergency Management Agency (FEMA)
"Industry members" denotes all involved in the construction industry (Contractor, Designer, ER Provider, Building Official, etc.)
International Accreditation Service (IAS)
International Association of Plumbing and Mechanical Officials Uniform Evaluation Service (IAPMO UES)
International Building Code (IBC)
International Code Council (ICC)
International Code Council Evaluation Services (ICC-ES)
International Electrotechnical Commission (IEC)
International Organization for Standardization (ISO) NTA Inc. (NTA)
Pacific Earthquake Engineering Research (PEER)
product certification agency (PCA)
product certification provider (PC Provider)
product evaluation report (ER)
seismic force-resisting system (SFRS)
Structural Engineers Association of California (SEAOC)
Uniform Building Code (UBC)

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "An Evaluation of Current Practices Related to the Development and Implementation of Acceptance Criteria and Product Evaluation Reports," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Earthquake Design Documentation

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
11.2 12.8 13.3.2 13.2.7	107.2	2018 IBC 1603.1

Background

Inadequate design documentation can compromise safety and performance. It can also 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, should have a specific purpose for an intended audience. 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 and construction observations.

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 (SEAOC Seismology Committee 1999, section IB-7.2). Calculations are also required for most buildings by permitting authorities and by building codes.

Where the IBC and its standards refer to "submittal documents", they include construction documents, statements of special inspections, geotechnical reports, deferred submittals, and other data. "Construction document," 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 for use following earthquakes, and encourages engineers to convey to their clients the value of maintaining these drawings. Maintenance of original design documents also facilitates repairs, alterations, and additions through the life of the building. (On-site maintenance of documents is generally more convenient for immediate post-earthquake use. Off-site or cloud maintenance might be preferable in the event of severe structural damage.)

In California, the statute of limitations for building defects is 10 years. The SEAOC Seismology Committee recommends that the structural engineer of record maintain design drawings for a minimum of 12 years, because a lawsuit can be filed right before the statute of limitations expires, and it can be over a year before the engineer becomes aware of the lawsuit.

With regard to calculations, there is no consensus on the best approach. Some firms keep calculations for the same amount of time as drawings. Other firms do not keep calculations after a project is completed. The reasoning behind this is as follows: if there is a defect litigation issue, the plaintiff will have volumes of calculations to criticize. The plaintiff can then generate a list of errors that appears very significant, even if the original calculations followed the standard of care.

The Seismology Committee recommends that structural engineers consult with their attorneys and insurance brokers before deciding on a document retention policy.

Structural Drawings

ASCE 7-16 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, 2015 IBC section 107.2.1 requires 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 ensure that critical load path elements are adequately detailed. "All structures shall be provided with a continuous load path …and shall have a complete lateral force-resisting system with adequate strength" (ASCE 7-16 section 1.4).

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 303-16, 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 2015 IBC section 1603.1.5 lists 10 items (with 13 total design parameters) that must be shown on construction documents.

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 7-16 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 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.

The following information related to earthquake design are required to be shown on structural drawings:

- Material properties used, including soil properties (2015 IBC)
- Seismicity parameters (2015 IBC)
- Parameters needed to establish the seismic response coefficient (2015 IBC)
- The basic seismic force-resisting system(s), as tabulated in ASCE 7-16 Table 12.2-1 (2015 IBC)

It is the position of the Seismology Committee that, in general, the following basis for the earthquake design should also be stated on structural drawings:

- The date and edition of the governing building code
- Data needed for design by others (for example, seismic relative displacements per ASCE 7-16 section 13.3.2 needed for the design of nonstructural components and their attachments).
- Lower-bound and expected properties of existing materials (for a seismic retrofit)

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 7-16, and in model building codes, their production is implied and necessary for validation of compliance with the code and its referenced standards. Model code provisions that call for structural calculations include 2015 IBC section 107.1 ("other data"). 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 sufficiently clear to approve minor field revisions to the original design without reanalysis. The calculations should include a table of contents, and have all pages numbered. 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. 2016 CBC, section 1603A.3 also provides some guidance on best practices for engineering calculations, although this section is only enforced by certain enforcement agencies.

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:

- Images 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 as intended.
- 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.

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. The latest Tall Building Initiative guidelines also encourage the use of graphs to summarize large amounts of data (PEER TBI Version 2, Section 7.2).

Emerging Practices

New technologies are changing the nature of design documentation (Amor, 2013). 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.

References

AISC (2016). ANSI/AISC 303-16, *AISC Code of standard practice for steel buildings and bridges*, American Institute of Steel Construction, Chicago, IL, June 15.

Amor, T. (2013). "Challenging Traditional Design with BIM." *STRUCTURE Magazine*, January 2013, pp. 36-37.

ASCE (American Society of Civil Engineers) (2017). ASCE 7-16, *Minimum design loads for buildings and other structures,* American Society of Civil Engineers, Reston, VA.

California Building Standards Commission (2016). *2016 California building code* (California Code of Regulations, Title 24), California Building Standards Commission, Sacramento, CA.

ICC (International Code Council) (2015). *2015 International building code,* International Code Council, Falls Church, VA.

PEER Tall Buildings Initiative (2017). *Guidelines for Performance-Based Seismic Design of Tall Buildings*, Pacific Earthquake Engineering Research Center, Berkeley, CA.

SEAOC Professional Practice Committee (1999). *Recommended guidelines for the practice of structural engineering in California, fifth edition*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1996). *Recommended lateral force requirements and commentary, sixth edition*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). *Recommended lateral force requirements and commentary, seventh edition*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (2009). *SEAOC Blue Book: Seismic Design Recommendations, 2009 Compilation,* Structural Engineers Association of California, Sacramento, CA.

Keywords

calculations
computer-generated calculations
design documentation
drawings
engineer of record
specifications

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Earthquake Design Documentation," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2016 CBC reference section(s)	Other standard reference section(s)
11.2 12.2.1		1997 UBC Table 16-N

The structural systems most commonly used to resist earthquakes are listed in ASCE 7-16 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. That is, the SFRS is the "basic" system contemplated by ASCE 7. (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 7-16 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.

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. ASCE 7-16 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 7-16 in Table 12.2-1 are:

1. A response modification coefficient, R
2. An overstrength factor, Ω_o
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 ρ and Ie that account respectively for structural redundancy and the occupancy-defined seismic importance factors.

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 7-16, though design requirements have changed substantially based on updated records 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 7-16 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” (SEAOC Seismology Committee 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” (SEAOC Seismology Committee 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” (SEAOC Seismology Committee 1975, p. 15-C).

Still, the Committee acknowledged that K was largely a “judgment factor” (SEAOC Seismology Committee 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 7-16, 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 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 7-16, 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 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.

This 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 7-16 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 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.

At the time of this writing, there is a BSSC Ballot Proposal to effectively increase all drift requirements by R/Cd, so the ratios above would all change to 1.0. The Seismology Committee supports this change.

Height Limits

The height limits imposed by ASCE 7-16 Table 12.2-1 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 high-rise 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-ft 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 is 160 ft, a value established by the first Blue Book to supplement an earlier Los Angeles code requirement for buildings taller than 13 stories (SEAOC Seismology Committee 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 ft, it was 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 high-rises.

Thus the 160-ft 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" (SEAOC Seismology Committee 1960, p. 43). When ATC 3-06 revisited the system table, it retained the 160-ft 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 ft 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" (SEAOC Seismology Committee 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 ft. 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-ft 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.

System Attributes

Both ASCE 7 and the UBC group the seismic force-resisting systems into broad categories. Generic definitions for each category are given in ASCE 7-16 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 (SEAOC Seismology Committee 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" (SEAOC Seismology Committee 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 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 (SEAOC Seismology Committee 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" (SEAOC Seismology Committee 1960, p. 22). ASCE 7-16 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.

Regardless of how the code is interpreted, the Seismology Committee recommends that every load-bearing element be considered in seismic analysis. Failure to do so can be unconservative. Examples of this include upper level light-frame partition walls creating a soft-story mechanism below, and elevator/stair cores creating undesired load paths and torsional response due to their stiffness.

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 (SEAOC Seismology Committee 1967, p. 46).

Historically, gravity frames acted as moment frames with partial fixity. This likely contributed to their contribution to lateral stability.

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 (SEAOC Seismology Committee 1990, p. 12-C). Past Blue Books listed such systems specifically as "Braced Frames Where Bracing Carries Gravity Load" (SEAOC Seismology Committee 1990, Table 1-G), a distinction that persists in the 1997 UBC and the 1999 Blue Book (SEAOC Seismology Committee 1999, C104.6.2). Current codes, however, 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 7-16, 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

SFRS type[1]	ASCE 7-16			1997 UBC[1]		
	Bearing Wall R	Building Frame R	R_{BW} / R_{BF}	Bearing Wall R	Building Frame R	R_{BW} / R_{BF}
Ordinary steel concentrically braced frame	NA	3.25	NA	4.4	5.6	0.79
Special reinforced concrete shear walls	5.0	6.0	0.83	4.5	5.5	0.82
Special reinforced masonry shear walls	5.0	5.5	0.91	4.5	5.5	0.82
Light-framed walls with rated wood structural panels	6.5	7	0.93	5.5	6.5	0.85

[1] SFRS types per ASCE 7-16 Table 12.2-1. 1997 UBC SFRS type descriptions vary slightly from the ASCE 7 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" (SEAOC Seismology Committee 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 7-16 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 (SEAOC Seismology Committee 1967, p. 46). While neither ASCE 7 nor the 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 (SEAOC Seismology Committee 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.

New Thinking

The SEAOC Seismology Committee supports the calibration of new systems using the FEMA P-695 methodology. Tall, multi-story wood frame buildings are of particular interest in this regard.

References

ATC (Applied Technology Council) (1978). *Tentative Provisions for the Development of Seismic Provisions of Buildings.* ATC, Redwood City, CA.

ASCE (American Society of Civil Engineers) (2002). ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures.* American Society of Civil Engineers, Reston, VA.

ASCE (American Society of Civil Engineers) (2006). ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures, Including Supplement No. 1.* American Society of Civil Engineers, Reston, VA.

California Building Standards Commission (2001). *2001 California Building Code* (California Code of Regulations, Title 24). California Building Standards Commission, Sacramento, CA.

ICBO (International Conference of Building Officials) (1997). *1997 Uniform Building Code*. ICBO, Whittier, CA.

Berg, Glen (1983). *Seismic Design Codes and Procedures*. Earthquake Engineering Research Institute, Oakland, CA.

BSSC (Building Seismic Safety Council) (2001). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 2—Commentary* (FEMA 369). Building Seismic Safety Council, Washington, D.C.

BSSC (Building Seismic Safety Council) (2004a). *The 2003 NEHRP Recommended Provisions for Regulations for New Buildings and Other Structures, Part 1: Provisions* (FEMA 450). Building Seismic Safety Council, Washington, D.C.

BSSC (Building Seismic Safety Council) (2004b). *The 2003 NEHRP Recommended Provisions for Regulations for New Buildings and Other Structures, Part 2: Commentary* (FEMA 450). Building Seismic Safety Council, Washington, D.C.

Porush, A. and Zacher, E. (1987). "SEAOC, the Blue Book and seismic codes past, present, and future," in *Proceedings: 56th Annual Convention*, San Diego, October 8-10, 1987. Structural Engineers Association of California.

Reis, E. and Bonowitz, D. (2000). *State of the Art Report on Past Earthquake Performance of Moment-Resisting Steel Frame Buildings* (FEMA 355-E), Federal Emergency Management Agency, Washington, DC.

SEAOC Seismology Committee (1959). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1960). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1967). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1975). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1990). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, CA.

Keywords
bearing wall systems
building frame systems
combined systems
deflection amplification factor, C_d.
dual systems
ductility
height limits
overstrength factor, Ω_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

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 2019)

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

SEAOC Seismology Committee (2019). "Seismic Force Resisting Systems," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Development of System Factors

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.2.1 12.12.1 12.8.6		FEMA-368 5.2.2, 5.2.8, 5.4.1, 5.4.6.1 FEMA-369 5.2.7, 5.2.8, 5.4.1 FEMA P-695

Intent of System Factors

Past experience has shown that a structure can be designed for a fraction of elastic seismic design forces while still meeting the basic life safety performance objective. This design philosophy implies that inelastic behavior, and damage to the structure, is 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 linear elastic static analysis (i.e., the equivalent lateral force procedure) is needed for the majority of buildings. The design seismic force is, therefore, reduced from the elastic seismic force by a reduction factor R.

Design seismic forces and their associated elastic deformations are low, and do not represent the response expected during severe earthquake shaking. While deformation-controlled members that are detailed to provide ductility are expected to deform inelastically, other force-controlled members that are designed to remain elastic can experience significantly higher seismic force levels than those predicted by design seismic forces. To account for this effect, ASCE 7 uses a Seismic Force Amplification Factor, Ω_o, to allow the seismic force in these members to be calculated from the design seismic forces. (Ω_o is termed the Structural Overstrength Factor in ASCE 7.) To control drift or to check deformation capacity in deformation-controlled members, a similar approach is adopted in the code that uses a Deflection Amplification Factor to predict the maximum deformations produced by the expected seismic forces. (This factor is defined as C_d in ASCE 7.)

Implementation of System Factors

In 1960, Muto K. et al found that the maximum inelastic displacement of an SDOF system is similar to that obtained for elastic systems having the same initial period and damping. Later, Veletos and Newmark [Veletos and Newmark, 1960] reached similar conclusions. These findings are known as the equal displacement rule.

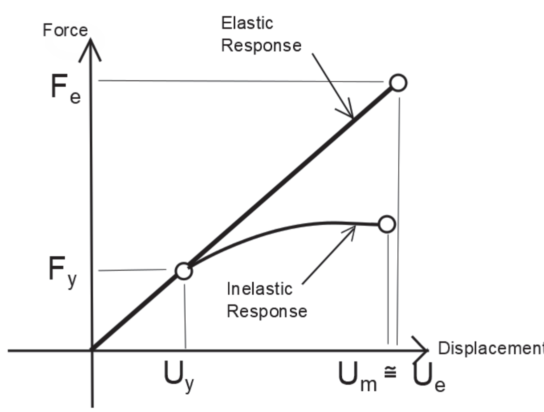


Figure 1. Equal Displacement Rule.

Figure 2, which shows a pushover curve of a structure, can be used to explain the R-factor seismic design procedure. This pushover curve can be established from either testing or a pushover analysis. The structure responds elastically until the first significant hinge forms in the structure, 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.

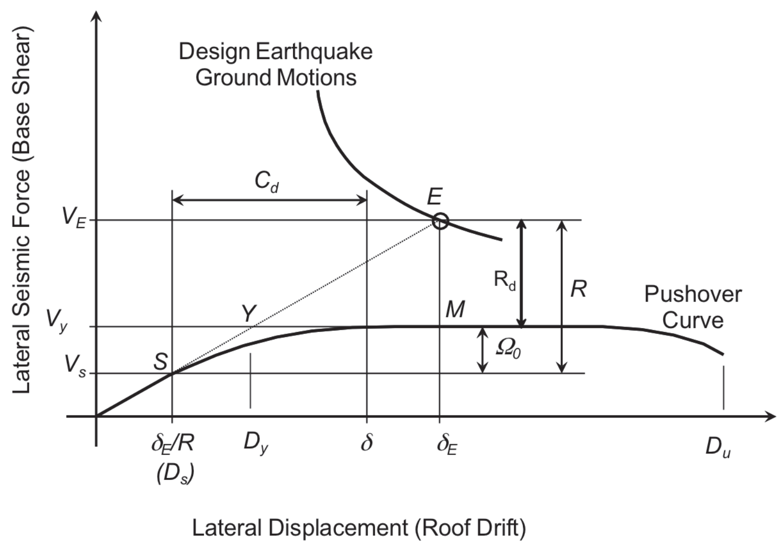


Figure 2. Inelastic force-deformation curve

Based on the fundamental period of the structure, the 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, Ω_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 inelastic drift, which cannot exceed the limit allowed by the code.

Historical Development of System Factors

In order to understand 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 a more explicit form.

In an implicit form, the general base shear formula of V_w =[a constant factor]×W for working stress design had been in use for many decades. The form of $V_w = (ZKC)W$ was used in the 1959 Blue Book recommendations, where

Z = Seismic Zone Coefficient,
K =Horizontal Factor (related to structural building type),

C = Period dependent coefficient.

This expression was later modified to include Importance and Site factors to become

$$V_w = (ZIKCS)W \quad \text{(Eq. 2)}$$

Where

S= Soil Profile Coefficient
I= Occupancy Importance Factor

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. (2). The K factor and the remaining terms ($ZICS$) individually did not have any physical meaning. However, the product, which was calibrated from experience and observed building performance in past earthquakes, did provide a reasonable estimate of 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 adopted:

$$V_w = \frac{V_e}{R_w} \quad \text{(Eq. 3)}$$

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) used the form in Eq. (1) to prescribe the design seismic forces, where $V_e = (C_v I / T)W$. In ASCE 7-02, V_e is expressed as $(S_{D1}I/T) \leq S_{DS}IW$

The system factor K is the forerunner of R. Originally, the basis for the K factor was the definition of four basic seismic 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)

$$R_w \approx \frac{8}{K}; \qquad R \approx \frac{R_w}{1.4} \quad \text{(Eq. 4)}$$

where the 1.4 reflects the conversion from working stress design to strength design. 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 seismic force-resisting system.
5. Multiplicity of lines on 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 systems

- 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
- Member sizes often governed by drift limits

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
- Drifts controlled by braces or walls

Building Frame System

- Performance varies depending on material and configuration
- Level of inelastic response depends on type of seismic force resisting system employed
- Damage to the lateral system should not lead to failure of vertical system

Bearing Wall and Bearing Braced Frame System

- Performance varies depending on material and configuration
- Lower level of inelastic response capability
- Seismic force resisting system failure could lead to vertical system failure.

Deflection amplification factor: Deformation checks of story drifts in the force-based design procedure have been performed in either of two formats in the US: serviceability and ultimate limit state. 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 was generally accepted (Freeman 1977). In the 1985 UBC, the story drift limit for the design seismic forces V_w in Eq. (2) is 0.005K; 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 3. As both the design base shear and the drift limit contain the K factor, the initial 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:

$$0.005K = 0.005\left(\frac{8}{R_w}\right) = \frac{0.04}{R_w} \qquad \text{(Eq. 5)}$$

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 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_u, 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) followed the same approach, except that C_d was replaced by 0.7R.

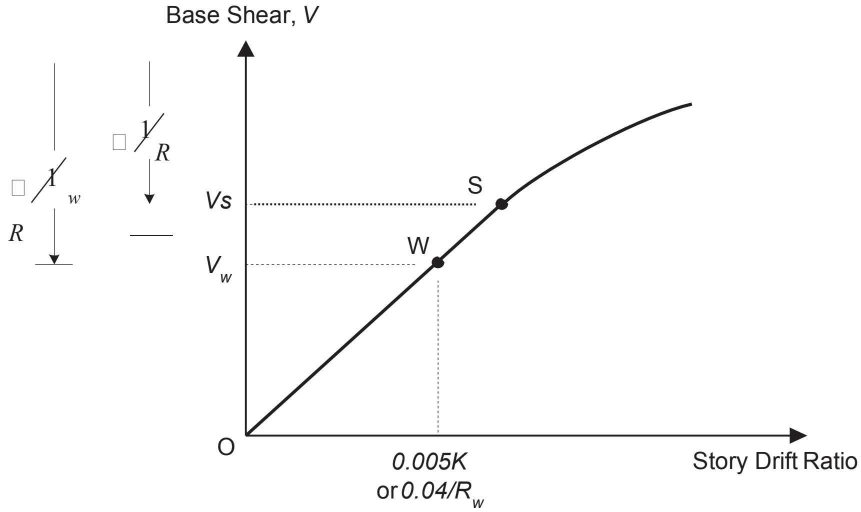


Figure 3. Story Drift Requirements

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, Ω_o, was used for the same purpose. The same factor was later adopted in the 2000 NEHRP (and subsequently ASCE 7).

Theoretical Relationship Between Elastic and Inelastic System Response

When a structural system has been designed for the design seismic force 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 2, by idealizing the actual response envelope by an elastic-perfectly-plastic response, the system ductility factor, μ_s, can be defined as D_u/D_y. 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 system, e.g., formation of a plastic hinge, occurs. The reserve of strength beyond V_s is defined as structural overstrength and is measured by the system overstrength factor Ω_o ($=V_y / V_s$). Therefore (Uang 1991a),

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

The C_d factor is

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

An alternate interpretation of Eq. (6) can be made with the aid of Figure 4. 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_s 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_s to the maximum strength V_y at point M is represented by the system overstrength factor Ω_o. In addition, the 5 percent damped elastic base shear demand (V_E) for the design earthquake increases in damping, thus reducing demand 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).

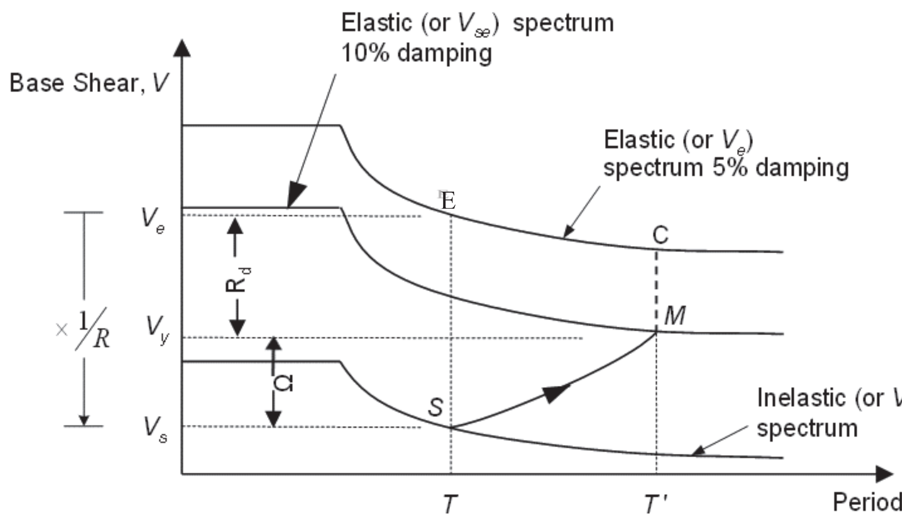


Figure 4. Seismic resistance versus demand

Other forms for the R factor have also been proposed. For example, Whittaker et al. (1999) suggested that a redundancy factor, R_*, 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.

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

Several codes (Japan, New Zealand, and Eurocode) consider both the serviceability state (corresponding to a frequent, moderate earthquake ground motion) as well as the ultimate limit state (corresponding to a rare, severe earthquake ground motion) (Standards New Zealand 1992)(CEN 2004)(IAEE 1992).
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 has been 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. It is known that the spectral shape also depends 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_d, and Ω_o) are intended for the ultimate limit state requirements. The following issues need to be considered in improving the reliability of these system factors.

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 Ω_o for capacity design. The 1997 Blue Book assumes $\Omega_o = 1.1R_o$.

Some limited research has been conducted to evaluate the amount of system overstrength for certain types of seismic force-resisting systems, but a systematic study that includes all building types of different heights, width, seismic zones, non-seismic loadings (gravity and wind), etc. has 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 seismicity; the system overstrength in regions of low seismicity can be several times larger than that in regions of high seismicity because the effects of gravity loads are 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. **Error! Bookmark not defined.** 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 $\Omega_o = \Omega_D\Omega_M\Omega_S$ (see Fig. 5). Ω_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, Ω_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 Ω_D value varies between 2 and 3. Second, Ω_D is highly dependent on the region's seismicity.

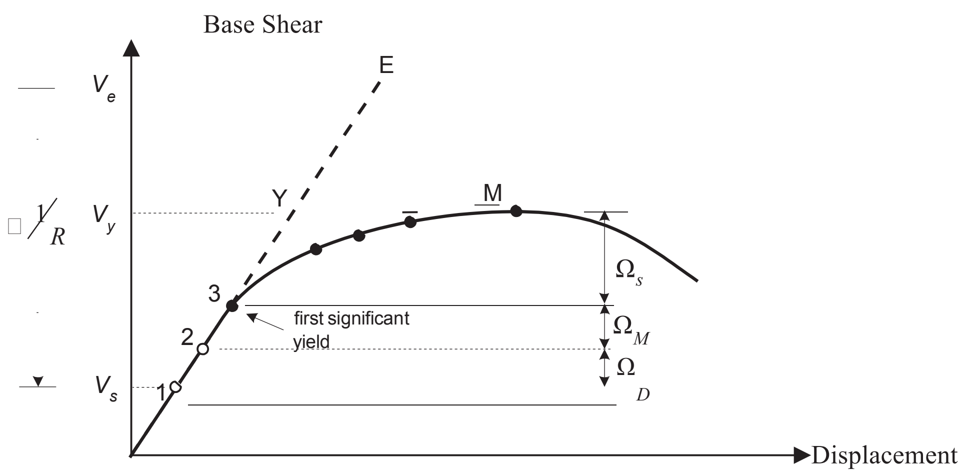


Figure 5. Components of System Overstrength

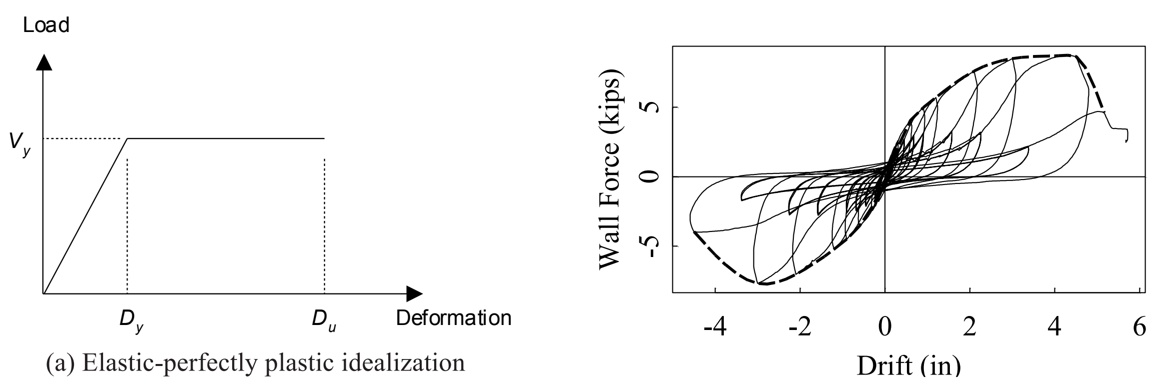
Ω_M represents material overstrength. This portion of the system overstrength, i.e., the ratio in lateral strength between Points 2 and 3 in Figure 5 reflects the difference between the nominal and actual material strengths, including strain rate effects. Reinforced masonry, concrete, and steel provisions 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, per AISC 341. Ω_S represents the system overstrength beyond the first significant yield point (Point 2 in the figure). It depends 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 seismic force resistance. See the NEHRP Provisions for further discussion on these components.

For residential construction, recent tests on a wood frame model house show 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

$$\mu_s = \frac{D_u}{D_y} \qquad \text{(Eq. 8)}$$

where for an idealized elastic-perfectly plastic system shown in Figure 6(a) the yielding displacement, D_y, is well defined, and D_u represents the maximum displacement. To establish R, μ_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 7). 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 6(b)-e shows the cyclic response of an isolated woodframe shear wall. The hysteresis 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 6 (a), the value of D_y 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 7(b). 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 similar consensus-based definition of D_y and D_u for the determination of ductility capacity, is in ASCE 41.



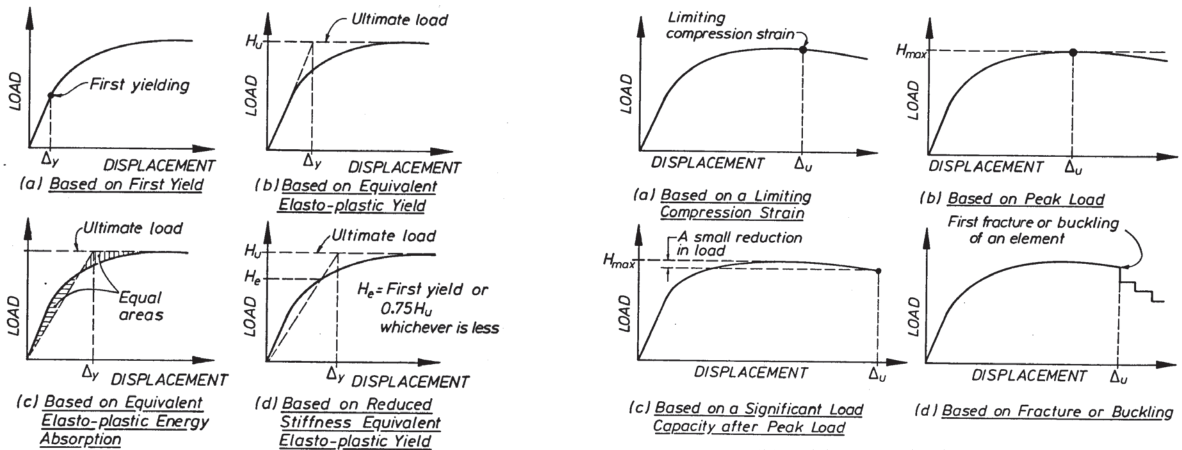
(a) Elastic-perfectly plastic idealization

(b) Hysteresis response of woodframe shear wall (Gatto and Uang 2001)

Figure 6. 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.



(a) Yield displacement

(b) Ultimate Displacement

Figure 7. Determination of yield and maximum displacements (Park 1989)

The earliest recommendation for the reduction factor R, goes back to the work of Veletsos and Newmark (1960), which is used to develop the inelastic design spectra developed by Newmark and Hall (1982). In this early study, R was determined to be function of the system ductility μ. The relationship for R as a function of μ, for short, intermediate and long period structures is presented below:

$$
\begin{array}{ll}
T < 0.2 \text{ seconds} & R\mu = 1 \\
0.2 < T < 0.5 \text{ seconds} & R\mu = \sqrt{2\mu - 1} \\
T > 0.5 \text{ seconds} & R\mu = \mu
\end{array}
\qquad \text{(Eq. 17)}
$$

A relationship was developed for the force reduction factor derived from the statistical analysis of 15 western USA ground motions with magnitude between 5.7 and 7.7 (Krawinkler and Nassar, 1992). The influence of response parameters, such as yield level and hardening coefficient α, were taken into account. A 5% damping value was assumed. The equation derived is given as:

$$
R_\mu = \left[c\,(\mu - 1) + 1\right]^{1/c}
$$

$$
c\,(T, \alpha) = \frac{T^a}{1 + T^a} + \frac{b}{T} \qquad \text{(Eq. 18)}
$$

where c is a constant which is dependent on period (T) and α which is the strain hardening parameter of the hysteretic model and a and b are regression constants. Values of the constants in above equations were recommended for three values of hardening α as in Table below:

HARDENING VALUE	MODEL PARAMETERS	
α	a	b
0 %	1.00	0.42
2 %	1.01	0.37
10 %	0.80	0.29

The equation for reduction factor introduced by Miranda and Bertero (1994) was obtained from a study of 124 ground motions recorded on a wide range of soil conditions. The soil conditions were classified as rock, alluvium and very soft sites characterized by low shear wave velocity. A 5% of critical damping was assumed. The expressions for the period-dependent force reduction factors Rμ are given by:

$$R_{\mu} = \frac{\mu - 1}{\Phi} + 1$$

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} \exp\left[-1.5(\ln T - 0.6)^2\right] \quad \text{for rock site} \qquad \text{(Eq. 19)}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} \exp\left[-2(\ln T - 0.2)^2\right] \quad \text{for alluvium site}$$

$$\Phi = 1 + \frac{T_1}{3T} - \frac{3T_1}{4T} \exp\left\{-3\left[\ln(T/T_1) - 0.25\right]^2\right\} \quad \text{for soft site}$$

where T1 is the predominant period of the ground motion. The latter corresponds to the period at which the relative velocity of a linear system with 5% damping is maximum within the entire period range.

Determination of System Factor by the Pushover Analysis

ATC-19 was published in 1995. One of the results was separating the *R* factor into three key components related to the structural system. The three factors, strength, ductility and redundancy, depended on characteristics of the particular structural system. Pushover analysis results indicated key characteristics of structural system behavior. The key points were the yield displacement, yield strength and strength at the maximum considered displacement. Another consideration of the structural system properties was inherent damping. Though this factor, R_{ξ}, was not included in the final calculation of *R*, ATC-19 recommended damping as a design consideration. This is illustrated in the graphic below.

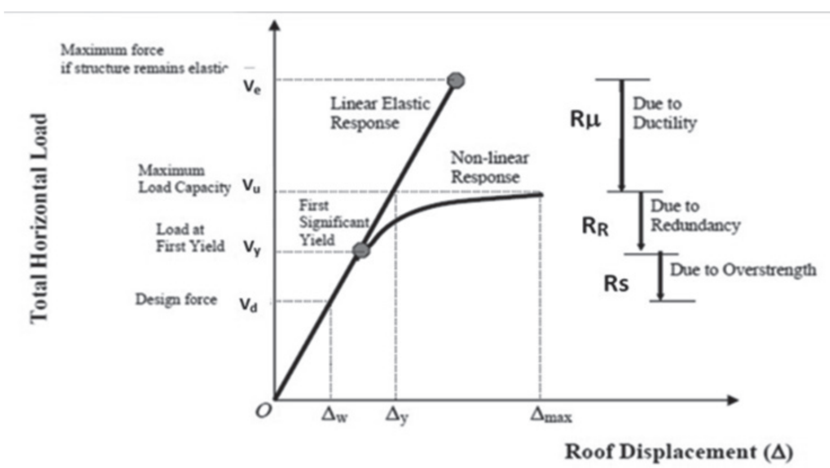


Figure 8. Push Components of R factor.

With: $R = R_s\, R_{\mu}\, R_R\, R_{\xi}$

Where:

R = Strength Level Response Modification Coefficient
R_s = Overstrength factor

R_μ = Ductility Reduction Factor (Rμ); Newmark-Hall, Miranda-Bertero, Krawinkler and Nassar, etc
R_R = Redundancy Factor
R_ξ = Damping Factor

The damping factor R_ξ accounts for the effect of added viscous damping and is primarily applicable for structures provided with supplemental energy dissipating devices. If such devices are not used, the damping factor is usually assigned a value equal to 1.0.

FEMA P-695 Approach to the determination of R Factor for systems

The FEMA P-695 process to determine the seismic parameters is also an iterative process. The structure is assigned an initial R value, which is used in the determination of the ASCE 7 seismic parameters. Associated to the R are detailing requirements that are used in the nonlinear model of the structure. P-695 defines acceptance criteria that are used to determine if the initial R factor is adequate. If the structure does not meet the acceptance criteria, the R factor needs to be adjusted accordingly. The process can be summarized as follows:

1. Selection of the initial R value and associated seismic parameters.
2. Ground Motion selection: FEMA P695 project (Federal Emergency Management Agency 2009) developed two suites of standardized ground motions that are recommended for use with that project's assessment procedure. One suite contains a set of 22 ground motions.
3. Incremental Dynamic Analysis (IDA) and Collapse Margin Ratio (CMR) Determination. A nonlinear model of the structure is created for dynamic analysis. One structure is analyzed under 22 different ground motions that are scaled up incrementally. In parallel, a collapse fragility curve can be defined through a cumulative distribution function (CDF) that relates the ground motion intensity to the probability of collapse until the building collapses under 50% of the ground motions.
4. Spectral Shape Adjustment: The unique characteristics of spectral shape for rare ground motions are captured using a spectral shape factor (SSF) that is a function of the fundamental period of the structure, ductility capacity and the seismic design category. This adjustment is necessary to account for the uniqueness of rare ground motions common in regions of high seismicity that are not captured in the FEMA P-695 set of ground motions.
5. Determination of Collapse Uncertainty: There are several factors considered as random variables in the analysis process of the FEMA P-695These are shown in Eq (20).

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \qquad \text{(Eq. 20)}$$

Where:
β_{TOT}= total system collapse uncertainty (0.275 - 0.950)
β_{RTR}= record-to-record collapse uncertainty (0.20 - 0.40)
β_{DR}= design requirements-related collapse uncertainty (0.10 – 0.50)
β_{TD}= test data-related collapse uncertainty (0.10 – 0.50)
β_{MDL}= modeling-related collapse uncertainty (0.10 – 0.50).

6. Expected System Collapse Margin Ratio: Using the Total Collapse uncertainty, FEMA P-695 defines Adjusted Collapse Margin Ratio (ACMR) at two performance levels: 1) The probability of collapse for MCE ground motions is approximately 10% or less, on average across a performance group. 2) The probability of collapse for MCE ground motions is approximately 20% or less, for each index archetype within a performance level.

7. Evaluate if the calculated *ACMR* is greater than the performance objective limits ($ACMR_{20\%}$ and $ACMR_{10\%}$). If acceptable, the seismic performance factors meet the collapse performance objectives and the initial or revised R is justified. ACMR goals are tabulated in Table 7-3 from FEMA P-695.

8. If the system does not meet performance objectives, the structural system concept and system design requirements must be redefined and reevaluated by modifying the proposed *R*-factor.

For more details, refer to the FEMA P-695 document.

ASCE 7-16 Alternative Definition of R Factor

ASCE 7-16 section 12.2.1.1 allows the EOR to propose unique structural system values for the response modification coefficient R, overstrength factor Ω_0, and deflection amplification factor Cd. Those parameters must be defined using nonlinear dynamic analysis to demonstrate that the proposed design results in a probability of collapse under MCE_R shaking not greater than 10% for Risk Category II structures. Also, ASCE allows the use of techniques similar to P-695. Refer to the Performance-Based Seismic Design Methodologies paper for a discussion of efforts to calibrate ASCE 7 and ASCE 41 using FEMA P-695.

Format of *R* factor. A review of seismic codes from other countries shows that the *R* factor is generally 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.

Period-dependent *R* factor. Use of a period-independent *R* factor is common engineering practice worldwide. However, Miranda and Bertero (1994), summarize the work of a number of researchers and 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 inelastic displacements tend to be larger than those of a linear elastic system in the 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 reason for the use of a period-independent R factor is based on the following: Refining the period dependency of response modification may not be justified, because predictions of periods for very stiff building structures, where the effect of soil compliance becomes more significant, is questionable using the period formulas in standards. The Chilean code does not allow R as a period dependent factor when using Equivalent Static Analysis, but it allows it when dynamic analysis is used. Current practice has started to recognize the contribution of the higher modes, for example, in diaphragm design. ASCE 7 is considering the use of multi point spectra that modifies the elastic response considering the effect of large vibration periods. However, further research and consensus on the behavior of representative building models and the review of observed performance of actual structures over a range of periods is necessary to calibrate a more refined period dependent-modification of the *R* factor.

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 Structural Response Modification Factors

Seismic Provisions	Response Modification factor	Deflection Amplification factor	$\dfrac{\text{Deflection Amplification Factor}}{\text{Response Modification factor}}$
UBC			
1994	R_w	(3/8) R_w	0.375
1997	R	0.7R	0.7
ASCE 7	R	C_d	0.5–1.0
Eurocode 8	q^a	q	1.0^c
Mexico	Q^a	Q	1.0^c
New Zealand	μ^b	μ	1.0^c
NBC of Canada			
1995	R/U	R	U (=0.7)
2015	R_dR_o	R_dR_o	1.0

[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 range from 0.5 to 1.25. Two observation can be made. First, the theoretical relationship between this ratio can be derived from Eqs. (6) and (7):

$$\frac{C_d}{R} = \frac{\mu_s \Omega_o}{R_d \Omega_o} = \frac{\mu_s}{R_d} \qquad \text{(Eq. 21)}$$

For an SDOF system, Newmark and Hall (1982) suggested that that the C_d/R ratio shouble 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 0.9R) 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).
As of this writing, there is a code change proposal to increase Cd to equal R. The Seismology Committee supports this change.

Remaining Challenges

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 shears and overturning moments have been used by some as force terms; roof lateral displacements and story drifts in critical stories have been used as deformation terms. 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 implemented for moderate and high seismic regions, appear effective in mitigating weak-story mechanisms. However, studies show that a global yielding mechanism may not always be achieved. The system ductility capacity can be reduced significantly if the damage is concentrated in a limited number of stories.

ATC, through FEMA P-695, has developed a systematic framework to define seismic performance parameters using nonlinear analysis with acceptance criteria that can be used to define R factors for either new or existing structural systems. However, this approach is based on a large suite of ground motions that are either historical records or artificially created due to the lack of site specific ground motions. The procedure is complex, and suitable only for large and well-funded projects.

One example that current standards still cannot address is the issue of residual drifts that 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 residual drifts that may not be acceptable to owners. Traditionally, the better estimated performance of dual systems has resulted in assigning high *R* values and more relaxed height limitations. While the strength requirement for the backup 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 residual drifts. No provisions for residual drift currently exist, except for some tall building designs. To incentivize better performance, the values of *R* factors for Building Frame Systems would need to be adjusted downward slightly and raised for Dual Systems.

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. The Seismology Committee prefers ductility capacity through proper detailing over increasing strength to resist seismic ground motions that are more severe than assumed. On this basis, although two systems just mentioned should conceptually have the same value of *R*, it would 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 structures, with simplified elastic analysis for routine designs. 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 may greatly complicate the design process and defeat the original intent. The biggest challenge is to address the owner's desire for a prompt recovery and reparability after a large seismic event.

References

Applied Technology Council (2007*). Quantification of building system performance and response parameters*. An in-progress study funded by the Federal Emergency Management Agency. Applied Technology Council, Redwood City, CA.

Applied Technology Council (1995). *Structural Response Modification Factors*. Applied Technology Council, Redwood City, CA.

ASCE (2017). ASCE 7-16, *Minimum design loads for buildings and other structures, including supplement no. 1,* American Society of Civil Engineers, Reston, VA.

Baker, William et al (2010). "Proposed Methodology to Determine Seismic Performance Factors for Steel Diagrid Framed Systems," Council on Tall Buildings and Urban Habitat, Chicago, IL.

BSSC (2015). *NEHRP recommended provisions for seismic regulations for new buildings and other structures,* (FEMA P-1050), Building Seismic Safety Council, Washington, D.C.

BSSC (2001). *NEHRP recommended provisions for seismic regulations for new buildings and other structures, 2000 edition, part 1: Provisions.* (FEMA 368), Building Seismic Safety Council, Washington, D.C.

BSSC (2001). *NEHRP Recommended provisions for seismic regulations for new buildings and other structures, 2000 Edition, part 2—Commentary* (FEMA 369), Building Seismic Safety Council, Washington, D.C.

CEN (2004). *Eurocode 8: Design provisions for earthquake resistant structures*, European Pre-standard ENV 1998-1-1, 1-2, amd 1-3, Bruxelles.

FEMA (2009). FEMA P695, Quantification of Building Seismic Performance Factors, Federal Emergency Management Agency, Redwood City, CA

Filiatrault, A., Fischer, D., Folz, B., and Uang, C.M., (2002). "Seismic testing of two-story woodframe house: influence of wall finish materials," *Journal of Structural Engineering*, Vol. 128, No. 10, pp. 1337-1345.

Freeman, S. A. (1977), "Racking tests of high-rise building partitions," *Journal of the Structural Division*, ASCE, 103, ST8, pp. 167385.

Gad, E.F., Chandler, A.M., Duffield, C.F., and Hutchinson, G.L. (1999). "Earthquake ductility and overstrength in residential structures," *Structural Engineering and Mechanics*, 8, (4,) 361-382.

Gatto, K. and Uang, C.M. (2003). "Effects of loading protocol on the cyclic response of woodframe shearwalls," *Journal of Structural Engineering*, 129 (10).

Heidebrecht, A.C. (2003). "Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada," *Canadian Journal of Civil Engineering*, vol. 30, pp. 241-254.

IAEE (1992). *Earthquake resistant regulations – A world list–1992*, International Association of Earthquake Engineering, Tokyo, Japan.

Iwan, W. D. (1980). "Estimating inelastic response spectra from elastic spectra," *Earthquake Engineering and Structural Dynamic*s, 8 (4) 375-388.

Jain, S. K. and Navin, R. (1995). "Seismic overstrength in reinforced concrete frames," *Journal of Structural Engineering*, 121 (3), 580-585.

Krawinkler, H. and Seneviratna, P. (1998). "Pros and cons of a pushover analysis for seismic performance Evaluation," *Engineering Structures*, 20 (4-6), 452-464.

Muto, K.,at al., 1960, "Non-linear Response Analysers and Appliaction to Earthquake Resistant Design" proc, 2WCEE, Vol.2 Japan, pp 649-668.

Miranda, E. and Bertero, V.V. (1994). "Evaluation of strength reduction factor for earthquake-resistant design," *Earthquake Spectra*, 10 (2), 357-379.

Newmark, N.M. and Hall, W.J. (1982). *Earthquake spectra and design*, EERI, Berkeley, CA.

Osteraas, J. and Krawinkler, H. (1990). "Seismic design based on strength of structures," *Proceedings*, *Fourth U.S. National Conference on Earthquake Engineering,* vol. 2, 955-964, Earthquake Engineering Research Institute, Oakland, CA.

Park, R. (1989). "State-of-the-art report: ductility evaluation from laboratory and analytical testing," *Proceedings*, *Ninth World Conference on Earthquake Engineering*, Vol. VIII, pp. 605-616.

Reyes-Salazar, A. (2002). "Ductility and ductility reduction factor for MDOF systems," *Structural Engineering and Mechanics*, 13 (4), 369-385.

Rojahn, C. and Hart, G.C. (1989). "U.S. code: focusing on R-factor of UBC, ATC-3 and NEHRP," *Proceedings of third US-Japan workshop on improvement of building structural design* (ATC-15-2*)*, 41-48, Applied Technology Council, Palo Alto, CA.

SEAOC (1995). *Vision 2000 - A framework for performance-based earthquake engineering*. Structural Engineers Association of California, Sacramento, CA.

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

SEAOC Seismology Committee (1985). *Recommended Lateral Force Requirements and Commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1997). *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California, Sacramento, CA.

Standards New Zealand (1992). *General structural design and design loadings for buildings*, NZS 4203:1992, Standards New Zealand, Wellington, New Zealand.

Uang, C.M. (1991a). "Establishing R (or Rw) and Cd factors for building seismic codes," *Journal of Structural Engineering* 177 (1) 19-28.

Uang, C.M. (1991b). "Comparison of seismic force reduction factors Used in U.S.A. and Japan," *Earthquake Engineering and Structural Dynamics*, 20, 389-397.

Uang, C.M. and Bertero, V. V. (1991). "UBC seismic serviceability regulations: critical review," *Journal of Structural Engineering*, 117 (7), 2055-2068.

Uang, C.M., Maarouf, A. (1994). "Deflection amplification factor for seismic design provisions," *Journal of Structural Engineering*, 120 (8), 2423-2436.

Veletsos, A.S. and Newmark, N.M. (1960). "Effect of inelastic behavoiur on the response of simple systems to earthquake motions", Proc. 2nd World Conference on earthquake Engineering, Japan, Vol. 2, pp 895-912.

Whittaker, A.S., Hart, G., and Rojahn, C. (1999). "Seismic response modification factors," *Journal of Structural Engineering*, 125 (4), 438-444.

Youssef, N. F. G., Bonowitz, D., Gross, J. L. (1995). "A survey of steel moment-resisting frame buildings affected by the 1994 Northridge earthquake," *Report No. NISTIR 5625*, National Institute of Standards and Technology, Gaithersburg, MD.

J.W.Baker. "Trade-offs in ground motion selection techniques for collapse assessment of structures", Vienna Congress on Recent Advances in Earthquake Engineering and Structural Dynamics 2013 (VEESD 2013)

X. Y. Ou, Z. He & J. P. Ou. "Seismic Collapse Margin of Structures Using Modified Mode-based Global Damage Model", 15 WCEE Lisboa 2012

Keywords

deflection amplification factor, C_d.
equivalent lateral force procedure
overstrength factor, Ω_0
response modification factor, R factor

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Development of System Factors," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.2.5.1 Table 12.2-1		UBC (1961, 1985, 1988, 1997) ASCE 7-02: 9.5.2.2.1, Table 12.2-1 2001 CBC: 1629.5, 1631.5.7, Table 16-N

Intent of System Factors

The first Blue Book commentaries 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 defined in the 1959 Blue Book (Table 23-C) and codified in 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* factor 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).

In the 1985 UBC (Table 23-I) the definition was 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 forces is provided by a specially detailed moment-resisting space frame (concrete or steel) that 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.

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 consistent with 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 (R = 8.5), while the value for dual systems with steel concentric braced frames was set at 11 (R = 8).

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 provided the background for the provisions in both ASCE 7 and the CBC. 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.

Starting with the 2007 edition, the CBC was based on the International Building Code (IBC), rather than the UBC. As a result, the CBC does not specifically address dual systems, but instead defers to ASCE 7. Although no longer used, the pre-2007 CBC/UBC definition and design of a dual system required such a lateral system to satisfy the following:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames are designed to independently resist at least 25 percent of the design base shear.
3. The two systems are designed to resist the total design base shear in proportion to their relative rigidities, considering the interaction of the dual system at all levels.

This definition/requirement is essentially the same as found in ASCE 7, except for the deletion of less ductile moment resisting frames (MMRWF and OMRFs) allowed in 2001 CBC.

Current Practice

Dual systems are defined in ASCE 7-16 as:

> A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment-resisting frames and shear walls or braced frames as prescribed in Section 12.2.5.1.

Section 12.2.5.1 of ASCE 7-16 further requires that:

> For a dual system, the moment frames shall be capable of resisting at least 25% 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.

Regarding dual systems, the current version of the CBC references ASCE 7 with no changes.

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, which were cited as the basis 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 behavior of 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 seismic force 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 needs to consider the bracing effects of cracked wall segments acting as compression diagonals within the infilled frame bays, which can be modeled using ASCE 41. 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 seismic forces meeting the 25% design requirement.

Determination of the 25% design requirement for the moment frame calls for some code interpretation. The 25% forces are traditionally based on the distributed base shear of the equivalent lateral force procedure (section 12.8), using tabulated R values for the dual system and T values for the total dual structure ($C_t = 0.02$ by default). The exponent k for the vertical distribution of seismic forces can be based on the period derived for the total structure (section 12.8.3).

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. Redundancy requirements in Section 12,3,4 apply to primary systems, but the Seismology Committee does not consider it necessary to include any special redundancy factor calculations for the stand-alone moment frames.

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 7-16 Table 12.2-1 lists 21 types of dual systems, 13 with SMFs and 8 with IMFS. It is based on the 2003 NEHRP Provisions (BSSC 2004), which list 13 types with SMFs and 8 with IMFs. The R factors for these systems range from 8 to 3.

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 2001 CBC/1997 UBC for Seismic Zones 3 and 4 or by ASCE 7-16 /2019 CBC 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 current and past reference documents. Several of the systems not permitted by ASCE 7 for high seismicity are simply not included in the older CBC. Similarly, composite systems were not included in the CBC until 2007 edition, although some had sufficient ductility for use in California prior.

Table 1 shows that many of the dual systems using IMFs have R factors that are about the same or only slightly higher 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 that incorporate IMFs 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.

Two new systems were added to ASCE 7-05 that resemble dual systems, although they are not categorized as such. Separate and greater R-values were 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. At the time of inclusion, no specific design rules had 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) had not been fully defined. At the time, the Seismology Committee recommended that the beam-column connections comply with special moment frame detailing provisions, and that potential interactions between braced frame connection plates and the connections be either specifically considered in the connection design or the condition avoided. ASCE 7-10 dropped requirements for moment connections for both frame systems but retained the higher R values. It appears that provisions in AISC 341-10 adequately address connection ductility requirements for the two frame systems, thus justifying the use of the higher R value found in ASCE 7-16.

Another structural system in ASCE 7 (added in 2005) 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; it is only permitted in SDC B.

ASCE 7 sets height limits on certain dual systems that use IMF 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. 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 why dual systems are used today.

Table 1. Comparison of dual systems in ASCE 7-16 and 2001 CBC

Shear wall or braced frame type	***R*** for non-dual building frame system ASCE 7	**ASCE 7-16** *R* value for dual system SDC D-F		**2001 CBC** *R* value for dual system		
		SMF	IMF	SMF	IMF	OMF
Steel EBFs	8	8	—	steel 8.5	—	steel 4.2
Steel SCBFs	6	7	6 in D, NP in E,F	steel 7.5	—	steel 4.2
Special reinforced concrete shear walls	6	7	6.5	8.5	concrete 6.5, NP	steel 4.2
Ordinary reinforced concrete shear walls	5	NP	NP	—	—	—
Composite EBFs	8	8	—	—	—	—
Composite SCBFs	5	6	5.5	—	—	—
Steel and concrete composite ordinary braced frames	3	—	NP	—	—	—
Steel and concrete composite plate shear walls	6.5	7.5	—	—	—	—
Steel and concrete composite special shear walls	6	7	—	—	—	—
Steel and concrete composite ordinary shear walls	5	NP	NP	—	—	—
Special reinforced masonry shear walls	5.5	5.5	—	5.5	concrete 4.2, NP	steel 4.2
Intermediate reinforced masonry shear walls	4	NP	NP	—	—	—
Steel bucking-restrained braced frames	8	8	—	—	—	—
Steel special plate shear walls	7	8	—	—	—	—
Steel ordinary CBFs	3.25	—	NP	steel 6.5	—	steel 4.2
Ordinary reinforced masonry shear walls	2	—	NP	—	—	—
Masonry shear walls	NA	—	—	—	MMRWF 6	—

Performance Issues

The nature and construction of dual-system structures have evolved significntly from the time when the system was first defined. 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, such 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 7-16

Primary System (Building Frame)	w/o Dual System	with Intermediate Moment Frames capable of resisting at least 25% of prescribed seismic forces	with Special Moment Frames capable of resisting at least 25% of prescribed seismic forces
Special reinforced concrete shear walls	B, C = NL D, E = 160 ft, F = 100 ft	B, C = NL, D = 160 ft, E, F = 100 ft	B, C, D, E, F = NL
Steel and concrete composite special concentric braced frames	B, C = NL, D, E = 160 ft, F = 100 ft	B, C = NL, D = 160 ft, E = 100 ft, F = NP	B, C, D, E, F = NL
Steel special concentrically braced frames (SCBF)	B, C = NL, D, E = 160 ft, F = 100 ft	B, C = NL D = 35 ft, E, F = NP	B, C, D, E, F = NL
Steel and concrete composite eccentrically braced frames	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Steel eccentrically braced frames (EBF)	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Steel and concrete composite steel plate shear walls	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Steel and concrete composite special shear walls	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Special reinforced masonry shear walls	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Buckling-restrained braced frames (BRBF)	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Steel special plate shear walls	B, C = NL, D, E = 160 ft, F = 100 ft	—	B, C, D, E, F = NL
Ordinary reinforced concrete shear walls	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP
Steel and concrete composite ordinary shear walls	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP
Intermediate reinforced masonry shear walls	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP
Steel and concrete composite ordinary braced frames	B, C = NL D, E, F = NP	B, C = NL D, E, F = NP	—

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 seismic forces 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 SMFs, 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 Table 12.2-1 allows several different dual systems with IMFs for SDC D, which covers standard occupancies in much of California. For a 12-story building (about 160 ft tall), the IMF, as described above, could be the unintended primary system in the upper floors. In the same table, however, steel IMFs are not permitted as stand-alone systems in multi-story SDC D buildings taller than 35 ft, and concrete and composite IRFs 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 SMF systems should be mandatory. It should be recognized that the height limits for a dual system with IMF 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 and standards (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 and standards 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 experienced significant motions in the 1989 Loma Prieta earthquake, but were not 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 should not be used in seismically active regions. 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 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 basis for the 25% rule 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.

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 ASCE 7 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 is based on judgment and has not been systematically verified by nonlinear analysis. Modern analytical techniques could be used to determine ideal strength and stiffness characteristics for moment frames. Such studies would likely indicate different design solutions for different building heights and system aspect ratios.

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

References

ASCE (2016). Minimum Design Load and Associated Criteria for Buildings and Other Structures, Provisions and Commentary, SEI/ASCE 7-16. American Society of Civil Engineers, Reston, VA.

ASCE (2017). *Seismic evaluation and retrofit of existing buildings*, SEI/ASCE 41-17. American Society of Civil Engineers, Reston, VA.

ATC (1978). *Tentative provisions for the development of seismic provisions for buildings, ATC-3-06,* Applied Technology Council, Redwood City, CA.

BSSC (2001). *NEHRP Recommended provisions for seismic regulations for new buildings and other structures, 2000 Edition, part 2—Commentary* (FEMA 369), Building Seismic Safety Council, Washington, DC.

BSSC (2004). *NEHRP recommended provisions for seismic regulations for new buildings and other structures, Part 1: Provisions* (FEMA 450-1/2003 Edition), Building Seismic Safety Council, Washington, DC.

CBSC (2019). *California Building Code*, Sacramento, California.

Degenkolb, Henry (1994). *Henry J. Degenkolb: Connections: The EERI Oral History Series*, Stanley Scott interviewer/editor*,* Earthquake Engineering Research Institute, Oakland, CA.

FEMA (2005). *Improvement of nonlinear static seismic analysis procedures*, FEMA 440, Federal Emergency Management Agency, Washington, DC.

ICBO (1961, 1985, 1988, 1997). *Uniform building code*, International Council of Building Officials, Whittier, CA.

Layne, H. et al. (1963). "Panel discussion: Section J of the SEA code—Should it be retained, modified, or eliminated?" *Proceedings of SEAOC Annual Convention,* Structural Engineers Association of California.

SEAOC Seismology Committee (1960). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1967). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1990). *Recommended lateral force requirements and commentary*, Structural Engineers Association of California, Sacramento, CA.

Keywords
dual systems
height limits
moment frames
overstrength factor, Ω_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 2019)

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

SEAOC Seismology Committee (2019). "Dual Systems," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

@Seismicisolation

Cantilever Column Systems

ASCE 7-16 reference section(s)	2019 CBC / 2018 IBC reference section(s)	Other standard reference section(s)
ASCE 7-16 11.2, 12.2.5.2, 12.2.5.5, 14.1. 14.2, 14.5, Table 12.2-1, Table 12.3-3	Section 2205	ACI 318-14, Chapter 18 AISC 341-16, Sections E5 and E6

Cantilever column systems are defined in ASCE 7-16 as "a seismic force-resisting system in which lateral forces are resisted entirely by columns acting as cantilevers from the base." Cantilever columns provide a simple alternative to a moment frame, braced frame, or shear wall for a variety of low-rise structures. In particular, cantilever column elements are useful in low-rise light-frame construction of two stories or less, in combination with other structural systems (mixed systems), to provide resistance within limited spaces. They also may be useful for seismic retrofits. Cantilever column elements, when used in mixed systems with light-frame shear walls, currently compete with proprietary prefabricated shear panels and proprietary light-gauge braced elements.

Historical Development of Cantilever Column Provisions

The seismic force-resisting system classification of cantilever columns was first introduced in 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-inch diameter gravity columns buckled due to accidental fixity at the base and 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 is 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 on the belief that cantilever columns have "very limited redundancy," "resist all lateral forces," and are used where there is "no independent vertical load-carrying system." As the design practice reacted to these changes twenty years ago, engineers began converting cantilever column elements to moment frames, or to proprietary narrow shear wall systems listed as having *R* factors equivalent to wood-sheathed shear walls. One of the major design drawbacks to the use of cantilever columns was that the low *R* factor of 2.2 was required to be applied to the entire structure along that axis. Reportedly, engineers complained that the cantilever column, as used in this typical class of structure, was unduly penalized in comparison to alternatives. Accordingly, the SEAOSC Light Frame Committee (June 2000), and thereafter the SEAOC Seismology committee, published a Position Paper titled "Cantilever Column Elements in Light Frame Shear Wall Systems" (SEAOC 2004). The paper suggested that, if certain conditions were met, it was appropriate to apply the more restrictive *R* factor only to the lateral resisting lines that used cantilever column elements along with a less restrictive *R* factor (appropriate to the seismic force-resisting elements used) for the rest of the structure. This relief from the full-building application of the restrictive *R* factor included requirements to perform a deflection check and a limit on axial load of 15 percent of capacity, as well as other qualifications. The position paper provided guidance for the practice of using cantilever column elements within a mixed seismic force-resisting system that was predominantly made up of light-frame wood shear walls. It also foreshadowed cantilever column system design requirements in ASCE 7 (-10 and -16) and AISC 341 (-10 and -16).

System Factors and Height Limits

ASCE 7-16 Table 12.2-1 identifies system factors for Cantilever Column Systems reproduced below as Table 1. Only the steel standard AISC 341 has specific requirements for cantilever column systems (Steel Special and Ordinary Cantilever Column Systems G.1 and G.2). In contrast, the concrete (ACI) and timber (NDS) material standards do not distinguish between cantilever column system elements and moment frame systems elements. For example, Cantilever Column System G.3 refers to Special (Concrete) Moment Frames. In these cases, when it comes to applying the other system requirements, little or no further guidance is provided, other than pointing to the respective materials reference standards. However, ASCE 7 does require application of section 12.2.5.2 for cantilever columns as well as their foundations and other elements used to provide overturning resistance. It is the SEAOC Seismology Committee's opinion that, when specific requirements for cantilever columns are not provided within the referenced material standard, the columns are to be designed to meet requirements as frame columns.

Table 1. Cantilevered Column Systems, from ASCE 7-16, Table 12.2-1

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_0[b]	Deflection Amplification Factor, C_d[c]	Structural System Limitations Including Structural Height, h_n (ft) Limit[d]				
					Seismic Design Category				
					B	C	D	E	F
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Steel special cantilever column systems	14.1	2½	1¼	2½	35	35	35	35	35
2. Steel ordinary cantilever column systems	14.1	1¼	1¼	1¼	35	35	NP[l]	NP[l]	NP[l]
3. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
7. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP

[a] Response modification coefficient, R, for use throughout the standard. Note R reduces forces to a strength level, not an allowable stress level.
[b] Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal 2 ½, Ω_0, is permitted to be reduced by subtracting the value of one-half for structures with flexible diaphragms.
[c] Deflection amplification factor, C_d , for use in Sections 12.8.6, 12.8.7, and 12.9.2
[d] NL = Not Limited, and NP = Not Permitted. For metric units use 10.6m for 35 ft.
[l] See Sections 12.2.5.6 for limitations in structures assigned to Seismic Design Category D, E, or F.
[m] In section 2.3 of ACI318. The definition of "special moment frame" includes precast and cast-in-place construction.

By its nature, a cantilever element does not provide much redundancy. That is the primary reason for the low R values and the severe building height restrictions for these systems. The R value for a cantilever column system ranges from a low of 1 for Ordinary Reinforced Concrete Moment Frames to a high of 2.5 for Steel Special Cantilever Columns and Special Reinforced Concrete Moment Frames. The Deflection Amplification Factor (C_d) is the same value as the R factor for each of these systems. Structural Overstrength Factors (Ω_o) are 1.25 for all systems except for timber frame systems, where the assigned overstrength factor is 1.5. Where permitted, cantilever column systems are limited to a maximum building height of 35 ft measured from the base of the structure. Only specially detailed systems, such Steel Special Cantilever Columns and Special Reinforced Concrete Moment Frames, are permitted in all Seismic Design Categories without restriction, while Steel Ordinary Cantilever Column systems must meet the limitations of ASCE 7 section 12.2.5.6 to do so. Timber frame systems are permitted through Seismic Design Category D. Other systems are not permitted in high to very high seismicity regions (SDC D-F).

Design Requirements

In addition to meeting the referenced material standards requirements for column elements, ASCE 7 Section 12.2.5.2 imposes additional design requirements and limit checks on cantilever columns. Section 12.2.5.2 limits the axial loads on individual cantilever column elements, including seismic overstrength (Ω_o), to no more than 15% of the available axial strengths. This axial load limit is meant to provide sufficient column ductility and enhance lateral stability under expected flexural yielding at the column bases.
In addition to the limit on axial load, Section 12.2.5.2 requires that elements providing flexural fixity at the base of the column (foundations) be designed to resist seismic load effects that include overstrength per Section 12.4.3. As the base connection is key to the performance of cantilever systems, the foundation and other elements providing

overturning resistance should have sufficient strength to resist the load combinations with the full capacity of the cantilevered column, including overstrength and strain hardening.

In addition to these two requirements, all other design and detailing requirements of ASCE 7 for diaphragm and collector designs, deformation compatibility, and drift compliance should also be met. One area that is not entirely clear within ASCE 7 is the importance of providing adequate stiffness of the base connection for the cantilever column. While ASCE 7 requires that elements providing overturning support be designed for forces including overstrength, it is the opinion of the SEAOC Seismology Committee that unconstrained flag pole footings and isolated spread footings should generally not be used for base connections, given the large contributions of foundation rotations and soil deformation expected for those foundation types. The exception would be where soil deformation and foundation rotations 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 typically easy to achieve. The adjacent vertical structural element(s) may be adjacent cantilever column(s), gravity columns(s), or building wall(s) that can provide restraint for the grade beam foundation.

In addition to ASCE 7 design requirements, each material standard has design and detailing requirements that must be met, depending on the seismic force-resisting system's designation.

Combinations with Other Seismic Force-resisting Systems

Historically, cantilever columns have been used by designers to provide alternatives to moment frames or shear walls for a variety of light-frame structures, including single and multi-family dwellings, and within storefronts of retail spaces. In particular, they are useful in providing seismic 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 for single-family dwellings where no qualifying seismic force-resisting elements were previously provided, such as first floor open parking under residential units ("tuck-under parking"). Cantilever column elements currently compete with similar seismic force-resisting elements such as proprietary prefabricated shear wall panels and proprietary light-gauge bracing elements in light-frame shear wall systems. Although the proprietary elements appear to be similar structurally to cantilever columns (narrow flexural elements with fix-base attachment), they are not categorized as cantilever column elements. Unlike cantilever column elements, proprietary prefabricated shear wall panels and bracing elements are tested to demonstrate that their seismic performance is equivalent to sheathed wall systems. In addition, proprietary elements are typically used in bearing wall systems where the primary vertical load carrying systems consist of distributed elements (wall studs).

ASCE 7, Section 12.2.3.3, provides some guidance regarding combinations of seismic force-resisting systems. In general, it requires that the lowest *R* value of any of the combined systems in a particular direction be used for the design in that direction. It also requires use of the largest overstrength factor and deflection amplification factor in the same direction of the system consistent with earlier codes.

ASCE 7, Section 12.2.4 requires that detailing provisions for seismic components common to different framing systems be designed using the detailing requirements of Chapter 12 that are required by the highest response modification coefficient, *R*, of the connected framing systems. For some lines of resistance with mixed systems that include cantilevered columns, this requirement can result in illogical detailing that standards writers may not have anticipated.

The use of the least value of *R* along an independent line of resistance is also addressed in the exception under ASCE 7 Section 12.2.3.3 that states: "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) Risk Category I or II building, (2) two stories or fewer above grade plane, 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 of R for any of the systems used in that same direction."

This essentially provides for the same provisions that were recommended by the 2004 SEAOC Seismology Committee position paper, mentioned earlier. The position paper provided necessary guidance for the existing practice of using cantilever column elements within a predominantly light-frame wood shear wall seismic force-resisting system. The position paper had four requirements, and presumed the 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, should 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 should 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 should be based on a K = 2.1 and should not exceed the force ratio of $Pu/\varphi Pn \leq 0.15$
4. A reinforced concrete grade beam should join cantilever columns to the adjacent vertical structural element(s) with sufficient stiffness to satisfy deflection limits along each line of resistance. Other reinforced concrete foundation systems may be used, provided 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 continues to be a part of the SEAOC Seismology Committee's 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

Cantilever Column Systems are sometimes confused with Inverted Pendulum Structures. An Inverted Pendulum Structure is not a designated seismic force-resisting system in ASCE 7 Table 12.2-1. Instead, it is a class of structures defined in ASCE 7 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 element" or portion of the structure. Inverted Pendulum Structures are defined by the specific geometric configuration of the structure, independent of the seismic force-resisting system and can include any system identified in ASCE 7 Table 12.2-1.

ASCE 7 Section 12.2.5.3 states: "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."

While some cantilever column systems may occur in buildings that are classified as inverted pendulum-type structures, the vast majority of buildings that use cantilever column systems do not fall into this category.

Reduction of the Redundancy Factor

ASCE 7, Section 12.3.4.2 Item a and Table 12.3-3 allow for the redundancy factor ρ 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 specifically addressed in ASCE 7. 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 a height restriction applicable to that cantilever system, as distinct from the height restrictions that pertain to the overall building.

Requirements for design of Cantilever Column Systems defined in ASCE 7 should be treated consistently across all material standards. For example, AISC 341-16 requires that steel cantilever columns not only meet detailing and bracing requirements, but also be designed for load combinations that include overstrength. However, systems that use other materials can be designed with load combinations that do not include overstrength. Because of this inconsistency, the SEAOC Seismology Committee recommends that ASCE 7 be modified to require the same load combinations for design of cantilever columns, regardless of material used.

References

ACI (2014). ACI 318-14: Building code requirements for structural concrete, with commentary, American Concrete Institute, Farmington Hills, MN.

AISC (2016). Seismic provisions for structural steel buildings (ANSI/AISC 341-16), July 12, 2016, American Institute of Steel Construction, Chicago, IL.

ASCE (2017). ASCE/SEI 7-16, Minimum design loads for buildings and other structures, American Society of Civil Engineers, Reston, VA.

ATC 2008. "Quantification of building system performance and response modification parameters," funded by the Federal Emergency Management Agency (FEMA) and being conducted by Applied Technology Council, Redwood City, CA.

BSSC (2004). The 2003 NEHRP recommended provisions for regulations for new buildings and other structures, Part 1: Provisions (FEMA 450), Building Seismic Safety Council, Washington, DC.

NEHRP Consultants (2008). "Quantification of building system performance and response parameters," an in-progress study funded by the National Institute of Standards and Technology (NIST) and being performed by the

NEHRP Consultants Joint Venture, a joint venture of the Applied Technology Council (ATC), Redwood City, CA, and the Consortium of Universities for Research in Earthquake Engineering (CUREE), Richmond, CA.

SEAOC Seismology Committee (1996). Recommended lateral force requirements and commentary, sixth edition, Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). Recommended lateral force requirements and commentary, seventh edition, Structural Engineers Association of California, Sacramento, CA.
SEAOC Seismology Committee (2004). “Cantilever column elements in light frame shear wall systems (LRFD).” Structural Engineers Association of California, Sacramento, CA; available via the World Wide Web at: http://www.seaoc.org/committees.html#positions

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 2019)

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

SEAOC Seismology Committee (2019). "Cantilever Column Systems." August 2019, *The SEAOC Blue Book: Seismic design recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2016 CBC reference section(s)	Other standard reference section(s)
12.2.3.1 12.2.3.2 12.12.5		1997 UBC: 1630.4.2, 1630.4.3, 1630.4.4, 1629.8.3 ASCE 7-02: 9.5.2.2.2.1, 9.5.2.2.4.4 2003 IBC: 1617.6.1.2 ASCE 7-05: 12.2.2

Background

Many buildings use a combination of seismic force-resisting systems (SFRSs). The SFRS is 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, collectors, and foundations. For combinations of SFRSs, building codes traditionally limit or modify design parameters such as R (response modification factor), Ω_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, Ω_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 with a weight less than 10% of the total weight) 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 7-16 section 12.2.3.1, as well as previous editions, 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 7-16 section 12.2.3.2 has now corrected the omission and added the two-stage procedure.

Effect of redundancy coefficient. The UBC and ASCE 7-16 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 exhibit significantly different elastic or inelastic behavior. 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 completely removed any orthogonal systems requirement. (This remains true in ASCE 7-16.) 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 7-16 is section 12.12.5 (for Seismic Design Category D, E, or F).

Exception for separate lines of resistance. ASCE 7-16 section 12.2.3.3 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 Category 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 least of all the 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 Ω_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.

References

ASCE (American Society of Civil Engineers) (2002). ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.

ASCE (American Society of Civil Engineers) (2006). ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures, Including Supplement No. 1,* American Society of Civil Engineers, Reston, VA

California Building Standards Commission (2001). *2001 California Building Code* (California Code of Regulations, Title 24), California Building Standards Commission, Sacramento, CA.

ICBO (International Conference of Building Officials) (1997). *1997 Uniform Building Code*, ICBO, Whittier, CA.

BSSC (Building Seismic Safety Council) (2001). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, Part 2—Commentary* (FEMA 369), Building Seismic Safety Council, Washington, D.C.

SEAOC Seismology Committee (1990). *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California, Sacramento, California.

Keywords

combined systems
deflection amplification factor, C_d.
lines of resistance
overstrength factor, Ω_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 2019)

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

SEAOC Seismology Committee (2019). "Combined Systems," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2016 CBC reference section(s)	Other standard reference section(s)
12.8.2 12.8.6 12.9.7		

Approximate Period Determination

ASCE 7-16 (ASCE 2016) 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 lower than the expected period of the building, 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 typical site characteristics 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: $T_a = C_t h_n^{3/4}$.

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 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 current building codes will have increased stiffness and correspondingly lower period values than the structures designed according to previous codes.

In the following 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). These changes were incorporated into ASCE 7-02, and remain essentially unchanged through ASCE 7-16. The values of the coefficient C_t and the exponent x given in ASCE 7-16 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 ensure that the estimated period is conservatively low.

Figure 1 compares the approximate periods obtained from the formulas used by ASCE 7-02 (and subsequent editions) 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.

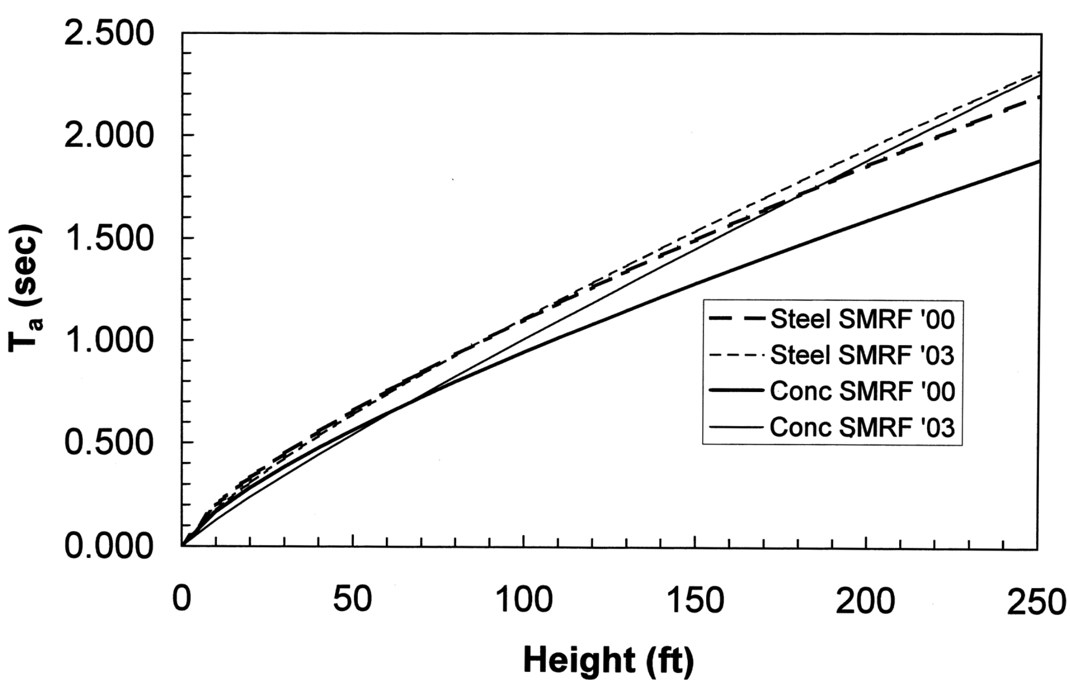


Figure 1. Comparison of Approximate Periods Determined by the 2000 and 2003 NEHRP Provisions

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:

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

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_S, 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.

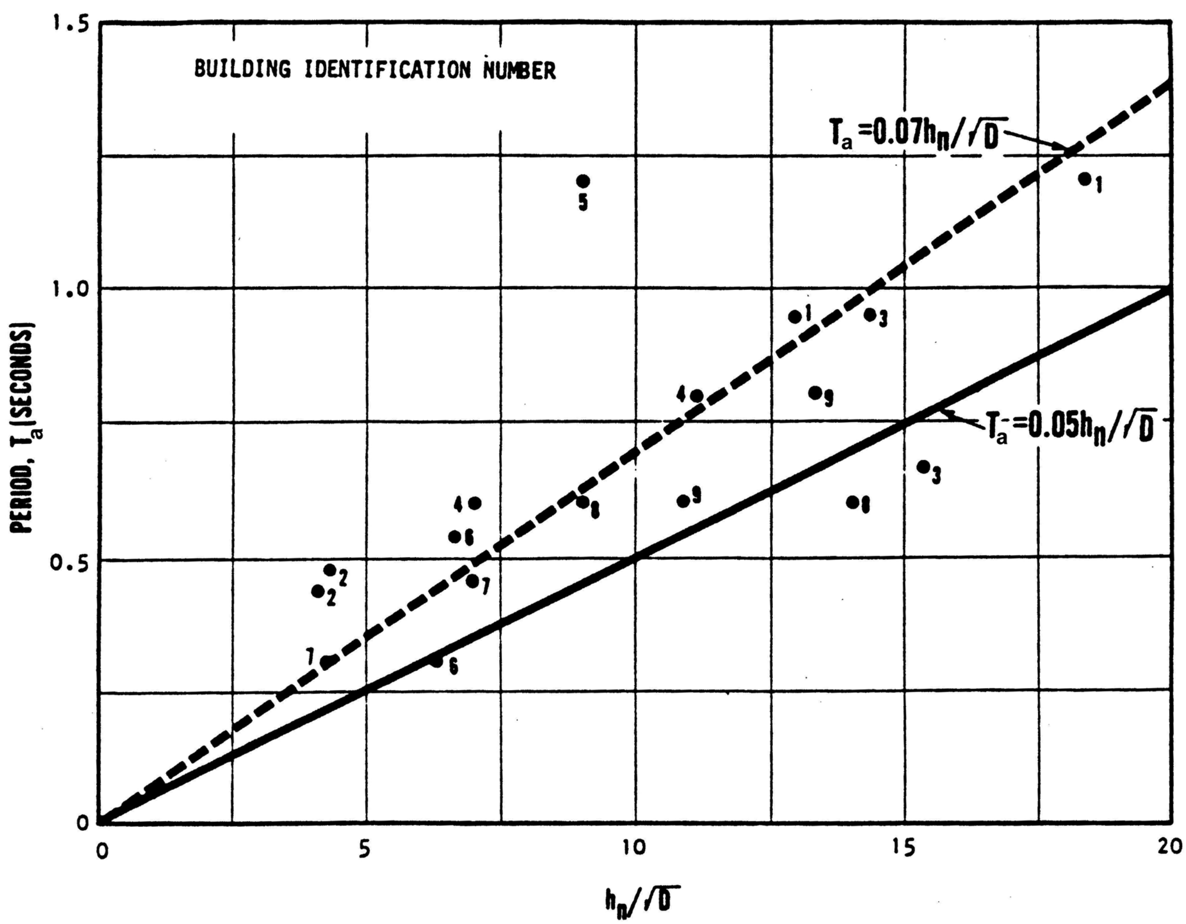


Figure 2. Comparison of Periods Calculated from Formulas with Periods Calculated from Strong Motion Records

The above formula was changed to $Ta = 0.02h_n^{3/4}$ in the 1988 UBC and subsequently in the 1994 NEHRP Provisions. Goel and Chopra (1997) 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.

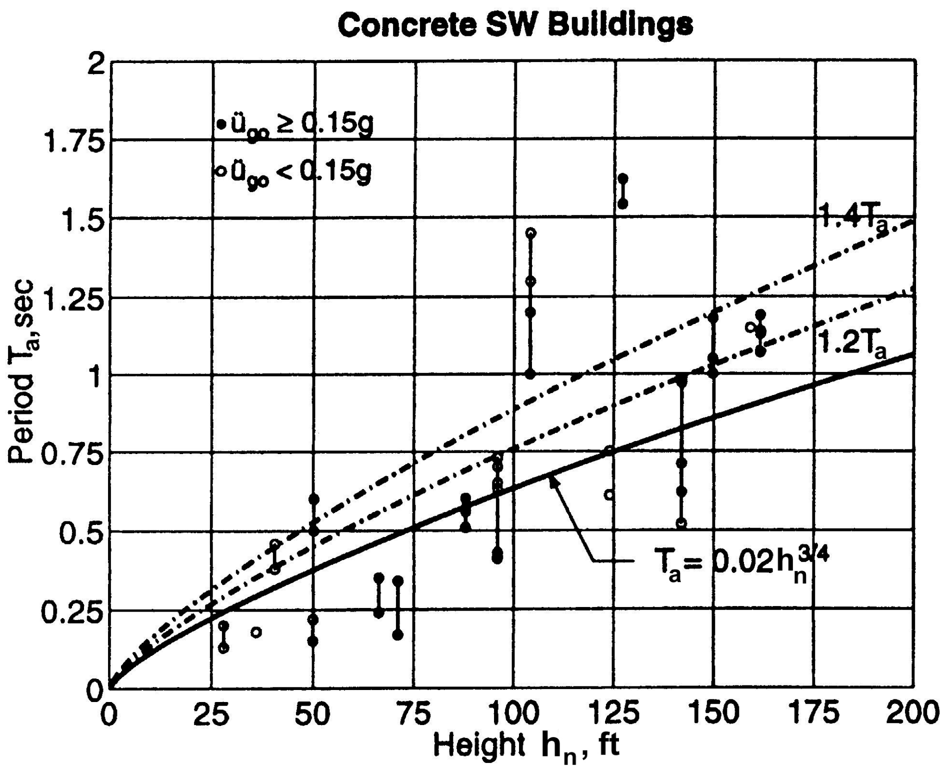


Figure 3. Comparison of Periods Calculated by Formulas
with Periods of Reinforced Concrete Shear Wall Building Strong Motion Records

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

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

$$A_c = A_e\,[0.2+(D_e/h_n)^2]$$

where A_e = 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) resulted in ASCE 7-02 Eq. 9.5.5.3.2-2 and 9.5.5.3.2-3, which provide a reasonably good fit to the data. (These equations are 12.8-9 and 12.8-10 in ASCE 7-16.) However, the form of these equations is rather complex, the simpler equation of ASCE 7-02 Eq. 9.5.5.3.2.1 was retained from the 1997 and earlier editions of the NEHRP Provisions, with the newer, more accurate information presented as an alternative (BSSC, 200l, Section 5.4.2). This remains true in ASCE 7-16; the simplified equation is 12.8-7.

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 (and subsequent editions). In the ASCE 7 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 ft^2 as well as for A_B = 20,000 ft^2. The comparisons are made for $D_e/h_n = D_i/h_i$ (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 $D_e/h_n = D_i/h_i = 0.1$, 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 7-16 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 later editions of the NEHRP Provisions.

The 2003 NEHRP provisions defined C_t for buckling-restrained braced frames (BRBFs) and eccentrically braced frames (EBFs) as 0.03. However, BRBFs were inadvertently left out of this same table in ASCE 7-05, which relegated the system to the "Other" classification with a value of 0.02. This omission was fixed in ASCE 7-10.

The optional use of $T_a = 0.1N$ (ASCE 7-16 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 was reintroduced 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 7-16 section 12.8.2.1 provide period estimates that are lower than most measured period values in the elastic range, and 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.

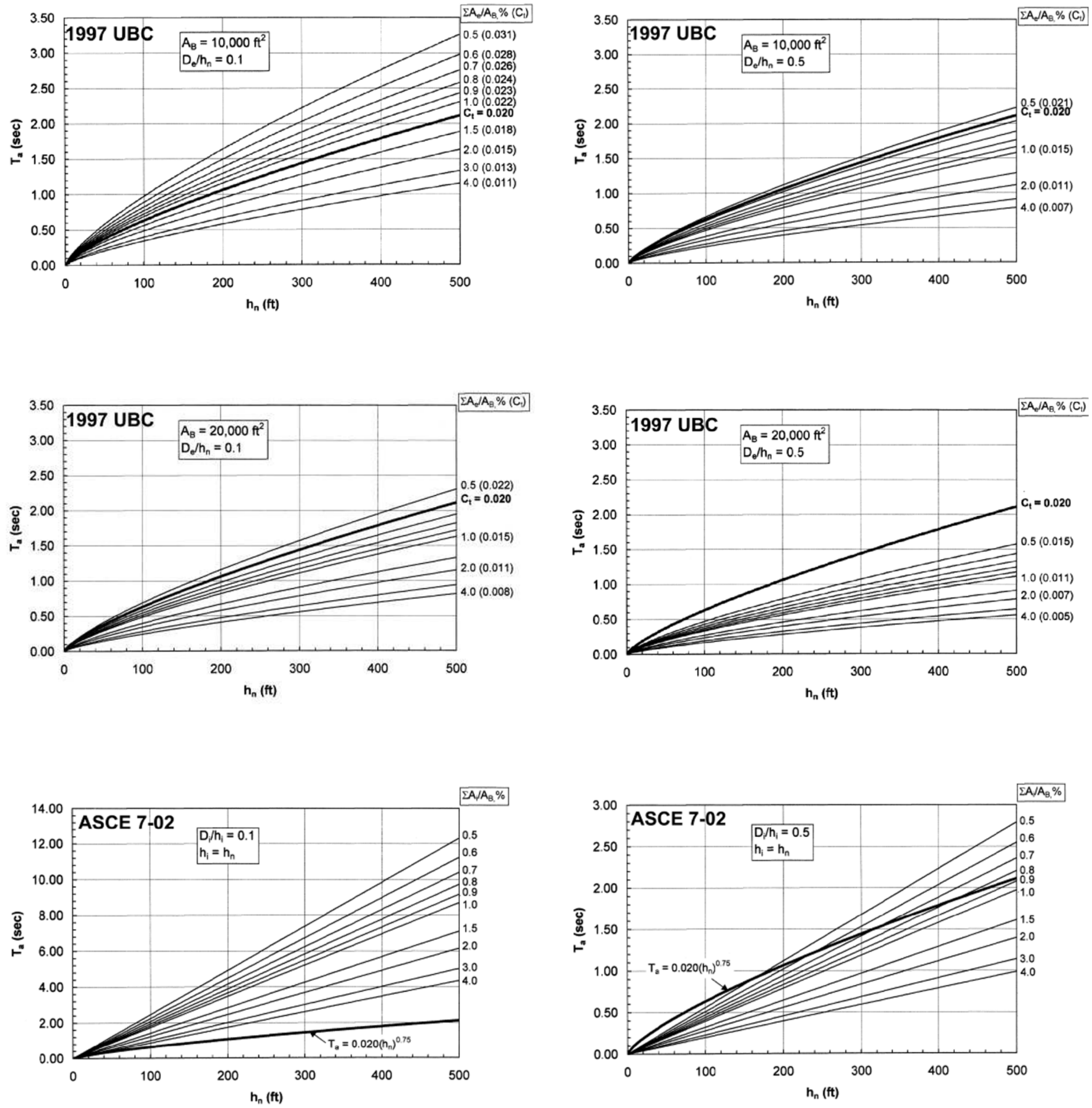


Figure 4. Comparison of 1997 UBC and ASCE 7-02 Period Calculations for Buildings with Concrete or Masonry Shear Walls

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. 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.

As of this writing, there is a BSSC proposal to update the NEHRP provisions to include periods for structures with flexible diaphragms. The SEAOC Seismology committee supports this proposal.

Rational Period Determination

ASCE 7-16 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 to 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 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 7-16 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 7-16 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 these 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 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 7-16 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.

If the computed period exceeds the limits given in ASCE 7-16 Table 12.8-1, it is likely that the model incorporates inappropriate assumptions regarding stiffness and/or mass.

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_s."

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 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 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 7-16 Section 12.8.6.2 waives the upper limit on calculated period for purposes of drift analysis. 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_uT_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 $T \leq C_uT_a$ permits computation of drift based on forces and stiffnesses that are consistent.

According to ASCE 7-16 Section 12.8.6.1, 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. 12.8-5 ($V \geq 0.044 I_e S_{DS}$), although the other lower bound limit ($V \geq 0.5 S_1/(R/I_e)$ 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; it was added as a subsequent erratum after.

Soil-Structure Interaction

ASCE 7-16 section 12.9.7 allows modification of the building period in recognition of beneficial soil structure interaction effects. See Article 07.03.010 of the Blue Book for further information on this topic.

References

Applied Technology Council (1978). *Tentative provisions for the development of seismic regulations for buildings*, ATC-03-06. Applied Technology Council, Redwood City, CA.

ASCE (2006). *Minimum design loads for buildings and other structures,* ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA.

Bertero, V.V., Bendimerad, F.M., and Hoskins, T. (1988). *Fundamental period of reinforced concrete moment-resisting frame structures*, Technical report No. 87, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA, 1988.

Bendimerad, F.M, Shah, H.C., and Hoskins, T. (1991). *Extension of study on fundamental period of reinforced concrete moment-resisting frame structures,* Report No.96, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, CA, June 1991.

BSSC (2001). *NEHRP Recommended provisions for seismic regulations for new buildings and other structures, 2000 Edition, part 2—Commentary* (FEMA 369), Building Seismic Safety Council, Washington, DC.

BSSC (2004). *NEHRP Recommended provisions for seismic regulations for new buildings and other structures, 2030 Edition, part 2—Commentary* (FEMA 450), Building Seismic Safety Council, Washington, DC.

Goel, S. and Chopra, A.K. (1997). "Period formulas for moment-resisting frame buildings," *Journal of Structural Engineering*, ASCE, 123 (11).

Goel, R.K. and Chopra, A.K. (1997). "Period formulas for moment-resisting frame buildings, " *Journal of Structural Engineering*, ASCE, Vol. 123, No.11, November 1997, pp.1454-1461.

Goel, R.K., and Chopra, A.K. (1998). "Period formulas for concrete shear wall buildings," *Journal of Structural Engineering*, ASCE, Vol. 124, No. 4, April 1998, pp. 426-433.

ICC (2000). *2000 International building code*, International Code Council, Falls Church, VA.

Keywords
period of vibration

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Period Determination," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

@Seismicisolation

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.6 12.8 12.9 12.12 12.14 Chapter 16	1613	ASCE 41-17

Introduction

The purpose of structural analysis for new construction is to provide information necessary for producing an adequate design (typically forces and deformations). 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.

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 hazards. For most earthquake-resistant design, it is expected that ductility and detailing will accommodate uncertainties in ground motion estimates and in modeling idealizations. Whether analysis provides reliable information to the designer depends on whether the analytical model is able to capture key aspects of behavior.

In this respect, analysis for the design of new structures has a practical advantage over analysis of existing structures. Existing structures with non-ductile details, degraded materials, or irregular seismic resisting systems often require more accurate and careful analysis to gauge expected performance. Linear analysis of existing structures with non-ductile detailing can be unconservative and grossly inaccurate at stronger shaking intensities; typically, a non-linear analysis would be needed to more accurately estimate the distribution of forces in such systems.

The basic choices to make in determining which analysis procedure to use are whether to perform a static or dynamic analysis, and whether to solely rely on linear model or use both linear and nonlinear models. Linear procedures are not reliable for 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 that are substantially greater than estimates made using static procedures for buildings with more than 3 stories.

Analysis capabilities have improved considerably since the development of the digital computer in the 1960s, the advent of finite element methods, and the development of software for nonlinear analysis.

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 standards 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 warrant analytical results with greater accuracy. Performance expectations are being specified with greater precision as our analytical capabilities improve.

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 the known variabilities and uncertainties in actual capacities (e.g. member and material strengths and deformation limits) as well as demands (ground motions).

Code Approaches

ASCE 7-16 section 12.6 requires the proper structural analysis method to be selected from four 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.

		Structural Model	
		Linear	**Nonlinear**
Analysis Method	**Static**	Equivalent lateral force analysis (section 12.8)	
	Dynamic	Modal response spectrum analysis (section 12.9.1) Linear response history analysis (section 12.9.2, 16.1.2)	Nonlinear response history analysis (section 16)

Not listed in the above table are the nonlinear static procedures of ASCE/SEI-41. Other analysis procedures do exist and are allowed by ASCE 7-16 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 7-16 allows each of its four procedures for certain combinations of seismic design category (SDC), Risk Category, 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 7-16 Section 12.14).

ASCE 7-16 allows Simplified analysis for buildings not exceeding three stories in height in Risk 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 regular bearing wall or frame system buildings.

Equivalent Lateral Force Analysis

The Equivalent Lateral Force method is the most commonly used design procedure due to its simplicity and has been codified in various forms since the 1930s. 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 and roof 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 level equilibrate the base shear and are distributed over the height of the structure, roughly in proportion to the weight of each level and its height above the base. Inherent and accidental eccentricities are considered in the static analyses. (ASCE 7 Section 12.8) The effects of seismic actions are combined with the effects of gravity and other loads according to the load combinations of the governing standard to determine member design forces. Story drifts due to lateral forces are computed and must not exceed prescribed limits (ASCE 7 Section 12.12).

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 then, 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 significantly 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

ASCE 7 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 Risk 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 7-16 Table 12.6-1 indicates that the linear static procedures should not be used for buildings over 160 ft in height, with a fundamental period greater than 3.5 times the site period, Ts.

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.

The base shear calculated by a modal response spectrum analysis reduced by an R factor can be significantly smaller than the ELF base shear, therefore current codes require the modal results to be scaled to equal the ELF base shear. Note that previous versions of ASCE 7 only required that the modal response spectrum base shear be scaled to at least 85% of the ELF value. Currently, ASCE 7 sections 12.9.1.4 and 12.9.2.5 prohibit such reductions.

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 with 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. Similarly, since the analyses are based on a linear elastic model, the LDPs do not adequately address weak story mechanisms (ASCE 7 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 decreases.

Modal Response Spectrum Analysis

The orthogonality of the undamped modes of vibration allows the linear elastic response to be represented by superimposing the responses of the individual modes. Any number of modes may be used as long as at least 90% of the participating mass is represented (Section 12.9.1.1). 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. This combination of modal responses means that all results will be expressed as positive numbers, including member forces and displacements. For complex structures, this can obscure important aspects of building behavior, such as which elements undergo compression and which undergo tension. For this reason, the Seismology Committee recommends performing a parallel static analysis to confirm the sign values of the response spectrum analysis output.

Smoothed elastic design spectra (mainly, uniform hazard spectra) such as those specified by codes (ASCE 7-16 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.

Even if a response spectrum analysis is not used for design, performing a modal analysis on a computer model can be useful. The resulting periods and mode shapes can help determine whether the model assumptions, elements and their connections, and loads are properly represented. Modes with very long periods, or unusual mode shapes, can indicate that some model elements are not connected properly or assumed boundary conditions are unrealistic.

Linear Response History Analysis

In this 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 (Section 12.9.2).

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 cannot.

Nonlinear Static Procedures

Nonlinear static procedures comprise a variety of analysis methods that are based on static pushover analyses of a nonlinear model. 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. As a result, the nonlinear methods lack some of the conservatism that is inherently present in the linear approaches for design; whether such differences manifest in the actual design depends 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 nonlinear static procedures, a model of the structure is subjected to gradually increasing lateral forces. 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 of 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. This 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 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. Peer review is generally required for the nonlinear static and dynamic procedures.

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). 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.

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. However, the point at which inaccuracies become excessive is not clear. Table 12.6-1 does not allow nonlinear static procedures for these reasons (FEMA P-750, 2009). Where higher modes contributions are significant and accuracy is important, nonlinear dynamic analysis should be conducted.

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 inelastic response, and has become the benchmark by which the accuracy of other procedures is evaluated. Nevertheless, the results obtained are sensitive to particular ground motions used and to 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). 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. To ensure that nonlinear models are accurate, ASCE 7-16 requires a linear analysis and independent reviews (Sections 16.1.2 and 16.5).

As with linear response history analysis, the solution may be determined using modal superposition or by direct integration. Modal superposition is significantly faster than direct integration; however, it is less reliable. If an analysis is performed using modal superposition, the Seismology Committee recommends benchmarking the results of several time history records against a direct integration analysis. Key parameters to compare include overall building displacements and key member forces.

Chapter 16 of ASCE 7-16 gives detailed requirements for nonlinear response history analysis. For performance-based seismic design, some engineers follow the requirements of Section 7.4.4 in ASCE 41-17, adopting the appropriate performance objectives required for new buildings.

Nonlinear Dynamic Procedures are frequently employed for the analysis of tall buildings. ATC 72-1, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings*, provides detailed guidance for this analysis.

NIST has also published a useful guide in NEHRP Seismic Design Technical Brief No. 4, "Nonlinear Structural Analysis for Seismic Design."

New Thinking

Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2002) consists of a succession of independent nonlinear dynamic analyses using a suite of ground motions scaled to different amplitudes. Peak responses, such as peak roof displacement or maximum interstory drifts, are plotted as a function of the intensity of the excitation, which may be expressed by the scaled spectral acceleration at the first mode period, spectral velocity, 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.

Performance-Based Seismic Engineering is one of the most significant new developments in structural analysis. For example, it has been used extensively in FEMA P-695 studies to justify R values for new systems.

For a detailed discussion of performance-based seismic design, see article 01.04.010.

References

ASCE 7-16 (2017). Minimum Design Loads for Buildings and Other Structures, Supplement 1, American Society of Civil Engineers, Reston, Virginia.

ASCE/SEI 41-17 (2017), Seismic Rehabilitation of Existing Buildings, Supplement 1, American Society of Civil Engineers, Reston, Virginia.

Aschheim, M., and Black, E. (2000). "Yield Point Spectra for Seismic Design and Rehabilitation," Earthquake Spectra, Earthquake Engineering Research Institute, Vol. 16, No. 2, May, pp. 317-335.

Aschheim, M.A. (2002). "Seismic Design Based on the Yield Displacement," Earthquake Spectra, Earthquake Engineering Research Institute, Vol. 18, No. 4, Nov., pp. 581-600.

Aschheim, M.A., Black, E. F., and Cuesta, I. (2002). "Theory of Principal Components Analysis and Applications to Multistory Frame Buildings Responding to Seismic Excitation," Engineering Structures, Elsevier Science, 24(8):1091-1103.

Aschheim, M., Tjhin, T., Hamburger, R., Comartin, C., and Inel, M. (2007). "The Scaled Nonlinear Dynamic Procedure," Engineering Structures, 29(7):1422-1441.

ATC (1996), "Seismic Evaluation and Retrofit of Concrete Buildings," ATC-40, Proposition 122 California Seismic Safety Commission & Applied Technology Council, Redwood City, California.

ATC (2010), Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings," ATC 72-1, Applied Technology Council, Redwood City, California.

Aydınoğlu, M.N. (2003). "An Incremental Response Spectrum Analysis Based on Inelastic Spectral Displacements for Multi-Mode Seismic Performance Evaluation", Bulletin of Earthquake Engineering, Vol. 1, No. 1, pp. 3–36.

CBC (2007). 2007 California Building Code, California Building Standards Commission, Sacramento, California.

CBC (2019). 2019 California Building Code, California Building Standards Commission, Sacramento, California published by the International Code Council.

Chopra, A. K., and Goel R. K. (1999). Capacity-Demand-Diagram Methods Based on Inelastic Design Spectrum, Earthquake Spectra, Vol. 15, No. 4, pp. 637-656.

Chopra, Anil K.; Goel, Rakesh K., A modal pushover analysis procedure for estimating seismic demands for buildings, Earthquake Engineering & Structural Dynamics, 31, 3, Mar. 2002, pages 561-582.

Clough, R. W., and Penzien, J. (1993). Dynamics of Structures, 2nd edition, McGraw-Hill, New York.
Deierlein, Gregory G., Reinhorn, Andrei M., and Willford, Michael R. (2010). "Nonlinear structural analysis for seismic design," NEHRP Seismic Design Technical Brief No. 4, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 10-917-5.

Eberhard, M. O.; Sozen, M. A. , Behavior-based method to determine design shear in earthquake-resistant walls, Journal of Structural Engineering, 119, 2, Feb. 1993, pages 619-640.

Fajfar, P. (1999). Capacity Spectrum Method Based on Inelastic Demand Spectra, Earthquake Engineering and Structural Dynamics, Vol. 28, No. 9, pp. 979-993.FEMA 273 (1997). NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273, October 1997, National Earthquake Hazard Reduction Program, Washington, D.C.

FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, November 2000, National Earthquake Hazard Reduction Program, Washington, D.C.

FEMA, (2003) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA 450, National Earthquake Hazard Reduction Program, Washington D.C.

FEMA P-750, (2009) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,

FEMA P-750, National Earthquake Hazard Reduction Program, Washington D.C.

FEMA 440 (2005). Improvement of Nonlinear Static Seismic Analysis Procedures, Report No. FEMA-440, Federal Emergency Management Agency, Washington, D.C.

Freeman, S. A. (1978). "Prediction of response of concrete buildings to severe earthquake motion," Special Publication SP-55, American Concrete Institute, p. 589–605.

Hernández Montes, E., Kwon, O-S, and Aschheim, M., (accepted 2003), "An Energy-Based Formulation for First- and Multiple-Mode Nonlinear Static (Pushover) Analyses, Journal of Earthquake Engineering.

Inel, M., Tjhin, T., and Aschheim, M.A. (2003). "The Significance of Lateral Load Pattern in Pushover Analysis, Fifth National Conference on Earthquake Engineering, Istanbul, Turkey, May 26 – 30.

Kunnath, Sashi K.; et al., Validation of evaluation methods and acceptance criteria in evolving performance-based seismic codes, SMIP2000 Seminar on Utilization of Strong-Motion Data: Sacramento, California, September 14, 2000: Proceedings, California Division of Mines and Geology, Sacramento, [2000], pages 39-63.

Krawinkler, Helmut; Seneviratna, G. D. P. K., Pros and cons of a pushover analysis of seismic performance evaluation, Engineering Structures, 20, 4-6, Apr.-June 1998, pages 452-464.

Lew, H. S.; Kunnath, Sashi K., Evaluation of analysis procedures for performance-based seismic design of buildings 12th World Conference on Earthquake Engineering [Proceedings] [computer file], New Zealand Society for Earthquake Engineering, Upper Hutt, New Zealand, 2000, Paper No. 1005.

Scott, Kimberly; Roeder, Charles; MacRae, Gregory, Seismic assessment of concrete-filled steel frame substations, Technical Council on Lifeline Earthquake Engineering Monograph No. 16, Optimizing Post-Earthquake Lifeline System Reliability, Proceedings of the 5th U.S. Conference on Lifeline Earthquake Engineering, American Society of Civil Engineers, Reston, Virginia, Aug. 1999, pages 257-266.

Vamvatsikos, Dimitrios; Cornell, C. Allin, Incremental dynamic analysis, Earthquake Engineering & Structural Dynamics, 31, 3, Mar. 2002, pages 491-514.

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Analysis Procedures," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2015 IBC reference section(s)	Other standard reference section(s)
12.3.4.1 12.4.3 12.8 12.10	1605.1	ACI 318-14, 12.5.4, 18.12.7

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 in addition to 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.

In the 1994 Northridge Earthquake, failure of collector elements, which contained insufficient reinforcement, was observed in multiple pre-cast parking structures. Collector elements were observed to have yielded early on, rendering the collector elements unable to transmit the lateral forces to walls.

The intent of the building code design provisions is to ensure that collector elements remain essentially elastic during a design basis earthquake. Unfortunately, these provisions are an accumulation of different requirements spanning the past 60 years, resulting in a set of complex and counterintuitive rules. For some building configurations, nonlinear response history analysis will result in collector design forces that are significantly higher than those given by the code. The Seismology Committee supports ongoing research into collector behavior, with the ultimate goal of more rational collector design requirements in the building code.

This paper only applies to cast-in-place concrete collectors. Diaphragms, including those with cast-in-place concrete collectors, are permitted to use new alternative provisions of ASCE 7-16, Section 12.10.3. Precast concrete collectors are required to comply with new requirements in Section 12.10.3. However, that section is beyond the scope of this paper.

Recommended References. The Second Edition of NEHRP Technical Brief No. 3 provides an extensive discussion of the concepts underlying collector design according to ASCE 7-16. However, minor referencing errors exist in the brief's Section 5.1.2 (a) and (b) where references to Section 12.4.3.2 should be to 12.4.3. Also, the references to four load combinations should be to five load combinations.

In addition, the 2015 Edition of the SEAOC Seismic Design Manual provides a detailed design example for concrete collector design according to ASCE 7-10. However, applications of overstrength and redundancy to collectors have changed in the 2016 edition of ASCE 7.

Applications of Overstrength, Redundancy and the 25% Irregularity Increase

ASCE 7-16, Section 12.10.2.1 requires that collectors be design for the maximum of the following:

1. Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by the equivalent lateral force procedure of Section 12.8 or the modal response spectrum analysis procedure of Section 12.9.1;
2. Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by Eq. (12.10-1); and
3. Forces calculated using the load combinations of Section 2.3.6 with seismic forces determined by Eq. (12.10-2).

#1 is relatively straightforward. In some multi-story buildings, this provision will govern collector design forces, particularly at the upper levels and in buildings with low R values.

#2 requires that diaphragm design forces Fpx be considered for collector design forces per equation 12.10-1.

12.10.1.1 Diaphragm Design Forces. Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis but shall not be less than that determined in accordance with Eq. (12.10-1) as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10\text{-}1)$$

where

F_{px} = the diaphragm design force at level x;
F_i = the design force applied to level i;
w_i = the weight tributary to level i; and
w_{px} = the weight tributary to the diaphragm at level x.

The force determined from Eq. (12.10-1) shall not be less than

$$F_{px} = 0.2 S_{DS} I_e w_{px} \qquad (12.10\text{-}2)$$

The force determined from Eq. (12.10-1) need not exceed

$$F_{px} = 0.4 S_{DS} I_e w_{px} \qquad (12.10\text{-}3)$$

Note that Equation 12.10-3 does not apply to collector design. It is intended to apply to other diaphragm elements, such as chords. Also, note that collectors located at the perimeter of a building can act as chords. In this case, these elements must be checked against each set of forces.

#3 requires use of the load combinations of Section 2.3.6. This section includes load combinations with and without overstrength. However, Section 12.10.2.1 does not specify "including overstrength." Therefore, the omega factor does not apply to #3.

These interpretations are consistent with NEHRP Technical Brief No. 3.

There are two additional factors to consider in the calculation of collector forces: the redundancy factor (per Section 12.3.4.1), and the additional 25% increase per Section 12.3.3.4.

The tables below summarize the applicable factors for collector design.

Table 1: Collector Force Factors per 12.10.1.1, 12.10.2 & 12.3.3.4 for Structures Not Braced Entirely by Light-Frame Walls

Force Type	Modifiers in 12.10.2 and 12.3.4.1		Do Irregularities: H1a, H1b, H2, H3, H4 or V4 Occur per 12.3.3.4?	
	Overstrength Ω_o	Redundancy ρ	Yes	No
1. Fx from ELF 12.8 or MRSA12.9	Ω_o	1.0	1.0	1.0
2. Fpx Diaphragm Force Eq. 12.10-1	Ω_o	1.0	1.0	1.0
3. Fpx, min Eq. 12.10-2	1.0	ρ	1.25	1.0
Transfer Forces Only in Type H4 Irregular Structures per 12.10.1.1	Ω_o	1.0	Add Modified Transfer Forces to Inertial Forces from Modified Collector Forces from 1, 2, or 3 above	

Table 2 Collector Force Factors per 12.10.1.1, 12.10.2 & 12.3.3.4 for Structures Braced Entirely by Light-Frame Walls

Force Type	Modifiers in 12.10.2 and 12.3.4.1		Do Irregularities: H1a, H1b, H2, H3, H4 or V4 Occur per 12.3.3.4?	
	Overstrength Ω_o	Redundancy ρ	Yes	No
1. Fpx from Diaphragm Force, Eq. 12.10-1	1.0	ρ	1.25	1.0
2. Fpxmin, Eq. 12.10-2	1.0	ρ	1.25	1.0
3. Fpxmax Eq. 12.10-3	1.0	ρ	1.25	1.0
Transfer Forces Only in Type H4 Irreg. Structures per 12.10.1.1	1.0	ρ	Add Modified Transfer Forces to Inertial Forces from Modified Collector Forces from 1, 2, or 3 above	

Effects of Building Geometry. Consider the building plan shown in Figure 1, below. This is a concrete shear wall building with a central core.

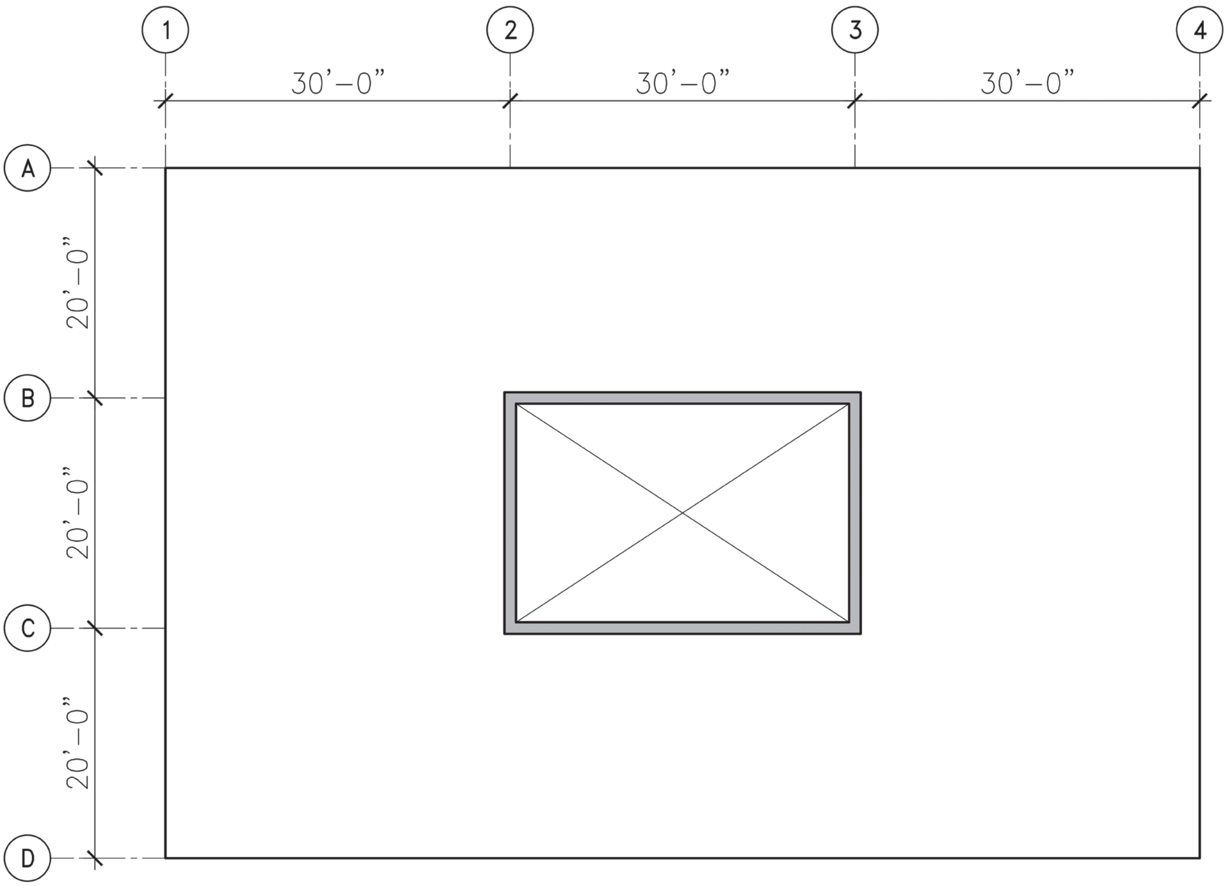


Figure 1. Building Layout

This discussion will focus on the collector along line C.

For the purposes of this example, assume that the total seismic force (with the Ω_0 factor included) acting on the diaphragm at this level is 190 kips. Due to accidental torsion effects, walls B and C each resist 100 kips. Furthermore, assume that there is slab reinforcing in excess of what is required for gravity. In this case, it can be desirable to not run the collectors the full length of the building. (This assumes that the excess slab reinforcing has sufficient lap length with the designated collector elements, per ACI requirements.) In this case, the designated collector elements might look like the layout shown in Figure 2.

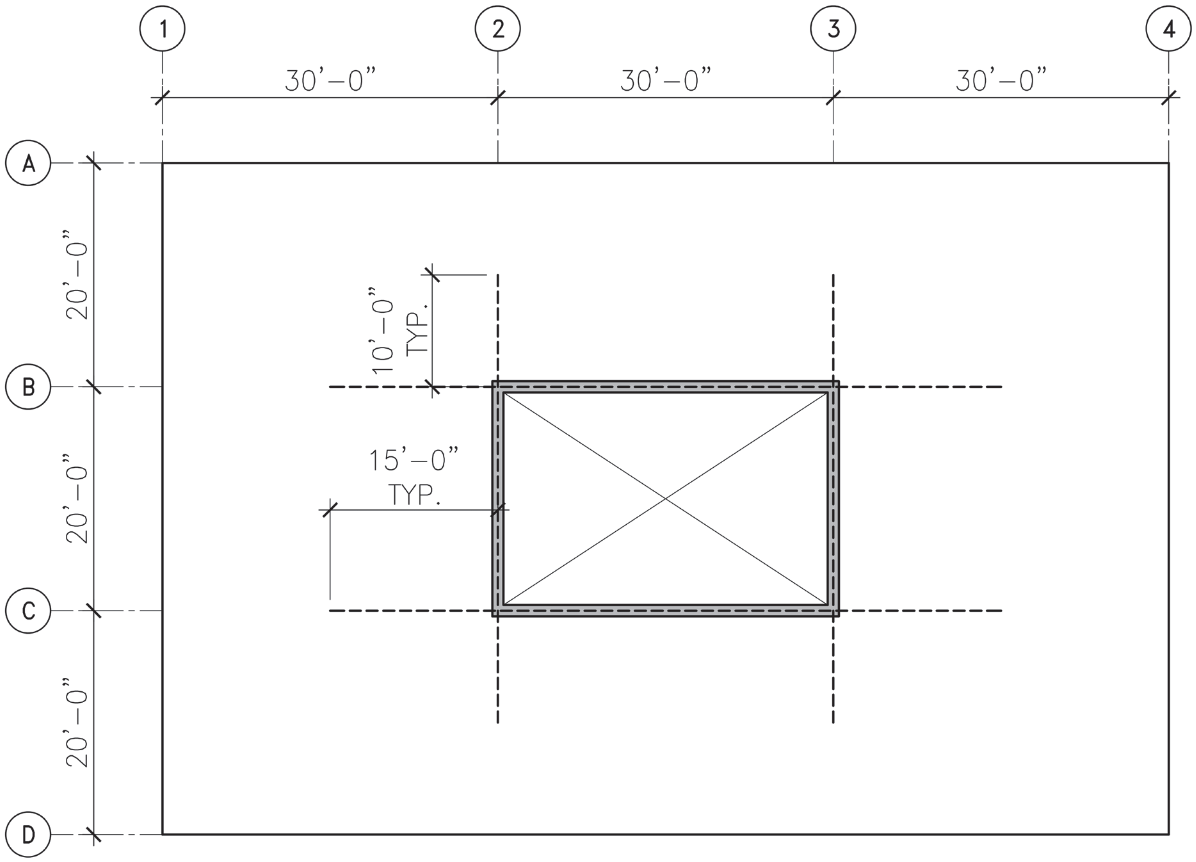


Figure 2. Partial Length Collectors

Applying classical collector design to this layout would result in the following diagrams for the collector along line C:

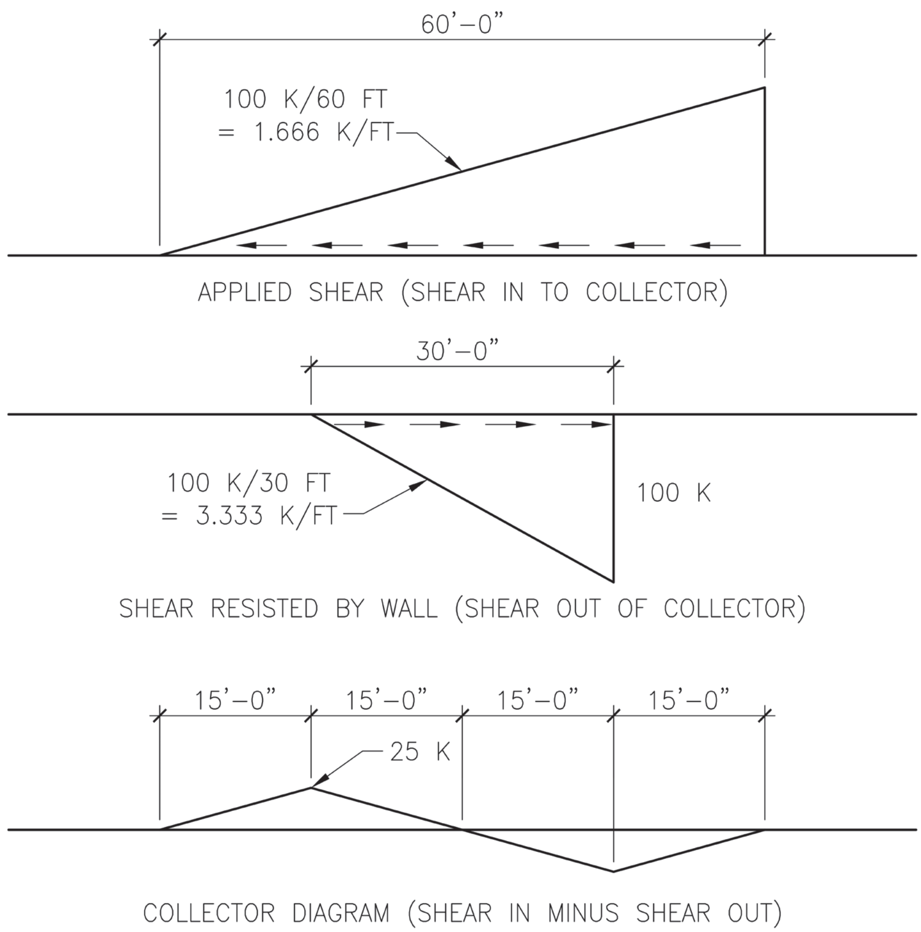


Figure 3. Partial-Length Collector Diagrams

This results in a maximum collector force of 25 kips. However, this is unconservative.

Regardless of whether the collector consists of distributed slab reinforcing or designated collector bars, the true collector force depends on the total collector length. Applying proper collector design to a full-length collector would result in the following diagrams:

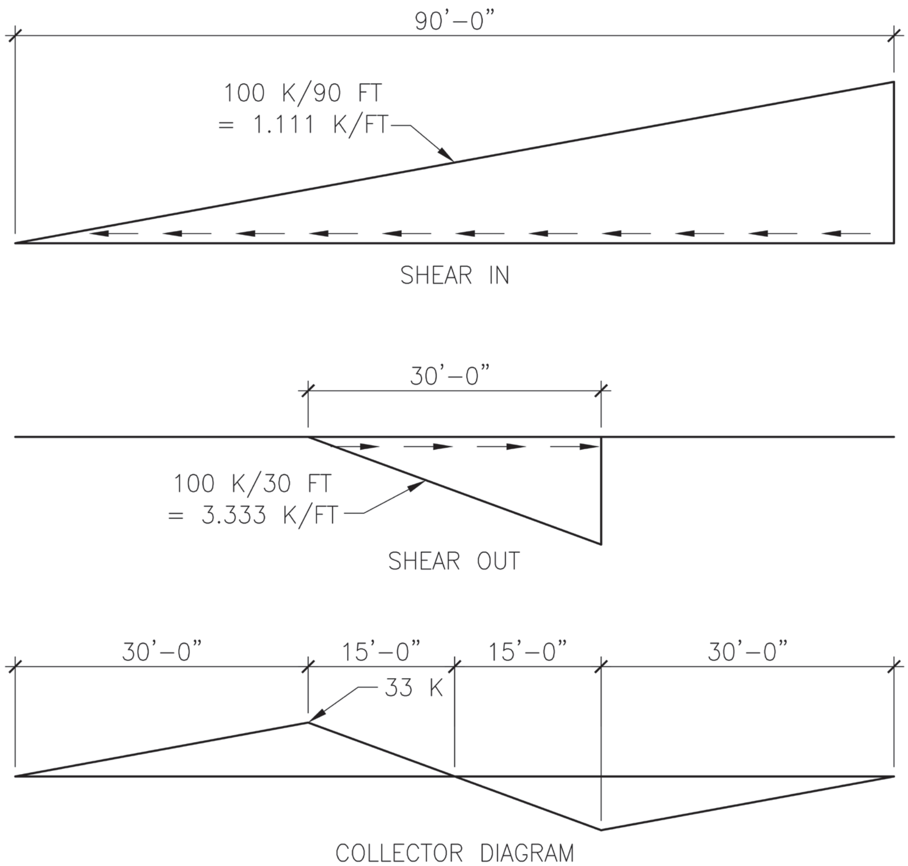


Figure 4. Full-Length Collector Diagrams

This results in a maximum collector force of 33 kips. However, this is also unconservative.

The "shear in" to the collector depends on the mass tributary to each portion of the collector. This tributary mass is shown as a shaded region in Diagram 5, below.

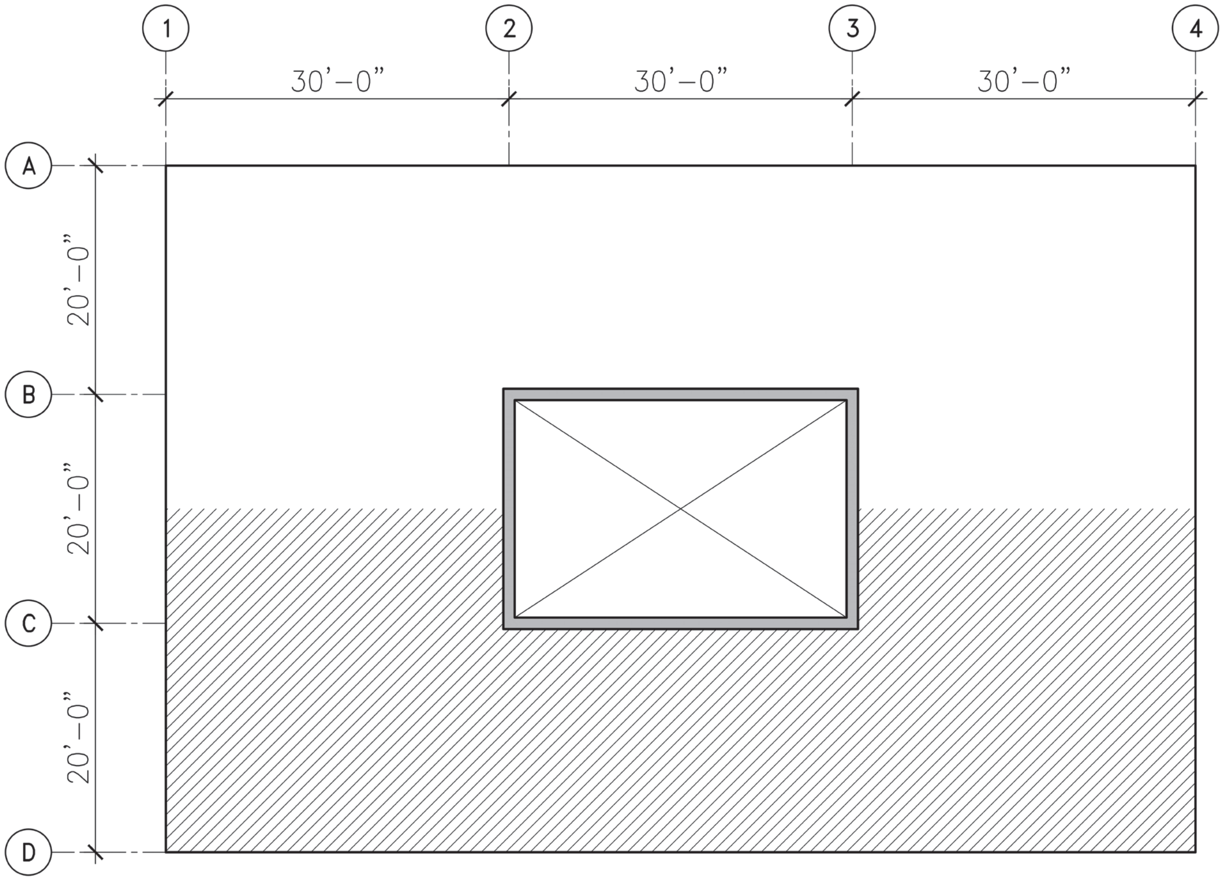


Figure 5. Mass Tributary to Collector at Line C

(Note: This analysis assumes that the collector tributary mass is proportional to tributary area. If cladding loads are significant, or if the slab thickness varies, this should also be considered.)

The slope of the "shear in" line is 50% higher in the end bays than it is in the center bay, because the tributary width is 50% greater for these two bays. This results in the collector diagrams shown in Figure 6.

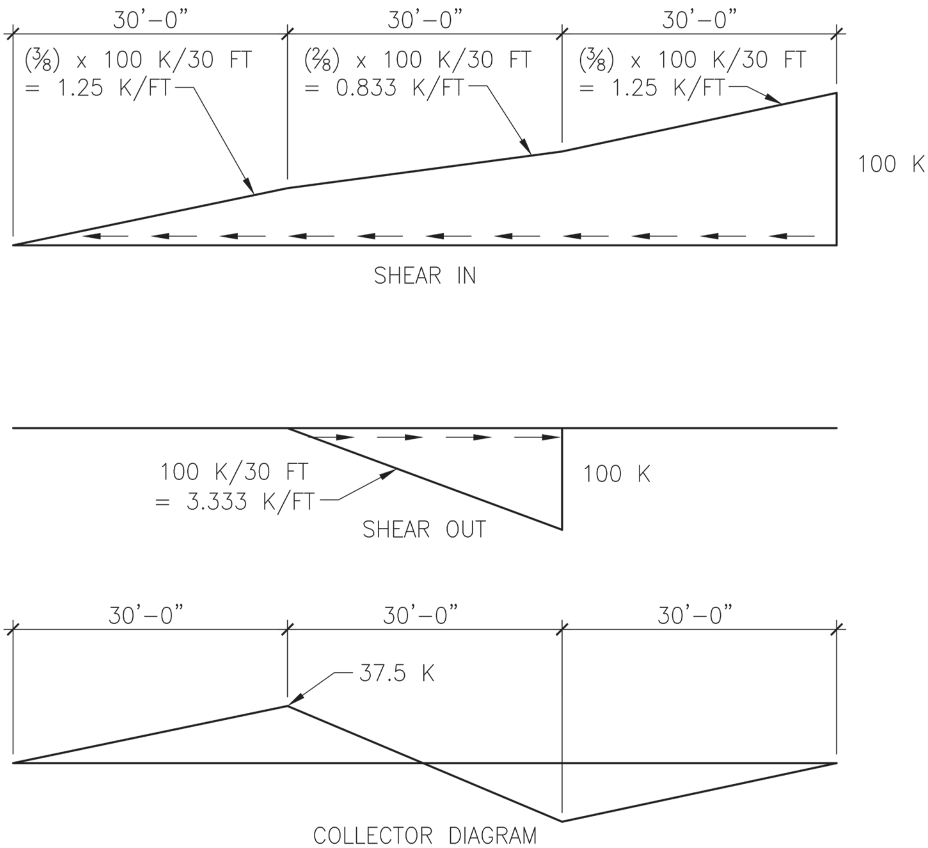


Figure 6. Collector Diagrams Incorporating Tributary Mass

This results in a maximum collector force of 37.5 kips. While this is a modest increase in collector force, the effect of tributary mass can be significant for some building configurations.

Now, assume that the wall above the diaphragm level we just analyzed has an opening in it, as shown in Figure 7.

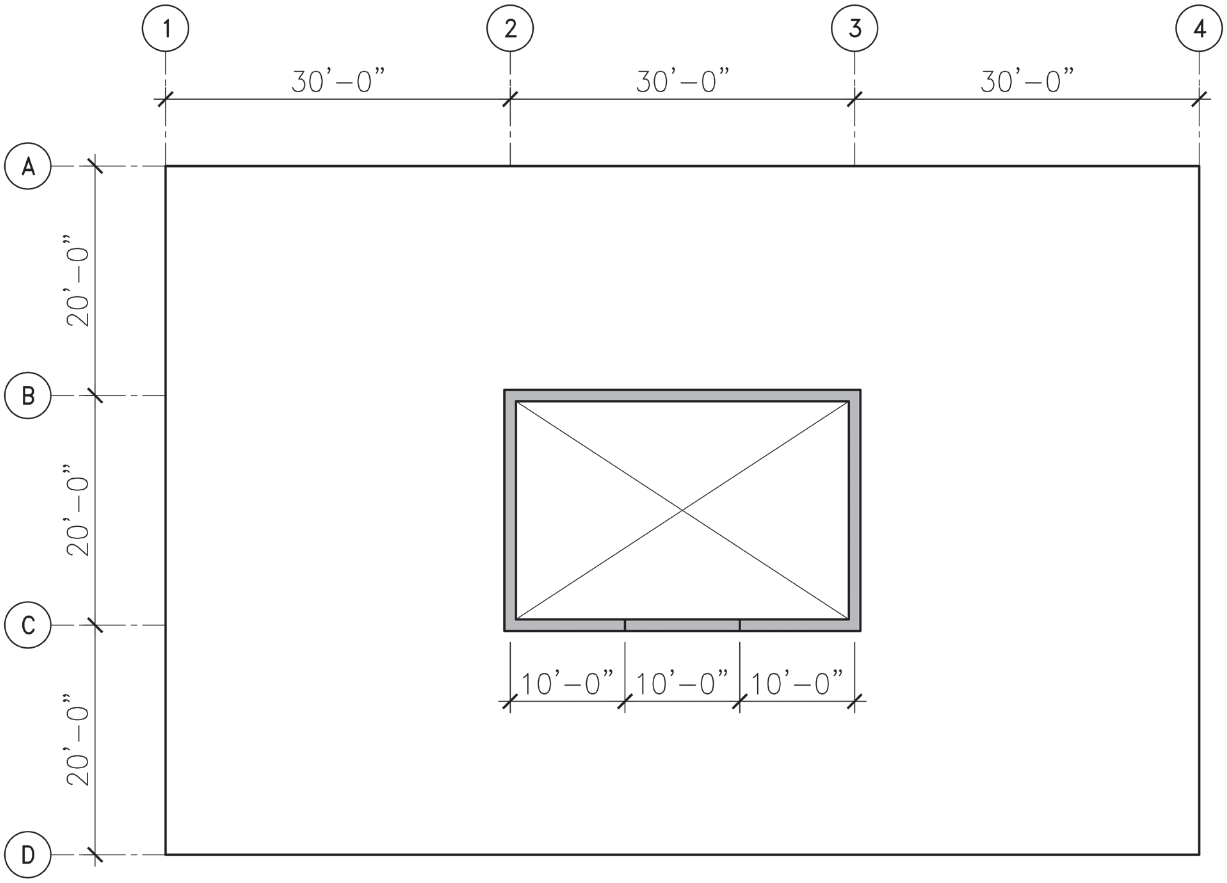


Figure 7. Floor Plan Showing Wall Opening

A partial wall elevation, including wall shear forces, is shown in Figure 8.

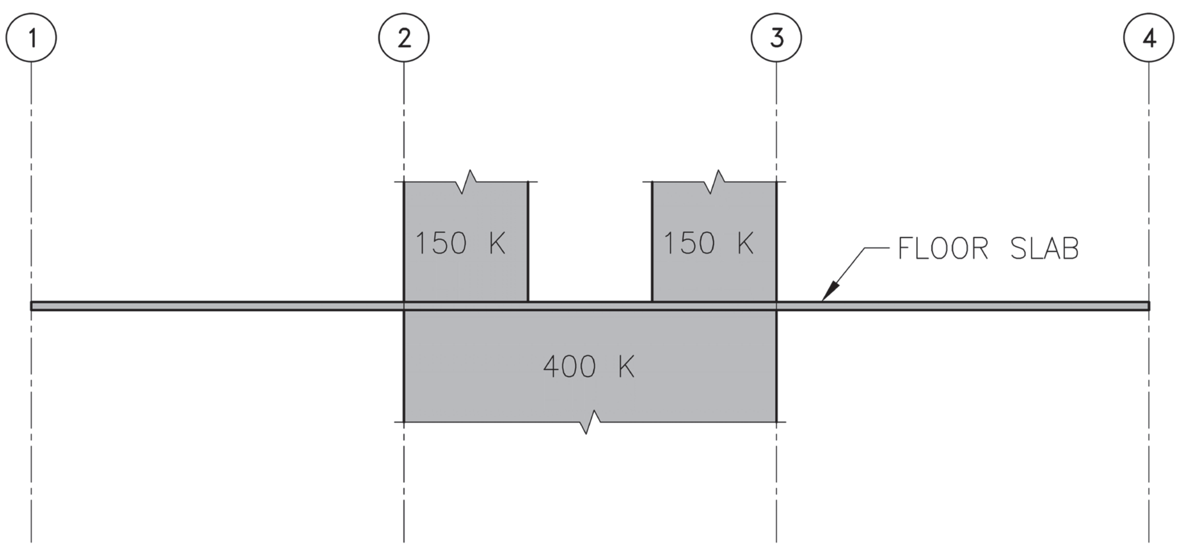


Figure 8. Wall Elevation Showing Shear Forces

For the purposes of this example, assume that the wall shears have been multiplied by the appropriate factors to design the collector.

In this case, it is necessary to explicitly consider the shear in the walls above as an additional force, as shown in Figure 9.

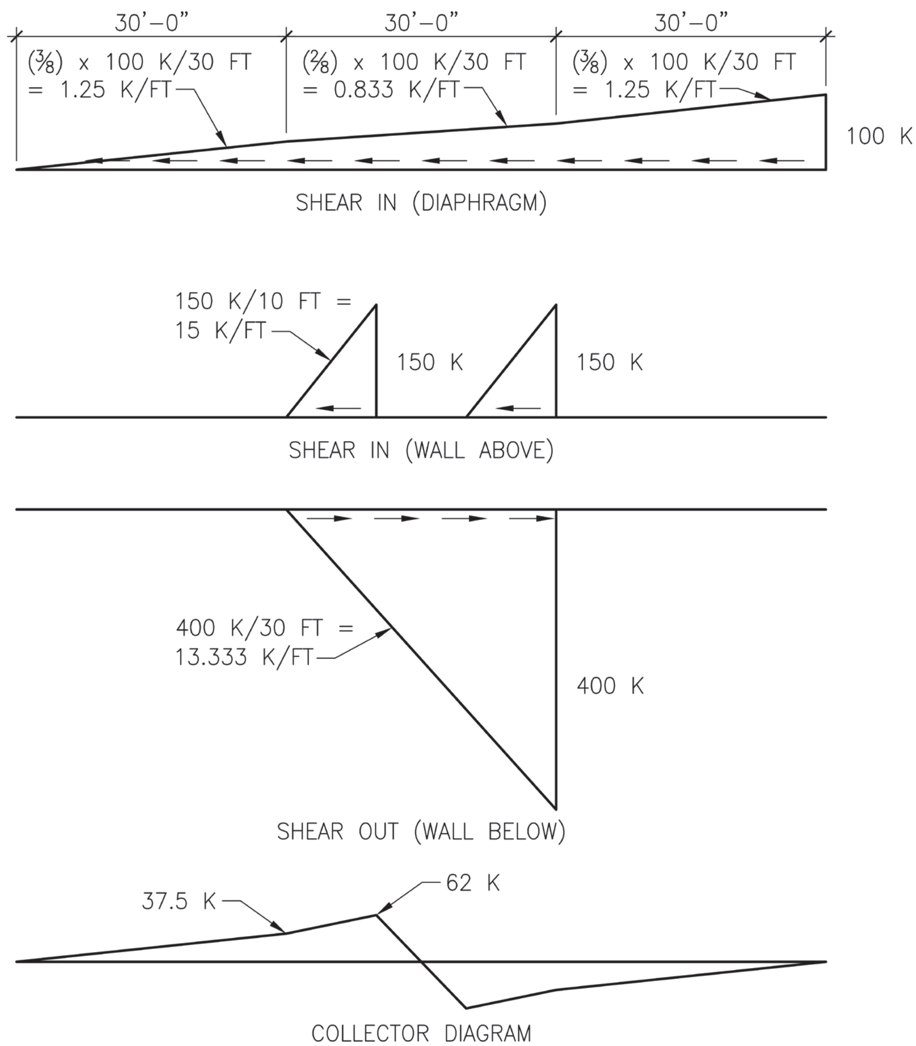


Figure 9: Collector Diagrams Incorporating Tributary Mass and Wall Opening

In this example, the maximum collector force occurs within the wall. This method of understanding shear in and shear out is very useful for complex wall elevations with multiple openings.

It is also important to consider the slope of the collector diagram segments. This is particularly important for collector designs that rely on bolted connections to diaphragms. The shear demand on these anchors is equal to the slope of the collector diagram. (For example, if the slope of the diagram is 2 kips per foot, and the anchors are spaced at 1 foot on center, each anchor will resist a shear demand of 2 kips.)

Transfer Forces in Diaphragms. 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. 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. When elements of the SFRS shed load to the surrounding diaphragm, the corresponding axial elements are designated as distributors, rather than collectors.

A detailed discussion of transfer forces in diaphragms is beyond the scope of this article. The following documents address design considerations for transfer forces:

- ASCE 7-16, Sections 12.3.3.4, 12.10.1.1, 12.10.2, and 12.3.4.1
- NEHRP Seismic Design Technical Brief No. 3, Second Edition "Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors", Section 5
- PEER TBI Guidelines, Chapter 4 and Appendix D

Response History Analysis. In both linear and nonlinear response history response analysis, collector forces are explicitly calculated from the model output, rather than using the equations given in ASCE 7-16 Section 12.10.1.1. This presents some additional complications for the designer. (The following discussion assumes a concrete shear wall building; similar concepts would apply to different seismic systems.)

In order to determine the shear in at a given level, it can be tempting to calculate the maximum shear force in the wall below the level, and subtract it from the maximum shear force in the wall above. However, this is only correct if the maximum shear in the wall above occurs *at the same timestep* as the maximum shear in the wall below. Due to dynamic effects, this isn't necessarily correct. Instead, the designer must calculate the difference in wall shears *at each time step*, and then use the maximum value of this difference. This is true for each element of the SFRS, at each level.

The situation becomes more complex if multiple SFRS elements exist along the same gridline. Again, the shear imparted to each element of the SFRS is dependent upon the timestep. In such cases, it may be necessary to automate a program that calculates the collector demands for each timestep, and then designs the collector for the resulting envelope of forces. Refer to ASCE 7 Section 12.9.2.6. Note however, that scaling collector design forces by Ie/R is not necessarily conservative and that transfer forces for collectors should be determined by the SRSS of collector forces from each vibration mode. For more advice, see NEHRP Seismic Design Technical Brief No. 3, Sections 5.1.4 and 5.2.

Direct Connection of Diaphragm to SFRS. ASCE 7-16 Figure 12.10-1 indicates that no "collector" is required when the diaphragm is directly connected to a full-length shear wall. The overstrength factor does not apply to this direct connection.

Strut and Tie Models. An alternate approach to collector design would be to use the strut and tie model, as covered in detail in Chapter 23 of ACI 318-14. It is the opinion of the SEAOC Seismology Committee that the seismic load effect including overstrength Factor of ASCE 7-16 Section 12.4.3 should 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.

Detailing of Reinforcement

Planar elements, such as shear walls and diaphragm slabs, have 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.

Compression and Confinement. ACI 318-14 Section 18.12.7.5 specifies that collector elements with compressive stresses exceeding 0.5f'c (when overstrength applies) and 0.2f'c (when overstrength factors do not apply) at any section shall have confining reinforcement. (This requirement is similar to the confined boundary element provisions of 18.10.6.3.)

Confinement of highly stressed collector elements has a secondary benefit. When these elements resist tension, the confining reinforcement helps to prevent longitudinal splitting of the concrete slab. If collectors are sized to avoid confinement reinforcement, excessive cracking may still occur in regions of high collector strain and its consequences should be considered.

Considerations for Eccentric Collectors. The preceding discussion assumes that each collector element is centered on the corresponding the SFRS element. In some cases, it is advantageous to locate collector elements offset from the wall. (For example, this can mitigate problems with congestion of reinforcement at the wall/slab interface.)

If a collector is not centered on the corresponding SFRS element, the designer must consider two additional load path issues.

1) The slab must be able to deliver the entire "shear in" force to the wall. Where a concrete slab meets a shear wall, this can be achieved through dowels acting in shear friction.
2) The eccentricity between the center of the collector and the center of the wall results in a moment that must be resisted by additional slab reinforcement. The designer is cautioned against "double counting" shear friction reinforcement as moment-resisting reinforcement. In addition, the designer is cautioned against "double counting" reinforcement provided for seismic forces and reinforcement required to resist gravity forces.

Other Design Considerations. Other issues to be considered in collector design include displacement compatibility, accumulated strain in long collectors, and re-entrant corner considerations. These are all covered in detail in the NEHRP Seismic Design Technical Brief No. 3.

References

ACI (2014). *Building code requirements for structural concrete (ACI 318-14) and commentary*, American Concrete Institute, Farmington Hills, MI.

ASCE (2017). ASCE/SEI 7-16, *Minimum design loads and Associated Criteria for Buildings and Other Structures,* American Society of Civil Engineers, Reston, VA.

ATC (1978). *Tentative provisions for the development of seismic provisions for buildings*, ATC-3-06, Applied Technology Council, Redwood City, CA.

Corley, G., 1996, "Northridge Earthquake of January 17, 1994 Reconnaissance Report – Concrete Parking Structures," *Earthquake Spectra*, EERI Publication 95-03/2, Earthquake Engineering Research Institute, Oakland, California.

Moehle, Jack P., Hooper, John D., and Meyer, Thomas R. (October 2016). "Seismic design of cast-in-place concrete diaphragms, chords, and collectors: a guide for practicing engineers," NEHRP Seismic Design Technical Brief No. 3, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 16-917-42

SEAOC Seismology Committee (2007). *Recommended lateral force requirements and commentary*, eighth edition, Structural Engineers Association of California, Sacramento, CA.

Keywords
collectors
concrete slab

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Concrete Slab Collectors," March, 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Fire Sprinklers

ASCE 7-16reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
13 13.6.7	1632 1632.5	NFPA 13 (2016)

Background

Fire sprinkler systems in buildings are unique among distributed piping systems. Other piping systems (water distribution, waste, process etc.) are usually in discrete areas of the structure. Fire sprinklers are, by design, ubiquitous in the building. This presents design challenges to ensure that the system has sufficient clearances from adjacent structural and nonstructural components to avoid damaging impacts and failures of the sprinkler system. Particular attention to the support and bracing of fire sprinkler systems in both ordinary occupancy and enhanced performance buildings is required to ensure their post-earthquake function.

> "Failures of suspended fire protection piping have resulted in both direct and indirect property loss following earthquakes. Some of these systems have failed or fallen and had to be replaced. More costly are the failures of sprinkler piping, connections, or sprinkler heads. These have resulted in the release of great volumes of water in plenum or occupied spaces. Flooded plenums have resulted in collapsed ceilings which cause the consequent loss of property and disruption of operations. In extreme cases, entire floors or buildings were abandoned as a result of the water damage. Flooding in occupied spaces has resulted in water damage to furniture, files, computer equipment, and interior finishes. As fire sprinkler lines are widespread in occupied spaces, this type of failure has been one of the most costly types of nonstructural damage." (FEMA E-74, 2012)

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. A loss of building power can render a sprinkler system inoperable, since most fire pumps and control panels rely on electricity to function. 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 a separation joint 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) and the 2014 South Napa Earthquake (FEMA P-1024, 2015). 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).

A complete fire sprinkler system includes components outside the typical purview of the structural engineer, such as the underground supply lines and their sources of pressurized water in the local water utility system. Most fire protection systems include pumps, control panels and other components that require power to operate. Tall buildings may contain firewater storage tanks. Other components of the overall fire sprinkler system include alarms, gauges, and valves. The scope of this article is limited to the piping in the building or industrial structure, and its associated supports and seismic bracing.

Design Issues

Piping that is hung from the floor or roof and 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 can be minimized by a robust bracing design. Braced pipes may also be vulnerable to damage if they can vibrate and displace excessively between brace points or if braces have too little rigidity.

Braces must be designed for seismic forces 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 should be designed with reduced capacities, perhaps by as much as 25 percent. 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 motions. A braced system can also be damaged by impacts from adjacent inadequately braced nonstructural systems such as suspended ceilings. Even if ceiling bracing is present, ceiling movements may be excessive compared to a stiffer piping system.

General design strategies for seismic protection of piping systems include:

- Braces to control relative displacements
- Flexible connections to relieve stresses at critical locations
- Providing clearance between sprinkler heads and adjacent components such as ceilings to minimize impact damage to piping and sprinkler-heads (NFPA 13 section 9.3.4 sets out minimum clearances)
- 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 near changes in direction and at ends, and not exceeding 80 ft spacing in straight runs
- Branch line restraint
- Adequate clearance of pipes through floors
- 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, suspended mechanical units, other nonstructural components and building contents that could affect sprinkler piping

Code Approaches and Interpretations

ASCE 7-16 section 13 provides general design requirements for nonstructural components. Section 13.6.7 states "Fire protection sprinkler piping, pipe hangers and bracing designed and constructed in accordance with NFPA 13 shall be deemed to meet the force and displacement requirements of this section." NFPA section 9.3.5.9 is used to determine the seismic forces for design of fire sprinkler systems. There is a method described in NFPA 13 Section 9.3.5.9.3, or the designer can use forces determined in accordance with section ASCE 7 Section 13.3.1 (per NFPA 13 Section 9.3.5.9.4). Per NFPA 13 Section 9.3.5.9.2 Wp is taken as 1.15 times the weight of the water filled pipe.

Brace type and size can be selected from the tables in NFPA 13 Section 9.3.5.11 or be designed by a qualified engineer as specified in section 9.3.5.11.9. The brace arm types in the tables are limited to Schedule 40 pipe, hot rolled steel angles, all thread rods and steel plate straps. The tables values are in allowable stress design and vary based on the installed angles and kl/r ratios.

Brace attachments are discussed in section 9.3.5.12 and can be chosen in tables 9.3.5.12.2(a) through 9.3.5.12.2 (i). 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 and based on the force levels calculated in ASCE 13.3.1 including the appropriate overstrength factors as required.

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. In most areas of the Western United States, such "sway bracing" wasn't installed until the 1940s. Because of insurance industry requirements, some seismic bracing may be found in older buildings that otherwise had no other nonstructural bracing. This is because the building codes at that time 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 and Upcoming Code Provisions

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 estimating fragility curves (Bachman et al. 2004). Research using those protocols is outside the scope of that FEMA-funded project.

In 2018 NIST published document GCR 17-917-44 (ATC 120) titled "Recommendations for Improved Seismic Performance of Nonstructural Components". It recommends changes to both design philosophy and design equations for nonstructural components including fire sprinkler systems. Strategies suggested in the report include providing flexible sprinkler head connections, providing a restraint system to limit pipe system movement and prevent adverse interactions with adjacent systems, and providing additional and more accessible shut off valves for sprinkler systems. The NEHRP IT5 committee is currently proposing a revised nonstructural seismic force design equation for consideration by the ASCE 7 Standards Committee that intends to more closely factor in the building's seismic force-resisting system when determining forces for nonstructural components.

References

ASCE (American Society of Civil Engineers) (2010). *ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, Including Supplement No. 1*. American Society of Civil Engineers, Reston, VA

Ayres, J. M., Sun, T.Y., and Brown, F.R. (1973). "Nonstructural damage to buildings," *The Great Alaska Earthquake of 1964: Engineering*, National Academy of Sciences, Washington, D.C.

Ayres & Ezer Associates Consulting Engineers (1996). *Northridge Earthquake Hospital Water Damage Study*, California Office of Statewide Health Planning and Development, Sacramento, CA. Similar documentation is provided in Ayres, J. M., and Philips, R. J. (1998), "Water Damage in Hospitals Resulting From Northridge Earthquake," *ASHRAE Transactions*, American Society of Heating, Refrigerating and Air-Conditioning Engineers, vol. 104, part 1.

Bachman, R. et al. (2004). "Engineering demand parameters report for nonstructural components." Applied Technology Council, Redwood City, CA.

California Building Standards Commission (2016). *2016 California Building Code* (California Code of Regulations, Title 24). California Building Standards Commission, Sacramento, CA.

FEMA E-74 (2012). *Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide,* Federal Emergency Management Agency, Washington, D.C.

Fleming, R. P. (1998). *Analysis of Fire Sprinkler Systems Performance in the Northridge Earthquake*, NIST-GCR-98-736, National Institute of Standards and Technology, Gaithersburg, MD.

FM Global (2001). *Seismic Sway Brace Components for Automatic Sprinkler Systems, Approval Standard Class 1950*, FM Global Research, Norwood, MA.

FSAB (Fire Sprinkler Advisory Board) (1994). *A Report on the Performance and Damage Associated with the Automatic Fire Sprinkler Systems Resulting from the Northridge Earthquake (6.7 Magnitude) and the Subsequent Aftershocks* (FSAB-2), Fire Sprinkler Advisory Board, Cerritos, CA.

Malhotra, P. K., Senseny, Paul, Braga, A. C., and Allard, R. L. "Testing sprinkler-pipe seismic-brace components," *Earthquake Spectra*, 19 (1), February 2003.

Maragakis, E., A. Itani, and E. Goodwin, 2003, "Seismic behavior of welded hospital piping systems," *Proceedings of Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities*. Applied Technology Council, Redwood City, CA.

Maragakis, E. E. Goodwin, and A. Itani, 2005. "Experimental evaluation of the seismic response of welded and threaded hospital piping subassemblies," *Proceedings of the Structural Dynamics Eurodyn 2005 Conference*, vol. 3, 1841-1846, September, 2005, Paris, France.

NFPA (National Fire Protection Association), 1990. "Hearing on the performance of automatic fire sprinkler systems during the loma prieta, California earthquake," January 24, 1990, Society of Fire Protection Engineers, Bethesda, MD.

NFPA (National Fire Protection Association), (2015). *Standard for the Installation of Sprinkler Systems,2016 Edition*, (NFPA 13), National Fire Protection Association, Quincy, MA.

NIST GCR 18-917-43 (2018). "Recommendations for Improved Seismic Design and Installation of Nonstructural Components" (ATC 120) Applied Technology Council, Redwood City, CA.

Reitherman, R. and Sabol, T. (1995). "Nonstructural damage," *Northridge Earthquake of January 17, 1994*, John Hall, ed., Earthquake Engineering Research Institute, Oakland, CA.

Sekizawa, A., Ebihara, M., and H. Notake, 1998. "Development of seismic-induced fire risk assessment method for a building," *Proceedings of the Seventh International Symposium on Fire Safety Science*, pp. 309-320.

Todd, D., Carino, N., Chung, R. M., Lew, H. S., Taylor, A. W., Walton, W. D., Cooper, J. D., and Nimis, R. (1994). *1994 Northridge Earthquake Performance of Structures, Lifelines and Fire Protection Systems*, NIST Special Publication 862, National Institute of Standards and Technology, Gaithersburg, MD.

UL (Underwriters Laboratory) (1995). *Outline of Investigation for Sway Brace Devices for Sprinkler System Piping* (203A), Underwriters Laboratory, Northbrook, IL.

Keywords
fire sprinklers
nonstructural
piping
water leakage

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 2019)

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

SEAOC Seismology Committee (2019). "Fire Sprinklers," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Stair and Ramp Design

ASCE 7-16 reference section(s)	2016 CBC reference section(s)	Other standard reference section(s)
4: 4.3.1, 4.4, 4.5, 4.7 13: 13.1.3, 13.3, 13.4, 13.5 C13: 13.5.10 (New in ASCE 7-16)	1607/1607A 1613/1613A 1615/1615A	ASCE41-13: 13.3, 13.4, 13.5, 13.5.10, 13.6.8

Background

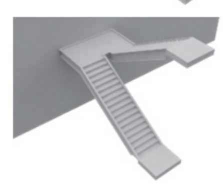
Most buildings have more than one floor, and thus require stairs or ramps. This article focuses on stairs, but generally also applies to ramps. The intricacies of stair design and detailing have not been sufficiently addressed in any of the building codes or commentaries prior to ASCE 7-16.

Stairs can be designed in different configurations, but generally function as a means of access between floor levels in a building or structure. In nearly all cases, they serve as the primary path of egress in the event of a fire, earthquake, or other event requiring evacuation. In some cases, stairs are provided for the occupants' convenience between floors and are not required for egress. Regardless, it is important that these elements maintain structural integrity and are functional after an earthquake for occupants' safety.

Each different stair geometry is accompanied by its own set of design implications and challenges. The more common stair configurations are discussed below. Stairs are typically constructed of steel, concrete or wood, however these configurations can be constructed of any structural material.

Straight runs have one or more flights of stairs that span either between floors and/or intermediate landings. Per code, landings are required for every vertical rise of 12 feet.

Switchback stairs consist of two or more flights of stairs that change direction 180° at intermediate landings. The intermediate landings are vertically supported with either posts to the structure below or hangers to the structure above. The flights of stairs are vertically supported on each end by floors and intermediate landings.



Cantilevered switchback stairs are a subset of switchback stairs. The difference is that for cantilevered switchbacks, the two flights of stairs and intermediate landing form a cantilevered structure that is vertically supported at the floor connections without any vertical support at the intermediate landing.

Angled stairs consist of two or more flights of stairs that change direction at an angle of less than 180°, instead of switchback stairs which turn around completely.

Curved stairs consist of flights of stairs that are curved in plan. Curved stairs can also have supported or unsupported intermediate landings.

Regardless of stair configuration, all stair members and connections must be designed to properly accommodate the forces and differential displacements that the stair will experience in a seismic event. In some cases, stairs and landings act as struts that transfer forces between the floors which the stairs connect. The most efficient way to mitigate these forces is through the incorporation of slip connections. When a stair is not detailed to accommodate any slip, the demands on the stair structure and the reaction forces at the attachment to the building structure can be significant. These demands are often problematic for new stairs in existing buildings. Additionally, these forces can result in heavier stair structures with expensive connections.

Though design for footfall-induced vibration is beyond the scope of this article, it can have consequences on the serviceability of the stair if not properly addressed. Footfall-induced vibration of stairs is not currently addressed in the building codes, but often governs the sizing of the stringers for stairs with long runs or without adequate intermediate supports. This becomes an issue when the vibrational response of the system is higher than the threshold for occupant comfort on and around the stair. The reader should see the reference by Wilford and Young (2006) for further information on vibration.

Design Considerations

ASCE 7-16 provides guidelines for the minimum design gravity and seismic loads on stairs.

Design Loads. Minimum design loads are addressed in ASCE 7-16 Chapters 4 and 13.
The typical types of loading that should be considered include:

- Dead Loads
- Live Loads
- Seismic Loads

These loads apply to both internal and external stairs. For external stairs, wind pressures should also be considered.

Design loads should be combined, as required per code, to capture the worst-case loading effects on the structural elements and all connections. It is important to remember that the same load case will not govern the design of all elements and connections.

Dead Loads. The dead loads consist of:

- Self-weight of all structural members
- Superimposed dead loads accounting for the weight of stair finishes, both on the walking surface and any finishes covering the structure underneath the stair, guard rails, and hand rails

Live Loads. Live Loads are found in ASCE 7-16 Chapter 4. The minimum live loads include:

- A uniformly distributed, reducible load of 100 psf or a concentrated load of 300 lb over a 2 in square area on the tread. For one- and two-family dwellings, the uniform reducible live load can be reduced to 40 psf. These uniform and concentrated loads are not required to be applied simultaneously.
- Hand rail loads of a 200 lb concentrated load or 50 lb/ft distributed load, applied at any location and in any direction on the hand rail. Consideration of guard rail loads is not required for one- and two-family dwellings or for factory, industrial and storage occupancies in areas with a maximum occupant load of 50. Typically, if the hand rails are supported on the stringers instead of on an adjacent wall, the out-of-plane guard rail loading produces the maximum load effect and can impose an additional torsion on the stringers if the treads do not create a couple between the two stringers.

Seismic Loads. Stairs and ramps that are either integral to the floor construction or are not designed to accommodate seismic relative displacements between floors should be designed as part of the primary structure and modeled in the global structural analysis. Stairs and ramps that are not part of the primary structure and are designed to accommodate seismic drifts should be designed using Chapter 13 for nonstructural components. Seismic design forces on nonstructural components (F_P forces) are prescribed in ASCE 7-16 Section 13.3.1. The seismic F_P force is determined through either static or dynamic analyses.

The magnitude of the design F_P forces is per ASCE 7-16 equations 13.3-1, 2, and 3. These equations are reproduced below.

The lateral F_P force is determined in accordance with Eq. 13.3-1:

$$F_P = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)$$

F_P is not required to be taken as greater than Eq. 13.3-2:

$$F_P = 1.6 S_{DS} I_p W_p$$

F_P shall not be taken as less than Eq. 13.3-3:

$$F_P = 0.3 S_{DS} I_p W_p$$

In the above lateral F_P force equations,

- Component amplification factor, a_p, and response modification factor, R_p, are calculated in accordance with ASCE 7-16 Table 13.5-1. For egress stairways not part of the building structure, the a_p and R_p factors are 1 and 2.5, respectively. Table 13.5-1 does not specifically reference non-egress stairs, so the same a_p and R_p factors should be used for all stairways.
- Component importance factor, I_p, is either 1.0 or 1.5, depending on the expected post-earthquake behavior and function. For an egress stairway required to function for life-safety purposes after a seismic event, the importance factor should be 1.5. For all other stairways, the code allows I_p to be taken as 1.0. However, the Seismology Committee recommends using an I_p factor of 1.5 for all stairs and ramps, not just egress stairs, to help avoid catastrophic failure or collapse of the stair structure.
- The height in the structure of the point of attachment of the component with respect to the base, z, can conservatively be taken as the highest point of connection to the structure (i.e. at the top of the stair run). Alternatively, if slip connections are provided, z may be the highest point along the height of the stair that is fixed to the structure.
- The ratio of the height of attachment, z, to the average roof height of the structure, h, need not exceed 1.0.

Where the acceleration at the levels of attachment of the stair is determined by an approved dynamic analysis method, it is acceptable to use the equation 13.3-4 below for the design F_P force:

$$F_P = \frac{a_i a_p W_p}{\left(\frac{R_p}{I_p}\right)} A_x$$

where,

a_i is the acceleration at the highest level to which the stair is fixed, as determined from the dynamic analysis method;
A_x is the torsional amplification factor determined by ASCE 7-16 equation 12.8-14.

When the demands on the stair are determined by dynamic analysis, the maximum and minimum values of F_P from equations 13.3-2 and 13.3-3 are still applicable.

The stair fasteners and attachments are designed for a higher F_P force with both the a_p and R_p factor equal to 2.5. Additionally, at locations where the stair structural elements are attached with non-ductile anchorage to concrete or masonry structure, these seismic demands are amplified by an overstrength factor, Ω_o, of 2.5.

Seismic F_P forces should be applied independently in two orthogonal horizontal directions. In addition to lateral F_P forces, a concurrent vertical force of $0.2S_{DS}W_p$ is applied in either the up or down direction to produce the worst possible load effect in each of the design load combinations.

Seismic Loads: Changes in ASCE 7-16. The magnitude of the F_P force has been increased for the attachments and connections of stairs. The design-level force for connections is 2.5 times higher in ASCE 7-16 than in ASCE 7-10. The reasoning for this change is to prevent failure of the stair system due to brittle connection failure. In recent shake table tests performed at UCSD, connections and welds of steel stairs experienced forces exceeding the anticipated design-level forces. These results are consistent with damage observed in past earthquakes; refer to the Lessons Learned section for further details and case studies.

Lateral Design & Drift Compatibility. In addition to the forces listed above, the compatibility of the stair structure to relative inter-story drifts must also be considered, according to ASCE 7-16 Section 13.3.2. The stair structure, connections, and any additional elements or attachments that will distort or may be damaged as a result of inter-story drift must be designed to accommodate the relative inter-story drifts. Distortion of the structure due to inter-story drift will often govern detailing and structural member sizing, so careful consideration should be given to this issue before the bulk of the analysis and design is performed. Based on the stair geometry, even relatively small inter-story drifts can impose very high forces on the stringers and their connections.

The code explicitly states that "the effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate." This means that the stair should be designed for the demands resulting from drifts in combination with the factored gravity loads.

When assessing drift compatibility, the inter-story drifts and any local diaphragm deformation are combined and amplified by the importance factor of the building, I_e, to determine the design-level relative displacements, D_{pl}. Local diaphragm deformation is included in the ASCE 7-16 provisions, as local deformations may increase the movement of the stair in a seismic event. If the actual drifts of the structure at the points of attachment are unknown, it is conservative to use the allowable inter-story drift of the building prescribed in ASCE 7-16 Table 12.12-1.

The Seismology Committee recommends slip connections as an effective way to detail the structure to accommodate inter-story drifts. These connections allow for movement of the stair in one or both orthogonal horizontal directions, and in turn significantly reduce the demands on both the stringers and their respective connections. Slip joints tend to result in a more economical solution, but require additional coordination with the architect to ensure that movement is compatible with architectural finishes. Where slip connections cannot be achieved, connections should be designed to be ductile.

The ASCE 7-16 provisions in Section 13.5.10 address design drift compatibility for egress stairs and ramps, though the Seismology Committee recommends that these provisions be applied to all stairs and ramps as best practice, as noted above. There are three ways in which drift compatibility is considered in the analysis and design of a stair and its supporting structure:

1. Stairs that allow for movement through a ductile connection or a slip connection that accommodates limited lateral sliding before a positive, fail-safe stop is activated are designed for a maximum movement of D_{pl} or 0.5 in. Examples of these are slip connections with bolts in slotted or oversized holes, sliding bearing connections seated in pockets or which contain end stop, or a ductile fuse that yields to allow for slip but retains vertical support of the stair.
2. Stairs with sliding bearing supports that allow for unrestrained lateral movement, where the stair does not have a stop, are designed for the maximum movement of 1.5 D_{pl} or 1.0 in. An example of this type of connection would be a precast concrete stair supported in vertical bearing only on a seat. In these cases, the

stairs could slide off their respective vertical supports if the inter-story drifts were excessive. Additionally, metal supports of stairs should have adequate rotational capacity to accommodate these seismic displacements of 1.5 D_{pl} or 1.0 in. The strength of the ductile metal supports cannot be governed by any brittle failure modes.

3. For stairs that do not utilize either slip connections or ductile connections, and thus do not accommodate the seismic relative displacements, the stair is required to be added to the global structural analysis model and designed for building overstrength Ω_0 level forces. This requirement addresses the fact that the stair may act as a bracing element between floors, and depending on the lateral system of the building, could attract significant load.

All of these connection types must maintain vertical support and stability after a seismic event. This is a change to the code from ASCE 7-10. These changes are to ensure life-safety and have been derived from recent research testing and observed failures, which are further discussed in the Lessons Learned section of this article.

Slip Connections. When selecting a type of slip joint, it is imperative that the engineer understands the impact of the lateral drifts on the stair. Slip connections can be provided in one or both horizontal directions, depending on the structural behavior of the stair when subject to imposed displacements. The loads imposed on the stair structures differ for each of the various stair types. The response of each stair type is as follows:

- Straight runs act as a flexural beam in the transverse direction and as axial compression struts longitudinally. Since the stair runs are significantly stiffer axially than flexurally, at minimum a slip connection at one end parallel to the axis of the stair should be provided to release the large forces that would occur from such a stiff system.
- Switchback stairs are similar to straight runs in that they tend to provide high axial stiffness. Additionally, they are pulled in opposite directions as the two parallel runs move independently, and high flexural demands are induced in the stair stringers and may cause warping and torsion of the shared intermediate landing. Therefore, the optimal slip connection would allow for movement in both directions, to relieve high axial and flexural stresses on the stair structure.
- Cantilevered switchback stairs behave similarly to switchback stairs. However, to maintain stability of the stair under gravity loads, a slip connection can only be provided in the direction transverse to the stair.
- Angled stairs and curved stairs also act similarly to switchback stairs, though the degree to which the flexural behavior governs the stair response and design is dependent on the angle or the degree of curvature.

Slip connection details: Examples of slip joints are provided in this section.

The detail below allows for movement at the top of the ramp or stair in both horizontal directions while maintaining vertical stability. A shear plate with long-slotted holes is bolted to the stringer to allow for movement in the longitudinal direction. The shear plate is welded to a face plate with long-slotted transverse holes to accommodate movement in the transverse direction.

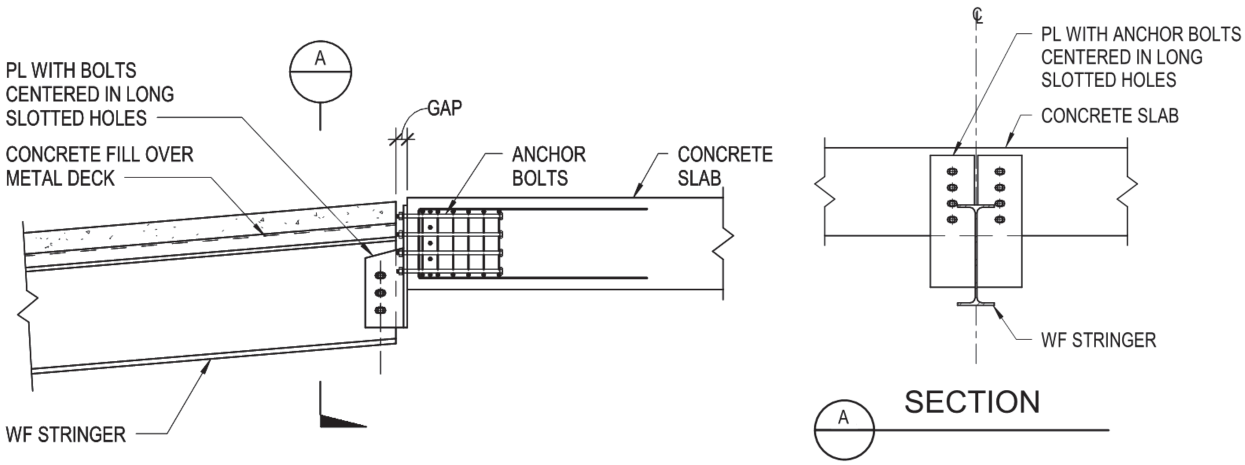


Figure 1. Examples of Slip Connections at Top of Ramp – Longitudinal and Transverse Direction

The detail below allows for movement at the bottom of the stair in only the longitudinal direction of the stair. The stair stringer bears on a concrete slab, and an angle is welded to the stringer and anchored to the slab with bolts through long-slotted holes in the angle.

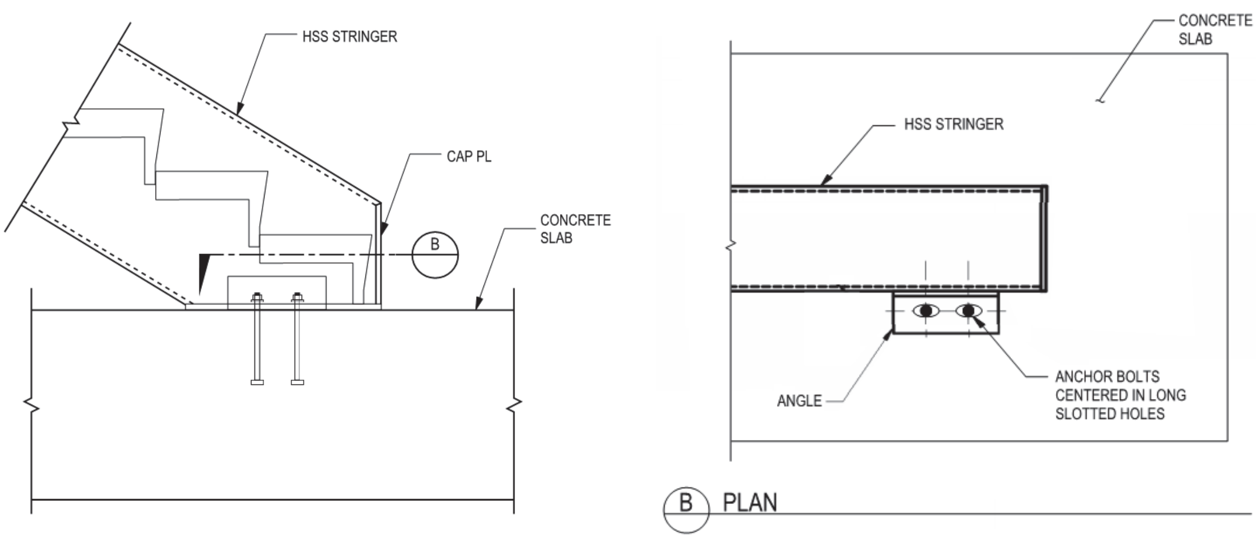


Figure 2. Examples of Slip Connections at Base of Stair – Longitudinal Direction

The details in Figure 3 allow for movement at the bottom of the stair in only the direction transverse to the stair. A detail is shown for connection to both concrete and steel base elements.

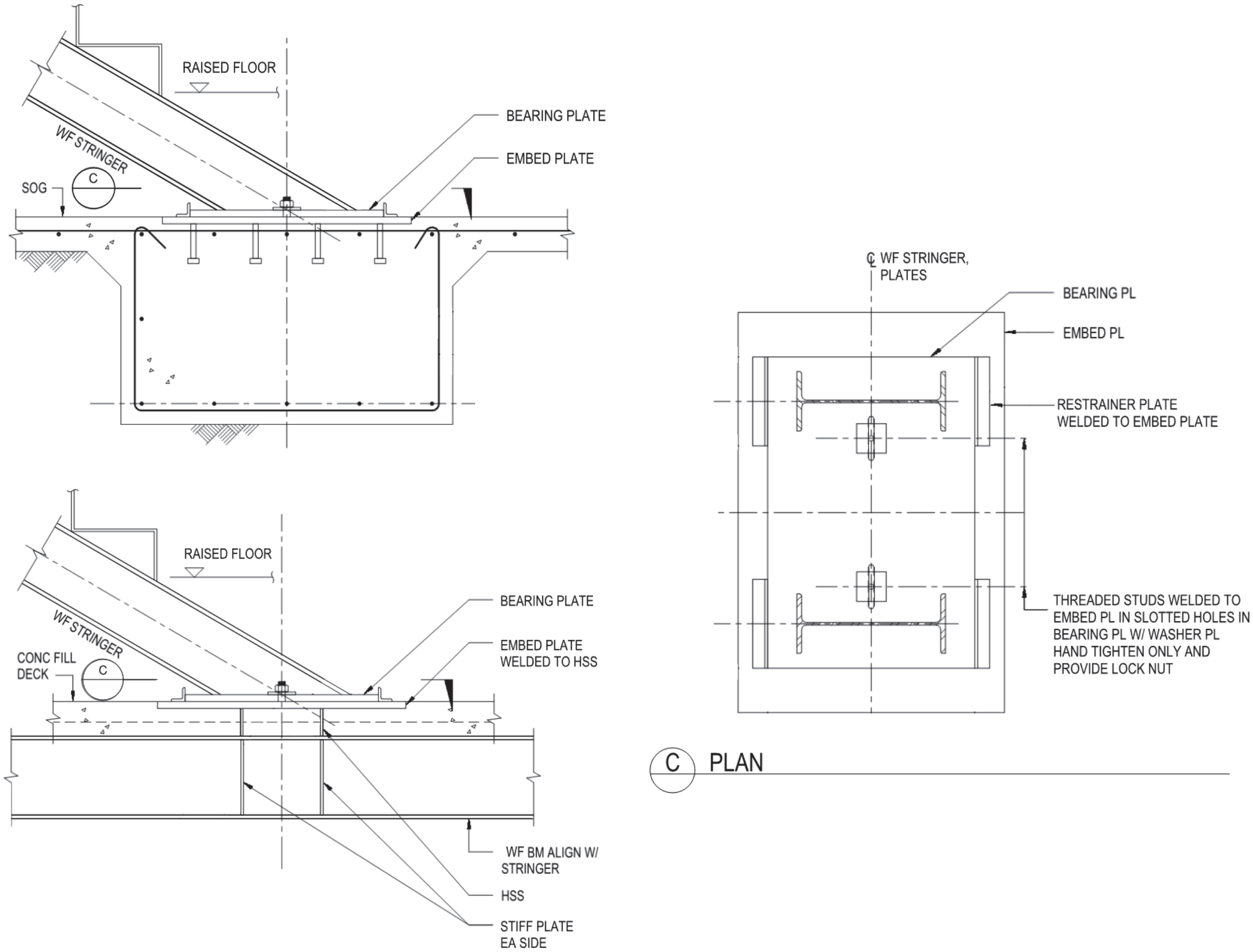


Figure 3. Examples of Slip Connections at Base of Stair – Longitudinal and Transverse Direction

The details shown thus far are for steel stairs and ramps, however conceptually these details can also be applied to other materials. For wood stairs, the slip connections can be achieved with a combination of steel plates and bolts, like the details shown. For concrete stairs, slip joints can be provided in one or both orthogonal horizontal dimensions by resting the stair on a Teflon bearing pad at one end of the stair.

Properly detailed slip connections require coordination with the architect to ensure that the stairs are able to move as designed. This is important for stairways where guardrails and/or handrails continue on to a fixed floor or landing. At these locations, all components attached to the stair also need to be able to slip laterally with the stair. Gaps provided in slip connections should be designed and maintained such that finishes or other material are not inadvertently inserted in the gaps which will compromise the movement required.

Similarly, adequate space needs to be provided around the moving elements in order to properly accommodate the lateral movements of the stair. This is important at stair cores that are pressurized, as any damage caused to the partitions may cause a loss in pressurization. If the stair is not able to move as needed, the elements will experience higher forces than were designed for, and are likely to be damaged.

Ductile Connections. An acceptable alternative to providing slip connections is to utilize ductile connections to accommodate drifts, and limit the transfer of excessive forces to the structural system at the floor. A ductile connection is governed by ductile failure modes, such as flexural yielding of a connection plate. This ductile mechanism limits the connection capacity while maintaining stability of the stair after a seismic event. Unacceptable mechanisms include all brittle failure modes such as bolt shear failures or weld fracture. There is currently no published literature on successful tests of ductile connections, and thus specific examples of details are not provided in this article.

Drift Compatibility: Changes in ASCE 7-16. The key revision is the magnitude of the stair design drifts. Egress stairs that are not provided with stops are now designed for the seismic relative displacements amplified by a factor of 1.5, which represent MCE-level drifts. This change is made in order to prevent catastrophic failure such as loss of seating and vertical supports. This has been observed in recent earthquakes, where drifts experienced were significantly greater than the design drifts; refer to the Lessons Learned section for further detail and case studies.

In ASCE 7-16, the definition of the seismic relative displacements is expanded to include diaphragm deformation. In the Christchurch earthquake, it was observed that in the Clarendon Tower the location of the stair supports were in areas associated with larger diaphragm cracking and deformations. The exact implications of the diaphragm cracking remain unclear, but it raises concern about the potential impacts and additional displacements of the stairs due to local diaphragm deformation.

Lessons Learned

The changes to the provisions in ASCE 7-16 arise from recent studies and observed failures of stairs in the Christchurch, New Zealand earthquakes in 2010 and 2011. Two buildings, the Forsyth Barr and the Hotel Grand Chancellor buildings, saw multiple cases of stair damage and at least four stair collapses. In the Forsyth Barr building, the precast stair runs were seated on sliding bearing connections at the base of the runs. During the earthquake, the building experienced higher drifts than the movement joint allowed for, which prevented complete movement of the stair and imposed large compression forces in the stair runs. The high compression forces caused compression failures of some stairs and axial shortening of the stair runs. The shortened stairs then did not have the necessary bearing length required when the seismic load was reversed, so these stairs slid off the bearing seat, since there was no stop provided at the landing in that direction. Since the stairs were then hanging from the top landings, several stairs hinged at the top, which caused progressive collapse of several staircases. Similar behavior was observed at the Hotel Grand Chancellor.

It is important to note that, in both of these buildings, the stair lengths and stair joints were not constructed according to the original construction drawings, though later inspection of the stairs showed that these mistakes were corrected in field. Regardless of these construction issues, the buildings experienced higher inter-story drifts than anticipated, which jeopardized the life-safety of the occupants. These observed failures show the importance of robust stair connections and effective sliding details to allow for adequate movement of the stair while maintaining stability and means of egress.

Structural Design Responsibility for Design-Build Stairs

For some projects, the stair design is performed by an entity other than the engineer of record (EOR) for the building. Current building codes are not clear on the requirements for the approval and review of these stair designs, and thus the responsibility of defining the criteria is left to the EOR and the construction documents. The Seismology Committee recommends that the construction documents include language that requires the stair design to be performed by a licensed engineer and be reviewed by the EOR for the building. Sample specification language is provided below:

> "Design build stair and railing structural calculations and drawings shall be stamped and signed by a civil or structural engineer registered in the state of California. Design build stairs shall be designed to meet all

relevant Code requirements. This includes all Code mandated vertical and lateral loads, and deformation compatibility. The stair framing elements and their connections shall be designed and detailed to be adequate to maintain support of the design dead plus live loads during the expected lateral deformations of the primary structure in a seismic event. Design stairs to accommodate the seismic story displacements given in the design criteria in the construction documents.
Design build stair design shall clearly indicate a complete load path for both lateral and vertical loads to the primary structural elements shown in these drawings. The calculations and drawings shall show the magnitude, location and direction of all design loads imposed by the stair structural elements onto the primary structure. Shop drawings and calculations of stairs and railings shall be approved by the Engineer of Record prior to start of fabrication."

Future Research

To utilize stair connections that do not allow for slip, additional research on ductile details is required. Ductile details for steel and wood stairs have been envisioned and checked through calculations, but it is difficult to demonstrate that they will behave as expected without experimental testing.

Although beyond the scope of this document, it is recommended that research be performed on the assurance of egress paths to and from the stairs, as these pathways should be designed to assure that occupants can access the stairs in a required evacuation. Without access to the stairs, the usefulness of the stairs will be limited.

References

ACI (2014). ACI318-14, *Building Code Requirements for Structural Concrete*, September 2014, American Concrete Institute, Farmington Hills, MI.

AISC (2010). *Steel Construction Manual* (ANSI/AISC 360-10), February 2012, American Institute of Steel Construction, Chicago, IL.

ASCE (2010). ASCE/SEI 7-10, *Minimum design loads for buildings and other structures, including supplement no.1 & 2,* American Society of Civil Engineers, Reston, VA.

ASCE (2017). ASCE/SEI 7-16, *Minimum design loads for buildings and other structures,* Society of Civil Engineers, Reston, VA.

ASCE (2013). ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings,* American Society of Civil Engineers, Reston, VA.

Black, Cameron J., Ian D. Aiken, Kevin W. Smith, Robert J. Belvin, and Anthony J. Peachy. "Seismically-Resilient Stair Systems for Buildings." 2017 SEAOC Convention Proceedings.

Bull, Des. *Report to the Royal Commission: Stairs and Access Ramps between Floors in Multi-Storey Buildings*, University of Canterbury, New Zealand.

Canterbury Earthquakes Royal Commission (2012). *Final Report, Volume 2: The Performance of Christchurch CBD Buildings*, Canterbury Earthquakes Royal Commission, New Zealand.
Fleischman, Robert B., Jose I. Restrepo, Stefano Pampanin, Joseph R. Maffei, Kim Seeber, and Franz A. Zahn. "Damage Evaluations of Precast Concrete Structures in the 2010–2011 Canterbury Earthquake Sequence." Earthquake Spectra 30.1 (2014): 277-306.

Higgins, Christopher. "Prefabricated Steel Stair Performance under Combined Seismic and Gravity Loads." Journal of Structural Engineering 135.2 (2009): 122-29.

NAAMM (1992). AMP 510-92, *Metal Stairs Manual,* Architectural Metal Products Division of the National Association of Architectural Metal Manufacturers, Chicago, IL.

NDS (2015). ANSI/AWC NDS-2015, *National Design Specification for Wood Construction*, September 2014, American Wood Council, Leesburg, VA.

Pantoli, Elide, Michelle C. Chen, Xiang Wang, Rodrigo Astroza, Hamed Ebrahimian, Tara C. Hutchinson, Joel P. Conte, José I. Restrepo, Claudia Marin, Kenneth D. Walsh, Robert E. Bachman, Matthew S. Hoehler, Robert Englekirk, and Mahmoud Faghihi. "Full-Scale Structural and Nonstructural Building System Performance during Earthquakes: Part II – NCS Damage States." Earthquake Spectra 32.2 (2016): 771-94.

Wilford, Michael, and Young, Peter (2006). *A Design Guide for Footfall Induced Vibration of Structures,* The Concrete Centre, London, UK.

Keywords
stairs
stair stringers
stair design
nonstructural components
F_P forces
drift compatibility
slip connection
slip joint
vibration

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Stair Design," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Geotechnical Investigations

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
11.4 11.8, 11.9 12.1.5 12.13 20, Table 20.3-1 21	1613 1803 1803A	FEMA P-1050-1/2015 (NEHRP 2015) 11.4.2, 11.4.3, 11.4.7, 11.8.3, 12.1.5, 12.13

Introduction

A geotechnical investigation is performed to evaluate the soil, rock, ground water, geologic, and seismologic conditions that potentially affect the siting, design, and construction of a proposed development or improvement. Specifically, the geotechnical investigation provides recommendations for design of foundation systems and subterranean structures, earthwork, 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 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 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 consultant should be made aware of any unusual structural features, such as:

- New structural technologies being considered in the proposed buildings, such as seismic isolation or energy dissipation devices (dampers), which typically require dynamic or nonlinear structural analysis. This may require site-specific ground motions.
- Heavy column loads immediately adjacent to much lighter loads, creating the possibility of significant differential settlements between adjacent columns.
- High uplift demands under seismic loading.

When projects proceed to final design, geotechnical 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 structural failure 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, cause structures to be designed for unrealistic forces, or even economically threaten or terminate projects. Excessive excavation and replacement due to conservatism can also lead to greater impact to adjacent properties.

Changing Practice

Geotechnical investigations vary 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 include in-situ Standard Penetration Tests (SPT), where the blow counts (N-values) are recorded, and limited laboratory testing of soils for classification and measurement of engineering properties. The geotechnical recommendations are then based on empirical correlations between 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).

For more critical projects or those of greater scale, or 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 measured by laboratory testing of the "undisturbed" samples and/or remolded samples of the soils.

In addition to normal static design considerations, geotechnical reports should address the potential for geologic and seismic hazards including:

- Evaluations of Site Class factors (ASCE 7 Chapter 20),
- Liquefaction (ASCE 7 Section 12.13.9, CBC Sections 1803.5.11, and 1803.5.12),
- Total and differential settlements (CBC Sections 1803.5.11, 1803.5.12 and 1803.6),
- Dynamic seismic lateral earth pressures on basement and retaining walls (CBC Section 1803.5.12), and
- Slope instability, surface lateral displacements due to faulting, seismically induced lateral spreading or lateral flow (CBC Section 1803.5.11).

The 2019 California Building Code requires that these hazards be evaluated for projects with Seismic Design Categories C through F.

Additional tests, such as seismic velocity profiling or cone penetration tests (CPTs) may be performed, in addition to dynamic testing of soil samples. The geotechnical recommendations are then developed from calculations using the soil properties measured and estimated from the field investigation and laboratory testing.

As the practice of civil engineering encompasses many disciplines, geotechnical investigations should be performed by licensed 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 licensed geologists or earth scientists. In California, these individuals should be Certified Engineering Geologists.

Where geotechnical and/or geological reports contain specialized seismic design information or where substantial professional judgment needs to be applied, peer reviews of 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.

Geotechnical consultants should also continue to educate themselves in the evolving practice of identifying and mitigating seismic hazards.

Code and Standard Approaches

Chapter 18 of the California Building Code requires geotechnical investigations based on observations and tests deemed necessary to the satisfaction of the Building Official. In addition, state amendments in Chapter 18A apply to DSA and OSHPD 1 and 4 regulated occupancies (CBSC 2019).

The ASCE 7-16 earthquake provisions are based on the 2015 NEHRP provisions. Foundation design requirements are found in Section 12.13, which addresses only those foundation requirements related to seismic resistant construction. Section 11.4 provides the procedures for determining the Maximum Considered Earthquake and design earthquake ground motion accelerations and response spectra. The main differences from previous ASCE 7 versions are that the short-period and long-period site coefficients, F_a and F_v, have been modified (Tables 11.4-1 and 11.4-2) and that the requirements for site-specific ground motion analyses have been expanded. Although not required in the provisions, the site class, based on definitions in Table 20.3-1, should be established by a geotechnical consultant. In Section 11.8.2, an investigation for structures with Seismic Design Category C or higher should also include the potential geologic and seismic hazards due to slope instability, liquefaction, total and differential settlement, and surface displacement caused by faulting or seismically induced lateral spreading or lateral flow, and the report should include appropriate recommendations for foundation design or other measures to mitigate the effects of these hazards. The design professional will need to inform the geotechnical consultant of the Seismic Design Category of the project to decide the need for seismic hazards evaluation. A site-specific geotechnical report is permitted to be waived by the Building Official, when prior evaluations of nearby sites with similar soil conditions provide direction relative to proposed construction.

The NEHRP Commentary also provides procedures for evaluating liquefaction and slope instability. The Resource Paper, RP-12, in the 2009 NEHRP Provisions summarizes alternatives for evaluating liquefaction triggering, methods for evaluating the possible consequences of liquefaction and method for mitigating these consequences. Another important report sponsored by NASEM (2016) makes a general recommendation to "refine, develop, and implement performance-based approaches to evaluating liquefaction, including triggering, the geotechnical consequence of triggering, structural damage, and economic loss models to facilitate performance-based evaluation and design." Within the discussion of performance-based design, the report suggests that probabilistic liquefaction hazard analyses (PLHA) are useful tools to better understand uncertainties and risks associated with liquefaction.

ASCE 7-16 requires the evaluation of liquefaction triggering the use of PGA_M, which is the MCE_G (Maximum Considered Earthquake Geometric Mean) peak ground acceleration (PGA) adjusted for Site Class effects. There are not considerations for performing the evaluation at different levels of seismic hazard (i.e., MCE and DE levels) or performance-based criteria. In addition, the only ground motion parameters to be used are the PGA_M and associated magnitude. Currently, there are alternative ground motion parameters for liquefaction evaluation, such as spectral acceleration at a particular period, Arias Intensity (AI), and cumulative average velocity (CAV).

ASCE 7-16 includes new requirements for structure foundations on liquefiable sites. Tables 12.13-2 and 12.13-3 present permissible limits for lateral spreading horizontal ground displacements and differential settlements for shallow foundations. There is an exception to these requirements for those structures on shallow foundations where the geotechnical investigation report indicates that there is negligible risk of lateral spreading, no bearing capacity loss, and differential settlements of site soils or improved site soils do not exceed one-fourth of the differential settlement threshold specified in Table 12.3.-3.

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. The geotechnical investigation report for structures assigned to Seismic Design Category D, E or F, should calculate dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions, not by MCE_G ground motions. See article 09.10.010, "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls," for further information.

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. Seismic demands 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 nature of the building codes inherently encourages the use of geotechnical investigations to provide realistic and reasonable design values.

Geotechnical reports should include the seismic and geologic information necessary for structural design. Detailed criteria in Chapters 18,18A, and 12.13 provide effective checklists to help ensure that reports are complete. Structural engineers should 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 analyses have been performed by geotechnical consultants and whether the proposed solutions or mitigations are reasonable and appropriate. Design professionals should consider if geotechnical consultants have sufficient knowledge and experience in seismic issues to provide recommendations that are state of the practice without being overly conservative.

Geotechnical/geologic consultants should be aware of additional requirements that a state or local authority having jurisdiction may have in addition to the CBC and ASCE 7 provisions. Examples of jurisdictions that have formal manuals for the preparation of geotechnical reports are the City of Los Angeles (2014), County of Los Angeles (2014), and City of San Diego (2018). Note 48, published by the California Geological Survey (2013), 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 insights 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 engineering methods or scopes 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 regarding behavior of structures under extreme loading conditions including earthquakes, traditional geotechnical analysis may not provide adequate information for foundation design. 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 would be the likelihood for liquefaction when there is variability in the ground motions as well as variability in the ground-water level.

The evaluation of liquefaction triggering has most commonly been performed in practice using the empirical stress-based approach, also called the "simplified approach," which was originally developed in the 1970's. Although there have been some updates to the procedure over the past 40 years by various researchers, the overall approach has remained the same. A Cyclic Resistance Ratio (CRR) is developed based on the soil profile, and an earthquake-induced Cyclic Stress Ratio (CSR) is developed to represent seismic demand based on PGA (maximum amplitude of earthquake acceleration) and magnitude M_w (a proxy to duration and energy). The biggest question surrounding the seismic demand then is what combination of PGA and M_W to use at what hazard level. Depending on the type of project and agency jurisdiction, how the PGA and magnitude are selected can vary. Although the guidelines and codes generally specify ground motions that should be used in the evaluation, some defer the details to geotechnical engineers performing analyses, while others are more prescriptive in their requirements.

Table 1 summarizes the ground motions parameters used for the evaluation of liquefaction triggering from several standards and regulations.

Table 1 – Ground Motion Parameters for Liquefaction Triggering Evaluation

Standard	Ground Motion Parameter
ASCE 7-10 (2019 CBC)	PGA_M and M_w from MCE_G
CGS Note 48 (2013)	PGA_M and modal M_w from MCE_G
City of Los Angeles (2014)	Level 1: PGA=2/3 PGA_M; modal or mean M_w from 475-yr Level 2: PGA=PGA_M; modal or mean M_w from 2475-yr
Caltrans (2014)	PGA larger from deterministic or 975-yr probabilistic; M_w either deterministic or larger of mean or modal from 975-yr
CAHSR (2012)	MCE = PGA greater of 950-year probabilistic or deterministic OBE = PGA from 50-year probabilistic
NAVFAC (ordinary buildings) (1997)	Level 1: PGA and M_w from 72-yr probabilistic Level 2: PGA and M_w from 475-yr probabilistic
ASCE 61-14 ("High") for Piers and Wharves	OLE: PGA and M_w from 72-yr probabilistic CLE: PGA and M_w from 475-yr probabilistic DE: PGA=PGA_M; modal or mean M_w from 2475-yr

Symbols:
MCE = Maximum Considered Earthquake
MCE_G = Maximum Considered Earthquake Geometric Mean
PGA = Peak Ground Acceleration
PGA_M = The MCE_G peak ground acceleration adjusted for site effects
M_w = Moment Magnitude
OBE = Operating Basis Earthquake
OLE = Operating Level Earthquake
CLE = Contingency Level Earthquake
DE = 2/3 of MCE_R

Review of Table 1 indicates that, aside from ASCE 7 and Caltrans, nearly all other approaches consider at least two seismic hazard levels – one associated with "design" ground motions at a longer return period (typically on the order of 500 years or greater) and one associated with more frequent shaking.

Although PGA has been conventionally used to represent the ground motion intensity for liquefaction triggering procedures and evaluating the consequences of liquefaction, this parameter has its limitations. One of the most significant limitations is that PGA is a scalar value; it represents the largest transient acceleration in an earthquake record. However, it does not capture the overall behavior of the earthquake record very well and may not accurately capture the duration or energy associated with shaking.

Alternative intensity measures (IMs) have been proposed for use in liquefaction evaluations by various researchers. There are generally two types of IMs – peak transient IMs and evolutionary IMs. Peak transient IMs, such as PGA or spectral acceleration at a particular period (Sa (T)), represent the largest amplitude in an earthquake record but may not accurately represent duration or frequency content. Evolutionary IMs, such as Arias Intensity (I_A) or cumulative absolute velocity (CAV), accumulate throughout earthquake shaking. In recent studies, the evolutionary

IMs appear to be better predictors of liquefaction triggering and consequences (Greenfield and Kramer, 2018; Bullock et al. 2018; Bray and Macedo, 2017).

Seismic ground motions for evaluating liquefaction vary depending on the structure type (building, bridges, piers, etc) and seismic hazard level of interest, and are generally represented by PGA and M_W based on deterministic or probabilistic seismic hazard analyses. Even though ASCE 7 currently uses only a single hazard level to evaluate liquefaction, most other codes consider a minimum of two seismic hazard levels. When probabilistic ground motions are used, care should be taken to adopt a consistent magnitude and PGA pairing. In our opinion, the use of two seismic hazard levels (such as a Maximum Considered Earthquake level, and a more frequent earthquake within the lifetime of the structure) may be a better approach to understanding liquefaction risks.

Probabilistic liquefaction hazard analysis (PLHA) has several advantages over the conventional practice using the "simplified procedure." PLHA is consistent with a Performance-Based Design approach, it can prevent the erroneous pairing of PGA obtained from PSHA with an inconsistent magnitude, and since a wide variety of ground motions can contribute to the evaluation, the risk of liquefaction at a particular seismic hazard level can be evaluated directly. In our opinion, simplified PLHA may be a path forward to better understand liquefaction hazards.

Regarding evolutionary IMs to represent ground motions, it is interested to note that none of the approaches have used the same software, models, or methods, and yet their results tend to converge – evolutionary intensity measures, particularly CAV, appear to be better at representing liquefaction and its effects. In areas where the "simplified procedure" may not be as effective (such as megathrust earthquakes), representing the seismic ground motions by an evolutionary IM instead of PGA may be a good alternative.

References

ASCE 2010. ASCE Standard ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures. Reston, Virginia: American Society of Civil Engineers.

ASCE 2014. ASCE/COPRI 61-14, Seismic Design of Piers and Wharves. Reston, Virginia: American Society of Civil Engineers.

ASCE 2017. ASCE Standard ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures. Reston, Virginia: American Society of Civil Engineers

Blake, T.F., R.A. Hollingsworth, and J.P. Stewart, editors (2002). Recommended procedures for implementation of DMG special publication 117 Guidelines for analyzing and mitigating landslide hazards in California, ASCE Los Angeles Section Geotechnical Group, Southern California Earthquake Center, University of Southern California.

Bray, J.D. & Macedo, J. 2018. Evaluating liquefaction-induced building settlements. Proceedings of the 11th National Conference in Earthquake Engineering, Los Angeles, CA: Earthquake Engineering Research Institute, Los Angeles, CA.

BSSC (2004). NEHRP recommended provisions for seismic regulations for new buildings and other structures, Part 1: Provisions, Part 2: Commentary (FEMA 450-1,2/2003 Edition), Building Seismic Safety Council, Washington, DC.

Bullock, Z., Dashti, S., Liel, A., Porter, K. & Karimi, Z. 2018. Efficiency, Sufficiency, and Predictability of Intensity Measures for Predicting Liquefaction Consequences. Proceedings of the 11th National Conference on Earthquake Engineering, Los Angeles, CA: Earthquake Engineering Research Institute.

CAHSR (2012), California High-Speed Train Project Design Criteria.

Caltrans (2014), Caltrans Geotechnical Manual, Liquefaction Evaluation, December 2014.

CBSC (2019). California Building Code. California Building Standards Commission, Sacramento, CA.

CDMG (1997). Guidelines for evaluating and mitigating seismic hazards in California, Special Publication 117, California Division of Mines and Geology (California Geological Survey), Sacramento, CA.

CGS (2002). Note 49 – Guidelines for evaluating the hazard of surface fault rupture, California Geological Survey, Sacramento, CA.

CGS (2013). Note 48 – Checklist for the review of engineering geology and seismology reports for California public schools, hospitals, and essential services buildings, October 2013, California Geological Survey, Sacramento, CA.

City of Los Angeles 2014. Letter to Geology and Soil Engineering Firms Practicing in the City of Los Angeles from the Grading Division, Los Angeles Department of Building and Safety, July 16, 2014.

Coduto, Donald P. (2001). Foundation design – Principles and practices, second edition, Prentice Hall, Upper Saddle River, NJ.

Greenfield, M.W. & Kramer, S.L. 2018. The Effects of Long-Duration Ground Motion on Soil Liquefaction. Proceedings of the 11th National Conference in Earthquake Engineering, Los Angeles, CA: Earthquake Engineering Research Institute.

Hart, Earl W. (1988). Fault-rupture hazard zones in California. Special Publication 42, California Division of Mines and Geology (California Geological Survey), Sacramento, CA.

Hart, Earl W. and Bryant, William A. (1997, supplements added 1999). Fault-rupture hazard zones in California. Special Publication 42, California Division of Mines and Geology (California Geological Survey), Sacramento, CA.

IBC (2018), International Building Code. International Code Council, ICC.

LADPW (2001). Manual for preparation of geotechnical reports, County of Los Angeles Department of Public Works, February 2000, revised May 8, 2001.

Martin, G.R. and M. Lew, editors (1999). Recommended procedures for implementation of DMG special publication 117 Guidelines for analyzing and mitigating liquefaction hazards in California, Southern California Earthquake Center, University of Southern California.

NASEM (2016) National Academies of Science, Engineering and Mathematics. State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences, Committee on State of the Art and Practice in Earthquake-Induced Soil Liquefaction Assessment, Board on Earth Sciences and Resources, Division of Earth and Life Studies. Washington DC: The National Academies Press.

NAVFAC (1997), Seismic Design for Buildings, Technical Manual P-355.

NEHRP (2015), NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, 2015 Edition. (FEMA P-1050-1).

SMDBS (2002). Guidelines for geotechnical reports, City of Santa Monica Department of Building and Safety, March.

Youd, T.L. and I.M. Idriss, editors (1997). Proceedings of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils NCEER Technical Report NCEER-97-0022, December 31, 1997.

Youd, T.L. et al. (2001). "Liquefaction resistance of soils, summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, Journal of Geotechnical and GeoEnvironmental Engineering, ASCE (127) 4, pp. 297-131.

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 2019)

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

SEAOC Seismology Committee (2019). "Geotechnical Investigations," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.1.5, 12.7.1, 12.13.3, 19,		ATC-40 FEMA 356 4.4.2.1.2, 4.4.2.2 ASCE 41-13/17 8.5, 8.4.2 FEMA 440 2015 NEHRP, 12.13, 19

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, knowing full well for short period fixed base buildings on soft soils that including the foundation flexibility in the model will tend to relieve demands on the Seismic Force Resisting System. This has especially been the case for those engineers conducting sophisticated analyses in the nuclear power plant industry.

Today's widely available analytical methods and tools allow a greater number of structural engineers to include soil-foundation-superstructure interaction in their projects and, currently, where soil-structure interaction effects are considered, ASCE 7-16 (ASCE 2016) requires the analytical model to include horizontal, vertical and rotational foundation and soil flexibility. 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/SEI-13/17 (ASCE 2014), 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). ASCE/SEI 7-16 Chapter 19 represents a complete replacement of the chapter in ASCE 7-10 and is based on the Building Seismic Safety Council’s recommendations in the 2015 NEHRP Provisions (NEHRP 2015) with minor exceptions.

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 as an inertial interaction 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, as an inertial interaction effect; and (3) a kinematic interaction effect, dubbed as embedment and base slab averaging, is the filtering of the dynamic characteristics of ground shaking transmitted to the structure.

Now ASCE/SEI 7-16 Chapter 19 permits all three of the above categories to be used for the Nonlinear Response History Analysis procedure. The use of kinematic interaction effects is not permitted for the Equivalent Lateral Force Procedure or the Modal Response Spectrum Analysis procedures. All analysis procedures are required to include models with horizontal, vertical and rotational foundation and soil flexibility when considering soil-structure interaction. Soil Site Classes A and B are not included in ASCE/SEI 7-16 Chapter 19 for soil structure interaction.

Figure 1 illustrates options for structural analysis models, starting with a traditional fixed base assumption.

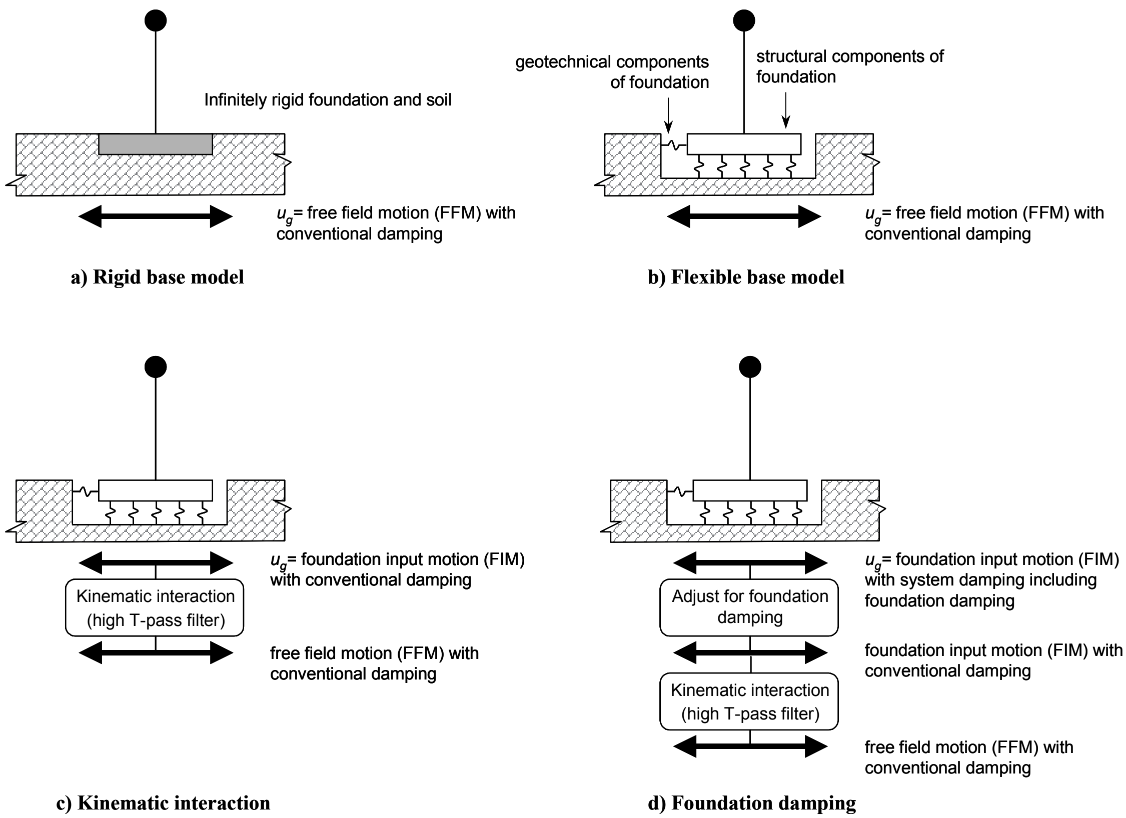


Figure 1. Foundation modeling assumptions

Rigid or 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.

- Such a fixed base assumption is not permitted by ASCE/SEI 7-16 when using Chapter 19. A fixed base assumption will also not quantify the deleterious effect of foundation flexibility on frame members adjacent to, but not part of the Seismic Force Resisting System.

Flexible Base or 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.

- ASCE 7-16 Chapter 19 requires the analytical model to include horizontal, vertical and rotational foundation and soil flexibility. For the Equivalent Lateral Force procedure or Modal Response Spectrum procedure, springs are placed in the model to approximate the effective linear stiffness of the deformations of the soil or geotechnical media and the structural foundation elements. For the Nonlinear Response History Analysis procedure, the nonlinear force-deformation and hysteretic characteristics of the geotechnical media and structural foundation elements can be modeled using a nonlinear finite beam, shell or solid element model of the combined structure, foundation and geologic media; or else springs and dashpots can be attached to the foundations and walls of the structure model.

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).

- In ASCE 7-16 Chapter 19, the use of kinematic interaction effects is not permitted for the Equivalent Lateral Force Procedure or the Modal Response Spectrum Analysis procedures since there is a potential to overestimate the reduction in response parameters for those R factor based procedures.

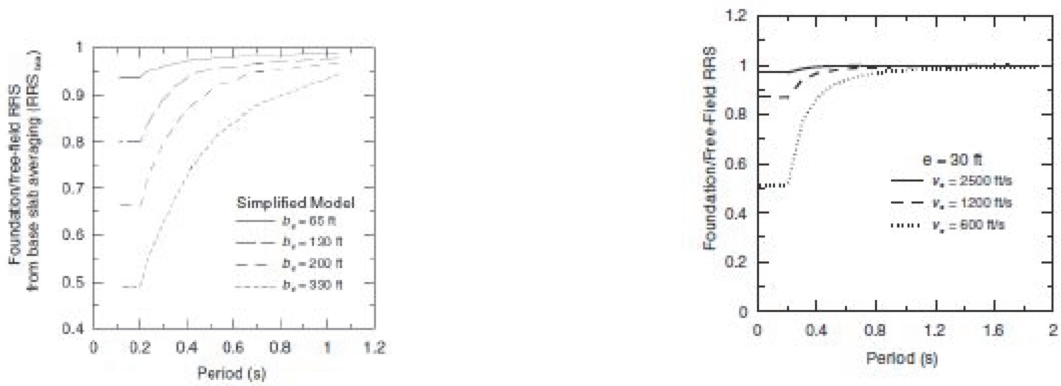


Figure 2. Spectral reduction factors for kinematic interaction

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.

- In ASCE 7-16 Chapter 19, the effective damping ratio, β_o, has distinct parts, β_f, the foundation soil-interaction consisting of the soil hysteretic damping, $\beta_{s,}$ and the radiation damping, β_{rd} in ASCE 7-16, and the effective damping of the structure, β. β is modified by the ratio of the fundamental period of the flexible base structure to the fundamental period of the fixed base structure and the expected ductility demand. β_s is also modified by the incremental ratio of the fundamental period of the flexible base structure to the fundamental period of the fixed base structure. β_{rd} can either be the radiation damping value from a rectangular or circular foundation, as applicable. Radiation damping in Sections 19.3.3 and 19.3.4 are for shallow or surface foundations. For pile foundations, detailed structural modeling of the soil and pile system is required to quantify radiation damping. The reader is referred to (NIST, 2012) in quantifying radiation damping for pile systems. β_s soil damping values may be taken from Table 19.3.3 provided in the Chapter 19 provisions based upon the effective peak acceleration of the site computed as $S_{DS}/2.5$. β_s increases with higher effective peak acceleration and for softer soils.

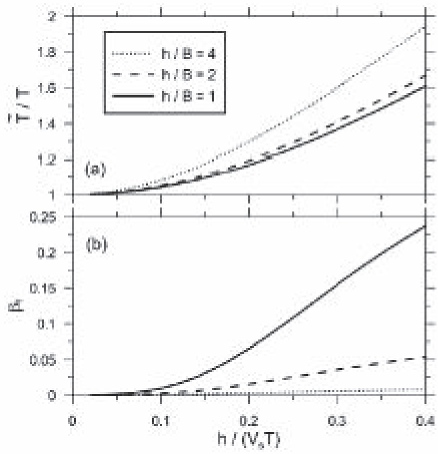


Figure 3. Foundation damping

Synopsis of Current Codes and Standards Related to Foundation Modeling

Recommendations for seismic regulations in building codes are adopted by *ASCE-7 Standard for Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 2016) a consensus national standard. The individual recommendations are initially developed, however, in the *NEHRP Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC 2015). Relevant parts of *ASCE 7-16* are summarized as follows with references to *ASCE 41-13 Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2014), *FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (ASCE, 2000), and *ATC-40 Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996).

ASCE 7-16 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 and Section 12.13.3 – Foundation Load-Deformation Characteristics may be used.

NIST GCR 12-917-21 provides more detailed guidance on SSI, including the following: 1) when the use of foundation springs and dampers are significant in an evaluation and what the effect on structural response parameters, 2) when is the consideration of differences between free-field ground motions and foundation input motions significant, and 3) what types of geotechnical exploration and testing are useful and/or needed for development of SSI modeling parameters

ASCE 7-16 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. With the current changes to integrate with Chapters 12 and 16 and to extend the procedure, it is expected that it will be used more frequently.

The substantial changes made to Chapter 19 in ASCE 7-16 include:

- The introduction of formulas for the stiffness and damping of rectangular foundations;
- Revisions to formulas for the reduction of base shear for soil-structure interaction for the linear procedures that includes the effect of the R factor;
- Reformulation of the effective damping ratio of a soil-structure interaction system including hysteretic foundation damping;

- The addition of a period lengthening ratio, associated with the effective damping ratio, based upon ductility demand and;
- The addition of kinematic interaction provisions (base-slab averaging and embedment).

Damped Structural Demands

The Chapter 19 Equivalent Lateral Force Procedure base shear equation 19.2-1 can be re-written as:

$$\tilde{V} = V - \Delta V = C_s W - \left[C_s - \frac{\tilde{C}_s}{B_{SSI}}\right]\overline{W} = C_s\left(W - \overline{W}\right) + \left(\frac{\tilde{C}_s}{B_{SSI}}\right)\overline{W} \geq \alpha V$$

The reduction in base shear used to be applied to the mass of building associated with the fundamental period of vibration, so the term $C_s\left(W - \overline{W}\right)$ represented the portion that is not subject to reduction. The term $\tilde{C}_s$ is calculated using the fundamental period from an analytical structure model, which directly incorporates horizontal, vertical and rotational foundation and soil flexibility. The analytical model should be a complete detailed model of the structure. The spectral acceleration associated with this adjustment for period lengthening is further reduced by the term $1/B_{SSI}$ that accounts for the increase in system damping (over the 5% normally assumed for fixed base) caused by the foundation damping, β_o . ASCE/SEI 7-16 permits $\overline{W}$ to be equal to W, therefore the modified base shear including SSI effects can be rewritten as:

$$\tilde{V} = \left(\frac{\tilde{C}_s}{B_{SSI}}\right) W \geq \alpha V$$

The introduction of the R factor to determine the soil-structure interaction base shear reduction limit in the Equivalent Lateral Force Procedure and the Modal Response Spectrum Procedure is intended to align the theoretical linear elastic models of the structure and geologic media used to the R factor seismic design methodology in ASCE 7. Larger soil-structure interaction reductions in base shear are permitted for lower R factors used in building seismic design since it is expected that those buildings will have limited inelastic response and align more with the soil-structure interaction equation derivations. However, there exists flexible, fixed-based Seismic Force-Resisting Systems with low R factors (such as Ordinary Moment Frames) that have limited soil-structure interaction due to their inherent flexibility.

For the nonlinear response history analysis procedure, it is important to note that in ASCE 7-16 Section 19.1.1, only the foundation and soil flexibility need be incorporated in the model of the structure. Soil damping is either added through ASCE 7-16 Section 19.3 or dashpots/nonlinear inelastic elements are explicitly modeled in the soil elements, but not both together.

Kinematic Interaction Modified Structural Demands

The ASCE/SEI 7-16 and ASCE/SEI 41-13/17 provisions for kinematic interaction 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. ASCE/SEI 7-16 limits the kinematic interaction effects to the nonlinear response history analysis procedures in conjunction with site specific response spectrum under Chapter 21. Kinematic interaction effects include Base Slab Averaging and Embedment. Kinematic interaction will typically reduce the spectral ordinates in the acceleration (or short) period range of that response spectrum. These effects result from modification of the ground motion waves by the foundation system and are reflected in response spectrum modification factors RRS_{bsa} for Base Slab Averaging and RRS_e for Embedment. Essentially, for Base Slab Averaging and Embedment, $RRS(ASCE\ 7\text{-}16) \approx 0.25 + 0.75\ RRS(FEMA\ 440)$. The conservatism indicated in the

ASCE 7-16 Commentary using the 0.75 factor has been neutralized by the addition of 0.25 in the equations. For example, the embedment factor result in ASCE 7-16 is greater than that from FEMA 440 for a period of 0.5 seconds, 20 feet embedment and a shear wave velocity of 650 feet/second by only about 2%.

The effects of kinematic interaction, or embedment and base slab averaging, are limited in ASCE 7-16 Chapter 19. The provisions for Kinematic Interaction cannot be used for soil site class F and for the Equivalent Lateral Force Procedure or the Modal Response Spectrum Analysis procedures, unless site specific study is required for soil site class F. In ASCE 7-16, the product of RRS_{bsa} and RRS_e cannot be less than 0.7. This limit mitigates steep theoretical acceleration reductions in the response spectrum in the short period range that have little experiential basis. The RRS_{bsa} and RRS_e reductions in the accelerations of the response spectrum should not be used for design of non-structural components using ASCE 7-16 Section 13.3.1.1 that determines F_p, but may be used indirectly based on structure response for design of non-structural components for seismic relative displacements.

To take advantage of this interaction the foundation components must be laterally connected to form a rigid diaphragm and meet specific soil site classifications. For Base Slab Averaging, the building should have rigid diaphragms. For embedment kinematic interaction, the building should have a basement, although it is not required by ASCE 7-16.

Where the nonlinear response history analysis procedure is used, it is implied that time history records are scaled to the reduced spectral acceleration ordinates resulting from the kinematic interaction. Note that under ASCE 7-16, the site specific response spectrum reduced for kinematic effects cannot be less than 80% of the site specific response spectrum alone and not less than 70% of the code design response spectrum and MCE_R spectrum where used. However, where the site specific response spectra and associated acceleration time histories are generated at the base of the structure, and not at the ground surface, the embedment kinematic interaction provisions and reduction cannot be used.

ASCE 7-16 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 should be used in conjunction with Chapter 19. This flexible foundation methodology can also be used for the seismic response history procedures in Chapter 16, as permitted by Chapter 19. 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 first started with the *NEHRP Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 2003), Appendix to Chapter 7, Section A7.2. The Appendix to Chapter 7 attempted 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.

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_o, 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_o 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/SEI- 16 Tables 19.3-1 and 19.3-2. Reduction factors for large strain G are identical to Table 4-7 given in FEMA 356 and Table 8-2 of ASCE/SEI 41-13/17. Further guidance for testing necessary for evaluating properties are provided in NIST GCR 12-917-21.

ASCE 41-17 Section 8.5 Soil Structure Interaction Effects. Section 7.2.7 requires Soil Structure Interaction effects be modeled explicitly, including the flexibility and damping of individual foundation elements. However, this Section also permits the use of β_{SSI} for the structure-foundation system damping for the LSP, LDP and NSP analyses. Explicitly accounting for damping in individual foundation elements in linear or static procedures is not recommended.

Kinematic interaction effects are similar to those given in ASCE 7-16 Chapter 19. The equations for base slab averaging are identical to those in ASCE 7-16 Chapter 19, except for the equation for b_o. The applicability of base slab averaging to soil site classes differ from that in ASCE 7-16 Chapter 19, where soil site class F is not permitted by ASCE 41-17. Also, for base slab averaging, rigid floor and roof diaphragms and interconnected foundation elements are required similar to ASCE 7-16 Chapter 19. However, ASCE 41-17 Section 8.5.1.1 for base slab averaging requires that the foundation elements be stronger than the vertical elements of the lateral force resisting system and that the flexible base period of the structure be multiplied by 1.5 when the LSP and LDP are used. The equation for embedment is identical to ASCE 7-16 Chapter 19. ASCE 41-17 Section 8.5.1.2 for embedment also requires that the foundation elements be stronger than the vertical elements of the lateral force resisting system and that the flexible base period of the structure be multiplied by 1.5 when the LSP and LDP are used. RRS_e cannot be less than 0.50, which is in agreement with FEMA 440 (ATC,2005), and the embedment provisions cannot be used for soil site class F, contrary to that in ASCE 7-16 Chapter 19. In ASCE 41-17, the product of RRS_{bsa} and RRS_e cannot be less than 0.5, which results in a more drastic acceleration reduction in the short period range of the response spectrum than permitted by ASCE 7-16 Chapter 19. In ASCE 41-17, reductions in the accelerations for the short periods of the response spectrum for kinematic interaction are permitted for all ASCE 41 analysis procedures as implied in Section 7.2.7.

The ASCE/SEI 41-17 provisions for foundation damping are also similar to that in ASCE 7-16, but without the limitation of reduction as a function of the R factor for the building. However, ASCE 41-17 does not contain radiation damping provisions for circular foundations as ASCE 7-16 does. ASCE 41 has expanded the provisions to all analysis procedures in ASCE 41. However pseudo-lateral force or target displacement demands due to kinematic interaction and damping effects using the LSP, LDP or NSP analysis procedures, cannot be less than 70% of that calculated without the inclusion of SSI effects.

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 for the NSP, LSP and LDP analyses are applied as a damping factor β_{SSI}, where the spectral acceleration ordinates of the response spectrum are divided by β_{SSI}. The β_{SSI} damping factor consists of the viscous damping of the structure β prior to yield plus the damping due to the radiation damping of the soil β_f. The β value is modified by the period lengthening due to the anticipated ductility demand on the structure. ASCE 41-17 Section 7.2.7 now requires that foundation damping be modeled explicitly in individual foundation elements for the NDP analyses. GCR 12-917-21 (NIST 2012) should be consulted on the procedure to model foundation damping in individual footing elements.

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 41-17 Figure 8-2. 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 ASCE 41-17 Figure 8-6 Passive Pressure 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 8.4.3.1. 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 41-13/17 Sections 8.4.2.4 through 8.4.2.6. 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 or trilinear. The ultimate soil bearing capacity should be taken as the bearing capacity failure load and not the soil bearing capacity as limited by long-term settlement. 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.

Acceptance criteria for rectangular and I shaped shallow footings on a flexible base as limited by soil rotation are given in ASCE 41-17 Table 8-4.

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.

Pile Foundations. 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. Refer to ASCE 7-16 Section 12.13.9.3.1 provides design requirements for downdrag.

- For additional information on pile foundations, see the EERI Seminar Series “Performance-Based Earthquake Engineering for Structural and Geotechnical Engineers: Seminar 2, Practical Applications to Deep Foundations, Buildings, Bridges, and Ports,” Los Angeles, 2008.

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. For additional information, see ASCE 7-16, Section 12.13.8.3, and the corresponding commentary for the evaluation of pile-soil interaction.

References

ASCE (2006). *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA.

ASCE, (2014), *Seismic Evaluation and Retrofit of Existing Buildings, ASCE 41-13,* Structural Engineering Institute, Reston, VA.

ASCE (2010). *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA.

ASCE (2017). *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

ATC (1996), *The Seismic Evaluation and Retrofit of Concrete Buildings,* volumes 1 and 2, ATC-40 Report, Applied Technology Council, Redwood City, CA.

ATC (2005), *Improvement of Nonlinear Static Seismic Analysis Procedures,* FEMA 440 Report, February 2005, Applied Technology Council, Redwood City, CA.

BSSC (2003). *NEHRP Provisions for Seismic Regulations for New Buildings and Other Structures*, Federal Emergency Management Agency, Building Seismic Safety Commission, Washington, DC.

Ensoft (2004). *Computer program LPILE plus version 5.0,* Ensoft Inc., Austin, Texas, July 2004.

FEMA (2015). *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, FEMA P-1050-1, Federal Emergency Management Agency, Building Seismic Safety Commission, Washington, DC.

FEMA (1997). *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, Federal Emergency Management Agency, Washington, DC.

FEMA (2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356. Federal Emergency Management Agency, Washington, DC.

FEMA (2005). *Improvement of Nonlinear Static Seismic Analysis Procedures*, FEMA 440, Federal Emergency Management Agency, Washington, DC.

SEAOC (2001). USD/LRFD/Limit state approach to foundation design. *Proceedings*, *70th Annual SEAOC Convention Proceedings*, Sacramento, CA.

NIST (2012). “Soil-Structure Interaction for Building Structures”, *GCR 12-917-21*, National Institute of Standards and Technology, Gaithersburg, MD.

SEAOC Seismology Committee (2019). “Limit state design of piles, pile caps, and grade beams,” 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Keywords
foundation modeling
foundations
free-field motion
soil-structure interaction

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). “Foundation Modeling,” August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Steel SFRS Connections to the Foundation

ASCE 7-16 reference section(s)	2019 CBC / 2018 IBC reference section(s)	Other standard reference section(s)
12.2.5.2 12.4.3 14.1.2.2 14.2.2.2	1809 2205.2.1 2205.2.2	AISC 341 (2016 Seismic Provisions Including Commentary), Chapters A through F ACI 318-14, Chapter 17, Chapter 18

System Considerations

It is important to recognize the expected yielding mechanism and ultimate limit state of a steel seismic force-resisting system (SFRS), so that connections to the foundation are designed and detailed to be compatible with performance expectations of the chosen system.

The connection of a steel SFRS to the foundation is critical to achieving the ductile performance desired. The initial yielding in a moment frame system will often occur at the base connection, unless explicitly designed otherwise. Cantilever column systems, unless designed to yield within the foundation system, will most likely yield at the column base, either through yielding of the column section or within the base connection. On the other hand, only in special circumstances should the base connection of a steel frame or cantilevered column to the foundation, when designated as fully-rigid, be designed as the primary yielding mechanism. It is difficult to ensure that ductile yielding will occur in this usually complex connection, especially under high inelastic rotations. In general, the foundation connection of moment frames should either be capable of accommodating the required inelastic rotations or have the strength and stiffness necessary 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 horizontal and vertical force components due to the full tensile strength of the brace(s). Likewise, 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 shear strength of all the links above, as determined by AISC 341, Section F3.3.

Due to its critical nature, the base connection of a cantilever column to the foundation, as well as the foundation, should be able to develop the full capacity of the column, including expected overstrength of the system.

Code Issues

CBC Section 2205 requires that steel structures be designed and detailed in accordance with AISC 360, and for seismic requirements, AISC 341. Anchor rods or bolts into concrete are to be designed and detailed per ACI 318-14, Chapter 17, as modified by AISC 341, Section D2.6. The SEAOC Seismology Committee recommends complying with ASCE 7-16 Section 14.2.2.2, which calls for seismic hooks on all transverse reinforcing around anchor bolts placed at the top of a column or pedestal, even though they are not required by the CBC. AISC 341 also has a general discussion on column bases in the Commentary for Section D2.6 that is summarized below.

Column Bases

Column bases should be designed using a capacity-based design approach, where the base connection should have adequate strength to permit the expected ductile behavior for the selected SFRS, the basis of design requirements in AISC 341.

Column bases are required to be designed for the same axial and shear forces as those required for 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 base connection must also be designed for those loads.

In many cases, AISC 341 requires that welds at column-to-baseplate connections be demand critical. System-specific requirements are described in the AISC 341 Sections listed below.

Moment Frame Systems

- For Special Moment Frames (SMFs) per Section E3, and Intermediate Moment Frames (IMFs) per Section E2, 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 D2.6 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 push yielding into the connecting element (column or foundation). Note that IMFs are not permitted in Seismic Design Categories E and F (see exceptions in ASCE 7).

- For Ordinary Moment Frames (OMFs), the connection to the foundation should be designed for equivalence with E1. OMFs are not permitted in Seismic Categories D, E, and F (see exceptions in ASCE 7).

- For Special Truss Moment Frames (STMFs), the connection to the foundation should be designed for equivalence with Section E4. STMFs are not permitted in Seismic Category F (see exceptions in ASCE 7).

Cantilevered Column Systems

- For Special Cantilever Column Systems (SCCSs), the connection to the foundation should be designed for the requirements of E6.6b and ASCE 12.2.5.2. SCCSs have limited applications for use in Seismic Categories D, E, and F as defined in ASCE 7 Table 12.2-1 (height limit of 35 feet). In addition to story height limitations, ASCE 12.2.5.2 limits the required axial strength of the column, including seismic effects, to no more than 15% of the available axial strength. ASCE 12.2.5.2 further requires that foundation and connections that provide overturning resistance be designed to resist seismic effects, including overstrength factor (Ω_0), or be designed using a capacity-based approach where the connection can develop the inelastic flexural capacity of the column.

- Ordinary Cantilever Column Systems are not permitted in Seismic Categories D, E and F per ASCE 7.

Braced Frame Systems and Special Plate Shear Walls (SPSW)

- For SCBFs, the connection to the foundation should be designed for equivalence with Section F2.6, 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:

  - The expected yield strength, in tension, of the bracing member, determined as $R_yF_yA_g$ (LRFD) or $R_yF_yA_g/1.5$ (ASD), as appropriate.
  - 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 F1.6. OCBFs are not permitted in Seismic Category F (see exceptions in ASCE 7).

- For EBFs, the connection to the foundation should be designed for equivalence with Section F3.6.

- For buckling-restrained braced frames (BRBFs), the connection to the foundation should be designed in a similar manner as other braced frame systems. In addition to meeting requirements of Section D2.6, column base connections that are connection points with a BRBF member should have the required strength as defined in Section F.6c, where the adjusted brace strength is defined in Section F4.2a. Here again, the design goal is to develop the full capacity of the system's defined yielding mechanism (axial yielding of the brace).

- For SPSWs, the connection to the foundation should be designed based upon Section F5.3. Connection to the foundation of vertical boundary elements (VBE) and, if present, horizontal boundary elements (HBE) should have the required strength based upon the load combinations that include amplified seismic force.

Anchor rods

The most common connections of steel frames to foundations involve cast-in anchor rods. Headed anchor rods or threaded rods with heavy hex nuts (with or without plate washers) generally provide uplift resistance. The failure modes (limit states) of anchor rods are defined in ACI 318-14 Chapter 17, Section 17.3.1. Whenever feasible, anchor rods should be sized and/or embedded sufficiently to preclude a brittle concrete pullout failure. The head of the anchor rod provides sufficient anchorage bearing, so the use of an additional plate washer, unless unusually thick, may not add significantly to the anchorage.

Anchor rods are available in a variety of ASTM specifications, including ASTM A307, ASTM A193-B7 and ASTM A449 high-strength anchor rods (minimum tensile strengths from 58 ksi up to 125 ksi). ASTM F1554 is the most common specification for anchor rods. ASTM F1554 anchor bolts may be hooked, headed, or threaded with nuts. Three strengths are available (yield strengths of 36 ksi, 55 ksi, and 105 ksi), and the specification covers the use of galvanized and weldable material (grade and weldability is specified).

There is extensive literature on designing anchor rods embedded in concrete to account for the combined effects of tension and shear. ACI 318-14, Section 17.3.1.3 requires that interaction of shear and tension be designed using tested data or the provisions of Section 17.6. 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 used for braced frame anchorage, with oversize holes and the rod passing through either a built-up grout pad or a grout pocket below the base plate. ACI 318 does allow the use of built-up grout pads, with a reduction of 20 percent in the nominal shear strength of the anchor rods (17.5.1.3), although no supporting commentary is supplied. It is not recommended to use anchor rods for the transfer of large shear loads with grout pads. Regardless of the use of grout pads, for large seismic 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 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 wall reinforcing. For high-strength applications of this type, threaded bars (ASTM A722) may be used. A special case occurs where anchor rods are expected to yield in tension. Adequate concrete embedment is required, but it is also required that 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 anchor rods up the side of the column flange and provide a bearing bracket on the face of the flange, which engages the anchor rods' nuts, will also provide strain lengths 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. Recommended maximum baseplate hole diameters and minimum plate washer sizes are provided in Table 14-2 of the AISC *Steel Construction Manual*. Oversized hole diameters are provided in Table C-9.1 of AISC 360. 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 holes. 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. Commentary in AISC 360 Section C-J9 explains that these recommendations are based upon the assumption that shear forces at the base plate are transferred to the foundation through friction, and not directly through the bolts in bolt bearing. It further cautions that bases of braced frame or moment frame columns, where larger shear forces and lower gravity loads are present, should be embedded into the foundation or that shear keys be used. Furthermore, using embedded base plates or shear keys will help to limit excessive lateral deflections at the foundation due to slip.

Rod bending should be considered when a column base plate is installed above the concrete surface and a built-up grout pad is used. Since a round shape is an inefficient bending member, only relatively small shear forces can be transferred by rods (see rod shear reduction in ACI 318 Section 17.5.1.3). 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 tensioning of high-strength anchor rods is performed, 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 Seismic Force Resistance

AISC 341-16, Commentary, C-D2.6, provides guidance on the design and detailing of high-seismic shear transfer mechanisms. Examples of base connection shear transfer mechanisms are shown in Figure C-D2.5 (AISC 341). Note that C-D2.5 identifies two conditions of high strength grout use; a grout pad, where the base plate sits upon a built up pad of grout, and a grout pocket, where the base plate (or shear key) in embedded within a pocket formed in the concrete. With a grout pocket, the space between the concrete surfaces and the base plate are filled with grout that is used to transfer shear forces into the foundation through bearing.

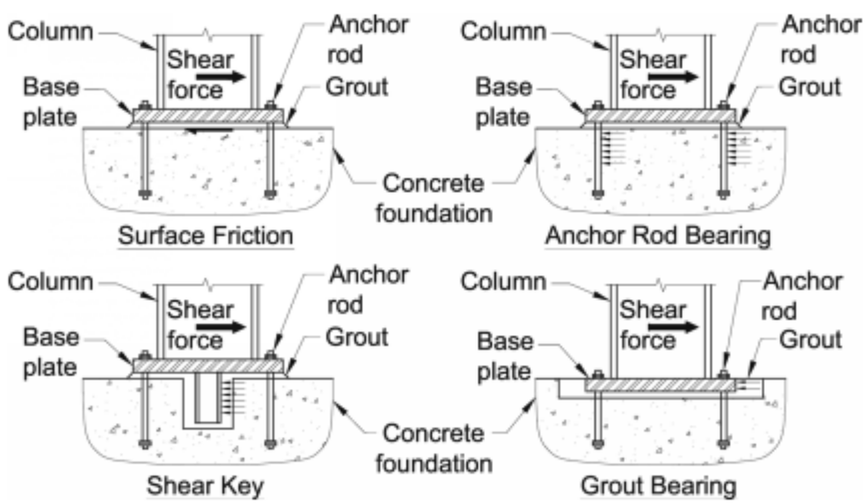


Fig. C-D2.5. Shear transfer mechanisms—column supported by foundation.

Shear lugs welded to base plates are often used when a direct transfer of shear to the foundation concrete is needed. Where the base plate itself is embedded, direct bearing on the edge of the base plate can be adequate for lower loads. The lug can be designed as a cantilever through the depth of the grout pocket 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. The anchor rods or other reinforcing crossing that plane can then be checked per 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 eliminates the need for weld washers and mitigates the grout placement issue associated with shear lugs. Shear studs may be designed utilizing PCI and AISC provisions.

Another effective method to transfer shear into the foundation is the use of steel shape or reinforcing bars welded directly to the brace gussets and/or base plates. An alternative approach is to provide a reinforced recess in the slab, so that direct bearing of the column on concrete may be used for shear transfer. In either case, reinforcing dowels adequate to transfer shear between the slab and the foundation system are required.

Flexurally Rigid, Partially-Rigid and Simple (Pin) Base Connections

In addition to base connections that transfer shear and axial load, AISC 341-16, Commentary, C-D2.6c, provides guidance on the design and detailing of high-seismic connections, where flexural strength is required. With steel systems using moment frames, cantilever columns, or brace frames requiring additional stiffness at the base, there is often a need to provide connections to foundations that can transfer significant bending moments in conjunction with large seismic shears. Historically, earthquakes including Northridge (1994) and Kobe (1995) have shown numerous cases where significant damage and failure have been observed to column base-plate connections (Grauvilardell, et al, 2005). These failures highlight the need to better understand the true behaviors of these connections; force transfer, the ability to resist loads while undergoing significant rotations (whether the design is Rigid, Simple or Semi-Rigid), and the ultimate failure mechanisms (limit state).

Where a flexurally stiff connection is desired to reduce drift, providing a base connection that is embedded into the concrete (especially within a concrete grade beam) has been shown to provide a significantly more rigid condition than a surface mounted base plate with bolts set to the outside of the column flanges. Although both details may be designed as a “Fixed-base” condition, they are likely to have significantly different rotational stiffness and ductility. Figure C-D2.7 and Figure C-D2.8 (AISC 341) provide examples of embedded and surface mounted rigid connections.

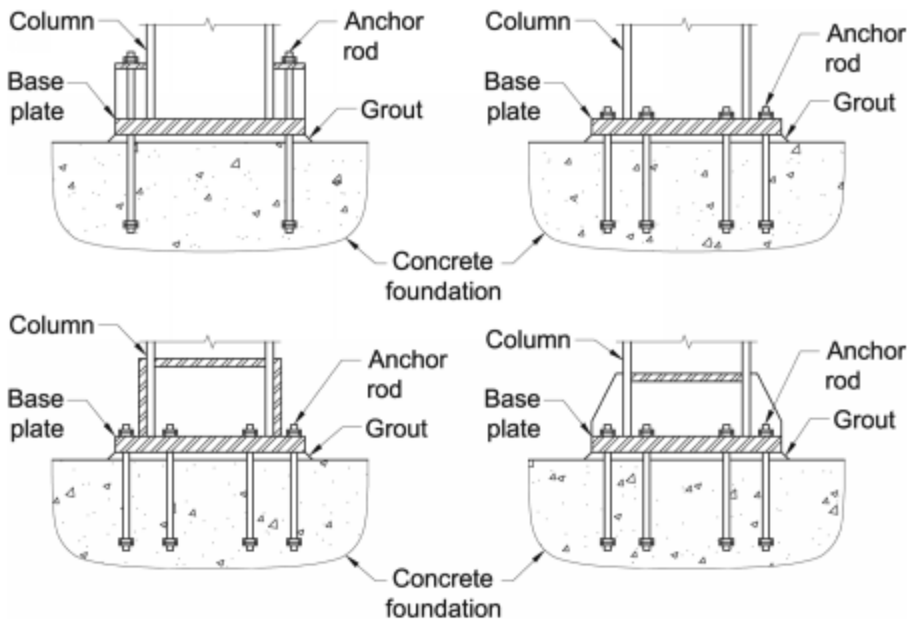
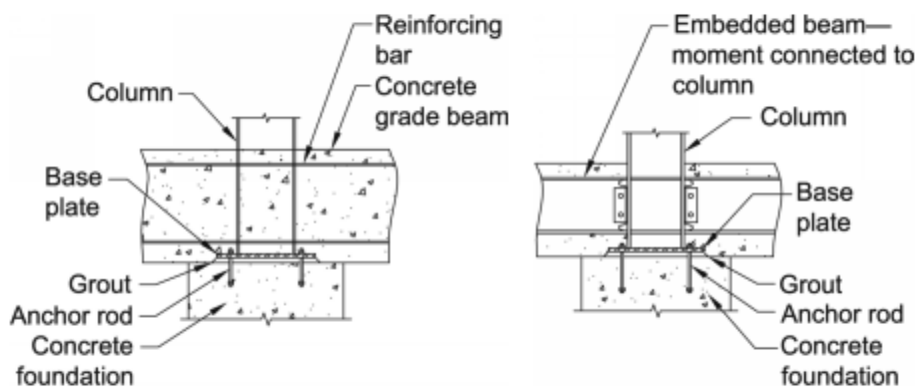


Fig. C-D2.7. Example of "rigid base" plate assembly for moment frames.



(a) (b)

Fig. C-D2.8. Examples of column base fixity in a grade beam.

Similarly, although a connection may be considered and designed as simple (pinned) with little or no moment carrying capacity, base plate connections are not true pins and will resist some flexural forces. It is important that elements within the base connection be detailed to minimize, or be designed to resist, the secondary moments induced by the expected semi-rigid behavior.

Grauvilardell, et al, 2005, provides an informative synopsis of past research and analysis studies into the testing of column base plate connections used in seismic resisting steel systems.

References

ACI (2014). ACI 318-14: Building code requirements for structural concrete, with commentary, American Concrete Institute, Farmington Hills, MN.

AISC (2016). Seismic provisions for structural steel buildings (ANSI/AISC 341-16), July 12, 2016, American Institute of Steel Construction, Chicago, IL.

ASCE (2017). ASCE/SEI 7-16, Minimum design loads for buildings and other structures, American Society of Civil Engineers, Reston, VA.

Shipp and Haninger (1983). "Design of headed anchor bolts." Engineering Journal 2 (2, 2nd quarter) pp. 58-69, AISC.

J. Grauvilardell, D. Lee, J. Hajjar and R. Dexter (2005). "Synthesis of Design, Testing and Analysis Research on Steel Column Base Plate Connections in High-Seismic Zones." University of Minnesota Structural Engineering Report No. ST-04-02.

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 2019)

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

SEAOC Seismology Committee (2008). "Steel SFRS Connections to Foundations," August 2019, *The SEAOC Blue Book: Seismic design recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
ASCE 7-16 Chapter 12 Chapter 15 14.1.2	22, 22A	ANSI/AISC 341-16 ANSI/AISC 360-16 AISC 358-16 AISC-303-16 2018 IBC, section 2205.2

Steel Lateral Systems

Steel lateral resisting systems can be economically and architecturally desirable for numerous building configurations. There are three main lateral force resisting system types: braced frames, moment frames, and steel plate shear walls. Each have their advantages and disadvantages regarding design complexity, architectural desirability, post-earthquake functionality and repairability. Each system also has limitations within ASCE 7, including allowable building heights. The systems recognized by ASCE 7 are summarized below. For an in-depth history of the development of the AISC Seismic Design Manual, refer to the 2008 version of this article.

The design of flexible, ductile systems, including Buckling Restrained Braced Frames and Special Moment Resisting Frames, is frequently governed by drift. For these systems, residual drift can impede post-disaster recovery. The SEAOC Seismology Committee recommends that the designer consider more stringent allowable story drift limits if seismic resilience is a concern. For example, a Risk Category II building could be designed to the drift limits of a Risk Category III building per ASCE 7-16 Table 12.12-1.

Steel Ordinary Concentric Braced Frame (OCBF)

Advantages

- Relatively simple design and construction procedures, ideal for smaller buildings and non-building structures.
- Lower drift values may limit damage to finishes and nonstructural components in the building in comparison to moment frames.
- OCBFs tend to be more economical in material, fabrication, and erection than moment frame and eccentrically braced frame systems.

Disadvantages

- Reduced flexibility in floor layout compared to moment frame and eccentrically braced frame systems.
- R = 3.25, resulting in larger design loads in diaphragms, collectors, and footings.
- Limited allowable use in higher seismic design categories.
- Less reliable yielding and buckling modes of response than SCBF's or BRBF's; connection fractures.
- Global brace and brace-beam buckling and their effects on finishes.

Limits

- 35 feet in Seismic Design Category D and E. Per footnote f in Table 15.4-1, 65 feet in non-building structure pipe racks.160 feet in non-building structures with R = 2.5. No limit in non-building structures with R = 1.5, these frames are designed using AISC 360 only.
- Not permitted in Category F. Per footnote j in Table 12.2-1, certain single-story buildings up to 60 feet tall may use an OCBF system. Per footnote f in Table 15.4-1, 65 feet in non-building structure pipe racks. 100 feet in non-building structures with R = 2.5. No limit in non-building structures with R = 1.5, these frames are designed using AISC 360 only.

History and Performance in Past Earthquakes

- OCBF buildings have experienced extensive damage in past earthquakes.
- In use in the U.S. since primarily the 1940's.

References for More Specific Guidance and Relevant Research Results

1. "Comparative study of special and ordinary braced frames" The Structural Design of Tall and Special Buildings. 22. 10.1002/tal.750. Akbas, Bulent & Sutchiewcharn, Narathip & Cai, Wenyu & Wen, Rou & Shen, Jay. (2013).

2 "Performance Assessment of Concentrically Braced Steel Frames," 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada August 1-6, 2004 Paper No. 1639, Patxi URIZ1, Stephen A. Mahin.

3. "Seismic Evaluation and Upgrading of Chevron Braced Frames" Journal of Constructional Steel Research 59 (2003) 971–994 D.C. Rai, S.C. Goel, 6 January 2003.

Steel Special Concentric Braced Frame (SCBF)

Advantages

- Lower drift values may limit damage to finishes and nonstructural components in the building than compared to moment frames.
- More economical in material, fabrication, and erection than moment frame and eccentrically braced frame systems.
- R = 6, resulting in lower design forces than OCBF's for collectors, diaphragms and foundation elements.
- Allowable in all Seismic Design Categories.
- Higher allowable height limits in Seismic Design Category D and E than ordinary concentric braced frames.
- More reliable yielding and buckling modes of response than OCBF's

Disadvantages

- Reduced flexibility in floor layout compared to moment frames and eccentrically braced frames.
- Lower height limits than moment frames.
- Global brace buckling and their effects on finishes

Limits

- 160 Feet in Seismic Design Category D and E, 100 feet in Category F.

History and Performance in Past Earthquakes

- No information currently available from past earthquakes.
- In use since the late 1990's

References for More Specific Guidance and Relevant Research Results

1. NEHRP Seismic Design Technical Brief No. 8 "Seismic Design of Steel Special Concentrically Braced Frame Systems" Rafael Sabelli, Charles W. Roeder, Jerome F, Hajjar

2. "Seismic Performance of Concentrically Braced Frames With and Without Brace Buckling." Engineering Structures. 141. 461-481. 10.1016/j.engstruct.2017.03.043. Shen, Jay & Seker, Onur & Akbas, Bulent & Seker, Pinar & Momenzadeh, Seyedbabak & Faytarouni, Mahmoud. (2017).

3. "Comparative Study on the Seismic Behavior of MRF Systems with SCBF and EBF Systems" 14th European Conference on Earthquake Engineering (ECEE), At Macedonia, Kioumarsi, Mahdi & Emdad, Reza & Taghvaee, Ali & Tehranizadeh, Mohsen.

Steel Eccentrically Braced Frame

Advantages

- R = 8, resulting in lower design loads on diaphragms, collector elements, and smaller footings.
- Yielding generally concentrated in links
- Global brace buckling and their effects on finishes avoided

Disadvantages

- Takes up more visual “space” in the building than a moment frame system, but perhaps less than other brace frame systems.
- Large beam sizes to accommodate large seismic loads.
- Lower height limits than moment frame systems.
- More expensive than other braced frame systems in fabrication and erection.

Limits

- 160 feet in Seismic Design Category D, E, and F.

History and Performance in Past Earthquakes

- Mixed performance in the Christchurch New Zealand earthquakes in 2010/2011.
- In use since the 1970s in Japan and the 1980s in the U.S.

References for More Specific Guidance and Relevant Research Results

1. “A Review of Research on Steel Eccentrically Braced Frames” Journal of Constructional Steel Research. 128. 53-73. 10.1016/j.jcsr.2016.07.032. Kazemzadeh Azad, Sina & Topkaya, Cem.

2. “Comparative Study on the Seismic Behavior of MRF Systems with SCBF and EBF Systems” 14th European Conference on Earthquake Engineering (ECEE), At Macedonia, Kioumarsi, Mahdi & Emdad, Reza & Taghvaee, Ali & Tehranizadeh, Mohsen.

3. “Seismic Eccentrically Braced Frames,” Journal of Constructional Steel Research. Vol 10, pp 321-324, Popov & EngElhardt.

Buckling Restrained Braced Frame

Advantages

- R = 8, resulting in lower design loads on diaphragms, collector elements, and smaller footings.
- Yielding generally concentrated in BRB's and effects on finish due to global brace buckling avoided.
- Higher ductility and energy dissipation than other braced frame systems.

Disadvantages

- Primarily proprietary systems that require project team lock in with particular designers/manufacturers.
- Reduced flexibility in floor layout than moment frame and eccentrically braced frame systems.
- Lower height limits than moment frame systems.

Limits

- 160 Feet in Seismic Design Category D and E, 100 feet in Category F.

History and Performance in Past Earthquakes

- In use since the late 1990's in the U.S. and since the 1970's in Japan
- No earthquake performance observations currently available

References for More Specific Guidance and Relevant Research Results

1. NEHRP Seismic Design Technical Brief No. 11" Seismic Design of Steel Buckling Restrained Braced Frames, A Guide for Practicing Engineers" NIST GCR 15-917-34 Ryan A. Kersting Larry A. Fahnestock Walterio A. López 2015.

2. Buckling-Restrained Braced Frame Connection Performance" Journal of Constructional Steel Research Volume 66, Issue 1, January 2010, Victoria R. Wigle, Larry A Fahnestock

Special Plate Shear Wall

Advantages
- Thinner walls than an equivalent concrete shear wall system, resulting in more useable floor area.
- Lower building weight than an equivalent concrete shear wall system.
- R = 7, resulting in lower design loads on diaphragms, collector elements, and smaller footings.

Disadvantages
- Contractor unfamiliarity with construction, as this is a rarely used system.
- Lower height limits than moment frame systems.
- Higher drifts than braced frame system, unless alternate methods of stiffness are added.

Limits
- 160 Feet in Seismic Design Category D and E, 100 feet in Category F.

History and Performance in Past Earthquakes
- In limited use since the 1970s.
- Sylmar Hospital in the 1994 Northridge Earthquake experienced no significant structural damage but wide-ranging nonstructural damage. Minor damage in the 1985 Kobe Earthquake.

References for More Specific Guidance and Relevant Research Results
1. AISC Design Guide 20: Steel Plate Shear Walls.

2. "Steel Plate Shear Walls: Practical Design and Construction" Modern Steel Construction April 2005 Ignasius F. Seilie, P.E. and John D. Hooper, P.E.

3. "Experimental Seismic Study on Shear Walls with Fully-connected and Beam-only-Connected Web Plates", Journal of Constructional Steel Research Volume 141, February 2018, pp 204-215, B.Shekastehband, A.A. Azaraxsh, H. Showkati

Steel Ordinary Moment Frame (OMF)

Advantages

- Simple design and fabrication of connections.
- Economical for construction.
- Provides flexibility in wall locations and routing of MEP systems.

Disadvantages

- R = 3.5, resulting in larger design loads in diaphragms, collectors, and footings.
- Higher building drifts require design considerations in nonstructural components.
- Fractures can propagate through or near beam-column joints

Limits

- Not permitted in Seismic Design Categories D, E and F. There are exceptions to these limits for special use buildings outlined in ASCE 7-16 section 12.2.5.6. For non-building structures, footnote h in Table 15.4-1, permits use up to 65 feet in pipe racks "where the moment joints of field connections are constructed of bolted end plates". Footnote i, permits use up to 35 feet in pipe racks with no field connection exceptions. There is a 100 feet limit in Seismic Design Category D and E non-building structures with R = 2.5. No limit in Seismic Design Categories A-F for non-building structures using R = 1, these frames are designed using AISC 360 only.

History and Performance in Past Earthquakes

- Ordinary Moment frames have performed poorly in past earthquakes. However, when restrained by nonstructural components in buildings that may perform like dual systems, they have provided reliable backup resistance after stiffer elements failed in many past earthquakes, notably during the 1906 San Francisco earthquake.
- Some older pre-Northridge Earthquake welded steel moment frames may perform similar to ordinary moment frames
- Information about past earthquake performance of pre-Northridge moment frames documented in FEMA 355E after the Northridge Earthquake
- In use since the 1880s with bolted or riveted frames prior to the 1960s and welded frames since the 1960s

References for More Specific Guidance and Relevant Research Results

1. AISC Design Guide 3: "Serviceability Design Considerations for Steel Buildings"

2. "A Policy Guide to Steel Moment Frame Construction" SAC Joint Venture November 2000, Ronald O Hamburger

Steel Intermediate Moment Frame

Advantages

- Less complicated design and fabrication of connectors.
- Provides flexibility in wall locations and routing of MEP systems.

Disadvantages

- R = 4.5, resulting in larger design loads in diaphragms, collectors, and footings.
- Higher building drifts require design considerations in nonstructural components.
- Larger and heavier columns and beams than equivalent braced frame system.
- Lower height limits than Steel Special Moment Frame. Fractures can propagate through or near beam-column joints

Limits

- 35 feet in Seismic Design Category D.
- Not permitted in Seismic Design Category E and F. There are exceptions to these limits for special use buildings outlined in ASCE 7-16 section 12.2.5.7.
- For non-building structures 35 feet in Seismic Design Category D, not permitted in Seismic Design Category E and F. Footnote h in Table 15.4-1, permits use up to 65 feet in pipe racks “where the moment joints of field connections are constructed of bolted end plates”. Footnote i, permits use up to 35 feet in pipe racks with no field connection exceptions. There is a 160 feet limit in Seismic Design Category D and E and 100 foot in Seismic Design Category F for non-building structures using R = 2.5. No limit in Seismic Design Categories A-F for non-building structures using R = 1, these frames are designed using AISC 360 only.

History and Performance in Past Earthquakes

- Ordinary Moment frames have performed poorly in past earthquakes.
- Some older pre-Northridge Earthquake welded steel moment frames may perform similar to intermediate moment frames
- Information about past earthquake performance of pre-Northridge moment frames documented in FEMA 355E after the Northridge Earthquake
- In use since the late 1990’s.

References for More Specific Guidance and Relevant Research Results

1. “Behaviour of Steel Intermediate Moment Frames Designed According to Iranian National Building Code Under Lateral Load” 15th World Conference on Earthquake Engineering, H. Ramezansefat, A.A. Aghakouchak & S. Shahbeyk

Steel Special Truss Moment Frame

Advantages

- Longer floor and roof spans than traditional moment frame systems.
- R = 7, resulting in lower design loads on diaphragms, collector elements, and smaller footings.

Disadvantages

- Higher building drifts require design considerations in nonstructural components.
- Larger and heavier columns and beams than equivalent braced frame system.
- Lower height limits than special steel moment frame.
- Global buckling of link members can affect finishes.

Limits

- 160 Feet in Seismic Design Category D and E, 100 feet in Category F. There are no special use building exceptions for this system.

History and Performance in Past Earthquakes

- No earthquake performance information currently available.
- In use since the mid-1990s.

References for More Specific Guidance and Relevant Research Results

1. NEHRP Seismic Design Technical Brief 2: "Seismic Design of Steel Special Moment Frames A guide for Practicing Engineers" NIST GCR 16-917-41 Ronald O Hamburger, James O. Malley.

2. "Performance-Based Plastic Design of Special Truss Moment Frames," Engineering Journal, American Institute of Steel Construction, Vol. 45, pp. 127-150. Chao, Shih-Ho; Goel, Subhash C. (2008).

3. "Seismic performance improvement of Special Truss Moment Frames Using Damage and Energy Concepts" Earthquake Engineering and Structural Dynamics Volume 44 Issue 7 2014 Ahmad Heidari Sadjad Gharehbaghi

Steel Special Moment Frame

Advantages
- R = 8, resulting in lower design loads on diaphragms, collector elements, and smaller footings.
- Only lateral system with no height limits in current Code.
- Higher ductility and energy dissipation than other moment frame systems.

Disadvantages
- Expensive connection design and fabrication.
- Higher building drifts require design considerations in nonstructural components.
- Larger and heavier columns and beams than equivalent braced frame system

Limits
- No limits to height in any Seismic Design Category

History and Performance in Past Earthquakes
- Good structural performance in the South Napa Earthquake of 2014 for one Reduced-Beam-Section (Dog-Bone) System that experienced significant nonstructural drift-related damage
- In use since the late 1990's

References for More Specific Guidance and Relevant Research Results
1. NEHRP Seismic Design Technical Brief No. 2: Seismic Design of Steel Special Moment Frames A Guide for Practicing Engineers" NIST GCR 16-917-41Ronald O. Hamburger, James O. Malley

2. "What Makes a Special Moment Frame Special?" Modern Steel Construction 04 2017, Behzad Rafezy.

3. "A Policy Guide to Steel Moment Frame Construction" SAC Joint Venture November 2000, Ronald O Hamburger

References

AISC (2016) *Seismic Design Manual 3rd Edition* American Institute of Steel Construction, Chicago, IL.

AISC (2016). *Specification for Structural Steel Buildings ANSI/AISC 360-16*, American Institute of Steel Construction, Chicago, IL.

SAC Joint Venture (2000a). *Recommended Seismic Design Criteria for New Steel Moment Frame Buildings* (FEMA 350), Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000b). *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment Frame Buildings* (FEMA 351), Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000c). *Recommended Post-earthquake Evaluation and Repair Criteria for Welded, Steel Moment Frame Buildings* (FEMA 352), Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000d). *Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications* (FEMA 353), Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000e). *State of the Art Report on Base Metals and Fracture* (FEMA 355A), K. Frank and J. Barsom, team leaders, Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000f). *State of the Art Report on Welding and Inspection* (FEMA 355B), M. Johnson, team leader, Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000g). *State of the Art Report on Systems Performance* (FEMA 355C), H. Krawinkler, team leader, Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000h). *State of the Art Report on Connection Performance* (FEMA 355D), C. Roeder, team leader, Federal Emergency Management Agency, Washington D.C.

SAC Joint Venture (2000i). *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes* (FEMA 355E), E. Reis and D. Bonowitz, authors, Federal Emergency Management Agency, Washington, D.C.

SAC Joint Venture (2000j). *State of the Art Report on Performance Prediction and Evaluation* (FEMA 355F), D. Foutch, team leader, Federal Emergency Management Agency, Washington D.C.

Keywords

steel braced frame
steel plate shear wall
steel moment frame
steel seismic design

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Overview of Steel Lateral Systems," August 2019, The SEAOC Blue Book: Seismic Design Recommendations, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2016 CBC reference section(s)	Other standard reference section(s)
14.1	N/A	AISC 341-16 F4

Introduction

A steel buckling-restrained braced frame (BRBF) is a type of concentric braced frame with a unique type of brace. Unlike conventional concentric braced frames, buckling-restrained braces are not intended to buckle in compression at the expected force demands of the brace. This behavior is achieved by de-coupling the load-resisting component of the BRB (the steel core) and the buckling-restraining mechanism (the outer casing). Because the BRB core yields and strain hardens in both compression and tension, the hysteretic response of BRBs show large, symmetric, stable loops featuring positive post-yield stiffness and lack of significant strength degradation.

BRBs were first used in Japan in the 1980s, where they were intended to stiffen primary moment-frame lateral systems and provide additional energy dissipation. In the United States, BRBFs were first used in the 1990s. SEAOC, in collaboration with AISC, developed standards and provisions for BRBF design in the late 1990s and early 2000s to provide design professionals, plan reviewers, and peer-reviewers with a set of guidelines for BRBF performance and design (Sabelli and Aiken 2003).

These design provisions were incorporated into design standards released in 2005 (AISC 341-05 and ASCE 7-05) and continued to be updated. As of 2018, BRBFs are common in high-seismic regions of the United States. Design and performance of BRBFs is well-documented in a variety of design guides, including the AISC Seismic Design Manual, NEHRP Seismic Design Technical Brief No. 11, and the 2012 SEAOC Design Guide, Volume 4. This guide is intended to supplement existing references and guides by providing additional information about recent BRBF developments.

Existing BRBF Resources

Several resources exist that describe in detail the BRBF system. A list of selected resources is below. Additional resources may be found in the references section of this report.

- NEHRP Seismic Design Technical Brief No. 11 - This is a comprehensive guide on BRBF development and performance. The document includes specific recommendations for BRBF analysis and design. (NIST 2015)
- Steel Tips July 2004 - *Seismic Design of Buckling-Restrained Braced Frames* – This is a comprehensive design guide that includes design examples. Note that this is an older publication (2004) and may contain outdated design requirements. Nonetheless, it provides valuable insight and guidance. (SSEC, 2004)
- AISC Seismic Design Manual, Second Edition – This manual includes design examples for BRBFs and supplements the Seismic Provisions (AISC 341-10), which contains prescriptive building code requirements for BRBF design. The commentary for AISC 341-10 contains useful information about the history of the prescriptive requirements (AISC, 2012).
- 2012 IBC SEAOC Structural/Seismic Design Manual, Volume 4 – This manual provides a step-by-step design example for a BRBF system. (SEAOC 2013)

Developments in Analysis, Design, and Detailing

While BRBF design has become codified over the past few decades, there is ongoing research to improve BRBF reliability and seismic performance. This section focuses on recent developments relating to BRBF analysis, design, and detailing.

Orthogonal Effects in Columns in Intersecting Frames: Buildings in high-seismic regions often require columns in intersecting frames to be designed for the effects of seismic design motions in more than one direction. One method commonly used to meet this requirement is to design columns for forces from 100% of the design motion in one direction in conjunction with 30% of those in the orthogonal direction (ASCE). While the implications of this requirement are clear for force-based design methods, it is less clear how to apply these requirements when using the capacity-design requirements that are required for BRBF columns. Capacity design refers to the idea that certain elements must not yield and therefore must be designed to remain elastic under the maximum forces that can be delivered by the yielding portion of the lateral system. For a BRBF, this corresponds to the scenario where the forces in all braces are at their yielded and strain-hardened strengths.

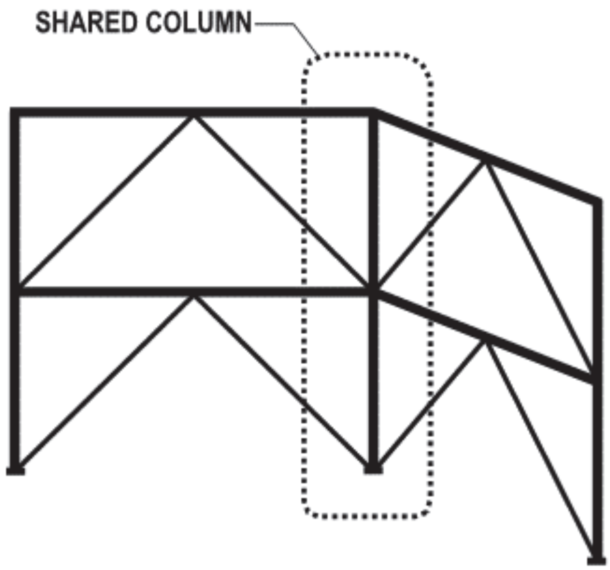


Figure 1. 3D View of Orthogonal Frames with Shared Column (Sabelli et al. 2003)

AISC 341 D1.4a provides some direction as to how such columns should be designed:

> *"For columns that are common to intersecting frames, determination of the required axial strength, including the overstrength seismic load or the capacity-limited seismic load, as applicable, shall consider the potential for simultaneous inelasticity from all such frames.*

Commentary D1.4a goes on to clarify:

> *"For systems with high R values, even the 30% design motion is likely capable of yielding the structure, and considering that the 100% motion may occur in any direction relative to a given axis of the structure, it is clear that simultaneous yielding of orthogonal systems is likely and should be considered in the design. Determination of the need to combine axial forces from simultaneous yielding of intersecting frames is left as a matter of judgment. The extent to which simultaneous yielding of orthogonal lateral frames is of concern is a matter of configuration and design, and depends upon the expected deformations and the story drift at which the system used is expected to start yielding. Depending upon stiffness and overstrength, moment frames generally remain elastic until they reach 1% story drift, whereas braced frames generally will yield before reaching half that drift."*

The commentary explains that the provisions leave this design decision to the designer's judgment; it is unclear what design criteria should be used to meet the minimum building code requirements.

A designer might seek to meet the code requirements by designing shared corner columns for 100% of the capacity requirement in one loading direction, and 30% of the capacity requirement from the other direction. Research has shown that this design criteria may be unconservative, especially for low-rise structure (Sherman and Taichiro 2010).

SEAOC Seismology recommends that designers avoid intersecting frames wherever possible. If intersecting frames are present, due to the high R values associated with BRBFs, SEAOC takes the position that designers should, at a minimum, design intersecting columns for axial forces corresponding to 100% of yielded/strain-hardened BRBs in one direction, and 100% of yielded (not strain-hardened) BRBs in the other direction.

This subject is the topic of ongoing research which seeks to identify reasonably conservative methods to extend the 100%+30% rule to linear, capacity-based design methods. Several factors impact the likelihood of simultaneous yielding and/or strain-hardening of BRBs in intersecting frames, including ground motion input and building height. For example, simultaneous yielding and/or strain-hardening of all BRBs may become less likely in taller buildings with significant higher-mode effects (Richards 2009, Sabelli et al, 2003). For designers using linear design methods, SEAOC recommends using the design methodology above for intersecting frames. Alternatively, designers could use inelastic analyses to justify lower demands for columns of intersecting frames, as allowed by AISC 341 F4.3; such methods may justify significant savings in taller structures.

It is important to note that this subject is relevant to all high ductility steel lateral systems, including moment frames and special concentric braced frames.

Length Effects in BRBs: AISC 341 requires that BRBs used in practice be similar to the BRBs used during qualifying cyclic tests. These similarities include BRB core cross-section shape and orientation, method of separating the BRB core and casing, and core yield strength. Regarding core yield strength, AISC 341 also provides a range within a designer may interpolate BRB test results. However, AISC 341 does not explicitly require that BRBs used in practice have a similar length to those used during qualifying tests.

Recent research suggests that long BRBs experience increased compression overstrength relative to shorter BRBs (Saxey and Richards, 2016). One hypothesis for this phenomenon is that longer braces have longer cores and as BRBs experience high-mode buckling within the BRB casing, there is more surface area over which friction can develop between the core and the BRB casing; this increased friction causes higher compression overstrength. This phenomenon has several implications which should be understood by both designers and BRB manufacturers.

1. BRB overstrength factors ($\omega\beta$ and β) are typically determined by the BRB manufacturer by using BRB testing results to establish a relationship between overstrength and BRB core strain, and these overstrength values are communicated to the designer based on the expected core strain for each BRB in a project. The strain/overstrength relationship may be based on shorter tested braces than the braces used in a given project. Therefore, a designer should be aware of the limitations of the data received from a BRB manufacturer and be prepared to engage with the BRB manufacturer to verify that the non-yielding elements of the BRBF system are designed for adequate overstrength.
2. Longer BRBs may require larger casings to preclude overall buckling under this increased compression overstrength.

SEAOC recommends that designers understand the length of BRBs that have been tested and the applicability of those test results to a given project. This information can be found online for both commercially-available BRB manufacturers. For projects that call for BRBs longer than those used during qualification testing, SEAOC Seismology cautions the designer to be aware of the phenomenon described above to ensure that adequate BRB compression overstrength is accounted for in the BRBF design.

Beam-Column Connection and Joint Fixity in Analysis: AISC 341 allows the designer some flexibility in choosing how to detail a BRBF beam-column connection. The designer may choose from a pinned or a moment-resisting connection, each with specific requirements:

Moment-Resisting Connections: A typical moment-resisting connection for a BRBF is shown in Figure 2 below. The beam is joined to the column with a WUF-W-type connection and the BRB gusset is joined to the beam and column with fillet welds. This type of connection has been shown by testing (Palmer et al. 2014) to perform well under design-level demand.

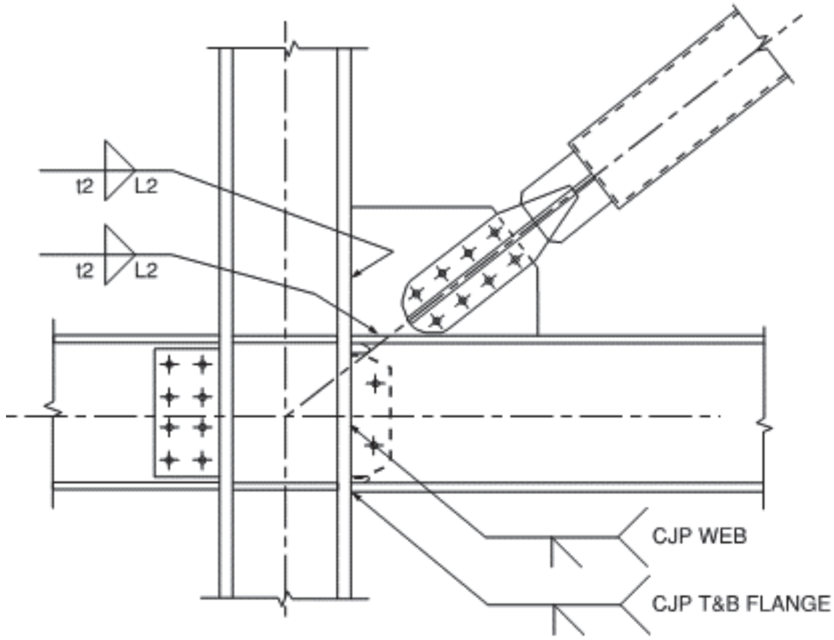


Figure 2. Typical BRBF Beam-Column Moment Connection (SEAOC, 2013)

With special detailing and proportioning requirements, a typical WUF-W connection exhibits reliable hysteretic behavior without significant damage. However, the introduction of a gusset plate to WUF-W-type connection results in less predictable behavior and makes the connection susceptible to damage at high story drifts (greater than 2%) beyond those typically required by code-level design (Palmer et al. 2014, Mahin 2004). The extent of damage is highly dependent on beam-column connection detailing and can include local yielding of beams and columns, beam-column weld rupture, and gusset plate buckling (Uriz 2005, Palmer 2012). It is important to note that damage at beam-column connections is relevant to all ductile steel braced frame systems and not just BRBFs.

This issue is complicated by the number of variables that contribute to the performance of gusseted beam-column connections, including frame geometry, brace type, gusset geometry, gusset thickness, beam and column size, weld detailing, presence of stiffener plates and/or web doubler plates. However, recent research suggests several design recommendations to improve BRBF beam-column connection performance (Palmer et al. 2016):

- Beam and column web thicknesses should not be significantly thinner than the gusset plate thickness; web doubler plates are recommended where beam and column webs are thin relative to the gusset.
- Continuity plates should be provided at the beam-column connection.
- Gusset-to-column and gusset-to-beam welds designed using the Uniform Force Method may not be adequate to prevent weld tearing at large story drifts; the authors recommend designing these welds for the capacity of the gusset.

Joint fixity in a designer's analysis model must be carefully considered when using this detail. An analysis model with rigid beam-column connections will resist some portion of story shear via frame action rather than truss action typical of braced frames. Depending on the designer's choice of how to analyze and design the BRBF beams and columns, there may or may not be significant flexural forces in each member.

The default choice of joint fixity would be for the analysis model to match the detail and assume the joint is fixed. This choice would result in a small portion of story shear being distributed to the beam and column via frame action. If BRBF beams and columns are designed for these forces in combination with the required forces specified by AISC (note that this generally complicates beam and column design), and with prequalified connections, this choice of joint fixity may be acceptable. Although frame action draws story shear away from BRBs and may result in slightly smaller BRBs, it is generally accepted as a design method and is presented in at least one design guide (SSEC, 2004).

Another choice of joint fixity would be to pin the beam-column connections and design the BRBF assuming that there is no frame action, even though the joint will be detailed as fixed. In this context, BRBF beams and columns are designed for primarily axial seismic forces, and this choice simplifies beam and column design. Analyzing and designing around this assumption has generally been deemed to be acceptable for linear design (NIST 2015) and AISC allows BRBF column design to neglect flexural forces resulting from seismic drift (AISC 2012). For non-linear analysis, joint fixity must be considered.

Pinned Connections: The second type of beam-column connection option for a BRBF designed per AISC 341 is a simple connection with a required rotation capacity of 0.025 radians. Several schematic-level connections are shown in Figures 3a-3d below.

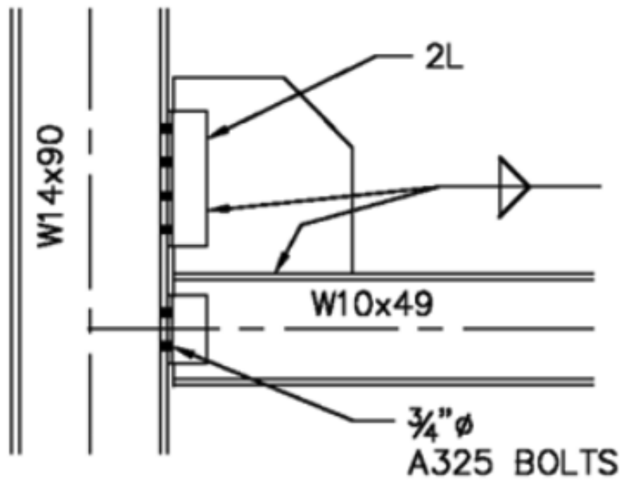


FIGURE 3a

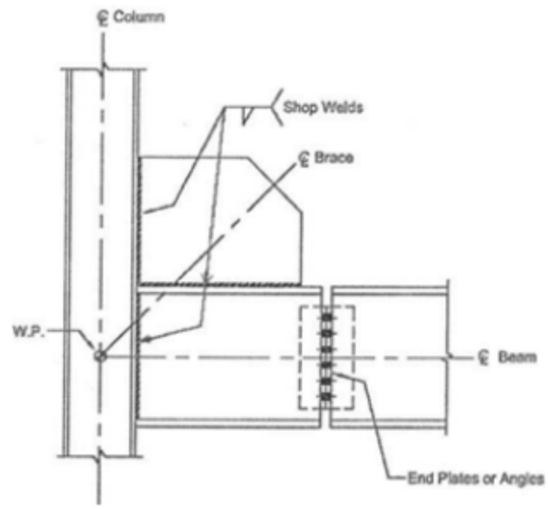


FIGURE 3b

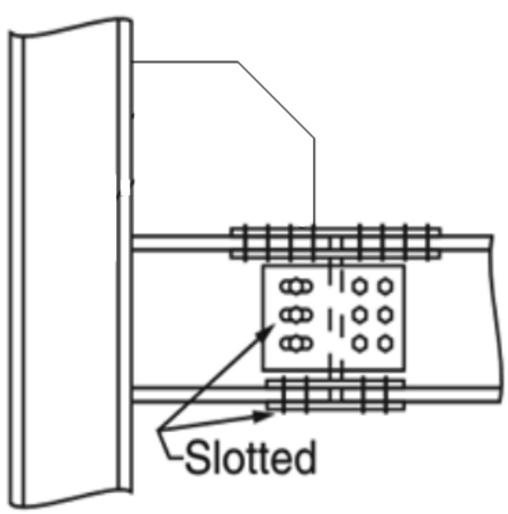


FIGURE 3c

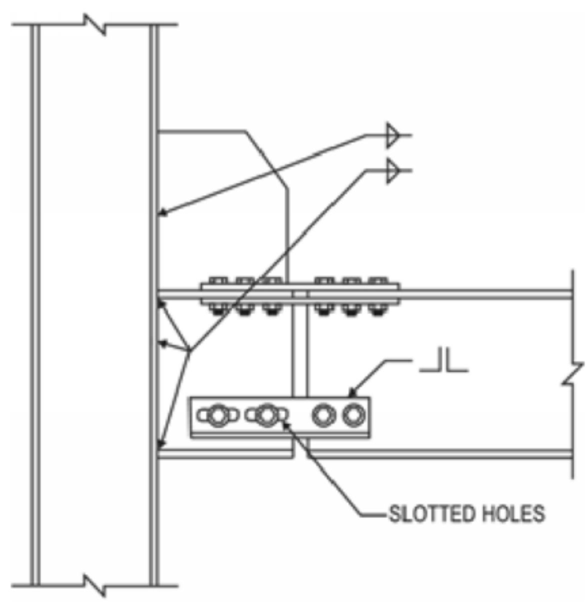


FIGURE 3d

Figure 3a-3d. Typical BRBF Beam-Column Pinned Connection (References below).

Because early full-scale BRBF tests indicated that moment-connected beam-column connections were susceptible to damage (see above), engineers have developed beam-column connection that can accommodate rotation. The figures shown above represent several examples of those connections.

The brace connection shown in Figure 3a (Stoakes and Fahnestock, 2010) accommodates rotation at the beam/gusset-to-column interface via a relatively flexible double-angle connection. Figure 3b (Thornton and Muir, 2008) illustrates a connection that features a beam stub fully welded to the column but features a pinned connection just beyond the edge of the gusset located along the beam centerline; beam-column relative rotation occurs at this connection. Figures 3C (Walters et al. 2004) and 3D (Prinz et al. 2014) illustrate a slight variation of the connection shown in 3B; whereas a moment couple will form between a concrete slab (not shown) and the pinned connection shown in Figure 3B, connections 3C and 3D locate the stub-to-beam connection at the top flange to minimize the moment arm between the steel centerline and the slab. Several of the connection schemes shown above have been experimentally validated and shown to mitigate many of the failure mechanisms associated with moment-connected connections (see references above).

If using this type of detail, a designer's analysis model should feature pinned beam-column connections. In contrast to a moment-connected connection, there will be no frame action and BRBF beams and column design is somewhat simplified.

In summary, the WUF-W-type connection has the following advantages:

1. This connection type can transfer large collector forces.
2. The rigid beam-column connection allows the BRBFs to act as a moment frame that can help provide story stiffness after BRBs have yielded. The resulting stiffness can help reduce peak and residual drifts, and may also help reduce formation of a soft story mechanism (Fahnestock et al. 2007).

A pinned beam-column connection has the following advantages:

1. It reduces field welding and weld inspection.
2. Whereas a gusseted WUF-W-type connection may incur significant damage after a seismic event, a pinned-type connection will see minimal damage. This minimizes damage in hard-to-repair elements like beams and columns and drives damage almost solely into BRBs which are comparatively easy to replace.
3. This connection affords the designer a higher degree of control over the performance of a structure and may be desirable for projects utilizing performance-based design methods.

Therefore, a pinned beam-column connection may yet be advantageous for certain projects. Where the BRBF is expected to exceed 2% story drift, SEAOC Seismology recommends use of this detail.

Gusset Plate and Out-of-Plane Buckling: Gusset plates are not expected to buckle in BRBFs. However, several full-scale tests have shown BRBFs can be susceptible to gusset plate local buckling and combined gusset and BRB out-of-plane buckling under certain circumstances. Illustrations of these failures are presented in Figure 4. To reduce the likelihood of buckling, AISC 341-16 F4.6c.2 requires that: "Lateral bracing consistent with that used in the tests upon which the design is based shall be provided."

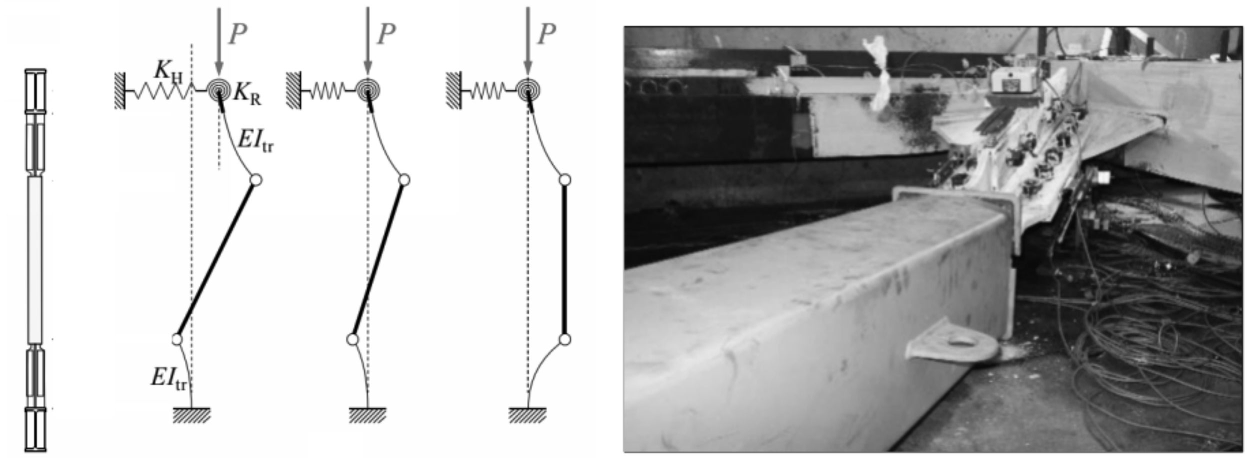


Figure 4. Analytical depiction of out-of-plane buckling of BRB and gusset for a chevron brace configuration (Okazaki et al. 2012) and out-of-plane buckling exhibited during full-scale BRBF testing (Palmer et al. 2014).

Research has shown that several factors affect the likelihood of these failures (Takeuchi et al 2013):

1. Detailing of the transition zone between the gusset and the BRB core and the transition zone's embedment within the BRB casing.
2. The connection of the BRB to the gusset
3. The size of the gusset plate
4. The presence of gusset free-edge stiffener plates
5. The type of beam-column connection
6. The frames' expected inelastic story drift.

The many factors impacting these phenomena make them difficult to model and predict.

Nonetheless, researchers have suggested a variety of design methods to minimize the likelihood of gusset plate and out-of-plane buckling, including using an effective length factor (K) equal to 2 (Tsai et al., 2008) and/or utilizing edge stiffeners (Chou and Chen 2009). A comprehensive list of all such methods is presented in (Westeneng et al. 2015). While some research (Chou and Liu, 2012) has found that free-edge stiffeners are effective in preventing gusset buckling, there is some concern about how these stiffeners would impact the beam-column joint. Additional recommendations and design suggestions can be found in: Lin et al. 2015, Chou et al, 2012, and Berman and Bruneau, 2009). This is a complicated issue; SEAOC Seismology recommends that designers consider using free edge stiffeners for gussets (see Figure 5 below) and consult research publications as they become available.

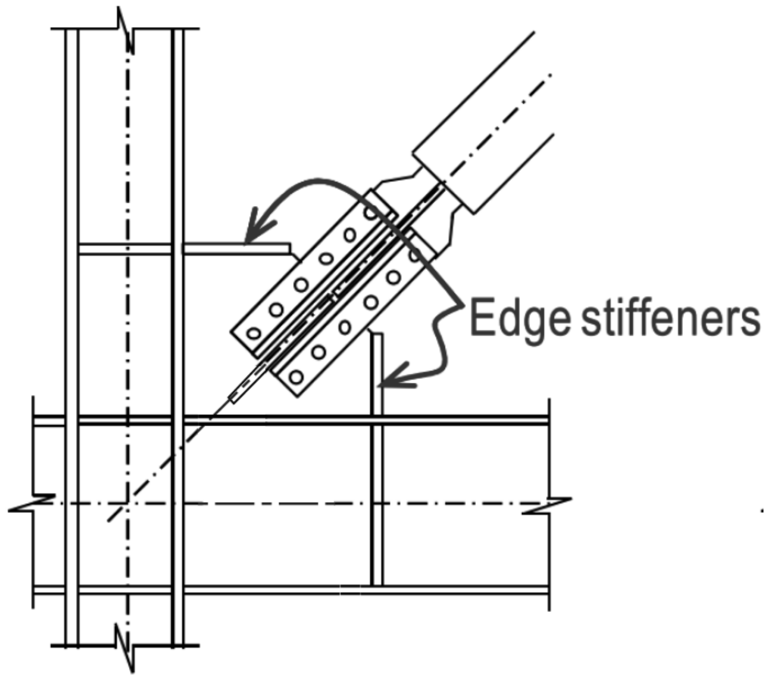


Figure 5. Gusset plate free-edge stiffeners. Hikino, et al. 2013.

BRBF Beam and Column Compactness: Per AISC 341-16, BRBF columns and beams are required to meet the width-to-thickness requirements of moderately-ductile members. This represents a change from AISC 341-10, which required BRBF beams and columns to be highly-ductile members. It is worth noting that this relaxation of code requirements does not apply to other highly-ductile braced frame systems, including Special Concentrically Braced Frames (SCBF) and Eccentrically Braced Frames (EBF).

This code change allows designers to choose from a wider range of beam and column sizes. For example, under AISC 341-10, there are no W14 sizes between W14x82 and W14x132 that meet the ductility requirements of the code (assuming A992, Gr. 50 steel). Under AISC 341-16, use of W14x120 and W14x109 sections are permissible, giving the designer additional flexibility during design.

State-of-the-Art Research

There are several active areas of research into improving BRBF performance:

- Self-Centering BRBFs: Several researchers are exploring the use of self-centering BRBFs consisting of a traditional BRB acting in combination with a shape-memory alloy in order to reduce post-earthquake residual drift. These BRBs exhibit behavior that allows for energy to be dissipated via traditional BRB yielding, while the shape-memory alloy remains elastic and forces the BRB back to its original length. For more information, see: Miller et al. 2012.
- Mitigation of Soft-Story Mechanism: Numerous studies have shown that braced frames may be susceptible to soft-story mechanism formation during large earthquakes, where damage is concentrated in one or a few stories. One development to mitigate this behavior is the strongback braced frame, which consists of an elastic vertical truss to force a uniform drift distribution and distribute yielding over all stories. For more information on this research, see: Simpson and Mahin, 2018,

References

AISC (2012) *Seismic Design Manual* (2nd ed.). (2012). Chicago: American Institute of Steel Construction.

AISC 341 2016. *Seismic Provisions for Structural Steel Buildings, July 12, 2016, American Institute of Steel Construction, Inc., Chicago, IL.*

AISC 341 2010. *Seismic Provisions for Structural Steel Buildings, 2010*, American Institute of Steel Construction, Inc., Chicago, IL

ASCE. 2016. Minimum Design Loads for Buildings and Other Structures. ASCE/SEI Standard 7-16

Berman, Jeffrey W., and Michel Bruneau. "Cyclic Testing of a Buckling Restrained Braced Frame with Unconstrained Gusset Connections." *Journal of Structural Engineering*, vol. 135, no. 12, 2009, pp. 1499–1510., doi:10.1061/(asce)st.1943-541x.0000078.

Chou, Chung-Che, and Pei-Jin Chen. "Compressive Behavior of Central Gusset Plate Connections for a Buckling-Restrained Braced Frame." *Journal of Constructional Steel Research*, vol. 65, no. 5, 2009, pp. 1138–1148., doi:10.1016/j.jcsr.2008.11.004.

Chou, Chung-Che, and Jia-Hau Liu. "Frame and Brace Action Forces on Steel Corner Gusset Plate Connections in Buckling-Restrained Braced Frames." *Earthquake Spectra*, vol. 28, no. 2, 2012, pp. 531–551., doi:10.1193/1.4000007.

Chou, Chung-Che, et al. "Compressive Behavior of Dual-Gusset-Plate Connections for Buckling-Restrained Braced Frames." *Journal of Constructional Steel Research*, vol. 76, 2012, pp. 54–67., doi:10.1016/j.jcsr.2012.03.003.

Fahnestock, L. A., Ricles, J. M., and Sause, R. (2007). "Experimental evaluation of a large-scale buckling-restrained braced frame," Journal of Structural Engineering, 133 (9), pp. 1205-1214.

Hikino, Tsuyoshi, et al. "Out-of-Plane Stability of Buckling-Restrained Braces Placed in Chevron Arrangement." *Journal of Structural Engineering*, vol. 139, no. 11, 2013, pp. 1812–1822., doi:10.1061/(asce)st.1943-541x.0000767.

Lin, Pao-Chun, et al. "Seismic Design and Experiment of Single and Coupled Corner Gusset Connections in a Full-Scale Two-Story Buckling-Restrained Braced Frame." *Earthquake Engineering & Structural Dynamics*, vol. 44, no. 13, 2015, pp. 2177–2198., doi:10.1002/eqe.2577.

López, W.A., Gwie, D.S., Saunders, C.M., and Lauck, T.W. (2002). "Lessons learned from large-scale tests of unbonded braced frame subassemblages," in *Proceedings of the 71st Annual Convention*, pp. 171-183, Structural Engineers Association of California, Sacramento, CA.

Mahin, Stephen, et al. "Seismic Performance of Buckling Restrained Braced Frame Systems," *13th World Conference on Earthquake Engineering*, 2004.

Miller, David J, et al. "Development and Experimental Validation of a Nickel–Titanium Shape Memory Alloy Self-Centering Buckling-Restrained Brace." *Engineering Structures*, vol. 40, July 2012, pp. 288–298.

NIST (2015). Seismic design of steel buckling-restrained braced frames: A guide for practicing engineers, GCR 15-917-34, NEHRP Seismic Design Technical Brief No. 11, produced by the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering for the National Institute of Standards and Technology, Gaithersburg, MD.

Okazaki, T., et al. "Out-of-Plane Stability of Buckling-Restrained Braces." 15th World Congress on Earthquake Engineering. 2012.

Palmer, K.D. (2012), "Seismic Behavior, Performance, and Design of Steel Concentrically Braced Frame Systems," Ph.D. Dissertation, University of Washington, Seattle, WA.

Palmer, Keith D., et al. "Experimental Evaluation of Cyclically Loaded, Large-Scale, Planar and 3-d Buckling-Restrained Braced Frames." *Journal of Constructional Steel Research*, vol. 101, 2014, pp. 415–425., doi:10.1016/j.jcsr.2014.06.008.

Palmer, Keith & Roeder, Charles & Lehman, Dawn. (2016). Connection Design Recommendations for Improved BRBF Performance. Engineering Journal (New York). 53. 29-45.

Prinz, Gary S., et al. "Experimental and Numerical Investigation of Ductile Top-Flange Beam Splices for Improved Buckling-Restrained Braced Frame Behavior." *Journal of Structural Engineering*, vol. 140, no. 9, Sept. 2014.

Richards, Paul W. "Seismic Column Demands in Ductile Braced Frames." *Journal of Structural Engineering*, vol. 135, no. 1, 2009, pp. 33–41., doi:10.1061/(asce)0733-9445(2009)135:1(33).

Sabelli, R., Kirschenbaum, D. S., & Polat, S. (2003). Orthogonal Effects: Seismic Axial Forces in Columns Shared by Orthogonal Frames. In *SEAOC 72nd Annual Convention Proceedings* (pp. 197-209). SEAOC.

Sabelli, R., & Aiken, I. (2003). "Development of building-code provisions for buckling-restrained braced frames," in Mazzolani (ed.), Behavior of Steel Structures in Seismic Areas: STESSA 2003, pp. 813-818, Mazzolani, ed., Balkema, Rotterdam, Netherlands.

Saxey, Brandt, and Paul Richards. "SEAOC 2016 Convention." *The Effect of Increasing Brace Length on the Expected Design Values for Buckling-Restrained Braces*, 2016.

SEAOC Seismology Committee (2008). "Buckling-restrained braced frames," March, 2008, The SEAOC Blue Book: Seismic design recommendations, Structural Engineers Association of California, Sacramento, CA; accessible via the world wide web at: http://www.seaoc.org/bluebook/index.html.

SEAOC. (2013). *2012 IBC SEAOC Structural/Seismic Design Manual* (Vol. 4, Examples for Steel-Framed Buildings). Sacramento, CA: Structural Engineers Association of California.

Sherman, Joel, and Taichiro Okazaki. "Bidirectional Loading Behavior of Buckling-Restrained Braced Frames." *Structures Congress 2010*, 2010, doi:10.1061/41130(369)320.

Simpson, Barbara G., and Stephen A. Mahin. "Experimental and Numerical Investigation of Strongback Braced Frame System to Mitigate Weak Story Behavior." *Journal of Structural Engineering*, vol. 144, no. 2, 2018, doi:10.1061/(asce)st.1943-541x.0001960.

SSEC (2004). *Seismic design of buckling-restrained braced frames,* López, W.A., and Sabelli, R., "Steel Tips", July 2004, Structural Steel Education Council, Moraga, CA.

Stoakes, C. D. and Fahnestock, L.A. (2010), "Flexural Behavior of Concentrically-Braced Frame Beam Column Connections," Proceedings, 2010 Structure Congress, ASCE, Orlando, FL

Takeuchi, Toru, et al. "Out-of-Plane Stability of Buckling-Restrained Braces Including Moment Transfer Capacity." *Earthquake Engineering & Structural Dynamics*, vol. 43, no. 6, 2013, pp. 851–869., doi:10.1002/eqe.2376.

Thornton, W.A. and Muir, L.S. (2008), "Vertical Bracing Connections in the Seismic Regime," *Proceedings, Connections VI: Sixth International Workshop on Connections in Steel Structures,* AISC, Chicago, IL.

Tsai, K. C., and Hsiao, P-C. 2008. Pseudo-dynamic test of a full-scale CFT/BRB frame—Part II: Seismic performance of buckling-restrained braces and connections. Earthquake Engineering and Structural Dynamics, 37, 1099-1115.

Uriz, Patxi (2005). *Towards earthquake resistant design of concentrically braced steel frames*, doctoral dissertation, December 2005, Department of Civil and Environmental Engineering, University of California, Berkeley, CA.

Walters, M. T., Maxwell, B. H., and Berkowitz, R. A. (2004). "Design for improved performance of buckling-restrained braced frames." Proc., 2004 SEAOC Convention-Monterey, Structural Engineers Association of California, Sacramento, CA, 507–513.

Westeneng, B. A., et al. "Steel Innovations Conference 2015." *Early Failure of Gusset Plates in Buckling Restrained Braced Frames*, 2015.

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 2019)

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

SEAOC Seismology Committee (2019). "Buckling-Restrained Braced Frames," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
ASCE 7-16 12, 14	2205	ANSI/AISC.341-16 ASTM A 500, A 501

Introduction and History

Concentrically braced frames are vertical truss systems that resist lateral forces in the elastic range, primarily through axial forces in members. Members intersect at a point, or with small eccentricities that are not sources of inelastic deformation. In the inelastic range, braced frames may involve the flexure of frame members, but the inelastic behavior is expected to be dominated by brace buckling in compression and yielding in tension.

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 reductions in calculated brace strengths, which resulted in raising the elastic force capacities 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 2005 and later AISC seismic provisions (AISC, 2005, 2016) make more rational distinctions 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. SCBFs are expected to achieve trilinear hysteretic behavior by accommodating cyclic brace buckling, and withstanding forces corresponding to the tension 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. General summaries of the most recent code changes regarding OCBF and SCBF systems are shown in Table 1 and Table 2.

SCBF requirements for braces are intended to prevent undesirable modes of brace behavior. Analytical studies on bracing systems designed in accordance with earlier code requirements anticipated 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 can occur at the ends of buckling braces, and at brace midspans. 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. Standards revisions for OCBF systems

AISC SEISMIC 341-02	*AISC SEISMIC 341-05*	*AISC SEISMIC 341-10*	*AISC SEISMIC 341-16*
• Member and connection demands (non-brace connections) shall include Amplified Seismic Forces (See UBC 1997) • Braces with KL/r $\langle 4.23\sqrt{E/Fy}$ shall not be used in V or inverted-V configurations. • Brace connection demand = $R_yF_yA_g$ of brace.	• No Amplified Seismic Force demands, but reduced R-value (See IBC 2006). • Braces in K, V, or inverted-V configurations must have $KL/r \leq 4\sqrt{E/Fy}$. • For K, V, and inverted-V configurations, beams (columns for K) must be designed for unbalanced brace forces, $R_yF_yA_g$ vs. 0.3Pn and full gravity loads. • Brace connection demand = $R_yF_yA_g$, except for the bolt-slip limit state, or if max. system capacity or Amplified Seismic Force demands are used. • OCBF above Seismic Isolation System: KL/r $\leq 4\sqrt{E/Fy}$; no K frames; continuous beams between columns for V and inverted-V configurations. • Defined a new term, "demand critical welds" • Defined a new term, "protected zones"	• Prohibition of K-type braced frames • For V and inverted-V configurations, clarified the unbalanced brace tension forces on the beams are to use the least of: $R_yF_yA_g$, amplified seismic loads, or the max force that can be developed by the system • Introduced term of "moderately ductile members" (similar to previously used "seismically compact"). • Brace connection demand = Amplified Seismic Force but need not exceed: i) $R_yF_yA_g$ in tension ii) lesser of $R_yF_yA_g$ or 1.14FcreAg in compression iii) bolt slip limit state need not use Amplified Seismic Force where oversized holes used	• Introduced new provisions for "multi-tiered braced frame (MT-OCBF) • Clarified tension only bracing systems need not meet slenderness requirements.

Past observations of performance in earthquakes have revealed that some concentrically braced frames did not achieve 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 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, SCBF braced frames can sustain cycles of large inelastic drift without brittle failures. Proper design of SCBF systems requires consideration of the geometric configuration, element proportioning, connection detailing and analytical modeling.

Table 2. Standards revisions for SCBF systems

AISC SEISMIC 341-02	*AISC SEISMIC 341-05*	*AISC SEISMIC 341-10*	*AISC SEISMIC 341-16*
• Braces shall have KL/r $\leq 5.87\sqrt{E/Fy}$. • Width-thickness ratios per table I-8-1, and AISC 360-99, table B5.1. • Alternative requirement of gusset plate to accommodate inelastic post-buckling rotation (no guidelines given as to how). • Beam bracing axial demand (at V and inverted-V config.'s) shall satisfy 2% of $F_y b_f t_{bf}$. • Column splices shall develop nominal shear strength of smaller connecting member. • No K brace systems allowed.	• Braces shall have KL/r $\leq 4\sqrt{E/Fy}$, unless columns have available strength for brace capacities, $R_y F_y A_g$. • Width-thickness ratios per table I-8-1 (no changes), and AISC 360-02, table B4.1 (changes to numerous section types, including HSS). • Alternative requirement of gusset plate to accommodate inelastic post-buckling deformations (typical condition detailed in commentary). • Beam bracing (at V and inverted-V config.'s) must meet max. spacing criteria, and axial demand shall satisfy $P_{br} = 0.02 M_r C_d / h_o$, where M_r is the required flexural strength, C_d is a curvature factor, and h_o is the distance between flange centroids • Exception for penthouses, ones story structures, and top stories of buildings removed for special beam design requirements for V and inverted-V configurations. • Column splices shall develop approximately 50% of the member flexural capacity.	• Added analysis requirements to address the inelastic response by requiring two force distributions be checked (amplified seismic forces and brace full tension yielding with compression brace post-buckling) • Connection requirements increased to accommodate drift • Demand Critical welds required at column splices and bases • No PJP groove welds at column splices • Alternate Elliptical yield line procedure presented in the design examples	• Introduced new provisions for "multi-tiered braced frame (MT-SCBF) • Brace to beam connection requirements expanded to cover the "Chevron Effect" (i.e. increased forces in beam at the connection location)

CBF Configurations

Figure 1 shows common concentric braced frame configurations.

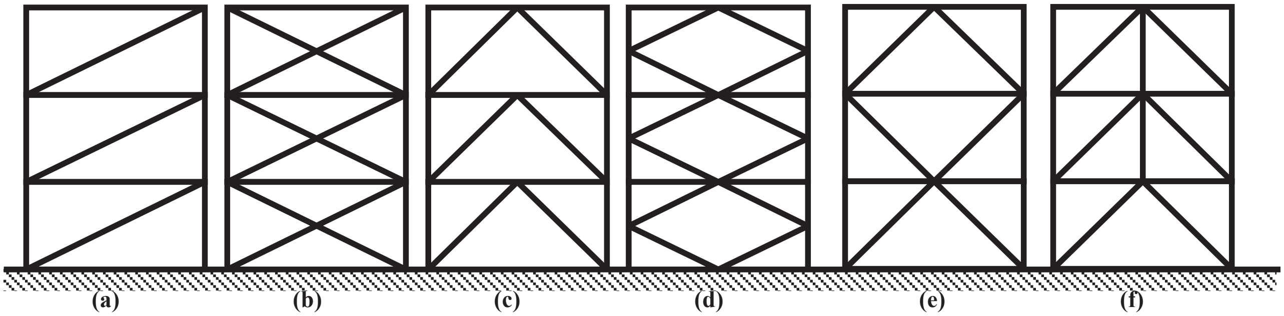


Figure 1. Typical braced frame configurations: (a) diagonal bracing; (b) cross or X bracing; (c) chevron bracing; (d) K-bracing, (e) two-story X bracing, and (e) "zipper" braced frame

Chevron Bracing. Traditional chevron-braced (V- and inverted-V-braced) frames have been shown to have undesirable post-buckling behaviors characterized by beam flexure rather than truss action (Figure 2). In the post-buckling range, the forces in the buckled braces diminish with additional deformations, and the vertical components of the compression brace and tension brace forces no longer balance; the beams must then resist the unbalanced components. 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.

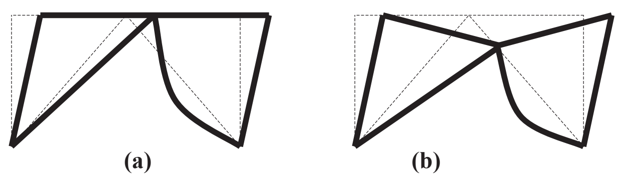


Figure 2. Expected deflected lateral shape for (a) a strong and stiff beam, and for (b) a beam not designed to carry unbalanced load.

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 (Section F2.3, AISC 341). The flexural demand should be combined with appropriate gravity loads.

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 forces 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 forces. 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 demands calculated by a linear analysis or proportioned to avoid a single-story mechanism using a nonlinear analysis. The requirements in AISC 360 provide for a minimum strength of the beams to support gravity loads in the event of loss of brace capacities.

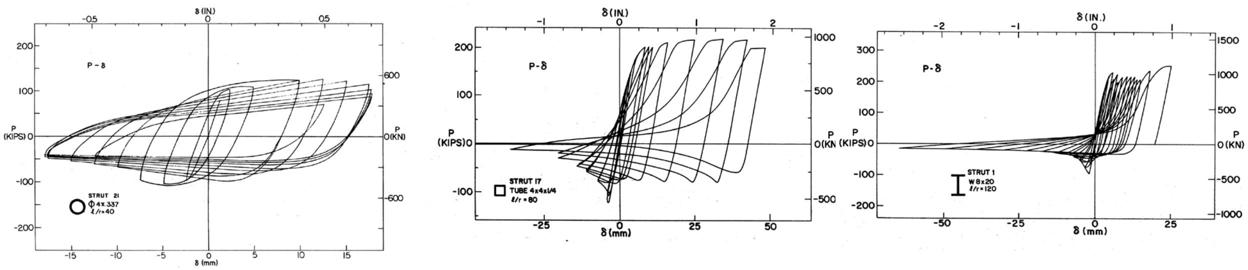
K-Bracing. K-bracing is prohibited for seismic applications, because columns that are subjected to unbalanced seismic forces from the braces in the post-buckling range are susceptible to buckling. Similar to the unbalanced forces on the beam in chevron bracing, columns would be expected to react to very 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 gravity load-path instabilities.

Multi-Tiered Braced Frames. Multi-tiered braced frames (MTBF) are braced frames with two or more levels (or ties) of bracing points between horizontal diaphragms or out-of-plane supports. They can be built using a variety of bracing configurations and multiple bays. They are common in industrial and structures with tall story heights (such as stadia and concert halls), when it is not practical to use single bracing members spanning from the roof to the foundation. Contrary to conventional braced frames, there are no floor diaphragms laterally to brace the columns at every tier; therefore, lateral forces such as wind and seismic events can induce out-of-plane deformations on the columns which may impact their stability. Inelastic response of MTBF also result in additional in-plane demands that may impact the frame stability, especially when unbalanced horizontal loads of the tension and compression braces are considered. Different from conventional braced frames, MTBF columns must be designed to explicitly consider bending demands. New requirements are now included in the 2016 Seismic Provisions (AISC 341-16, Section F2.4e) to assess and address this demand and other aspects of MTBF's. The Commentary of these Provisions contains approximately eleven pages of material covering the various analysis cases and failure modes that are to be considered. The requirements for multi-tiered ordinary concentrically braced frames (MT-OCBF) are simpler than the MT-SCBF requirements, due to the reduced level of ductility required.

Brace Elements

Braces in SCBFs must withstand cycles of compression 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.

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. 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 360 permits designing compression members with slenderness ratios of 200 due to the inherent overstrength in the tension capacity. Research has shown that these systems can perform well under seismic excitations; see Tremblay (2000).



(a) (b) (c)

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 demands 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).

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 to form 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 recommended 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.

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."

Table 3. Effective Length Factors for Cross-Braced Frames.

Splice Condition[4]	Fixed In-Plane Fixed Out-of-Plane		Fixed In-Plane Pinned Out-of-Plane			
Brace	Continuous Brace or Spliced Brace		Continuous Brace		Spliced Brace	
End Condition[3]	Theoretical K value	Recommended K value[6,7]	Theoretical K value	Recommended K value[6,7]	Theoretical K value	Recommended K value[6,7]
Fixed Out-of-Plane	0.70[8]	0.8	Note 9.	(1.0)[10]	0.708	0.8
Pinned Out-of-Plane	1.00[8]	1.0	Note 9.	(2.0)[10]	1.008	1.0
Fixed In-Plane	0.54[11]	0.7	0.54[11]	0.7	0.54[11]	0.7
Pinned In-Plane	0.80[11]	0.9	0.80[11]	0.9	0.80[11]	0.9

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.
6. Values combine theoretical values with increases in effective length recommended in AISC LRFD Table C-C2.1.
7. 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 Stoman (1989).
9. See Kitipornchai and Finch (1986); el-Tayem and Goel (1986); Picard and Beaulieu (1987).
10. Recommended values for this splice condition are conservative and are not meant to preclude designers from calculating lower values. See note 9.
11. See Sabelli and Hohbach (1999).

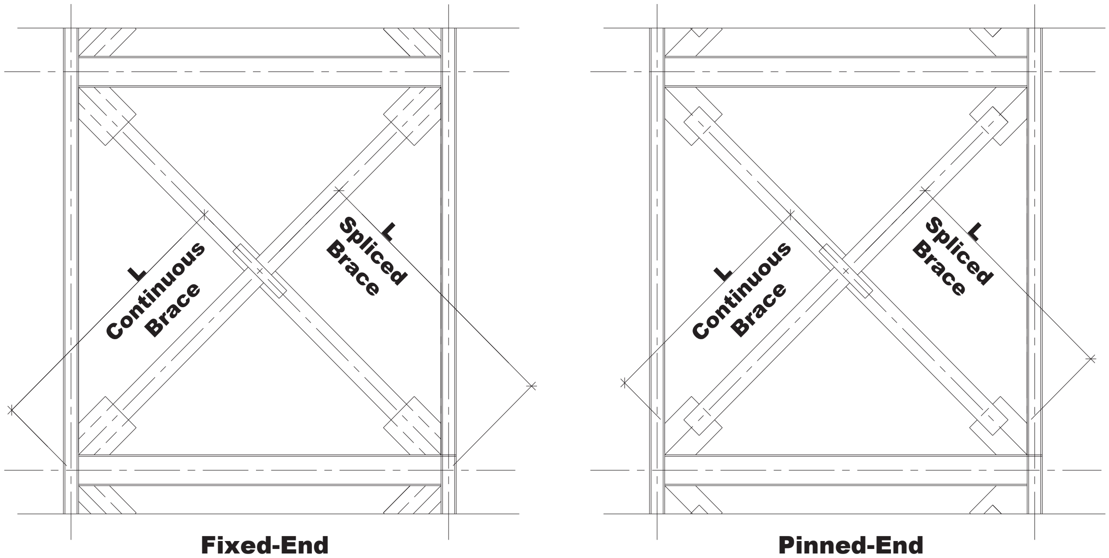


Figure 4. Brace lengths to be used with effective length factor in Table 1.

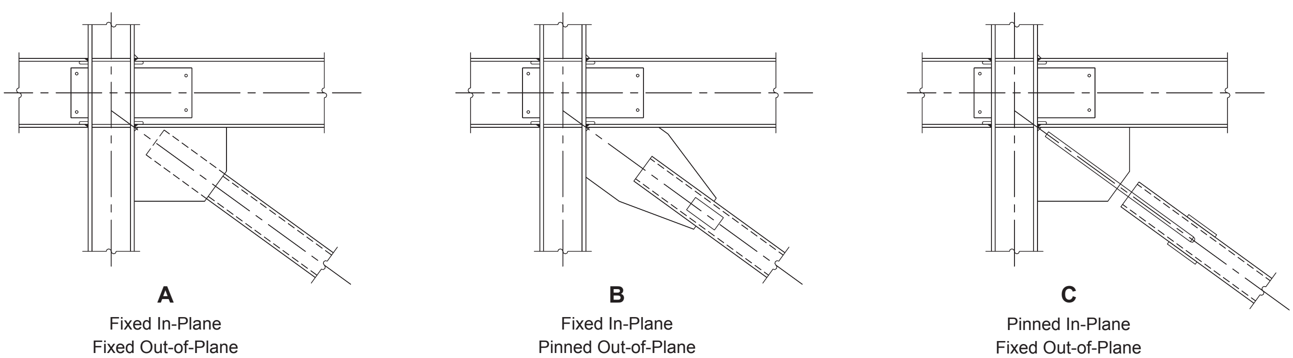


Figure 5. Gusset connections providing differing end-fixity conditions.

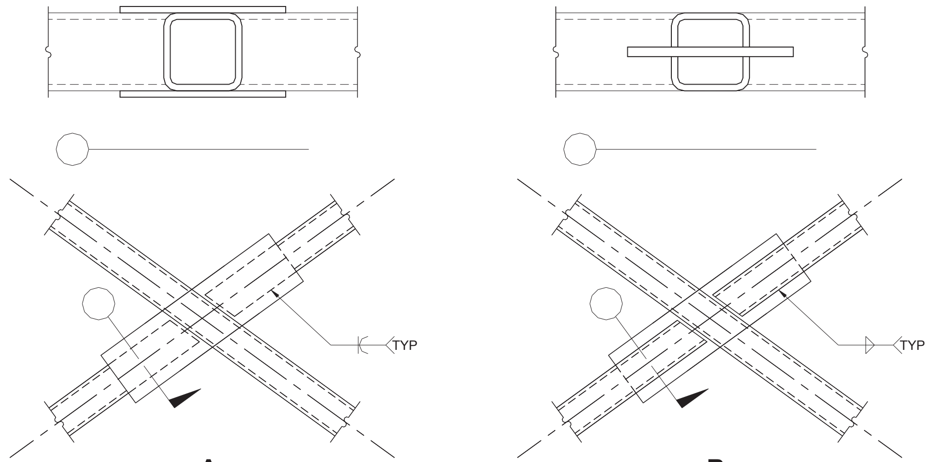


Figure 6. Splice details with differing continuity conditions.

Cross Section. During design, three important aspects of a brace's cross section should be considered: 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.

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). 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. However, the maximum compressive force that the brace can impose on the connections and columns should include the strengthening effect of the concrete (Liu and Goel 1988). 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.

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, 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 forces, 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 demands. 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. It is assumed that the controlling yield mode of an SCBF building is brace yielding. It is not valid to limit force demands because of some other yield mode, such as column buckling, collector failure, or foundation uplift. 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 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. The SEAOC Seismology Committee recommends 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.

The detailing of connections with hinge zones leads to gusset plates that are larger than those designed to meet but not exceed code-prescribed forces. 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. 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 SCBF structures. The current 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. This elliptical model is included in the design examples of the 2010 and 2016 *AISC Seismic Design Manual*.

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.

Beams and Columns

Columns. In 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 ($\Omega_o = 2.0$). Additionally, many steel materials commonly used as braces exhibit higher material overstrength than that assumed in the determination of the overstrength factor. The SEAOC Seismology Committee recommends using a capacity-design approach for sizing columns in concentrically braced frames.

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 forces, potentially overloading them. The AISC Seismic Provision Sections H2.3 and F2.3 require capacity based load cases to be considered in order to account for the impact of unbalanced forces on the columns, beams, struts, and connections of SCBF's.

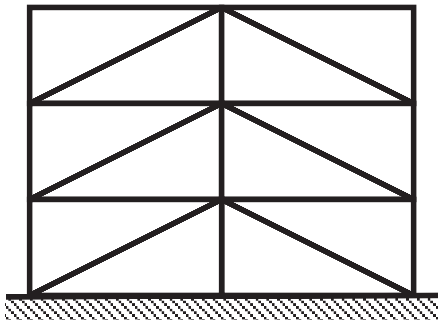


Figure 7. Single diagonal configuration sharing a center column.

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. An additional consideration for chevron brace configurations is designing the beams for the unbalanced tension-yielding compression-buckling forces 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 refine 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 building heights, configurations and analytical models 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) can result in median drift values ranging as high as 6%, further complicated by the fact that some of the models predict "collapse" of structures. The scatter in response data is significantly greater at larger intensities.

Modeling. Non-linear dynamic analyses using appropriate models, including the effects of fractures due to fatigue (for example see: Uriz 2005, Sabelli 2001, Tremblay and Poncet 2004) are paramount to understanding global responses 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 selected earthquake response histories. 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 (See Uriz 2005).

Recent efforts have been made to investigate the calibration of ASCE 41 methodologies with the design provisions of ASCE 7; however, the results for SCBF's have been somewhat inconclusive (Harris, J.L. and Speicher, M.S. 2015b). The studies have found that components of the SCBFs that do not satisfy the collapse prevention acceptance criteria would need to be strengthened to achieve the performance required by ASCE 41. However, results can be inconsistent for different analysis techniques (i.e., LSP vs. NDP or ELF and RSA). Future research is needed to evaluate the collapse probability of new systems designed by ASCE 41 compared to the seismic performance objectives of ASCE 7.

References

AIJ (1995). Reconnaissance report on damage to steel building structures observed from the 1995 Hyogoken-Nanbu (Hanshin/Awaji) earthquake, Architectural Institute of Japan, Steel Committee of Kinki Branch, Tokyo, Japan.

AISC (1997a). "Advisory statement on mechanical properties near the fillet of wide flange shapes and interim recommendations, January 10.

AISC (1997b). Technical bulletin 3, March (ASTM Gr50 with special requirements).

AISC (2005). Seismic provisions for structural steel buildings, American Institute of Steel Construction, ANSI/AISC.341-05

AISC (2010). Seismic provisions for structural steel buildings, American Institute of Steel Construction, ANSI/AISC.341-10

AISC (2016). Seismic provisions for structural steel buildings, American Institute of Steel Construction, ANSI/AISC.341-16

ASCE (2006). Minimum design loads for buildings and other structures, ASCE/SEI 7-05, Structural Engineering Institute, 2005.

ASTM (2001). "Standard specification for cold-formed welded and seamless carbon steel structural tubing in rounds and shapes," A500, American Standards and Testing and Materials, 2001.

ASTM (2001). "Standard specification for hot-formed welded and seamless carbon steel structural tubing," A501, American Standards and Testing and Materials, 2001.

Aslani, F. and Goel, S. C. (1991). "Stitch spacing and local buckling in seismic-resistant double angles," Journal of Structural Engineering, ASCE, Vol. 177, no.8.

Astaneh-Asl, A. (1998). Seismic behavior and design of gusset plates, Structural Steel Education Council.

Astaneh-Asl, A., Goel, S. C., and Hanson, R. (1985). "Cyclic out-of-plane buckling of double-angle gusset plates," Journal of Structural Engineering, ASCE, Vol. 111 no. 5.

Astaneh-Asl, A., Goel, S. C. and Hanson, R. (1986). "Earthquake-resistant design of double-angle bracing." Engineering Journal, AISC, fourth quarter.

Black, R.G., Wenger, W. A. B. and Popov, E. P. (1980). Inelastic buckling of steel struts under cyclic load reversals. UCB/EERC-80/40, Berkeley: Earthquake Engineering Research Center, University of California.

Bruneau, M., Uang, C.M., and Whittaker, A. (1998). Ductile design of steel structures, McGraw-Hill, New York, NY.

Callister, J. T., and Pekelnicky, R. G. (2011). "Seismic evaluation of an existing low ductility braced frame building in California."

Celik, O., C., Berman, J. W., and Bruneau, M. (2004). "Cyclic testing of braces laterally restrained by steel studs to enhance performance during earthquakes." MCEER-04-0003, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.

DeWolf, J. T., and Pelliccione, J. F. (1979). "Cross-bracing design." Journal of the Structural Division, ASCE, 105(7), 1379-91.

El-Tayem, A. and Goel, S.C. (1986). "Effective length factor for design of x-bracing systems," Engineering Journal, AISC, 24(4).

Fell, B., Kanvinde,A., Deierlein, G., Myers,A., Fu, X. (2006). "Buckling and fracture of concentric braces under inelastic cyclic loading" Structural Steel Education Council, Steel Tips No.94.FEMA. (2000).

FEMA (2000). Prestandard and commentary for the seismic rehabilitation of buildings, FEMA 356, Federal Emergency Management Agency, Washington, D.C.

Goel, S.C. (1992). Cyclic post-buckling behavior of steel bracing members: Stability and ductility of steel structures under cyclic loading, CRC Press, Boca Raton, Florida.

Goel, S.C., and Lee, K. (1992). "Fracture criterion for concrete tubular bracing members under cyclic loading." Proceedings of ASCE Structures Congress, San Antonio, TX, April 13-15, 1992.

Hanson, R. D., and Martin, H. W. (1987). "Performance of steel structures in the September 19 and 20, 1985 Mexico Earthquakes." Earthquake Spectra, 3(2), 329-346.

Harris, J.L. and Speicher, M.S. (2015b). Assessment of First Generation Performance-Based Design Methods for New Steel Buildings, Volume 2: Special Concentrically Braced Frames – NIST TN 1863-2. National Institute of Standards and Technology, Gaithersburg, MD.

Hassan, O.F. and Goel, S. C. (1991). Modeling of bracing members and seismic behavior of concentrically braced steel structures. Report No. UMCE 91-1. University of Michigan, Ann Arbor, MI.

Imanpour, A., Stoakes, C., Tremblay, R., Fahnestock, L. and Daravan, D. (2013), "Seismic Stability Response of Columns in Multi-Tiered Braced Steel Frames for Industrial Applications," Proceedings Structures Congress, ASCE, pp. 2,650–2,661.

Imanpour, A. and Tremblay, R. (2014), "Seismic Design of Steel Multi-Tiered Braced Frames: Application of Incremental Static Analysis for Design of Steel Multi-Tiered Braced Frames," Proceedings Eurosteel, Naples, Italy, Paper no. 688.

Imanpour, A., Tremblay, R., Fahnestock, L. and Stoakes, C. (2016a), "Analysis and Design of Two-Tiered Steel Braced Frames Under In-Plane Seismic Demand," Journal of Structural Engineering, ASCE, Vol. 142.

Imanpour, A., Tremblay, R., Davaran, A., Stoakes, C. and Fahnestock, L. (2016b), "Seismic Performance Assessment of Multitiered Steel Concentrically Braced Frames Designed in Accordance with the 2010 AISC Seismic Provisions," Journal of Structural Engineering, ASCE, Vol. 142.

Jain, A. K. and Goel, S. C. (1978). Hysteresis behavior of bracing members and seismic response of braced frames with different proportions, Report No. UMCE 78-3. University of Michigan, Ann Arbor, MI.

Kato, B., Tanaka, H., and Yamanouchi, H. (1980). "Field work investigation of steel building Damage due to the 1978 Miyagi-ken-oki Earthquake." Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, 479-486.

Kelly, D. J., Bonneville, D. R., and Bartoletti, S. J. (2000). "1994 Northridge earthquake: damage to a four-story steel braced frame building and its subsequent upgrade," Proceedings of the 12th World Conference on Earthquake Engineering, New Zealand Society for Earthquake Engineering, Upper Hutt, New Zealand.

Khatib, I.F., Main, S.A. and Pister, K.S. (1988). Seismic behavior of concentrically braced steel frames, Report No. UCB/EERC-88/01. University of California, Berkeley.

Kim, H. I. and Goel, S.C. (1992). A seismic evaluation and upgrading of braced frame structures for potential local failures, UMCEE 92-24, University of Michigan, Ann Arbor.

Lee, S. and Goel, S.C. (1987). Seismic behavior of hollow and concrete-filled square tubular bracing members, UMCE 87-11; Report UMCE 87-11, Dept. of Civil Engineering, University of Michigan, Ann Arbor, Mich.

Lehman, D.E, Roeder, C.W., Herman, D., Johnson S. and Kotulka, B. (2008), “Improved Seismic Performance of Gusset Plate Connections,” Journal of Structural Engineering, Vol. 134, No. 6, pp.890-901.

Liu, Z., and Goel, S. C. (1988). “Cyclic load behavior of concrete-filled tubular braces,” Journal of Structural Engineering 114 (7), 1488-1506.

Naeim, F. (1997). Performance of extensively instrumented buildings during the January 19, 1994 Northridge Earthquake, John A Martin And Associates, Los Angeles, CA.

Naeim, F. (1998). "Performance of 20 extensively instrumented buildings during the 1994 Northridge Earthquake." The Structural Design of Tall Buildings, 7(3), 179-215.

Osteraas, J., and Krawinkler, H. (1989). "The Mexico Earthquake of September 19, 1985 -- Behavior of steel buildings." Earthquake Spectra, 5(1), 51-88.

Popov, E., and R.G. Black (1981). “Steel struts under severe cyclic loadings.” Journal of the Structural Division, ASCE, vol. 107, September.

Popov, E.P. (1976). Mechanics of Materials, 2nd edition, Prentice-Hall.

Rabinovitch, J.S. and Cheng, J.R. (1993). Cyclic behavior of steel gusset plate connections, University of Alberta. Dept. of Civil Engineering. Edmonton, Alberta. Structural engineering Report no. 191.

Remennikov, A.M. and Walpole, W. P. (1998). “Seismic behavior and deterministic design procedures for steel V-braced frames” Earthquake Spectra. EERI, Vol 14, no.2.

Sabelli, R. (2001). “Research on improving the design and analysis of earthquake-resistant steel braced frames,” EERI/FEMA NEHRP Fellowship Report, EERI, Oakland, California.

Sabelli, R., Wilford, C. Abey, K., Pottebaum, W. and Hohbach, D. (1998). “Pushover analysis of inverted-V braced frames and its implications for the design of multistory systems,” Proceedings of the 67th Annual Convention. Structural Engineers Association of California, Sacramento, CA.

Sabelli, R., and Hohbach, D. (1999). “A design of cross-braced frames for predictable buckling behavior,” Journal of Structural Engineering, ASCE, Vol 125, no.2.

Shaback, B., and Brown, T. (2003). “Behavior of square hollow structural steel braces with end connections under reversed cyclic axial loading.” Canadian Journal of Civil Engineering 30, 745-753.

Stoakes, C. and Fahnestock, L. (2013), “Three-Dimensional Finite Element Simulation of the Seismic Behavior and Performance of Multi-Tier Braced Frames,” Interim Report to AISC, September 3.

Tang, X. and Goel, S.C. (1987). Seismic analysis and design considerations of braced steel structures, Report UMCE 87-4. University of Michigan, Department of Civil and Environmental Engineering, Ann Arbor, MI.

Tang, X. and Goel, S. C. (1989). “Brace fractures and analysis of Phase I structure,” Journal of Structural Engineering, ASCE, Vol.115, no.8.

Tremblay, R., et al. (1995). "Performance of steel structures during the 1994 Northridge earthquake." Canadian Journal of Civil Engineering, 22(2), 338-360.

Tremblay, R., and Filiatrault, A., (1996). "Seismic impact loading in inelasticc tension-only concentrically braced steel frames: Myth or reality?" Earthquake Engineering and Structural Dynamics, 25(12), 1373-1389

Tremblay, R. (2000). “Influence of brace slenderness on the seismic response of concentrically braced steel frames,” Behaviour of Steel Structures in Seismic Areas – STESSA 2000: Proceedings of the Third International Conference, August 21-24, 2000, pp527-534, Montreal, Canada

Tremblay R. and Tirca, L. (2003). “Behaviour and design of multi-storey zipper concentrically braced steel frames for the mitigation of soft-storey response” STESSA, 2003, Balkema, Naples.

Tremblay, R. and Poncet, L. (2004). “Improving the seismic stability of concentrically braced steel frames,” Proc. 2004 SSRC Annual Technical Session & Meeting, Long Beach, CA, 19-38.

Uriz, Patxi (2005). Towards earthquake resistant design of concentrically braced steel frames, Doctoral Dissertation, Structural Engineering and Mechanics of Materials, University of California, Berkeley, December 2005

Whitmore, R.E. (1952). “Experimental investigations of stresses in gusset plates,” Engineering Experiment Station, Bulletin No. 16. University of Tennessee.

WJE (1998). "Oviatt Library, East and West Wings investigation of base plate damage caused by the Northridge Earthquake." Wiss, Janney, Elstner Associates Inc., Emeryville, CA.

Xu, P. and S. C. Goel (1990). Behavior of double channel bracing members under large cyclic deformations, Report UMCE 90-01, Dept. of Civil Engineering, University of Michigan, Ann Arbor, Mich.

Yang, F., and Mahin, S. (2005). “Limiting Net Section Fracture in Slotted Tube Braces,” Steel TIPS Report, Structural Steel Education Council, Moraga, CA

Yoo, Jung-Han; Roeder, Charles W., Lehman, Dawn E. (2008). “Analytical performance of special concentrically braced frames,” Journal of Structural Engineering, American Society of Civil Engineers, Vol. 134, No. 6, June 1, 2008.

Zayas, V. A., Popov, E. P., and Mahin, S. A. (1980). Cyclic inelastic buckling of tubular steel braces, UCB/EERC-80/16, University of California, Berkeley, Berkeley, California

Zayas, V.A., P. Shing, B. Shum, S.A. Mahin, and E.P. Popov (1981). Inelastic structural modeling of braced offshore platforms for seismic loading, UCB/ EERC-81/04: Earthquake Engineering Research Center, University of California.

Keywords
braced frames
concentrically braced frames

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Concentrically Braced Frames," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.2, 14.2	1908	ACI 318-14 Chapter 18

Introduction

This article summarizes key provisions, provides recommendations on seismic issues related to reinforced concrete elements and systems, and identifies design issues in need of further study, clarification, and/or research.

Special Moment Resisting Frames (SMF)

General. ACI 318-14 Section 18.6 provides prescriptive detailing requirements for beams of special moment frames. Similarly, Section 18.7 provides requirements for 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 than that provided in 18.6.4 is required. As the axial load in a member increases, the required confinement needs to increase 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 18.6 (beam requirements) have been used in the past to exempt columns from strong-column/weak-beam requirements. The SEAOC Seismology Committee does not recommend this approach. Instead, the committee recommends that only top-story columns be exempted from strong-column/weak-beam requirements.

Geometric Considerations for SMF Beams. Based on experimental evidence (Hirosawa 1977) and engineering judgement, stocky frame members with length-to-depth ratios of less than four, under reversals of displacement into the nonlinear range, may respond differently from slender members. Design rules derived from experience with relatively slender members, therefore, may not apply to members with length-to-depth ratios less than four, especially those pertaining to shear strength and ductility. As a result, ACI 318 requires the depth of the beams to be less than one quarter 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. Ultimately, 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 reinforcement ratio of 0.025 for the beams of a SMF. Generally, beams with high reinforcement ratios are less ductile. As a result, the SEAOC Seismology Committee recommends that the reinforcement ratio in beams the resist seismic forces be limited to 0.015. 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 a SMF, are 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 the SEAOC Seismology Committee, particularly if upper limits of the reinforcement ratio are used. Without nearly equal positive and negative reinforcement, the hysteretic behavior is prone to unidirectional post-elastic strain accumulation that may result in residual deformation. (Fenwick, Dely, and Davidson 1999)

Transverse Reinforcement for SMF Beams. ACI 318 section 18.6.4 prescribes the use of hoops in the potential hinge regions of frame beams. The beam hinge region is taken as the 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 in section 25.7.2.3 and 25.7.2.4. 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 will likely 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 will then be compressed back toward its original length when the earthquake reverses direction. It is this cyclical stretching and compression of rebar that 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 from buckling. ACI 18.6.4.4 prescribes a spacing of the lesser of d/4, 6 times the diameter of the smallest primary flexural reinforcing bars (excluding longitudinal skin reinforcement of Section 9.7.2.3) or 6 in. 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, because the 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 that the compression zone of frame beams should be detailed with an appropriate level of confining reinforcement. The compression zone is limited to the portion of the beam where the compression stress block occurs and defines the location of the neutral axis. The depth of the compression zone is typically small compared to the overall depth of the beam. 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 bar, 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 Section 9.7.6.4.1. This additional horizontal tie should have seismic hooks at each end and not be spaced further than 6” inside both the top and bottom horizontal legs of the outer hoop. Confinement will enhance beam performance under large inelastic rotational demands imposed during a major seismic event, which analyses have shown to be on the order of 4% in frame beams.

Transverse reinforcement for beams should also be designed to satisfy the entire shear demand without considering any contribution from concrete shear strength. This is because the 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 seismic 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. Nonlinear analyses of frame structures have shown that current strong-column/weak-beam provisions can be inadequate in preventing story collapse mechanisms (Bracci and Dooley 2001, Kuntz and Browning 2003). Inelastic analyses of frames have 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 for an entire story. The SEAOC Seismology Committee continues to support this position. This recommendation approximately equates to $\Sigma M_c / \Sigma M_b$ of 2 for the story, 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. In some cases, additional analysis can reduce this margin. This additional analysis should consider the following factors:

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 that calculated. (See the section later in this paper titled: “Consider Variations in Axial Loads for Columns of SMF’s”.)
2. Column overstrength and limited yielding may be considered in reducing the margin.
3. Accurate assessment of column moment magnification through inelastic analysis may indicate 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. Furthermore, the columns permitted to be weaker than the beams should 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_g; rather a curvature analysis should be used (Mander, Priestley, and Park 1988).

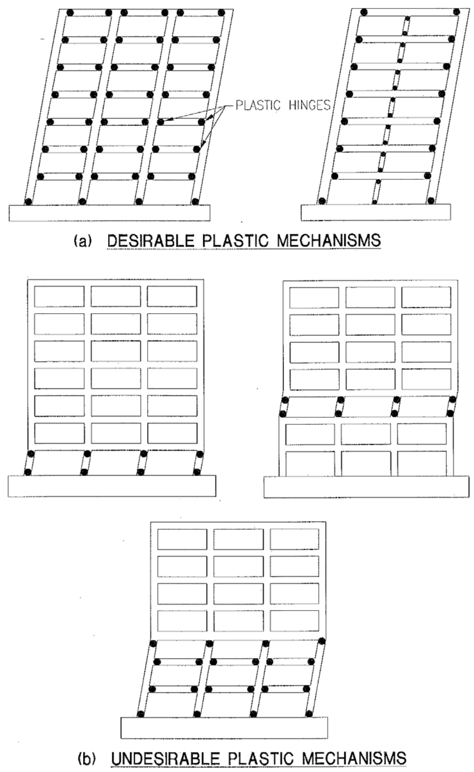


Figure 1. Desirable and undesirable plastic mechanisms.

Transverse Reinforcement for SMF columns. ACI 318 provides several equations for determining the amount of transverse reinforcement in frame columns. Eqs. (a), (b), (d) and (e) in Table 18.7.5.4 seek to maintain the axial load capacity of the column, even if the column concrete cover were to spall off. Equations (a) and (d) seldom govern the design, except for small column cross sections. Eqs. (b) and (e) in Table 18.7.5.4 govern where column sections are large, as is commonly 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 direct proportion to concrete strength.

Experimental and analytical studies have illustrated the need for increased confinement as axial load increases (Bayrak and Sheikh 1998, Brachmann et al. 2004; Li and Park, 2004, Paultre and Legeron 2008, Saatcioglu and Razvi 2002, Sheikh and Khoury 1997, Watson et al. 1994, Wehbe et al. 1999). Building codes outside the United States have adopted provisions for confinement that depend on the level of axial load (CSA, 2004, NZS 2006). ACI provisions currently address this in equations (c) and (f) to ensure a drift capacity of at least 0.03 for columns with high axial loads.

Story Drift. Drift demand is an important 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.

ASCE 7 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 code prescribed seismic forces. The drifts are then amplified by C_d, the deflection amplification factor, to estimate the inelastic drifts in the structure. The C_d factor is on the order of 0.7 times R. Analytical studies by the BSSC Technical Committees, as well as other nonlinear response history analyses, have observed drifts higher than those estimated using 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. (As of this writing, there is a BSSC ballot to revise this section of ASCE 7 and make $C_d = R$.) Thus, structures designed to the 0.02 drift limit per ASCE 7 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 MCE shaking in Seismic Design Categories D, E and F the actual story drift demand could therefore be on the order of 0.04 for most code-based designs (approximately 0.029/(2/3)).

Thus, the SEAOC Seismology 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.

Confinement Efficiency. The configuration of the transverse reinforcement, whether all longitudinal bars are tied, the presence of 90-degree bends at the ends 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 crossties in regions of the column susceptible to inelastic yielding. This is based on observing strength degradation due to loss of the concrete cover and the subsequent longitudinal bar buckling at the 90-degree hook ends of cross ties.

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 columns. Pourzanjani & Englekirk (2000) found an approximate attainable drift level of 3.5% for columns tested with an axial service load level of 0.34 $A_g f'c$ (i.e. without load factor) versus a drift level of 6% for the same column with a lower axial load level of $0.2A_g f'c$.

Variations in Axial Loads for Columns of SMFs. Axial loading on frame columns is greatly impacted by the inelastic dynamic response of frame beams, because these 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 a column is from the center of a frame, the higher the axial loading on the column is. Thus, the end columns of the frame will experience the highest levels 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 the 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 appropriate levels of additional capacities 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 the increase in column concrete strength due to a fast rate of loading, overstrength of the column, an adjusted phi factor (ø =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. Future adjustments in codes and standards should only be implemented after further research and comparisons with field observations from earthquakes can be made.

Ø Factors for SMF Columns. ACI 318 Table 21.2.2 currently prescribes a different *ø* factor for compression–controlled members (i.e. most columns) for spiral and rectangular transverse reinforcement: *ø* is 0.75 for spirally reinforced members and 0.65 for members with rectangular tie reinforcement. This difference in the recommended phi factors is based on the enhanced post-elastic performance of the confined concrete afforded by spiral reinforcement. The SEAOC Seismology Committee believes that current design methods for rectangular hoops in SMF columns achieve equivalent confinement effectiveness to spiral reinforcement. Accordingly, the SEAOC Seismology Committee recommends that the *ø* factor for columns with spiral reinforcement and with rectangular hoop reinforcement should be identical, i.e. *ø* = 0.75. The ACI standard, however, does not yet allow this recommended relaxation to be applied.

Shear Walls

Shear walls respond differently to seismic forces than moment-resisting frames. This 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 governed by flexure, 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 their flexural responses. However, this is not practical or efficient in all conditions, in particular when considering low or squat shear walls. Shear-yielding can provide sufficient post-yield stiffness and ductility (ASCE 2017). A sliding shear mechanism, however, should be avoided if possible. This can be accomplished by well distributed vertical reinforcement and/or the addition of vertical dowels. 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 and limit rotations 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; this can help to prevent shear yielding in walls. To ensure this behavior, the shear capacity of the wall must exceed its flexural capacity. For special reinforced flexural walls (See ASCE 7-16 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. The SEAOC Seismology Committee recommends that V_E be calculated per Equation 1 when an analysis is undertaken using prescribed static earthquake forces in the code, or by using Equation 2 when modal dynamic analysis is used for design. V_E in Equation 1 is increased to account for inelastic dynamic amplification of shear. The shear amplification factor, ω_v, in Equation 1 accounts for inelastic dynamic effects that can cause the vertical distribution of 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 ω_v are taken from Paulay and Priestly (1992). Note that ASCE 7-16 prescribes a parabolic distribution of earthquake forces depending on the height of the structure. However, Equation 1 can still be used as an approximation. Equation 1 can capture the elastic dynamic response of the structures. It uses the relationship of story shear and moment obtained from analyses to obtain demand V_E corresponding to M_{pr}. Given that this procedure is based on an elastic analysis, a level of uncertainty warrants the use of ϕ =0.75 in accordance with Table 21.2.1 and Section 21.2.4.1 for the calculation of shear strength, ϕV_n.

Design wall shear force using Equivalent Lateral Force Procedures of ASCE 7 Section 12.8:

$$V_E = (V_u\, M_{pr}/M_u)(\omega_v). \qquad \text{(Eq. 1)}$$

Where

V_u = shear demand from a static seismic force analysis occurring simultaneously with Mu, and in accordance with factored load combinations

M_{pr} = flexural demand corresponding to the axial load case that results in the largest value of M_{pr}.

M_u = flexural demand from a static seismic force analysis in accordance with factored load combinations

ω_v = shear amplification factor for a static seismic force analysis, assumed as $(0.9+N/10)$, for buildings up to 6 stories, and $(1.3+N/30)$, for buildings over 6 stories.

Design wall shear force using Linear Dynamic Analysis of ASCE 7 Section 12.9:

$$V_E = (V_u\, M_{pr}/M_u)\,(\omega_d). \qquad \text{(Eq.2)}$$

Where

V_u = shear demand from a dynamic seismic force analysis occurring simultaneously with Mu, and in accordance with factored load combinations

M_{pr} = flexural demand corresponding to the axial load case that results in the largest value of M_{pr}.

M_u = the flexural demand from a dynamic seismic force analysis in accordance with factored load combinations

ω_d = shear amplification factor for a dynamic seismic force analysis, assumed as $(1.2+N/50)$.

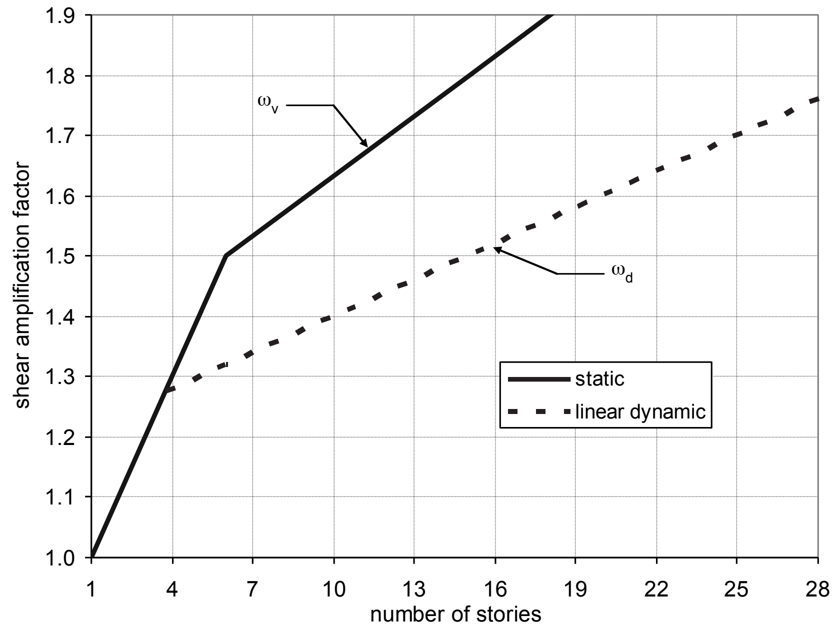


Figure 3. Relationship of the shear amplification factor over the number of stories.

***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. 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 undefined 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 7-16 (Table 12.2-1) 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's opinion that the R-value for bearing shear 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, assuming shear walls are properly detailed to maintain gravity axial load support under damaging cyclic loading. The Seismology Committee cautions that factored axial loads on shear walls should be limited to less than 0.35 $f'_c A_g$, unless ductility assessments using moment-curvature analyses are performed to justify higher axial load levels.

The 1999 Blue Book suggested placing gravity-load frame reinforcement within shear walls in order to artificially create a "building frame" wall. This has become popular with building officials and insurance underwriters, but it may not be necessary to maintain gravity load support, and it may adversely affect the seismic performance of walls. Where confined boundary elements are adequately designed within the shear wall, the addition of gravity frame reinforcement within 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 walls. The design engineer should consider the effect of heavy concentrated vertical loads or reactions within the wall length, for example from beams, columns, or intersecting walls, where additional tied vertical longitudinal reinforcement may be required. 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 consistent *R* values 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 gravity frames within walls, and without adding gravity load-bearing columns next to walls.

Shear wall response to earthquakes is greatly influenced by geometric configuration, as discussed above. Each type of wall configuration has a different degree of inherent ductility that warrants different detailing requirements. 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 walls 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 walls to promote shear yielding, except for limits of shear strength under section 18.10.4. Further studies on this topic are recommended.

In any event, walls that yield in shear are generally considered to have a greater propensity to exhibit strength degradation and low displacement capacity than flexurally-governed walls. As a result, the ductility demand on such walls should be limited. In multi-story 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 responses. 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, shear-yielding mechanisms may be an acceptable structural response.

As mentioned above, sliding shear mechanisms 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 in walls. The ASCE 7-16 requirements for load combinations result in some cases with only 60% of wall tributary dead loads 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 should be avoided at wall ends and instead should be distributed throughout walls to increase sliding shear resistance by increasing the lengths of the walls' compression zones (Paulay and Priestley 1992). This is shown in Figure 4. The effect of vertical accelerations is beyond the scope of this article, but is cited here because of the significant detrimental effect it can have on the performance of shear walls.

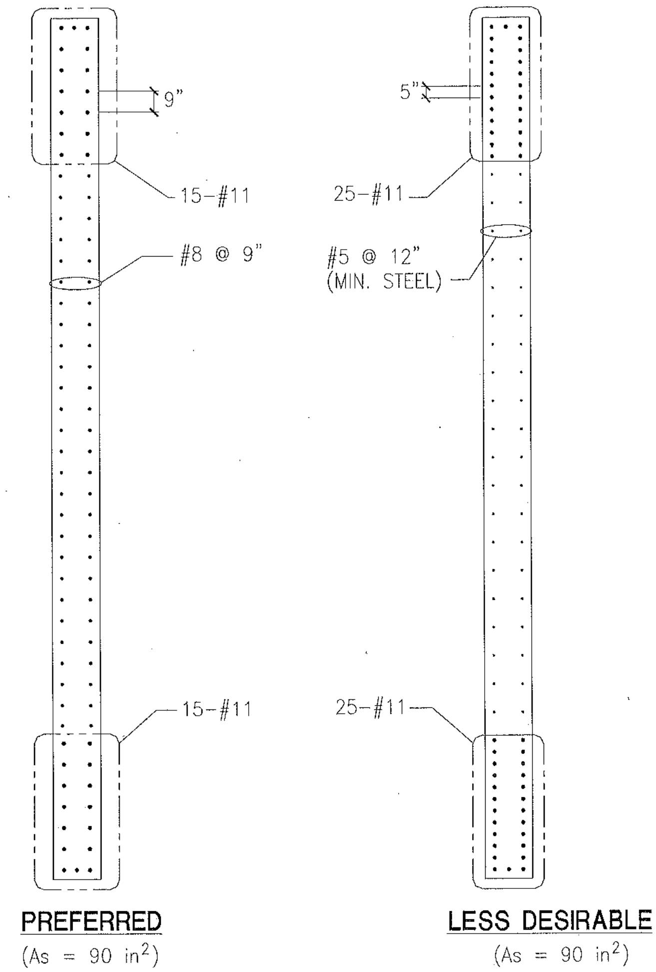


Figure 4. Distribution of vertical reinforcement along the length of shear walls

The SEAOC Seismology Committee recommends a study to develop appropriate *R* and C_d values and corresponding 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 configurations, more than one type of wall behavior may be present in a building. Unless the designer undertakes a more detailed study, the SEAOC Seismology Committee recommends that a single, lowest *R* value be used for the structural system and that the detailing provisions should be adjusted to account for the anticipated behavior.

Foundations of Shear Walls. The SEAOC Seismology Committee recommends that designers compare wall flexural and shear strengths to the capacity of the wall's foundations, and to the wall's strength in sliding shear. This is particularly important for low-rise walls. Specific recommendations are given in Paulay and Priestley (1992) and ATC (1998). Consistent with this, both OSHPD and DSA require foundations to be designed for amplified seismic forces in CBC Section 1617A.1.16.

Intersecting Wall Sections and Flanges. Connected or intersecting wall sections should be considered as integral units. The strength of flanges, boundary members, and webs should be evaluated on the basis of compatible interaction between these elements. The effect of wall openings should also be considered.

For walls connected to each other by beams, slabs, or coupling beams, the coupling action should 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 width as specified by ACI 318.

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 will not be capable of developing significant shear capacity until larger lateral deformation occurs. Vertically aligned wall openings create a coupled wall system that 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 should be exercised, as discussed by Taylor, Cote, and Wallace (1998).

Analyzing the flange effect can be difficult and time consuming for large, complex shear wall layouts. Note that flanged walls frequently behave asymmetrically; they can be compression controlled when the flange is in tension, and tension controlled when the flange is in compression. Also, note that ACI 318 requires that effective flange widths be calculated per Section 18.10.5.2.

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 simplification of coupling beam shear and confinement reinforcement is now allowed. In ACI 318, Section 18.10.7, the diagonal bars in the coupling beams no longer require transverse reinforcement directly bracing them. Instead, transverse reinforcement can alternatively be provided for the entire beam cross section, along with longitudinal reinforcement with spacing not exceeding smaller of 6 inches and $6d_b$, and hoop legs or cross ties both vertically and horizontally, with spacing not exceeding 8 inches. Each crosstie and each hoop leg should 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. The SEAOC Seismology Committee recommends against fully developing the top and bottom horizontal reinforcement into the wall piers, as this would increase the flexural strength, and consequently the shear demands, on the coupling beam. Instead, the SEAOC Seismology Committee recommends continuing the additional longitudinal reinforcement only 6 inches into the wall piers, which should be adequate to maintain the integrity of the coupling beam after cracking and spalling (Paulay and Priestley 1992).

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 minor strength degradation for coupling beams in the testing of four specimens designed to ACI 318-14. ASCE 41-17, Table 10-19, (ASCE 2017), 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 coupling beams. The SEAOC Seismology Committee recommends plastic hinge limitations of 0.03 to 0.05 radians 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 project. As additional testing data become available, they may justify higher rotational limits for coupling beams of certain reinforcement configurations.

Compatibility Requirements for Gravity Load-Resisting Elements

The seismic force-resisting systems in high seismic zones are designed and detailed to provide sufficient strength, stiffness, and ductility for anticipated earthquake-induced demands. However, elements that are not part of the seismic force-resisting system also experience seismic deformations and thus need to have proper ductility to withstand these. Concrete structures, in particular, will experience unintended frame-like behavior in the gravity load-resisting system when subjected to seismic forces. For example, in buildings with flat slab construction, the slab and column combinations will attempt to act as frames during earthquakes. 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. This is commonly referred to as the outrigger effect (PEER 2007).

ACI 318 Section 18.14 provides recommendations for deformation compatibility requirements for members not designated as part of the seismic-force-resisting-system, but further recommendations are warranted. The 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 columns. Alternatively, a subassembly representing the gravity frame of the structure may be subjected to the maximum anticipated drift, and the axial loads obtained may be incorporated into the column design.

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. ACI 318, Table 19.2.1.1, does not specify an upper limit on f'c for normalweight concrete structures. However, 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's 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. 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, 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 strengths of 6000 psi and higher consistent with ACI 363R-10.

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 stress-strain behavior for the mix. As a result, the C4 report recommends that the HSC mix has 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 ductility for concrete mixes in general but especially for HSC mixes.

Research indicates that HSC is significantly less prone to creep than normal weight concrete. However, it is somewhat more prone to shrinkage (Ngab et al, 1981; Marzouk, 1991).

HSC Special Moment Frames. The ITG, in Chapters 5 and 8, provides design provisions for HSC SMF's. 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 has found that 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.

ITG concluded 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 effectively confine joints to enhance performance, particularly in light of increased axial loading on the columns.

C4 report suggests that on lightly loaded joints with $P \leq 0.1f'cA_g$ 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 $1.25F_y$. SEAOC Seismology recommends that the joint overstrength factor be based on the $1.25F_y$ 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 determining shear strength in HSC members. It reports that the minimum shear reinforcement of equation in Table 9.6.3.3 in ACI 318-14 is appropriate for HSC. ITG emphasizes the importance of confinement in preventing rapid shear strength degradation in members subject to deformations in the inelastic range. 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 strength of 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, $V_n = ø0.09f'_c\, A_{core} \leq\ ø0.9\, A_{core}$. This is based on the concept 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. (b) in Table 18.7.5.4 for determining transverse reinforcement to provide a confinement pressure of 0.09 f'_c for the core of the column.

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 unconservative. 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 for the addition of transverse reinforcement over the splice lengths of bars located outside the compression zone of boundary elements in shear walls. ITG also indicates that the lengths of hooked 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 for determining the development of hooked 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 a study by Wallace (1998) and on other 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 failures due to rupture. This is particularly important 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 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 indicated that for HSC walls with $\rho_n f_y/f'_c \geq 0.08$, the current ACI 318 equations are unconservative. (This is because the stress-strain behavior of HSC is not compatible with the typical ACI compression block assumption.) The Wallace study also indicated 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 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. 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 recommends that for calculating the nominal shear strength limit, $8\sqrt{f'_c}$ (ACI 318 Section 18.10.4.4), the specified compressive strength of concrete shear walls in regions of high seismicity be limited to approximately $f'_c = 10$ ksi until further research provides more insight and recommendations for the design of HSC walls.

Pending further research and investigations, the committee recommends that transverse tie requirements in the boundary elements of shear walls be calculated based on current ACI 318 Ch. 18 requirements.

High Strength Reinforcement

Current ACI 318-14 provisions limit the yield strength of reinforcement used to resist earthquake forces in special moment frames and special structural walls to 60 ksi. The reason for this restriction is the lack of research supporting the use of high strength reinforcement in seismic zones. Since then, there have been documents published by ATC and NIST that investigate the feasibility of incorporating higher strength reinforcement (yield strength of 80 ksi or greater) into building codes, not just for seismic applications but for gravity-only members as well (ATC 2014 and NIST 2014). The last major change to US building codes in regard to reinforcement was made in 1971.

ATC anticipates high-strength steel to be specified using the existing ASTM A615 standard to cover yield strengths of up to 120 ksi for non-seismic applications. Bars to be used in seismic members would need to be specified under an entirely new standard. ASTM A706 would cover only grade 60 and 80 steel. When it comes to specifying high-strength steel, ATC recommends that producers and fabricators take the lead in establishing new standards and determining how this new steel will be specified. Producers and fabricators leading efforts to make high-strength rebar more widely accepted is a recurring theme throughout ATC's report. The role of practicing engineers and the research community would be to support and validate the use of high-strength steel across different applications (ATC 2014).

The fabrication of high-strength rebar presents several challenges. One of them is the added complexity introduced by having to sort, store, and ship several different grades of steel. Another one is the additional equipment that will need to be replaced or upgraded, not to mention the added difficulty of bending stronger bars. Once rebar begins arriving at a job site, contractors in charge of rebar placement run higher risks of using the wrong grades in the wrong places if many different bars are present and being handled simultaneously.

In terms of design, one potential concern with high strength reinforcement is the required length needed to develop these bars. Development length equations in ACI 318-14 are based on conventional grade 60 ksi reinforcement. Seliem et al. (2009) did lap splice tests with high strength reinforcement and compared the results to the equation in ACI 318. The tests found that Class A splices needed modification, but the 1.3 factor which is applied to Class B splices make them acceptable for high strength reinforcement. It was also found that confining the bars with transverse reinforcement could help reduce their development length. There should be special consideration for lap splice lengths for the following conditions: special moment frame beams and column longitudinal bars, structural wall special boundary elements, inner layers of structural wall typical reinforcement, outer layers of structural wall web reinforcement and reinforcement in columns that are not part of the seismic force resisting system.

When lap splice lengths become excessive, mechanical splices may be used. Currently, there are two types of mechanical splices defined in ACI 318. The NIST document recommends a third type of mechanical splice that would develop the tensile strength of the rebar for high strength reinforcement.

The required development for standard hooks and headed deformed bars are other areas where more information is required for high strength reinforcement. There were tests done in Japan investigating grade 100 hooked bars in beams. These tests found that the development lengths depended on a number of factors, such as concrete cover and concrete strength. It should be noted that the Japanese design provisions for development lengths are slightly different from those in ACI 318. Headed bars may be used instead of hooked bars where congestion may be an issue. At the time of writing this article, tests are being done at the University of Kansas on high strength (Grade 60 to Grade 120) hooked and headed bars. The results of these tests are not available at this time.

ACI 318-14 limits the spacing of transverse reinforcement in potential plastic hinge locations to six times the diameter of the longitudinal bar. This is done to restrain the longitudinal bars, and delay undesirable bar buckling during seismic events. Bar buckling can limit ductility and energy dissipation, which would result in an ineffective seismic force resisting system. Studies compared the theoretical compressive stress-strain curve of grade 60, grade 80 and grade 100 bars with different spacing-to-bar-diameter ratios. This study found that the grade 80 bars with an

s/d_b of 6 and the grade 100 bars with an s/d_b of 5 started to buckle at about the same strain as the grade 60 bars with an s/d_b of 6. Therefore, the NIST document recommends that the transverse bar spacing be set to 6 longitudinal diameters for grade 80 bars and 5 longitudinal diameters for higher grade reinforcement (NIST 2014). More generally, ATC recognizes that using high-strength steel in special moment frames and special shear walls is a viable option. At this time, more studies are still required to precisely evaluate, among other things, rotational capacities of joints and to quantify these capacities as a function of steel yield strength (ATC 2014). In addition, high strength bars are less ductile than typical reinforcing bars. For this reason, high-strength reinforcement can reduce the rotation capacity of intended plastic hinges (Bishaw 2016).

Construction tolerances should also be considered when incorporating these recommendations into code. Tanaka studied the how effective 90-degree hooks were in restraining longitudinal bars, and found that cross ties with 90 degree hooks on one end and 135 degree or 180 degree hooks on the other end were just as effective as 135 degree or 180 degree hooks on both ends. This study should be revisited for high strength reinforcement because the flexure stiffness of high strength reinforcement is probably less than what was studied by Tanaka. At the time this article was written, there have been limited tests on beams and columns detailed to develop plastic hinges.

The maximum permitted yield strength for shear reinforcement in ACI 318-14 is 60 ksi. Concrete with high strength shear reinforcement can potentially develop large diagonal tensional cracks. Ou et al observed that "The maximum strength did not occur at the same time with the yielding of the transverse reinforcement. The maximum stress of shear reinforcement in design should be limited." The NIST document recommends that the maximum permitted shear reinforcement allowed in design be increased to 80 ksi, and that more testing be done before grades higher than 80 ksi are allowed to be used in design. To account for higher yield strengths, current phi factors would likely need to be modified as well (NIST 2014).

The use of high-strength steel will need to be incorporated into building codes carefully in order to fully realize the benefits of using such steel. The reductions in the volume of steel required for projects need to be large enough to offset the premium that high-strength steel will command. From a constructability standpoint, increasing bar spacing is one of the best ways to lower costs and reduce construction times. Some estimates place the monetary savings when using grade 80 steel rather than grade 60 at 4% the cost of the concrete structure. While more research is required to begin using high-strength reinforcing at a large scale, the benefits are already clear.

References

ACI (2014). *ACI 318-14: Building Code Requirements for Structural Concrete with Commentary*, 2014 edition. American Concrete Institute, Farmington Hills, MI.

ACI (2007). *Report on structural design and detailing for high-strength concrete in moderate to high seismic applications*, ITG-4.3R-07, American Concrete Institute, Farmington Hills, MI.

ASCE (2017). *Seismic Evaluation and Retrofit of Existing Buildings,* ASCE/SEI 41-17. American Society of Civil Engineers, Reston, VA.

ATC (1978). *Tentative provisions for the development of seismic provisions for building codes*, ATC-3-06. Applied Technology Council, Redwood City, CA.

ATC (2014). *Roadmap for the use of high-strength reinforcement in reinforced concrete design,* ATC 115. Applied Technology Council, Redwood City, CA.

Bayrak O. and Sheikh, S. (1998). "Confinement reinforcement design considerations for ductile HSC columns." *Journal of Structural Engineering*, 124 (9), 999-1010

Bishaw, B.B. (2016). *Effect of High Strength Materials on the Seismic Performance of Reinforced Concrete Moment Resisting Frames*, University of Utah, Salt Lake City, UT.

Bracci, J.M. and Dooley, K.M. (2001). "Seismic evaluation of column-to-beam strength ratios in reinforced concrete frames." *ACI Structural Journal* 98 (6).

Brachmann, I., Browning, J., and Matamoros, A. (2004). "Relationships between drift and confinement in reinforced concrete columns under cyclic loading," Paper No.2531. *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada.

CBC (2019). *California Building Code*. California Building Standards Commission. International Code Council, Country Club Hills, IL.

Carpenters Contractors Cooperation Committee (2000). "High-strength concrete research." Carpenters/Contractors Cooperation Committee, Inc., Los Angeles, CA.

CSA (2004). "Design of concrete structures, A23.3-04." Canadian Standards Association, Mississauga, Canada.
Englekirk R.E. (2003). *Seismic design of reinforced and precast concrete buildings.* John Wiley & Sons, New York, NY.

Elwood, Kenneth J, Maffei, Joe, and Riederer, K. (2009). "Improving column confinement provisions in the ACI 318 building code." *Concrete International* (submitted for publication). American Concrete Institute, Farmington Hills Michigan

Fenwick, Richard, Dely, Raad, and Davidson, Barry (1999). "Ductility demand for uni-directional and reversing plastic hinges in ductile moment resisting frames." *Bulletin of the New Zealand Society for Earthquake Engineering*, 32 (1), Marach 1999.

Galano, L. and Vignoli, A. (2000). "Seismic behavior of short coupling beams with different reinforcement layouts." *ACI Structural Journal*, 97 (6).

Hirosawa, M., "Strength and ductility of reinforced concrete members," Report No. 76. Building Research Institute, Ministry of Construction, Tokyo, Mar. 1977 (in Japanese). See also related data in Civil Engineering Studies, Structural Research Series No. 452, University of Illinois, 1978.

Kabeyasawa T. and Hiraishi H. (1998). "Tests and analysis of high-strength reinforced concrete shear walls in Japan." *High-strength concrete in seismic regions*, SP-176. C.W. French and M.E. Kreger, eds., American Concrete Institute, Farmington Hills, MI, 311-328.

Kuntz, G.L. and Browning, J. (2003). "Reduction of column yielding during earthquakes for reinforced concrete frames." *ACI Structural Journal*, 100 (5).

Jury, R.D. (1978). *Seismic Load Demands on Columns of Reinforced Concrete Multistorey Frames*, Research Report 78-12. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Li, B., and Park, R. (2004). "Confining reinforcement for high-strength concrete columns." *ACI Structural Journal*, 101 (3), 314-324.

Maffei, Joe (1996). "Reinforced concrete structural walls: Beyond the code." *SEAONC 1996 Fall Seminar Notes*, Structural Engineers Association of Northern California, San Francisco, CA.

Mander, J.B., Priestley, M.J.N. and Park, R. (1988). “Theoretical stress-strain model for confined concrete.” *ASCE Journal of Structural Engineering*, 114 (8), 1804-1826.

Marzouk, H (1991). “Creep of High-Strength Concrete and Normal-Strength Concrete.” *Magazine of Concrete Research*, 34 (155): 121-126.

Megally, Sami, and Amim Ghali (2000). “Punching shear design of earthquake-resistant slab-column connections.” *ACI Structural Journal* 97 (5).

Ngab, A., Nilson, A., and Slate, F. (1981). “Shrinkage and Creep of High Strength Concrete.” *ACI Journal Proceedings*, 78 (4): 255-261.

NIST (2014). *Use of high-strength reinforcement in earthquake-resistant structures.* National Institute of Standards and Technology. Gaithesburg, MD.

Noguchi H., Fujii, S., and Teraoka, M. (1998). “Shear strength of beam-column joints with high-strength concrete.” *High-strength concrete in seismic regions*, SP-176, C.W. French and M.E. Kreger, eds., American Concrete Institute, Farmington Hills, MI. 329-356.

Standards Association of New Zealand (2006). “Concrete design standard, part 1” and “Commentary on the concrete design standard, part 2,” NZS 3101:2006, Wellington, New Zealand.

Pan, Austin D., and Moehle, Jack P. (1992). “An experimental study of slab-column connections.” *ACI Structural Journal*, 89 (6).

Paulay, T. (1977). “Seismic design of ductile moment-resisting reinforced concrete frames; Columns: Evaluation of actions,” *Bulletin of the New Zealand National Society for Earthquake Engineering*, (10) 2.

Paulay, T., and M.J.N. Priestley (1992). *Seismic design of reinforced concrete and masonry buildings*. John Wiley & Sons, New York, NY.

Paulay, T., M.J.N. Priestley, and Synge, A. J. (1982). “Ductility in earthquake resisting squat shear walls.” *ACI Structural Journal* 79 (4).

Paultre, P., Legeron, F. (2008). "Confinement reinforcement design for reinforced concrete columns." *Structural Journal*, ASCE, 134 (5).

PEER (2017).”Guidelines for Performance-Based Seismic Design of Tall Buildings,” Version 2.03, Pacific Earthquake Engineering Research Center, Berkeley, CA

Pourzanjani, M. and Englekirk R.E. (2000). “High-strength concrete applications in regions of high seismicity.” *Proceedings of the Fifth Conference on Tall Buildings in Seismic Regions*, Los Angeles, CA, May 5, 2000, 201-217.

Saatcioglu, M. and Ozcebe, G. (1989). “Response of Reinforced Concrete Columns to Simulated Seismic Loading.” *ACI Structural Journal*, January-February.

Saatcioglu, M. and Razvi, S.R. (2002) “Displacement-based design of reinforced concrete columns for confinement.” *ACI Structural Journal*, 99, 1, 3-11.

Saqan E.I.and Kreger, M.E. (1998). “Evaluation of U.S. shear strength provisions for design of beam-column connections constructed with high-strength concrete.” *High-strength concrete in seismic regions*, SP-176, C.W. French and M.E. Kreger, eds., American Concrete Institute, Farmington Hills, MI. 311-328.

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

SEAOC Seismology Committee (1999). *Recommended lateral force requirements and commentary*, seventh edition, Structural Engineers Association of California, Sacramento, CA.

Sheikh, S.A., and Khoury S.S. (1997). "A performance-based approach for the design of confining steel in tied columns." *ACI Structural Journal*, 94 (4) 421-432.

C. Taylor, P. Cote, and J. Wallace (1998). "Design of slender RC walls with openings," *ACI Structural Journal* 95 (4) July-August, 420-433.

Tegos, I.A. and Penelis, G. (1988). "Seismic resistance of short columns and coupling beams reinforced with inclined bars." *ACI Structural Journal* January-February 1988.

Wallace J.W. (1998) "Behavior and design of high-strength RC walls." *High-strength concrete in seismic regions*, SP-176, C.W. French and M.E. Kreger, eds., American Concrete Institute, Farmington Hills, MI, 259-279.

J. Wallace and K. Orakcal (2002). "ACI 318-99 provisions for seismic design of structural walls." *ACI Structural Journal*, 99 (4), July-Aug., 499-508.

Watson, S., Zahn, F.A., and Park, R. (1994). "Confining reinforcement for concrete columns." *Journal of Structural Engineering*, 120 (6), 1798-1824.

Wehbe, N., Saiidi, M.S., and Sanders, D. (1999). "Seismic performance of rectangular bridge columns with moderate confinement." *ACI Structural Journal* 96 (2), 248-258.

Wood S.L. (1990). "Shear strength of low-rise reinforced concrete walls." *ACI Structural Journal*, 87 (1) Jan-Feb, 99-107.

Xiao, Y., and Yun, H.W. (1992). "Full-scale experimental studies on high-strength concrete columns." University of Southern California, Report No. USC-SERP95/05, July.

Keywords
concrete
reinforced concrete

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

> (SEAOC Seismology Committee 2019)

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

> SEAOC Seismology Committee (2019). "Reinforced Concrete Structures," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	ACI 318-14 Reference section(s)	2019 CBC reference section(s)
12.11	11.8 18.10 18.11 18.5	1901.3 1905.1.3 1905.1.8 2305

Introduction

Reinforced concrete tilt-up buildings are the most popular form of light industrial and low-rise commercial construction in the Western United States, and 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.

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 wood framing whose width depends on the shear demand, 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.

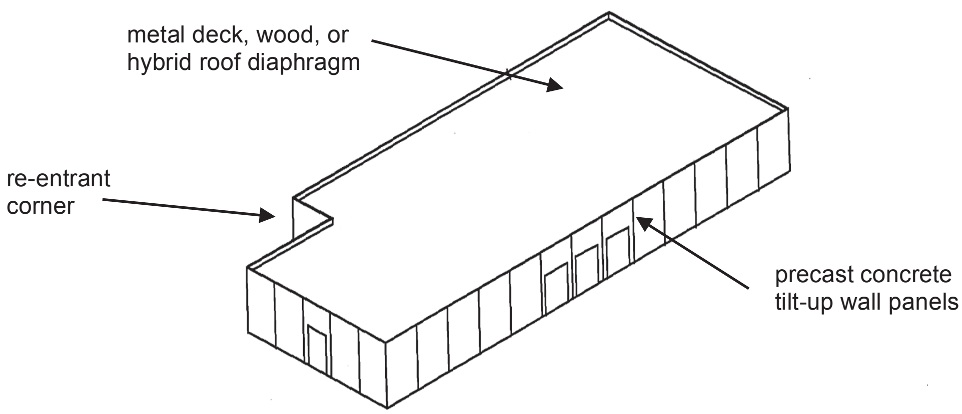


Figure 1. Typical structural features of a tilt-up building

The most critical component of tilt-up building seismic performance is the anchorage of the wall to the roof structure. This anchorage is typically accomplished with embedded concrete anchors or straps attached to the roof framing system.

Another critical component is the connection from the wall panels to the foundations. This is usually achieved through lapped reinforcing within a pour strip. It is strongly recommended that the wall panels be connected to the foundations in high seismic zones to provide a positive load path per ASCE 7-16 12.14.2.

Historical Background

In the 1950s and 1960s, reinforced concrete tilt-up walls were considered to be an innovative way to build economical structures. Over time, market demands pushed for taller buildings, which resulted in more slender tilt-up walls. In 1977, the SEAOSC Board appointed an Ad Hoc Committee to review the issues on appropriate design practice. The Committee concluded that the wall height-to thickness ratio, l_c/h, could be increased beyond the code requirement of 25, provided that P-Delta effects are included in the design. In 1979, SEAOSC published "Recommended Tilt-up Wall Design", also known as the Yellow Book, which pioneered an alternative design procedure for slender walls.

The Yellow Book helped to achieve uniformity in design practice in the Southern California area. A brief summary of the guidelines in this publication is given below:

- $l_c/h \leq 36$ for panels supported top and bottom,
- $l_c/h \leq 42$ for panels supported all four edges,
- Account for P-Delta effects, when determining the controlling deflection at the wall's nominal strength, Mn

The underlying philosophy of this methodology was based on the idealized deflection curve shown in Yellow Book Fig. 1.

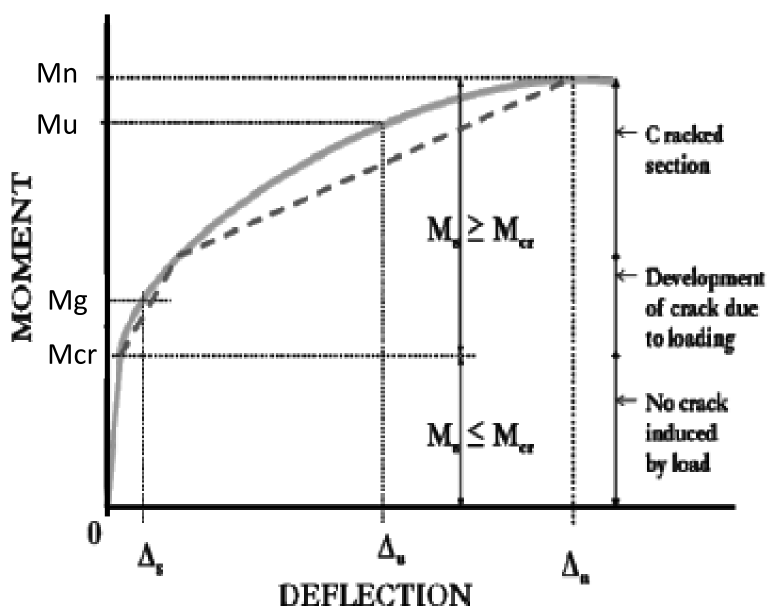


Fig. 1 – Idealized Moment Deflection Curve

The original equations and methods have been incorporated into ACI 318-14 Section 11.8. For an in-depth summary of the history of tilt-up research, performance observations and provisions, refer to the 2008 version of this paper SEAOC Seismology Committee (2008) Bluebook Tilt up Buildings article 9.02.010.

The evolution of the anchorage force levels between 1958 and 2018 for RWFD (Rigid Wall Flexible Diaphragm) buildings is illustrated in Figure 2 below from Filiatrault, A., Kelly, D., Koliou, M., Lawson, J (2018).

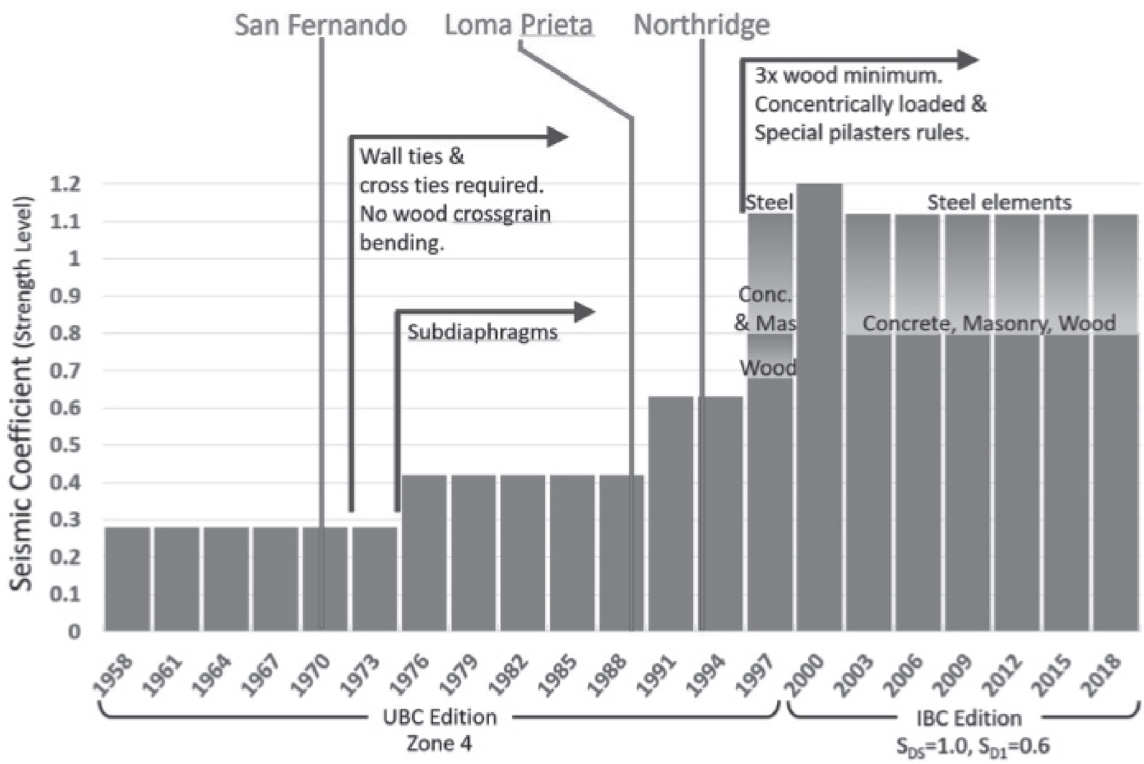


Figure 2. Evolution of Wall Anchorage Force Levels and other Provisions for RWFD Buildings in Western United States

Current Wall Anchorage Provisions

ASCE 7-16 Section 12.11.2.1 governs wall anchorage design for most tilt-up buildings in California, with additional requirements in Section 12.11.2.2 for structures assigned to Seismic Design Category C through F.

The wall tie force $F_P=0.4S_{DS}K_aIeW_P$ for a flexible diaphragm can be twice as much as the wall design force for a rigid diaphragm in Section 12.11.1, and four times the typical tilt-up building base shear, to account for the expected rooftop amplification associated with a flexible diaphragm. For structures in Seismic Design Categories C through F, see additional anchorage requirements in ASCE 7-16, Section 12.11.2.2

ASCE 7-16 implements very high wall anchorage forces 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 318-14 has combined the elevated force levels of the overstrength approach in ASCE 7-16 with new ductility requirements for seismic anchorage in ACI 318-14 Chapter 17. These ductility requirements were inadvertently added on top of the elevated anchorage force levels in conflict with the original intent of the ASCE provisions.

Wall anchorage forces act in compression as well as tension. Panelized wood roof systems, by their nature, are not erected tight against the perimeter wall ledger, leaving a small gap to potentially close during seismic compression forces. This gap is 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, wall anchorage is often 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 resulting force components induced by the configuration (Section 12.11.2.2.6). The bending induced by single-sided connections combined with the wall tie axial load may overstress the attached wood roof member and cause 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 past earthquakes (EERI 1996). Where pilasters occur, ASCE 7-16 Section 12.11.2.2.7 requires consideration of the force 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 minimum wall anchorage force elsewhere at a floor or roof is not permitted.

Anchorage to Wood Diaphragms. Wall anchorage to wood roof systems is not allowed to depend upon cross-grain bending, nailing in withdrawal, or diaphragm sheathing in tension (12.11.2.2.3). Wall anchorage loads are transferred into the main diaphragm with subdiaphragms and continuous crossties. These provisions are a direct result of the damage observed after the 1971 San Fernando Earthquake.

Subdiaphragms are provided under ASCE 7-16, 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 2½ to 1, which provides sufficient stiffness to limit potentially incompatible deflections between subdiaphragms and the main diaphragm. Tilt-up warehouse buildings today often have large column spacing of up to 70 feet, resulting in very large subdiaphragm spans and corresponding subdiaphragm depths. Pursuant to investigations of damage to wood diaphragms after the 1994 Northridge Earthquake, the joint task force committee recommended continuous ties at specified spacing to control cross grain tension in the interiors of diaphragms, and limited subdiaphragm shear to control combined orthogonal stresses within subdiaphragms. As a result, Los Angeles City and Los Angeles County jurisdictions have taken a conservative approach by limiting 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 2016 California Building Code, for structures assigned to Seismic Design Category D, E, or F, wood diaphragms supporting concrete or masonry walls shall comply with the following: 1. The spacing of continuous ties shall not exceed 40 feet. Added chords of diaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. 2. The maximum diaphragm shear used to determine the depth of the subdiaphragm shall not exceed 75 percent of the maximum diaphragm shear. (2016 LARUCP).

The benefit of the 300 plf upper bound on subdiaphragm shear strength is the reserve capacity available for orthogonal effects from seismic forces. Because subdiaphragms are a part of the main diaphragm, they are theoretically subject to shear forces from both orthogonal directions. Consideration of orthogonal loading effects in diaphragm shears is not normal practice today, however this approach may be more rational than arbitrary shear capacity limits. One such approach 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 dynamic amplification associated with flexible diaphragms amplifies the wall anchorage forces, but this increase is limited by yielding of roof diaphragms. 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 design is not excessively conservative. Because tilt-up buildings are often long and narrow, diaphragm designs are frequently 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. However, there are important detailing issues that must be carefully considered.

Metal deck can only provide continuous crossties parallel to the deck span direction. ASCE 7-16 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 forces 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 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 2016) 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 reduces 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 have been applied (2016 CBC Section 2207.2). 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. There are limits to the amount of force that manufacturers can transfer through these joist seats. The rapid growth of the use of steel joist systems in tilt-up buildings since the 1994 Northridge Earthquake will necessitate observations of their performance after future earthquakes.

Another result of the 1994 Northridge Earthquake was a new tilt-up retrofit ordinance by the City of Los Angeles. Because one third of the nearly 1200 tilt-up buildings in the San Fernando Valley suffered significant damage in that earthquake, Los Angeles developed an ordinance in conjunction with SEAOSC to require wall anchorage and continuity ties in existing pre-1976 tilt-up buildings. A similar ordinance was subsequently adopted in Los Angeles County and other jurisdictions. These ordinances do not attempt to force older tilt-up buildings to comply with current code requirements, but instead aim 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, ICC 2016, 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 roof diaphragms to span farther, with higher shear stresses. In the 2015 NDS SDPWS, the maximum allowable shear in horizontal blocked wood diaphragms is 820 plf (ASD) per NDS Table 4.2A. For special high-load wood diaphragms with multiple rows of fasteners per NDS Table 4.2B, the maximum allowable shear is 1565 plf (ASD). In large tilt-up buildings, engineers have relied upon these high-load diaphragm values with shear capacities up to 1565 plf. First introduced as an ICBO Evaluation Service ER document, these high-capacity wood diaphragms are now incorporated into the CBC. 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.

Metal deck roofs are more often common in California tilt-up buildings than in the past. 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. Research has shown that diaphragms connected using welds and button punches have a limited ability to dissipate energy achieving less inelastic deformation that those connected with nails and screws. In recent years, engineers have used power driven frame fasteners to connect decks to supports and have used special carbon steel self-driven screws for side lap connections (Massarelli 2010).

Since wall anchorage and collector design forces have been factored up to maximum expected levels, and perimeter shear walls often consist 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. In the past, structural chords typically consisted of special reinforcing bars embedded in the wall panels near the roofline, connected with welds 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 has caused concrete spalling and weld-breaks, even without earthquakes.

Prior to the late 1980s, reinforcing chord bars were generally ASTM A615 Grade 60, 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 AWS D1.4 discourages welding of ASTM A615 reinforcing by requiring a report of material properties from the producer and to establish proper preheating. ASTM A706 does not have these requirements.

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 skewed wall connections, and these forces result in inward or outward thrust components. Load paths into the main diaphragms for these components are difficult, and improper detailing can result 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 at re-entrant corners or interior braced frame elements. In perimeter walls lines where the panels are not connected to each other across vertical joints, lateral forces are distributed in proportion to each panel stiffness without the need for collectors. 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, a collector is connected to an embedded steel plate with drag reinforcing used to distribute forces into the shear wall line. The collector forces are normally dragged across panel joints in order to distribute the collected diaphragm force into sufficient wall length. The collector forces in the roof structure are subject to the special load combinations referenced in ASCE 7-16 Sections 12.10.2 and 12.4.3.

In-plane Diaphragm Deflections. In tilt-up shear wall buildings, diaphragm deflections are computed for the design of building separations and deformation compatibility. In-plane shear wall drifts are typically insignificant compared with diaphragm deflections and are usually ignored. Also ignored are wall panel out-of-plane deflections except when considering building separations. The estimated deflection of plywood diaphragms may be obtained from analysis per 2019 CBC Section 2305.2. Research indicates that under seismic forces, 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 that reduce actual diaphragm deflections (Lawson 2007c). Metal deck diaphragm deflections are computed using formulas furnished in their respective Evaluation Reports.

In tilt-up buildings, diaphragm deflections result in the columns and perpendicular walls rotating about their bases. 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 anchorage combined with any wall-to-footing anchorage. ACI 318-14 Section 11.8.1 requires slender tilt-up walls to be tension-controlled and limits the factored vertical concrete stress to $0.06f_c$'. This generally allows panels to better accommodate any localized yielding at the base while continuing to carry the vertical loads.

Diaphragm deflections are not normally included in the story drift limits of ASCE 7-16 Section 12.12.1. Story drift limits were developed with the intent to limit the deformation of the vertical elements of seismic force-resisting systems. In tilt-up buildings, these vertical elements generally deflect very little in-plane, with most of the translation occurring at other elements. Story drift limits do not apply to diaphragm deflections.

Over the past decade, most practitioners have calculated diaphragm deflections only for the purposes of building setbacks from property lines or structural separations from other adjacent buildings, such as that required in ASCE 7-16 Section 12.12.3. With larger and more flexible diaphragms being built today, CBC Section 2305.2 and ASCE 7-16 Sections 12.12.2 and 12.12.5 are becoming more important:

> Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

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 *P*-delta effects. 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 7-16 Section 12.8.7 can be used to investigate stability under *P*-delta effects.

Diaphragm Re-entrant Corners. Unfortunately, tilt-up design 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 diaphragm movements. 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 the provisions in tilt-up buildings (See Figure 3). 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 fin walls, and racked interior partition walls in contact with either the roof or tilt-up walls.

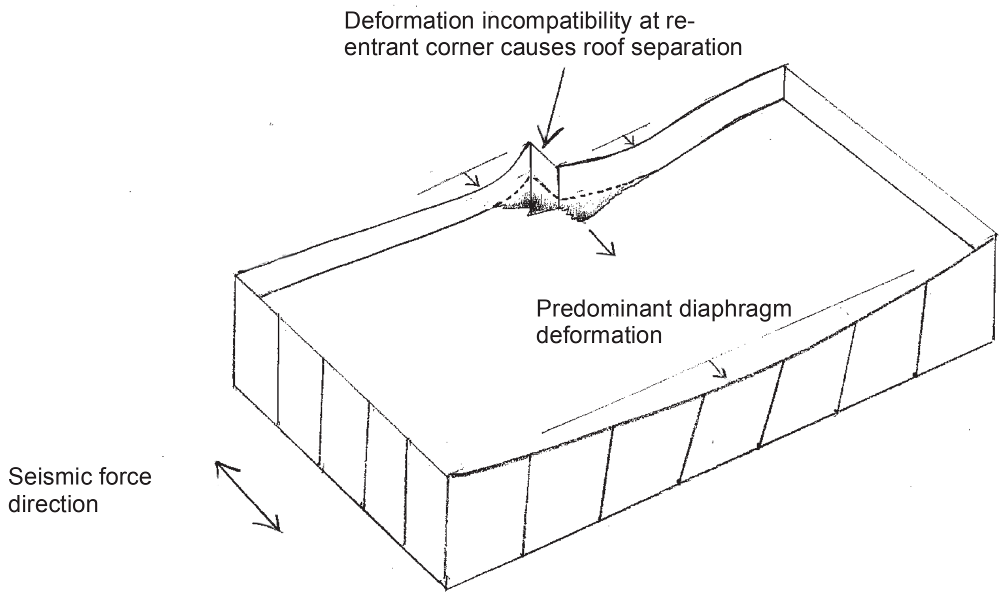


Figure 3. Effect of Shear Wall at Re-entrant Corner on Diaphragm Deformation

Depending upon the configuration, the incompatible re-entrant elements can be isolated from the main diaphragm movement with, for example, slip connections, however this requires the isolated portions of the building to remain independently stable.

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 incompatible with diaphragm drifts.

Where the re-entrant corner or stiff element length 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 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 fully accounted for. Supplemental vertical supports for major members framing into such re-entrant corners should be considered. The designer must also ensure that the wall base can resist the in-plane shear corresponding to the rocking force.

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 318-14 Section 18.14 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 force into the diaphragm necessary to achieve the pushover capacity of the piers.

In ASCE 7-16, there are no provisions that indicate the amount of plan offset or re-entrant depth that these re-entrant

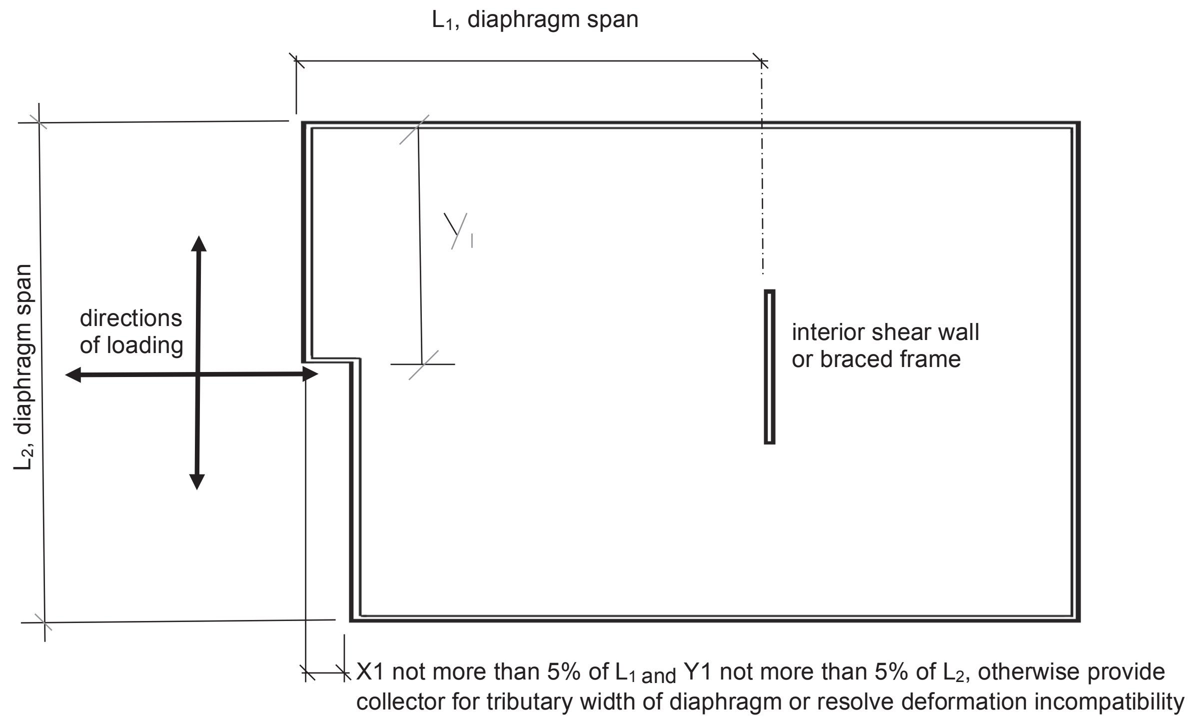


Figure 4. Plan of Typical Tilt-up Building
Illustrating Guidelines for Re-entrant Corner Considerations in Flexible Diaphragms

corners can reach before requiring investigation. ASCE 7-16 Table 12.3-1 identifies a reentrant corner irregularity as a condition where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction, regardless of diaphragm flexibility (ASCE 7-16 Table 12.3-1). Because of the more critical nature of re-entrant corners in flexible diaphragms, it can be 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 force, as shown in Figure 4.Engineers are encouraged to design irregular diaphragms based on simple statics, as described in recent publications (Malone R.T 2014) and (Malone, R.T, Rice, R.W 2012)

Multiple-Story Diaphragm Compatibility. Originally, tilt-up buildings were one-story warehouses with occasional mezzanines. 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 concrete floor systems over metal decks, while the roof system remains a panelized wood roof or metal deck roof.

With concrete panels extending full height 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 flexible roof and a more rigid 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 318-14 Section 11.8.1.1(d)) and the wall panel is sufficiently flexible out-of-plane. A worse scenario occurs 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 (See Figure 5). In conditions where diaphragm drift significantly varies from floor-to-floor or floor-to-roof, the designer should investigate the wall out-of-plane bending and anchorage capacity for this additional effect by analyzing deformation incompatibility or designing the floor or low roof to be structurally independent of the concrete wall supporting the high roof.

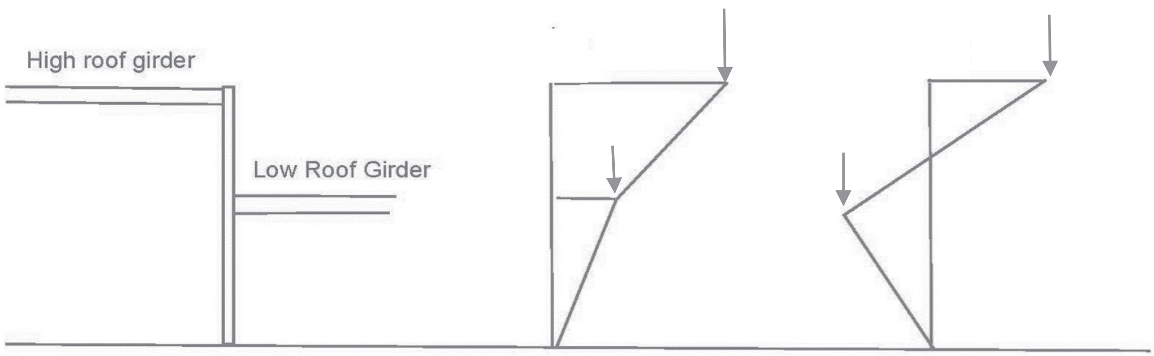


Figure 5. Section at Walls in a Tilt-up Building with a Low Roof that can respond out of Phase with the High Roof. Walls restrained at both roofs can experience large out of plane bending stresses and high anchorage forces. Similar conditions can also occur at tall walls shared by interior mezzanines.

Are Tilt-up Buildings Bearing Walls or Building Frame Systems?

In ASCE 7-16 Table 12.2-1, "Design Coefficients and Factors for Seismic Force-Resisting Systems," there are now three categories under Bearing Wall Systems and for Building Frame Systems that potentially apply to tilt-up buildings. 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. ASCE 7-16 places a height limit of 40 ft on the Intermediate Precast system in Seismic Design Categories D, E, and F (this height limit can be increased to 45 ft for single story warehouses facilities). Ordinary Reinforced Concrete Shear Walls are not permitted in Seismic Design Categories D, E, and F per ASCE 7-16.

ACI 318-14 Section 18.2 provides guidance for designing three categories of walls applicable to tilt up construction:

- Intermediate precast walls that satisfy Section 18.5
- Special Structural Walls that satisfy Sections 18.2.3 through 18.2.8 and 18.10
- Special Structural walls constructed using precast concrete that satisfy Sections 18.2.3 through 18.2.8 and 18.11.

Most tilt-up buildings are classified as bearing wall systems unless they are configured to have independent vertical load carrying systems that qualify as Building Frame Systems.

Precast concrete seismic design provisions are based on 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 ACI 318 seismic provisions for precast concrete systems.

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 318-14 Chapter 18 encourages ductile detailing, including confinement reinforcement and development of tensile reinforcement in high-seismic regions. SEAOC believes 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 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 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. In the past a series of individual panels modeled together with multiple opening configurations and numerous panel joints made modeling too complex for the average engineer without a powerful computer.

Today, engineers have enough computing power to better distribute shear forces along complicated shear wall lines in proportion to individual panel rigidities as described by ASCE 7-16 Section 12.8.4, considering both shear and flexural stiffnesses. Accurate distributions of in-plane shear forces along designated shear wall lines have become quite complex as buildings use panel configurations with numerous openings. Recognizing that stiff shear wall elements could become overloaded, crack, and redistribute their forces to other more flexible wall elements, wall piers are detailed to ensure flexural yield failure mechanisms.

Another approach is to designate solid panels along a wall line as the primary shear walls for the total wall line. This simplifies the distribution by ignoring the more flexible panels and transferring the entire seismic force to a few stiffer and stronger panels through a collector. By ignoring portions of the concrete wall line, these ignored elements are subject to the provisions of ACI 318-14 Section 18.14 for concrete members not designated as part of the seismic force-resisting system. The special detailing of these ignored members will provide more ductile behavior in the event that solid walls crack or rock under forces in excess of design levels and redistribute 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 have enough flexibility to isolate 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 resist chord forces without excessive diaphragm deflections, and analysis complications occur when collectors at re-entrant corners apply large seismic drag forces to the end panels of wall lines.

ASCE 7-16 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 ASCE 7 Commentary (Section C12.7.3) states, "For reinforced concrete buildings, it is important to address the effects of axial, flexural, and shear cracking in modelling the appropriate effective stiffness of the structural elements. Determining appropriate effective stiffness of the structural elements should take into consideration the anticipated demands on the elements, their geometry, and the complexity of the model." Section C12.8.4 states, "…the distribution of forces to the various structural elements depends on the strength of the yielding elements and their sequence of yielding (See C12.1.1). Such effects cannot be captured accurately by a linear elastic static analysis and a nonlinear dynamic analysis is too cumbersome to be applied to the design of most buildings. As such, approximate methods are used to account for uncertainties in horizontal distribution in an elastic static analysis."

Wall Pier and Shear Wall Classifications

Traditionally, tilt-up buildings contained many solid wall panels that easily fit the definition of special reinforced concrete shear walls. However, as greater architectural demands on tilt-up buildings pushed doors and window closer together and closer to panel joints, many 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.

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. ACI 318-14, Section 18.10, requires different wall detailing provisions depending on the demands and aspect ratios of the wall and individual wall piers.

Research done on thin tilt-up frame panels (similar to wall piers) has shown the benefits of close tie spacing in hinge zones 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).

Wall Connections to the Foundation

In the past, many tilt-up buildings have been constructed without any attachment from the concrete panels to the foundation. 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 the slab-on-grade that lap within a narrow concrete pour-strip in the floor parallel to the wall.

ACI 318-14 Section 16.3.6 allows precast concrete wall panels to forego the traditional footing connection requirements that cast-in-place walls have in Section 16.3.5 if load combinations result in no tension at the base of the walls, and vertical integrity ties are developed into the slab on ground. A rational load path is still required to transfer the in-plane and out-of-plane forces through the slab-on-grade and to the supporting soil consistent with Section 18.13. Slabs on ground that resist earthquake forces from walls shall be designed as diaphragms in accordance with 18.12 and construction documents shall clearly indicate that such slabs are part of the seismic force-resisting system in accordance with Section 18.13.3.4.

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, the SEAOC Seismology Committee issued a "Position Statement" strongly recommending that designs in 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 still quite common to see tilt-up panels with no direct foundation connections, 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, fully developing reinforcement required by Section 18.10.8 into the foundation.

New Thinking

Buildings with stiff shear wall systems supporting flexible diaphragms have several unique properties that make them behave substantially different from typical building types where the ductility occurs mostly in the vertical members of the seismic force-resisting system. Squat 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 have 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 7-16, as well as the 2016 CBC, estimates building seismic response as a direct function of building height and the seismic force-resisting system. In addition, current codes determine diaphragm and wall anchorage forces as a direct function of the Seismic Parameter Sds only, or sometimes in consideration of the stiffness of the vertical elements of the seismic force-resisting system, without any consideration of the dynamic response of the diaphragm. This inaccuracy in current codes has led some to recommend entirely new standards 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 7-16 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 practice.

Further research is justified for rigid shear wall / flexible diaphragm buildings, including tilt-up construction. A new approach to provisions governing this building type would allow a more rational consideration of building response, including appropriate provisions for redundancy coefficients, collector design, deformation compatibility, and diaphragm deflection.

Findings from FEMA P1026 (2015) indicate that the overstrength of the walls, combined with the required higher wall anchorage and collector force levels, could potentially make diaphragm yielding, foundation rocking or sliding, and global response more critical for RWFD (Rigid Walls Flexible Diaphragm) buildings in future earthquakes." "The simplistic model assumed by the Equivalent Lateral Force (ELF) procedure fails to capture the actual behavior of RWFD buildings." "Because RWFD buildings typically have excessive strength in the shear walls as compared with the diaphragm, it is unrealistic to expect the yielding to be in the walls instead of the diaphragm; despite the fact that the response modification coefficient R is selected based on that assumption." "Assuming that failure of out-of-plane wall anchorage is prevented, diaphragm yielding and potential failure may become the critical behavior. The ELF procedure of ASCE 7-16 as currently applied to RWFD buildings does not account for this diaphragm behavior."

FEMA P-1026 provides an alternate method for the design of diaphragms in tilt-up buildings to ensure more controlled yielding away from wall lines. The Building Seismic Safety Council's Issue Team 9 is developing a change proposal that, if approved will be provided to ASCE 7 Committee for its consideration in 2019 for the next cycle.

References

ACI-SEAOSC (1982). *Report of the task force on slender walls*. American Concrete Institute (Southern California Chapter) and the Structural Engineers Association of Southern California, Los Angeles, CA.

AISI (2001). *North American specification for design of cold-formed steel structural members*. American Iron and Steel Institute, Washington, DC.

APA (1994). *Northridge, California Earthquake*, Report T94-5. APA - The Engineered Wood Association (formerly the American Plywood Association), Tacoma, WA.

ASCE (2006). *Minimum design loads for buildings and other structures*. ASCE/SEI 7-10. American Society of Civil Engineers, Reston, VA.

Bischoff, P. and Scanlon, A. (2007). "Effective moment of inertia for calculating deflections of concrete members containing steel reinforcement and fiber-reinforced polymer reinforcement." *ACI Structural Journal*, 104 (1), January-February, 2007.

Bouwkamp, J., Hamburger, R.O., and Gillengerten, J. (1991). *Degradation of plywood roof diaphragms under multiple earthquake loading*. CSMIP90, California Department of Conservation, Division of Mines and Geology (now California Geological Survey), Sacramento, CA.

Brooks, H. (2000). *The tilt-up design and construction manual*, 5th ed. Tilt-up Concrete Association, Mt. Vernon, IA.

BSSC (2004a). "Chapter 9, concrete structure requirements and commentary," *NEHRP recommended provisions for seismic regulations for new buildings and other structures*, FEMA 450, 2003 ed., FEMA 450-1 and 450-2. Building Seismic Safety Council, Washington, DC.

BSSC (2004b). *NEHRP recommended provisions for seismic regulations for new buildings and other structures*, FEMA 450, 2003 ed., Part 2, Commentary, Building Seismic Safety Council, Washington, DC.

Çelebi, M. et al. (1989). "Seismic response of large-span roof diaphragms." *Earthquake Spectra*. 5 (2). Earthquake Engineering Research Institute, Oakland, CA.

CSSC, 1995. *Northridge Earthquake, turning loss to gain*, SSC Report No. 95-01. California Seismic Safety Commission, Sacramento, CA.

Dew, M., Sexsmith, R., and Weiler, G. (2001). "Effect of hinge zone tie spacing on ductility of concrete tilt-up frame panels." *ACI Structural Journal* 98 (6), November-December, 2001.

Eddington, Ronald (1990). "The flexible link." *From experience*. Structural Engineers Association of Central California, Sacramento, CA.

EERI (1985). "The Morgan Hill Earthquake of April 24, 1984," *Earthquake Spectra*. (1) 3. Earthquake Engineering Research Institute, Oakland, CA.

EERI (1988). "The Whittier Narrows Earthquake of October 1, 1987." *Earthquake Spectra* 4 (2). Earthquake Engineering Research Institute, Oakland, CA.

EERI (1990). "Loma Prieta Earthquake reconnaissance Report." *Earthquake Spectra*. Supplement to Volume 6. Earthquake Engineering Research Institute, Oakland, CA.

EERI (1996). "Northridge Earthquake reconnaissance report, vol.2." *Earthquake Spectra*. Supplement C to vol. 11. Earthquake Engineering Research Institute, Oakland, CA.

EQE (1989). *The October 17, 1989 Loma Prieta Earthquake*. EQE Engineering, San Francisco, CA.

EQE (1987). *Summary of the October 1, 1987 Whittier, California Earthquake*. EQE Engineering, San Francisco, CA.

FEMA P-1026(2015) Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure

Fonseca, F.S., Wood, S.L., and Hawkins, N.M. (1996). “Measured response of roof diaphragms and wall panels in tilt-up systems subjected to cyclic loading.” *Earthquake Spectra*. 12 (4). Earthquake Engineering Research Institute, Oakland, CA.

Fonseca, F.S., Hawkins, N.M., and Wood, S.L. (1999). “Calculated response of tilt-up structures.” *Proceedings of the 8th Canadian Conference on Earthquake Engineering*. Vancouver, British Columbia, June, 1999.

Freeman, S.A., Searer, G.R., and Gilmartin, U.M. (2002). “Proposed seismic design provisions for rigid shear wall / flexible diaphragm structures." *Proceedings of the Seventh U.S. National Conference on Earthquake Engineering* (7NCEE). Earthquake Engineering Research Institute, Oakland, CA.

Gilbert, R.I (2001). “Deflection calculation and control – Australian code amendments and improvements.” *Code provisions for deflection controls in concrete structures*, SP 203-4, American Concrete Institute, Farmington Hills, MI.

Ghosh, S.K., and Hawkins, N. M. (2006). “Codification of precast seismic structural systems: An update,” *PCI Journal* 51 (3), May-June, p. 2-5.

Hamburger, R.O. and Nelson, R.F. (1999). “Hold-down eccentricity and the capacity of the vertical wood member,” *Building Standards*, November-December, 1999. International Conference of Building Officials, Whittier, CA.

Harris, S. K. et al. (1998). “Response of tilt-up buildings to seismic demands: Case studies from the 1994 Northridge Earthquake.” *Proceedings of the NEHRP Conference and Workshop on Research on the Northridge, California Earthquake of January 17, 1994*. California (now Consortium of) Universities for Research in Earthquake Engineering (CUREE), Richmond, CA.

Hawkins, N. M. and Ghosh, S. K. (2004). “Acceptance criteria for special concrete structural walls based on validation testing.” *PCI Journal*, 49 (5), September – October, p. 78-92.

Hofheins, C.L., Reaveley, L.D., and Pantelides, C.P. (2002). “Behavior of welded plate connections in precast concrete panels under simulated seismic loads." *PCI Journal*, July-August 2002. Precast/Prestressed Concrete Institute, Chicago, IL.

ICBO (1967, 1973, 1976, 1985, 1988, 1991, 1994, 1997). *Uniform building code*. International Conference of Building Officials, Whittier, CA.

ICBO (2002). *Acceptance criteria for joist hangers and similar devices*, AC13. January 2002. ICBO Evaluation Service, Inc., Whittier, CA.

ICC (2000). *International building code*. International Code Council, Falls Church, VA.

ICC (2003). *International building code*. International Code Council, Falls Church, VA.

ICC (2006). *International building code*. International Code Council, Falls Church, VA.

ICC (2016). *International Existing Building Code, Appendix Chapter A2, “Earthquake Hazard Reduction in Existing Reinforced Concrete and Reinforced Masonry Wall Buildings with Flexible Diaphragms”*. International Code Council, Falls Church, VA.

Kripanarayanan, K. M. (1974). "Tilt-up load-bearing walls: A design aid." *PCA Engineering Bulletin*, Portland Cement Association, Skokie, IL.

LA City (2002). "Chapter 91 – Earthquake hazard reduction in existing tilt-up concrete wall buildings," *2002 City of Los Angeles building code*. International Conference of Building Officials, Whittier, CA.

LA County (2002). "Chapter 95 - Earthquake hazard reduction for existing concrete tilt-up buildings," *2002 County of Los Angeles building code*. International Conference of Building Officials, Whittier, CA.

LADBS (2002). "Wall anchorage assemblies to flexible diaphragms – Revised design criteria," Information Bulletin P/BC 2001-71 November 1, 2002. Los Angeles Department of Building and Safety, Los Angeles, CA.

Lawson, John (2007a). "A call for unified design – Deflection limits for tilt-up wall serviceability." *Structural Engineer*. January 2007, Zweig White Information Services, Chicago, IL.

Lawson, John (2007b). "Deflection limits for tilt-up wall serviceability – The history behind changes to Chapter 14 of ACI 318." *Concrete International,* 29 (9). American Concrete Institute, September 2007.

Lawson, John (2007c). "Thinking outside the box: New approaches to very large flexible diaphragms." *SEAOC 2007 Convention Proceedings,* Structural Engineers Association of California, Sacramento, CA.

Malone,R. Terry, Rice, R.W.(2012). The Analysis of Irregular Shaped Structures: Diaphragms and Shear Walls. McGraw-Hill and International Code Council publishers

Malone, R. Terry (2014). The Analysis of Irregular Shaped Diaphragms Wood Works, Wood Products Council.

Massarelli Robert (2010) Dynamic Behaviour and Inelastic Performance of Steel Roof Deck Diaphragms. McGill University, Montreal, Canada

Filiatrault, A., Kelly, D., Koliu, M., Lawson, J (2018). "The evaluation of Current Wall-to-Roof Anchorage Force Provisions for Single- Story Concrete and Masonry Buildings with Lightweight Flexible Diaphragms." *SEAOC 2018 Convention Proceedings,* Structural Engineers Association of California, Sacramento, CA.

Lemieux, K., Sexsmith, R., Weiler, G. (1998). "Behavior of embedded steel connectors in concrete tilt-up panels." *ACI Structural Journal*, July-August, 1998.

Mayo, John L. (2001). "Metal roof construction on large warehouses or distribution centers." *Steel Tips*. June 2001, Structural Steel Education Council, Moraga, CA.

NOAA (1971). *San Fernando, California, Earthquake of February 9, 1971,* vol. 1. National Oceanic and Atmospheric Administration, Washington DC.

Scanlon, A., and Murray, D. (1982). "Practical calculation of two-way slab deflections." *Concrete International*, 4 (11). American Concrete Institute, 1982.

Scanlon, A. and Bischoff, P. (2007). "Shrinkage restraint and loading history effects on deflection of flexural members." Submitted to *ACI Structural Journal*, February 2007.

SEAOC (1991). *Reflections on the Loma Prieta Earthquake October 17, 1989*. Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). *Recommended lateral force requirements and commentary – 7th ed.* Structural Engineers Association of California, Sacramento, CA

SEAOC Seismology Committee (2008) Blue Book, Tilt-up Buildings article 9.02.010, Structural Engineers Association of California, Sacramento, CA.

SEAOC (2000). *Position statement: Tilt-up wall connections to footings,* Structural Engineers Association of California, Seismology and ad-hoc Tilt-Up Committees, Sacramento, CA.

SEAONC (2001). *Guidelines for seismic evaluation and rehabilitation of tilt-up buildings and other rigid wall/flexible diaphragm structures*, Structural Engineers Association of Northern California and International Conference of Building Officials.

SEAOSC (1979). *Recommended tilt-up wall design*. Structural Engineers Association of Southern California, Los Angeles, CA.

SEAOSC (2006). *UBC 97 and ACI 318-02 code comparison summary report.* January 2006. SEAOSC Slender Wall Task Group, Structural Engineers Association of Southern California, Los Angeles, CA
.
SEAOSC (2007) "Diaphragm Design", SEAOSC Macroseminar 2007 Proceedings, Acero. G. and O'Dell, K.
Sheedy, P. and Sheedy, C. (1992). "Concrete and Masonry Wall Anchorage." *Building Standards*, July-August, 1992. International Conference of Building Officials, Whittier, CA.

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 2019)

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

SEAOC Seismology Committee (2019). "Tilt-up buildings," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC (2018 IBC) reference section(s)	ACI 318-14 reference section(s)	
12.3.1.3 12.10.1 12.10.2 12.10.3 14.2.4		18.10.6.4 18.12.6 18.12.7 18.14.2 18.14.3 18.14.4	21.2.4.2 25.5

Background

In the 1994 Northridge Earthquake, 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 25 years prior to the Northridge earthquake. They ranged in height between one and eight stories, and were constructed of cast-in-place concrete, precast concrete, or a combination of the two systems. The seismic 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 code's life safety objective. Observations from Northridge revealed that collapse of the gravity systems sometimes occurred while perimeter walls and frames were undamaged. Other observations included failures of diaphragm collectors and chords, large diaphragm deflections, and distress at precast connections due to lateral movements (Holmes 1996). 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 Northridge Earthquake, the following changes were implemented for concrete structures in regions of high seismicity:

Table 1. Changes to standards since 1994 intended to affect design of concrete parking structures

Structural Element	Intent of Code Change	ASCE 7-16	ACI 318-14
Diaphragm and Collectors	Specified the minimum thickness of topping slabs. Limited the spacing and bar size at lap splices for force transfer, Connections of diaphragms to vertical elements, chords and collectors	ASCE 12.10	ACI 18.12.6 ACI 25.5
Collector Design Forces	Increased collector design forces	ASCE 12.10.2 ASCE 12.10.3	
Prestress Tendons resisting collector forces, diaphragm shear, or flexural tension	Limited the stress in bonded tendons, and permitted pre-compression only from unbonded tendons in diaphragms.		ACI 18.12.7.2
Strength Factor, ϕ	Reduced ϕ from 0.85 to 0.60 for diaphragm shear in buildings with walls		ACI 21.2.4.2

Transverse Reinforcement of Frame Members	Prescriptive requirements for transverse reinforcement for frame members not proportioned to resist seismic forces.		ACI 18.14.2 ACI 18.14.3
Beam to Column Connections	Prescriptive requirements for precast concrete gravity frames for improved beam to column connections for frame members not proportioned to resist seismic forces		ACI 18.14.4
Transverse Reinforcement of Special Boundary Elements	Prescriptive requirements for transverse reinforcement for confinement of wall boundary elements		ACI 18.10.6.4(f)
Precast Concrete Diaphragms	Design and detailing requirements to achieve ductility and overstrength for expected diaphragm performance	ASCE 14.2.4	

Parking Structure Characteristics

Parking structures have several characteristics that make them more susceptible to seismic damage, as summarized below:

Typical parking structures 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 that are likely to be governed by shear rather than bending.

Parking structures are usually very large in plan, with relatively thin post-tensioned or precast concrete diaphragms compared to typical office buildings. Architectural, traffic, security, and economic demands push for long spans and large open areas. Prestressed concrete is a more suitable and economical system, due to long spans and smaller member sizes. The long-span floor systems tend to vibrate, but the resulting vibrations are acceptable for 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 results in fewer shearwalls and less redundancy.

Parking structures usually have few interior non-structural elements, such as partitions, ceilings, or mechanical systems. This inherently leads to lower damping than typical office buildings. Damping ratios ranging from 3% to 4% were recorded in an instrumented parking structure during the Northridge earthquake (Hilmy et al).

Ramps

Ramps can significantly affect the seismic behavior of parking structures by altering the distribution of seismic forces. In some cases, ramps can attract a significant percentage of the forces (Lyons, et al. 2003). The consequences of this varies between different parking structures depending on the selected seismic resisting system, the plan configuration, the structure height, and other factors.

Current building codes and standards do not provide specific guidelines suitable for analyzing the complex interactions that can occur in parking structures, nor do they include detailing provisions for ramps. Assuming discrete story levels may be too simplified an approach and could cause designers to overlook unintended structural deficiencies. The level of analytical sophistication needed to identify and address these concerns varies based on the complexity of the structural configuration.

Shortcomings of Current Code and Standards Provisions for Ramps. There are several reasons why the 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. However, ramps can be stiff and massive enough to interact with the designated seismic force-resisting system. A narrow interpretation of ASCE 7 Section 12.12.5 might designate ramps as not included in the seismic force-resisting system, like gravity columns and non-frame beams. When ramps are categorized as non-seismic elements, their effects on the seismic behavior of the structure could be inadvertently overlooked. Consequently, force distributions to the shear walls and/or frames might not accurately reflect the behavior of the building. However, Section 12.7.3 requires the stiffness and strength of all elements that are significant to the distribution of forces to be included in structural models. Furthermore, Section 12.7.4 requires the design of moment resisting frames adjoined by elements that are more rigid and not considered to be part of the seismic force-resisting system to be designed so that the action or failure of those elements will not impair the frames.

The 2019 CBC also 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 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 highly flexible diaphragms with perimeter-only seismic restraint systems (Fleischman 2002).

Parking structures often have spiral or split-level configurations that are 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 deviate from the idealized distinct story levels assumed in current standards.

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, the following additional issues should be considered:

- It is common practice to release ramps at grade, but to rigidly connect them to 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.
- 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.
- Stiff ramps can cause secondary torsion effects that redistribute the story forces, potentially increasing forces to specific seismic resisting elements.
- The top level may have shear resisting elements on three sides only when ramps with expansion joints are not designed to transfer diaphragm forces to the lower level, and thus rely on cantilever diaphragm rotation to distribute seismic forces. The horizontal irregularity types described in ASCE 7 lack guidelines to limit cantilever diaphragm spans. Providing expansion joints in ramps at each level tends to be a less common approach because of the added initial cost, ongoing maintenance of joints, and aesthetic drawbacks.

The 2019 CBC and ASCE 7 do not specifically address the need to evaluate these potential deficiencies that can occur in structures with ramps or sloping floors.

Current Practice for Evaluating Ramps. Currently, the challenge for designers of parking structures is to overcome the disparity between complex ramp configurations and simplified code-prescribed design procedures. It is common in the industry to consider neglecting the ramps' interconnectivity to the story levels in order to simplify analysis. But this approach risks unintended structural shortcomings noted above. Some practitioners believe that interconnecting sloped floors provide for structural "toughness," judging that well tied together structures are inherently more robust. While it is valid to assert that connected ramps provide reserve stiffness or redundancy, it also is true that concurrent load paths that are not included in structural models 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 energy (SEAOSC/LA City Task Force). If this were to happen, the ramps might fail before the ductile frames are engaged. Although possible, this type of failure has not yet been observed in earthquakes. Dissipating energy in a slab system is not a rational choice, because the diaphragm does not possess the same ductility as moment frames. This effect is not as prevalent in shear wall structures, given their increased stiffness. 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 forces imposed on short columns.

In any event, designers should be aware that ramps have two different characteristics: orthogonal and longitudinal. In the longitudinal direction, ramps can act as truss elements transmitting axial forces. However, the predominant concern is in the orthogonal direction, where the main deformations of the diaphragms occur. In typical parking structures with long ramps, particularly where intermediate lateral bracing is absent, the lateral deformations of the diaphragms could be significant and reach up to half of the total drift. Computer modeling of such ramps should be carefully considered with appropriate effective moments of inertia on the order of 50% I_g. Additionally, ductile detailing should be provided for the diaphragms in such cases.

For all parking structures, engineers should demonstrate rational representations of the seismic force paths. In the absence of published guidelines, the best approach currently being used is project-specific computer analysis, explicitly modeling the effects of each ramp's configuration. Previous analysis tools assumed rigid diaphragms at each discrete level. Today's computational tools permit more sophisticated analyses, including semi-rigid diaphragms and complex definitions of deck levels and load paths, including sloped ones. The corresponding interpretations of the code and standards, as well as applicable detailing, are at the discretion of design engineers.

Ramps in Moment Frame Buildings. Moment frames are inherently flexible seismic force-resisting systems that permit large story drifts. Stiff ramps may limit building movements and impair the frames in potential violation of ASCE 7 Section 12.7.4.

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 more like a typical structure, and the code provisions remain valid for detailing ductile moment frames. However, this solution can be impractical because of expansion joints, 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 one way 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 frames may be stiff enough to attract significant amounts of lateral force away from moment 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 frames to the moment frames due to the reduced stiffness of the gravity frames.

Ramps in Shear Wall Buildings. Continuous ramps are not likely to have a significant detrimental effect on the design of shear walls. The ramps acting in the strut and/or wall direction typically will not compete with stiffer shear walls. In some cases, ramps could shift the balance of the structure and cause a redistribution of the story forces among walls. The following guidelines have been useful in evaluating the forces in shear wall structures:

- If a structure is going to be designed with continuous ramps, they should be included in the seismic analysis models to rationally evaluate incidental loading and the ramps' effects on the distribution of story forces.
- In the current code and standards, no special detailing requirements are provided to address seismic actions 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 seismic forces of the structure.
- If the computer analysis indicates that the net stress in a ramp will be greater than 0.1f'c, then consider increasing the lateral stiffness of the structure. A heavily loaded ramp resisting earthquake forces can undermine the role of the shear walls as primary seismic resisting elements.
- If the net in-plane stresses in the ramps are negligible (less than 0.1f'c), 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.

Columns Not Designated as Part of the Seismic Force-Resisting System

Many columns intended to carry only gravity loads failed during the Northridge earthquake, because they did not possess sufficient strength or ductility to safely accommodate seismic drifts. Typical geometric configurations of parking structures tended to create short, brittle columns at ramp conditions and at exterior columns where spandrel rails were not sufficiently separated from the columns. Excessive diaphragm deflections in structures with large plan areas also contributed significantly to the large drifts that gravity columns were required to withstand. Even gravity columns that would normally be considered unrestrained failed, which could be attributed to inadvertent foundation restraint and vertical accelerations.

Recent standards provisions (ACI 318-14 section 18.14) now include prescriptive requirements for transverse reinforcement for gravity columns in areas of high seismicity. These requirements significantly improve the expected seismic performance of columns in parking structures. 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 seismic deformations.

The shear forces in short columns may exceed their design capacity. Based on ACI section 18.14.3, 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 moment frame structures with large inter-story drifts, the addition of confinement ties may not be enough to ensure ductile behavior. Engineers should perform deformation compatibility calculations to determine the forces that can be generated in short columns consistent with ASCE 7 Section 12.12.5.

A recommended detail for columns at or near ramps is to separate them from the ramps, allowing them to rotate over the full height of each 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 forces in collectors. Many examples of failed collectors and insufficient load paths strongly suggested inadequate code requirements, as well as flawed construction practices and ineffective plan reviews.

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 can place large ductility demands on columns and other non-seismic elements when the diaphragm deflections are added to those of the seismic force-resisting elements. Diaphragms that span long distances between seismic resisting elements may not perform in a rigid manner. ASCE 7-16 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 diaphragm deflections, cracked section properties (50% Ig) should be used. Adding interior seismic resisting elements with sufficient stiffness to reduce the spans of the diaphragms is the most cost-effective way to control excessive diaphragm deflections.

A consensus on recommendations to limit span lengths and aspect ratios of diaphragms is not available at this time. Research on long span diaphragms (Fleishman 2002) indicates that diaphragms spanning more than 180-feet with aspect ratios of 1:3 may not have acceptable response in regions of high seismicity at any design strength. This study recommended a performance-based design procedure for wall structures with flexible diaphragms. This study also recommended that a constant diaphragm force pattern be used for flexible diaphragms using the top-level force for all levels.

Modifications to standards for collectors and boundary elements are based on damage observed in Northridge. This damage included fractured collector reinforcement and diaphragm bars buckling out of the slab, especially in thin topping slabs over precast systems (Holmes 1996). As noted in the 1999 Blue Book, Appendix F and Commentary C407.6, there were also partial collapses due to failures in the load paths between diaphragms and shear walls. ACI 318-14's Commentary for section 18.12.7 explains restrictions added to section 18.12.7.3 through 18.12.7.6 at splices and anchorage zones for chord and collector reinforcement.

In response to lessons learned from the Northridge earthquake, overstrength factors were introduced to the code. These factors should be applied to critical structural elements where elastic behavior is required 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 large quantities of collector reinforcement directly into the shear walls. Instead, the bars are placed in the slabs alongside the walls. Since ACI 318-14 section 18.12.7.6requires 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 should be added to complete the load paths between the collector bars and the shear walls. These orthogonal bars need to be fully anchored into the walls and extended a development length beyond the most remote collector bar. Concrete shear stresses using the overstrength factor should be checked in the slab along the length of the shear walls. 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. Even though ACI 318-14 section 18.12.6 specifies 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 4" thick topping slab.

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 reduced values. Continuing the shear walls above the top level into parapets to fully develop wall bars above the slab is another option that allows full shear friction values.

ACI 318-14 section 18.12.7.2 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 failure observed due to the loss of anchorage of unbonded tendons. Per ACI 318-14 section 18.12.7.2 only precompression from unbonded tendons should be permitted to resist diaphragm design forces.

In the past, high strength tendons without 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 approach 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 standard limits the reinforcement characteristics for frame members and wall boundary elements in ACI 318- 14 section 18.2.6, but does not have similar limitations for diaphragm elements. ACI 318-14 section 18.12.7.2 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 long-span double-tees supported by short span girders. Precast columns are generally used to support the girders. Due to inherent long-term shrinkage in prestressed units, precast beams and girders are seldom connected to the supporting corbels at each end, and rely mainly on the thin concrete topping slabs to tie the structure together. This practice has led to visible damage of corbels or similar bearing supports caused by diaphragm displacements.

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 1994), and special attention to detailing is required 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. ACI 318-14 Section 18.14.4 includes provisions for deformation compatibility of precast concrete frame members. Perimeter gravity columns should have positive connections to the diaphragms, calculations for the deformation compatibility of the gravity system are required, and bearing lengths should be 2 inches more than required for bearing strength (PCI Design Handbook). 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 1998 California Building Code (CBSC 1998). Those modifications were not carried over into the IBC-based version of the CBC. Therefore, these improvements, which were developed and put into practice after the Northridge Earthquake, are no longer requirements in the latest standards. The UBC-based requirements not adopted in the 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 drifts 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 1998 Code adopted requirements to improve the performance of these diaphragms to a level equivalent to monolithic cast- in-place construction with two options aimed at increasing redundancy:

- 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 seismic 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 1997).

Other recommended seismic design provisions for the use of thin cast-in-place topping slab diaphragms over precast concrete members are in the 2015 *NEHRP Recommended Provisions* (FEMA P-1050, 2015) section C12.10.3 as discussed by (Hawkins 2000). These provisions are now required for precast concrete diaphragms and are acceptable alternatives for cast-in-place concrete diaphragms in ASCE 7-16 under a new section 12.10.3 titled Alternative Design Provisions for Diaphragms Including Chords and Collectors.

Vertical Accelerations

Effects of vertical acceleration should also 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

CBSC (1998). California Building Code, 1998 Edition. California Building Standards Commission, Sacramento, CA.

Corley, E., et al, January 1996, "Concrete Parking Structures", Earthquake Spectra, Volume 12 Issue S1, EERI.

FEMA, (2015) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, FEMA P-1050, National Earthquake Hazard Reduction Program.

Fleishman, Robert, Kenneth Farrow and Kristin Eastman, "Seismic Performance of Perimeter Lateral System Structures with Highly Flexible Diaphragms", Earthquake Spectra, Volume 18, May 2002.

Ghosh, S. K., Nakaki, S., Krishnan, K., "Precast Structures in Regions of High Seismicity: 1997 UBC Design Provisions", PCI Journal, November-December 1997.

Hawkins, N., Ghosh, S.K., "Proposed Revisions to 1997 NEHRP Recommended Provisions for Seismic Regulations for Precast Concrete Structures, Part 3 – Diaphragms", PCI Journal, November-December 2000.

Hilmy, S., Masek, J, "Failure Mechanisms of parking structures during the Northridge earthquake," Proceedings of the first Cairo Eartqhauke Engineering Symposium on Seismic Risk Assessment, Dec. 3-5, 1994.

Hilmy, S., Werner, Stuwart, Nisar, A., and Beck, J.," Analysis of Strong Motion Records from Parking Structure During the January 17th Northridge Earthquake, " Technical Report, California Department on Conservation, Division of Mines and Geology, Strong Motion Instrumentation Program, Data Interpretation Topic B2 of CDMG 1994 (Building Response), April 1996.

Holmes, W. and Somers, P., editors. "Northridge Earthquake of January 17, 1994 Reconnaissance Report", Volume 2. Earthquake Spectra, supplement C to volume 11, January 1996.

Iverson, J. and Hawkins, N., "Performance of Precast/Prestressed Concrete Building Structures During Northridge Earthquake", PCI Journal, March-April 1994.

Lyons, S., Bligh, R., Purinton, J., Beaudoin, D., "What about the Ramp? Seismic Analysis and Design of Ramped Parking Structures", Proceedings of 72nd Annual Convention, Lake Tahoe, 2003. Structural Engineers Association of California, 2003.

NISTIR 5396. "1994 Northridge Earthquake Performance of Structures, Lifelines and Fire Protection Systems". US Department of Commerce. March, 1994.

Portland Cement Association. "The Northridge Earthquake of 1994," Engineered Concrete Structures. Volume 7, No. 1, April 1994.

Portland Cement Association. "Major Changes in Seismic Design Provisions," Engineered Concrete Structures. Volume 10, No. 1, April 1997.

Structural Engineers Association of California – LA City/SEAOSC Task Force on Parking Structures, Seismology Ad Hoc Committee on Precast Concrete, 1994.

Wood, S., Stanton, J., Hawkins, N., "New Seismic Design Provisions for Diaphragms in Precast Concrete Parking Structures", PCI Journal, January-February 2000.

How to Cite This Publication

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

> (SEAOC Seismology Committee 2019)

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

> SEAOC Seismology Committee (2019). "Concrete Parking Structures," August 2019, The SEAOC Blue Book: Seismic Design Recommendations, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	**2019 CBC reference section(s)**	**Other Standard reference section(s)**
§11.8.3 §C11.8.3	§1605 §1610 §1613 §1807.2.2 §1807.2.3 §1803A.5.12 §1807A §1807A.2.3 §1807A.2.4	ASCE 41-17 §8.6 §C8.6

Introduction

Since the 1989 Loma Prieta earthquake, code requirements for the design of retaining structures have been significantly increased. However, case history data and recent experimental work have shown that the current methods for determining seismic lateral earth pressures lead to excessively conservative designs in regions where design PGA exceeds 0.4g. Specifically, the experimental data from seismic centrifuge tests shows that the seismic earth pressure distribution for moderate size retaining structures, on the order of 6-7 m high, is triangular, increasing with depth. Moreover, there is no significant increase in seismic earth pressure between retaining walls that are laterally supported at the top and walls that are not laterally supported at the top (e.g., basement walls with relatively rigid vs. flexible diaphragms, respectively). Also, the loads on freestanding cantilever structures (retaining walls) are significantly lower than expected, due to their ability to translate and rotate. As a result, the dynamic seismic lateral earth pressure resultant forces can be applied at 1/3H, as is done for static lateral soil loads, which substantially decreases the seismic moment on the walls.

For the readers' information, refer to *Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls,* by Professor Nicholas Sitar from UC Berkeley, as published in "Geotechnical Special Publication No. 226 ASCE" (Sitar 2012).

Performance of Retaining Walls and Basement Walls in Past Earthquakes

A summary of damage to retaining walls in recent earthquakes has been presented in Sitar, Mikola and Candia (2012). Although there are reports of damage and failure of retaining walls due to earthquakes in the United States, the distress has been attributed to some form of soil or foundation failure, such as slope instability or soil liquefaction.

There have been no reports of damage to building basement walls as a result of seismic earth pressures in recent U.S. earthquakes, including the 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta,1994 Northridge and 2014 South Napa earthquakes. Similarly, while there are many failures of walls during earthquakes outside of the United States, almost all are associated with some form of soil-related failure, with many in marine or waterfront structures (Whitman, 1991; Huang, 2000; Tokida et al., 2001; Abrahamson et al, 1999). Other failures have been attributed to inadequate structural detailing or load path (Iida, Hiroto, Yoshida and Iwafuji, 1996).

Determining the Magnitude of the Seismic Earth Pressure on Walls – the Mononobe-Okabe Method and more recent research

For many years, the Mononobe-Okabe (M-O) method – or some variation of it – has been the basis for determining the seismic lateral earth pressures on walls that retain earth. The M-O method is a pseudo-static analysis based on the Coulomb wedge theory for active and passive pressure and includes additional vertical and horizontal seismic forces (Sitar et al., 2012). It uses force equilibrium at the limit state of failure for the wedge of soil behind the wall to develop the total force behind the wall. Using the assumption that the wall has yielded enough to produce static active pressures, the net seismic soil pressure is determined.

The original tests that formed the basis for the M-O method were conducted on a sand-filled box shake table with hinged doors representing the walls. The method is based on the response of the small-scale cantilever bulkhead hinged at the bottom that retained a dry, medium dense cohesionless backfill, and was excited by a one gravity (1g) sinusoidal input on a shake table that was 4 feet high, 4 feet wide, and 9 feet long. The boundary conditions associated with the "wall", the lack of moisture and cohesion in the backfill, and the limitations of the shake table to produce results that can confidently be applied to full scale walls, have resulted in questions associated with the validity of the method in current practice, as well as questions about how to properly implement it for different conditions. Some of the key concerns come from the following conditions:

1. The walls in the Mononobe and Matsuo (1929) test were hinged at the bottom of the wall, thus allowing only for rotation and not for horizontal movement.
2. The walls in the Mononobe and Matsuo test had a free edge at the top, not a fixed or a pinned edge, as is the case in the intermediate or top levels of a building basement wall.
3. The physical scaling of the test wall may not be applicable to a full-size wall.

Additionally, Ostadan and White (1998) stated that "…the M-O method is one of the most abused methods in the geotechnical practice" for the reasons listed below:

1. The walls of buildings are often of the non-yielding type. Wall movement may be limited due to the presence of floor diaphragms, and displacements to allow limit-state conditions are unlikely to develop during the design earthquake.
2. The frequency content of the design ground motion is not fully considered, since a single parameter (peak ground acceleration) may misrepresent the energy content of the motion at frequencies important for soil amplifications.
3. Appropriate soil properties are not considered, as they are for soil dynamic problems. The most important property is the shear wave velocity, followed by the material damping, Poisson's ratio, and then the density of the soil.
4. Soil nonlinearity effects are not considered.
5. Soil-structure interaction (SSI) is not considered, such as building rocking motion, amplification and variation of the motion in the soil, geometry, and embedment depth of the building.

To validate and/or improve the M-O method, researchers have turned to centrifuge testing, which can simulate correct boundary and load conditions on large prototype structures. Centrifuge testing creates a stress field in a model that simulates prototype conditions such that proper scaling will provide correct strength and stiffness in granular soils. Granular soils, in a scale model with dimensions of 1/N of the prototype and a gravitational acceleration during spinning of the centrifuge at N times the acceleration of gravity, will have the same strength, stiffness, stress and strain of the prototype (Kutter, 1995).

Centrifuge tests by Ortiz, Scott and Lee (1983), Nakamura (2006) and Al Atik and Sitar (2008) resulted in the following observations:

1. "it is difficult or impossible to achieve in a (one-g) shaking table a pressure distribution which can be related quantitatively to that of the full-scale situation." (Ortiz et al., 1983)
2. "true representation of the dynamic prototype behavior cannot be attained in a (one-g) shaking table experiment, utilizing a reduced scale model and same soil as the prototype." (Ortiz et al., 1983)
3. "…under dynamic loading, the resultant acts very near to the where the static one acted." (Ortiz et al., 1983)
4. "…the earth pressure distributions are not linear with distance down the wall although a linear earth pressure distribution seems to be a reasonable "average" for the actual." (Ortiz et al., 1983)
5. The earth pressure distribution on the model gravity retaining wall is not triangular (as assumed by M-O), and that its size and shape will change with time. (Nakamura, 2006)
6. The earth pressure distribution for an input motion that was based on actual earthquake ground shaking was different from the distribution for sinusoidal shaking. The earth pressure in the bottom part of the wall, which greatly contributes to the total earth pressure, is not as great in earthquake loading as it is for sinusoidal loading. (Nakamura, 2006)
7. Earth pressure distributions at the time of maximum moment in the gravity wall generally increase with depth. (Nakamura, 2006)
8. The maximum dynamic earth pressures increase with depth and can be reasonably approximated by a triangular distribution analogous to that used to represent static earth pressure. (Al Atik and Sitar, 2008)
9. The maximum moment in the wall and the maximum earth pressure were out of phase and did not occur at the same time. (Al Atik and Sitar, 2008)
10. Al Atik and Sitar (2008, 2010) developed relationships for the "Dynamic Increment in Earth Pressure Coefficient, ΔK_{ae}," as defined by Seed and Whitman (1970) computed from the dynamic earth pressures at the time that maximum wall moments developed based on strain gauge data. Their research illustrates that the seismic earth pressures in the M-O method are very conservative if the actual peak ground acceleration is used.

The findings from these studies suggest that the seismic earth pressures predicted by the M-O method can be very conservative, and that the location of the resultant of the static and seismic earth pressures is closer to the one-third height from the base of the wall and not in the upper portion of the wall.

Regarding cohesion in the backfill or retained earth behind the building basement wall, it is commonplace to have backfill material or retained earth that has some cohesion, and the M-O method does not account for any cohesion at all, following Coulomb's assumptions. It is recognized in the geotechnical community that cohesion in the soil can reduce the static lateral earth pressures, and that some excavations can stand vertically without support if there is sufficient cohesion in the soil. It seems logical that since soil cohesion reduces the active lateral earth pressure, it would also reduce the lateral seismic pressures. A National Cooperative Highway Research Program (NCHRP) report (Anderson, Martin, Lam and Wang, 2008) provides guidance for use of the M-O method for soils with cohesion. Anderson et al. state that most natural cohesionless soils have some fines content that often contributes to cohesion, particularly for short-term loading conditions. Similarly, cohesionless backfills (for highway structures) are rarely fully saturated, and partial saturation would provide for some apparent cohesion, even for clean sands.

Regarding the appropriate ground acceleration for use in the M-O method, Whitman (1991) recommended that except where structures were founded at a sharp interface between soil and rock, the M-O method should be used with the actual expected peak acceleration. In high seismic regions, such as California, these peak ground motions could easily exceed 0.5g. However, Kramer (1996) refers to the M-O method as a "pseudostatic procedure" and these accelerations as "pseudostatic accelerations." Arulmoli (2001) comments on the use of the M-O method and states that it has limitations, including the observation that the M-O method "blows up" for cases of large ground acceleration. This last point is well illustrated in the findings of Sitar et al. (2012).

In practice, many geotechnical engineers have been using a seismic coefficient that is less than the expected peak ground acceleration for the design of building basement walls and other walls. The reasons for the reduced value of the seismic coefficient compared to the peak ground acceleration are as follows:

1. The M-O method is a pseudo-static method of analysis, similar to many traditional slope stability methods that use a pseudo-static coefficient to represent earthquake loading.
2. Intuitively, there should be a reduction based on the use of an effective ground acceleration rather than an isolated peak ground acceleration (to take into effect the "repeatable" ground motion).
3. There should be a reduction to account for the averaging of the lateral forces on the retaining wall over the height of the wall (because of the potentially out-of-phase nature of the ground movement as shear waves propagate vertically through the backfill soil; this effect increases with increasing height of the wall and reduced stiffness of the retained soils).

For many geotechnical engineers, the justification for a reduced seismic coefficient comes from a Federal Highway Administration (FHWA) design guidance document (Kavazanjian, Matasović, Hadj-Hamou, and Sabatini, 1997), which states "…for critical structures with rigid walls that cannot accommodate any deformation and partially restrained abutments and walls restrained against lateral movements by batter piles, use of the peak ground acceleration divided by the acceleration of gravity as the seismic coefficient may be warranted." The document also states that "…however, for retaining walls wherein limited amounts of seismic deformation are acceptable…, use of a seismic coefficient between one-half to two-thirds of the peak horizontal ground acceleration divided by gravity would appear to provide a wall design that will limit deformations in the design earthquake to small values." Thus, many geotechnical engineers have been using a seismic coefficient of one-half of the horizontal peak ground acceleration.

For level soil above the groundwater, ASCE 41 §8.6 recommends using $0.4(S_{xs}/2.5)$ times the unit weight of the soil times the height of the wall for addition earth pressure caused by seismic shaking and assumes it is applied uniformly over the height of the wall.

Provisional Recommendations for Design of Retaining Walls and Basement Walls

Although there is evidence that seismic earth pressures may not actually develop as predicted by the M-O method, it is premature to recommend that seismic earth pressures be neglected in design altogether. It is, however, reasonable to recognize the findings from more recent testing, and to incorporate them where responsible engineering judgement allows.

CBC §1613 requires that structures and their components be designed to resist the effects of earthquake motions in accordance with the applicable chapters of ASCE 7. For structures in Seismic Design Categories D, E, and F, CBC §1807.2.2 requires that retaining walls supporting more than 6 feet of backfill height be designed including the additional seismic lateral earth pressure, and ASCE 7 §11.8.3 requires that the geotechnical investigation reports include "…the determination of dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions." Although not explicitly stated, it is appropriate to recognize that the intent is to include seismic lateral earth pressures in the design of basement walls only for structures in Seismic Design Categories D, E, and F, and when backfill heights against the basement wall exceed 6 feet.

Cantilevered, Freestanding Retaining Walls

The load combinations used for checking the global stability of retaining walls are different than the load combinations specified in CBC §1605. CBC §1807.2.3 states: "…Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a minimum safety factor of 1.5 in each case. The load combinations of §1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 1.0 times other nominal loads… Where earthquake loads are included, the minimum safety of factor for retaining wall sliding and overturning shall be 1.1". CBC §1807A.2.3 includes similar requirements, and CBC §1807A.2.4 includes additional stability requirements for freestanding cantilever walls supported on isolated spread footings. The strength designs for walls and foundations (e.g., concrete or masonry) are still based on the load combinations in CBC §1605.

In some instances, the safety factors for sliding and overturning may already be accounted for by the geotechnical engineer in the soil loads provided. It is recommended that any factors of safety for soil loading be listed in the geotechnical report for comparison to code required factors of safety.

As noted above, if the depth of the retaining wall is less than 6 feet, the evaluation of seismic earth pressures is not required by CBC §1807.2.2; refer to Figure 1. (Note that this exemption is not stated directly in CBC §1807A; however, it is implied through the requirements of CBC §1803A.5.12(1).)

The SEAOC Seismology Committee recommends that for retaining walls in non-saturated conditions with level ground:

- If a dynamic seismic increment of earth pressure is determined separately by the M-O method, it should be added to the static active earth pressure and not to the at-rest static earth pressure.
- With concurrence from the geotechnical engineer of record, the location of the resultant of the active and seismic earth pressures may be taken at the one-third point from the base of the wall; refer to Figure 2.

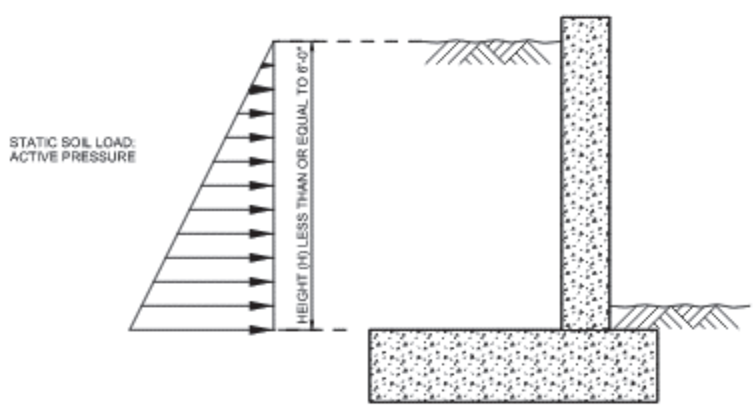


Figure 1. Retaining Walls with Backfill Height Less Than or Equal to 6'-0"

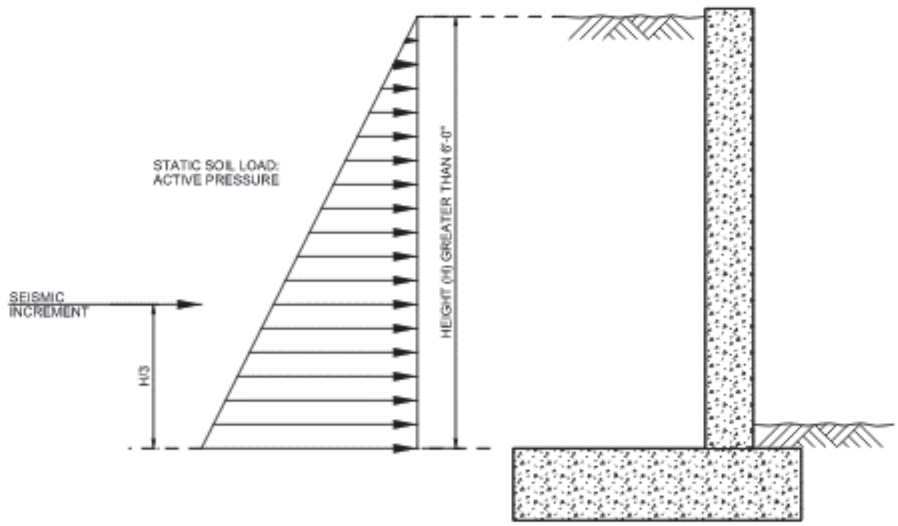


Figure 2. Retaining Walls with Backfill Height Greater Than 6'-0"

Basement Walls

The load factors and safety factors presented in CBC §1807.2.3 do not apply to basement walls; the load combinations required by §1605.2 or §1605.3 apply and include appropriate safety factors directly through the load factors. Regarding lateral soil loads, §1803.5.12 implies that seismic lateral earth pressures be included in the design of walls with more than 6 feet of backfill, and §1610 requires that basement walls be designed for at-rest earth pressures.

Another consideration applicable to structures with basements or other types of backfilled walls, such as parking garages or buried structures, is whether the seismic lateral earth pressures increase the total base shear. ASCE 41 §C8.6 indicates that seismic earth pressures should be considered when checking the capacity of the wall and its connections for out-of-plane loading, but the seismic earth pressures should not be used to increase the total base shear. On the other hand, ACI 350.3 "Seismic Design of Liquid-Containing Concrete Structures", published by the American Concrete Institute (2006), requires that seismic earth pressures be included in the base shear and overturning moments. When considering these different approaches, it is important to recognize the following:

- Structures designed using ASCE 41 are typically above grade structures that may include a below grade portion. The above grade height of the structure is generally greater than the below grade height, and the maximum base shear associated with the above grade structure acts in the opposite direction of the corresponding ground motion. Subsequently, dynamic earth pressures are not likely to add to the maximum base shear forces.
- Structures designed using ACI 350.3 that include a buried portion are typically relatively short structures, often fully buried or partially buried with differential backfill heights. The direction of the maximum base shear force is more likely to align with the direction of the dynamic earth pressure. ACI 350.3 recognizes that the frequencies driving the maximum response from the structure, its contents, and backfill soils, are not in phase; the square-root-sum-of-the-squares approach is used to combine these forces.

The SEAOC Seismology Committee recommends that for building basement walls founded in non-saturated conditions with level ground or retained earth conditions:

- If a seismic increment of earth pressure is determined separately by the M-O method, it should be added to the active earth pressure and not to the at-rest static earth pressure.
- With concurrence from the geotechnical engineer of record, the location of the resultant of the active and seismic earth pressures may be taken at the one-third point from the base of the wall; refer to Figure 4.
- The relationship between the direction of ground motion and the resulting seismic forces should be taken into consideration to determine whether seismic earth pressures should be included in the base shear and overturning moments. Special consideration for structures with differential backfill is warranted, especially when considering global stability.

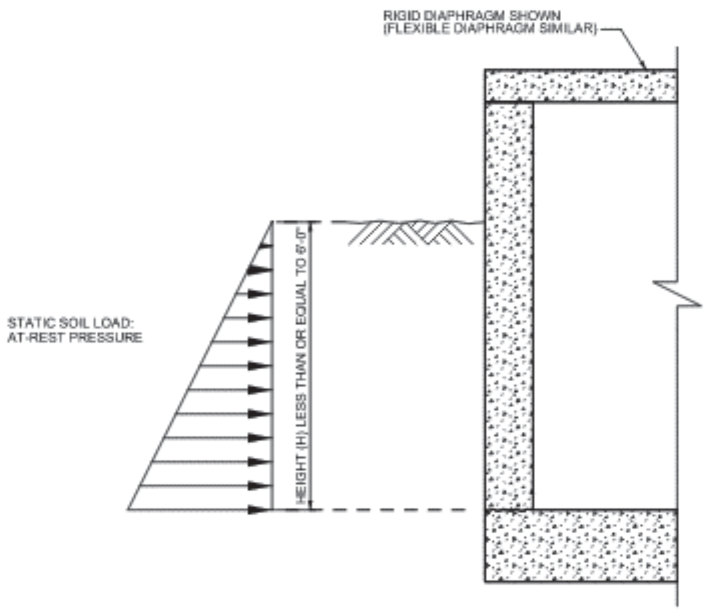


Figure 3. Basement Wall with Backfill height less than or equal to 6'-0"

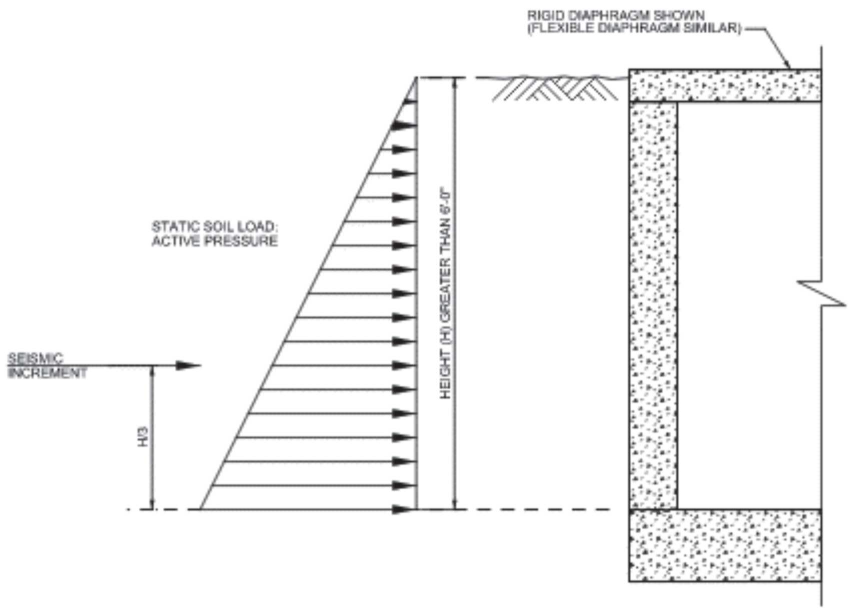


Figure 4. Basement Wall with Backfill height greater than 6'-0"

Load Factors for use with Strength Design or Load and Resistance Factor Design

The California Building Code prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. The primary load combinations associated with dead, live, soil and groundwater, and earthquake loads are:

$$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{[CBC Eq. 16-2] (1)}$$

$$1.2(D + F) + 1.0E + f_1 L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R) \qquad \text{[CBC Eq. 16-5] (2)}$$

$$0.9(D + F) + 1.0E + 1.6H \qquad \text{[CBC Eq. 16-7] (3)}$$

where

D = *dead load*
E = *earthquake load*
F = *load due to fluids with well-defined pressures and maximum heights*
f_1 = *1 for floors in public assemblies, live loads exceeding 100 psf and garage live load and,*
= *0.7 for other live loads*
f_2 = *0.7 for roof configurations that do not shed snow and,*
= *0.2 for other roof configurations*
H = *load due to lateral earth pressure, ground water pressure, or pressure of bulk materials*
L = *live load*
L_r = *roof live load*
R = *rain load*

S = *snow load*

For the static load combination (1), the at-rest lateral earth pressure is typically appropriate for use for walls laterally supported at the top, and static active lateral earth pressure is appropriate for walls that are not laterally supported at the top. See CBC Section 1807.2 and ASCE 7-16 Section 3.2. For the seismic load combinations (2) and (3), if the Mononobe-Okabe analysis is used to determine the dynamic seismic lateral earth pressure, the total lateral earth pressure should consist of the static active earth pressure and the seismic increment of earth pressure, as discussed in the previous section.

Based solely on the definitions of E and H in CBC Chapter 16, the load factor of 1.6 would be applicable to the total earth pressure. However, when considering the transitory nature of the seismic component and the low likelihood of the load maxima occurring simultaneously, a reduced load factor is justified. Accordingly, the SEAOC Seismology Committee proposes a load factor of 1.0 to be applied to the seismic increment component of earth pressure while the 1.6 load factor is applied to the static active pressure component. This is consistent with the application of the 1.0 load factor to other seismic loads, recognizing that all such loads are based on strength level forces, and it is consistent with the Commentary in ASCE 7 §C11.8.3.

Presented below are general provisional recommendations for load factors for use when considering lateral earth pressure in combination with seismic loads:

- A load factor of 1.6 should be applied to the static component of the lateral earth pressure.
- A load factor of 1.0 should be applied to the seismic component of the lateral earth pressure.

Advancing the State of Practice

At the time of this writing, the Building Seismic Safety Council (BSSC) Issue Team 7 (IT 7) has developed a proposal to include a new procedure for determining seismic earth pressures in the 2020 NEHRP Part 3 publication – Proposal IT-7-4 Rev.0-2019-02-22 *Seismic Lateral Earth Pressures*. A similar proposal is under development for retaining wall evaluations and retrofits in ASCE 41-22's Foundation Subcommittee.

The procedure estimates seismic earth pressures by considering relative displacements between the ground and the backfilled portions of the structure. It approaches the analysis as a soil-structure interaction problem, recognizing the potential for both kinematic and inertial components to be significant. Key factors used for determining the kinematic forces include the relative stiffness of the soil and wall, peak ground velocity, ground surface displacement, shear wave velocity and wavelengths. When wavelengths produce significant differential displacements in the soil along the wall height, the relatively stiff wall is forced to displace, resulting in potentially large seismic earth pressures. In addition, subgrade wall reaction forces develop when the inertial forces from the superstructure drive the foundation walls into the surrounding soil. (BSSC, 2019)

The paper developed by the BSSC IT 7 also includes validation of the method, as well as comparisons to results using the Mononobe-Okabe procedure.

The SEAOC Seismology Committee recommends the IT 7 procedure as an acceptable alternative for determining seismic earth pressures on retaining walls and basement walls.

References

Abrahamson, N., Bardet, J.P., Boulanger R., Bray, J., Chan, Y.-W., Chang, C.-H., Harder, L., Huang, A-B., Huang, S., Ju, J.W., Kramer, S., Kuo, M.-J., Lee, W.F., Lin, H-L., Loh, C-H., Ou, C-Y., Seed, R., Shen, J.-D., Sitar, N., Stewart, J., Teng, L., Sun, J.I., and Wright R. (1999). Preliminary Geotechnical Earthquake Engineering Observations of the September 21, 1999, Ji-Ji, Taiwan Earthquake, GEER Association Report No. GEER-002.

ACI (2006). ACI 350.3, Seismic Design of Liquid-Containing Concrete Structures and Commentary, 2006 edition. American Concrete Institute, Farmington Hills, MI.

Ahmadnia A., Taiebat M., Finn W.D.L., Ventura, C.E. and Devall, R.H. (2011). "Seismic Assessment of Basement Walls for Different Design Criteria," Proceedings of 2011 Pan-Am CGS Geotechnical Conference, Toronto, Ont.

Aitken, G.H. (1982). Seismic Response of Retaining Walls, MS Thesis, University of Canterbury, Christchurch, New Zealand.

Al Atik, L. and Sitar, N. (2008). Experimental and Analytical Study of the Seismic Performance of Retaining Structures. Pacific Earthquake Engineering Research Center, PEER 2008/104, Berkeley, CA.

Al Atik, L. and Sitar, N. (2010), "Seismic Earth Pressures on Cantilever Retaining Structures," Journal of Geotechnical and Geoenvironmental Engineering, October, (136) 10, pp. 1324-1333.

Anderson, D.G., Martin, G.R., Lam, I.P. and Wang, J.N. (2008). "Seismic Design and Analysis of Retaining Walls, Buried Structures, Slopes and Embankments", NCHRP Report 611. Transportation Research Board, National Cooperative Highway Research Program, Washington, D.C.

Anvar, S.A. and Ghahramani, A. (1995). "Dynamic Active Earth Pressure by Zero Extension Line", Proc. Third Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, USA.

Arulmoli, K. (2001). "Earthquake Simulation in Geotechnical Engineering: Applications and Research Needs to Improve State of Practice." Proceedings of NSF International Workshop on Earthquake Simulation in Geotechnical Engineering, Cleveland, OH.

ASCE (2017). ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA.

ASCE (2017). ASCE/SEI 41-17, Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers, Reston, VA.

Bolton M.D. and Steedman, R.S. (1982). "Centrifugal Testing of Micro-Concrete Retaining Walls Subject to Base Shaking," Proceedings of Conference on Soil dynamics and Earthquake Engineering, Southampton, 311-329, Balkema.

Bolton M.D. and Steedman, R.S. (1985). "The Behavior of Fixed Cantilever Walls Subject to Lateral Loading," Application of Centrifuge Modeling to Geotechnical Design, Craig (ed.), Balkema, Rotterdam.

Bray, J.D., Travasarou, T. and Zupan, J. (2010). "Seismic Displacement Design of Earth Retaining Structures," Earth Retention Conference, ER 2010, ASCE, Seattle, pp. 638-655.

Building Seismic Safety Council. (2004). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450), 2003 Edition, Part 1 – Provisions. BSSC, Washington, DC.

Building Seismic Safety Council. (2010). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 750), 2009 Edition, Part 1 – Provisions. BSSC, Washington, DC.

Building Seismic Safety Council. (2019). Proposal IT-7-4 Rev.0-2019-02-22 Seismic Lateral Earth Pressures. BSSC, Washington, DC.

California Department of Transportation (Caltrans). (2010). "Standard Plans: Retaining Wall Type 1 - H = 4' through 30', Plan No. B3-1", Standard Plans- U. S. Customary Units, Department of Transportation State of California.

Clough, G.W. and Fragaszy, R.F. (1977). "A Study of Earth Loadings on Floodway Retaining Structures in the 1971 San Fernando Valley Earthquake," Proceedings of the Sixth World Conference on Earthquake Engineering, Vol. 3.

Dewoolkar, M.M., Ko, H. and Pak R.Y.S. (2001). "Seismic Behavior of Cantilever Retaining Walls with Liquefiable Backfills," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, (127) 5, 424435.

Gazetas, G, Psarropoulos, PN, Anastasopoulos, I. and Gerolymos, N. (2004). "Seismic Behaviour of Flexible Retaining Systems Subjected to Short-Duration Moderately Strong Excitation," Soil Dynamics and Earthquake Engineering, (24), 537-550.

GEER 2009. Preliminary Report on the Seismological and Geotechnical Aspects of the April 6 2009 L'Aquila Earthquake in Central Italy(Ver. 2.0), Sept 2009, www.geerassociation.org

GEER 2010. Geo-Engineering Reconnaissance of the February 27, 2010 Maule, Chile Earthquake, Ver. 2.0. May 2010, www.geerassociation.org

Huang, C. (2000). "Investigations of Soil Retaining Structures Damaged During the Chi-Chi (Taiwan) Earthquake." Journal of the Chinese Institute of Engineers, (23) 4, 4 17-428.

Ichihara, M. and Matsuzawa, H. (1973). "Earth Pressure During Earthquake", Soils and Foundations, (13) 4, pp. 75-86.

Iida, H., Hiroto, T., Yoshida, N., and Iwafuji, M. (1996). "Damage to Daikai Subway Station." Soils and Foundations Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogoken-Nambu Earthquake, Japanese Geotechnical Society.

Ishibashi, I., and Fang, Y.S. (1987). "Dynamic earth pressures with different wall movement modes," Soils and Foundations, (27) 4, 11-22.

Ishii, Y. Arai, H. and Tsuchida, H. (1960). "Lateral earth pressure in an earthquake", In: Proceedings of the 2nd World Conference on Earthquake Engineering, Tokyo, Vol. 1, pp. 211-230.

Kavazanjian, E., Matasović, N., Hadj-Hamou, T., and Sabatini, P.J. (1997). Geotechnical Engineering Circular #3 – Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume 1 – Design Principles. Federal Highway Administration Report No. FHWA-SA-97-076.

Kramer, S.L., (1996). "Geotechnical Earthquake Engineering", New Jersey, Prentice Hall.

Krishna, J., Prakash, S. and Nandkumran, P. (1974). "Dynamic Earth Pressure Distribution Behind Flexible Retaining Walls," Journal, Indian Geotechnical Society, (4) 3, pp. 207-224.
\

Kutter, BL. “Recent Advances in Centrifuge Modeling of Seismic Shaking,” Proceedings, 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Vol. II, 927-941, 1995.

Lew, M., Sitar, N., and Al Atik, L. (2010a). “Seismic Earth Pressures: Fact or Fiction.” Invited Keynote Paper, Earth Retention Conference, ER 2010, ASCE, Seattle.

Lew, M., Sitar, N., Al Atik, L., Pourzanjani, M. and Hudson, M.B. (2010b). “Seismic Earth Pressures on Deep Building Basements.” Structural Engineers Association of California, Proceedings of the Annual Convention, 2010.

Matsuo, H. (1941). “Experimental Study on the Distribution of Earth Pressures Acting on a Vertical Wall during Earthquakes,” Journal of the Japanese Society of Civil Engineers, (27) 2.

Matsuo, H. and Ohara, S. (1960). “Lateral Earth Pressure and Stability of Quay Walls During Earthquakes,” Proceedings, Second World Conference on Earthquake Engineering, Vol. 1, Tokyo, Japan.

Mononobe, N. and Matsuo M. (1929). “On the Determination of Earth Pressures during Earthquakes,” Proceedings, World Engineering Congress, Vol. 9, 179- 187.

Mylonakis, G., Kloukinas, P. and Papatonopoulos, C. (2007). “An Alternative to the Mononobe-Okabe Equation for Seismic Earth Pressures”, Soil Dyn. and Earthquake Eng., (27) 10, 957-969.

Musante, H. and Ortigosa P. (1984). “Seismic Analysis of Gravity Retaining Walls”, Proceedings, Eighth World Conference on Earthquake Engineering, San Francisco, USA.

Nakamura, S. (2006). “Reexamination of Mononobe-Okabe Theory of Gravity Retaining Walls Using Centrifuge Model Tests,” Soils and Foundations, (46) 2, 135-146.

Nandakumaran, P. and Joshi, V.H. (1973). “Static and Dynamic Active Earth Pressure behind Retaining Walls”, Bulletin of the Indian Society of Earthquake Technology, (10) 3.

Nazarian, H.N. and Hadjian, A.H. (1979). “Earthquake Induced Lateral Soil Pressures on Structures,” Journal of Geotechnical Engineering Division, ASCE, (105) GT9: 1049-1066.

Newmark, N.M. (1965). “Effects of Earthquakes on Dams and Embankments,” Fifth Rankine Lecture, Geotechnique, (15) 2, 139-160.

Okabe S. (1926). “General Theory of Earth Pressure,” Journal of the Japanese Society of Civil Engineers, Tokyo, Japan, (12) 1.

Ortiz, L.A., Scott, R.F., and Lee, J. (1983). “Dynamic Centrifuge Testing of a Cantilever Retaining Wall,” Earthquake Engineering and Structural Dynamics, (11): 251–268.

Ostadan, F. (2005). “Seismic Soil Pressure for Building Walls – An Updated Approach,” Journal of Soil Dynamics and Earthquake Engineering, (25): 785- 793.

Ostadan, F. and White, W.H. (1998). “Lateral Seismic Soil Pressure-An Updated Approach”. In Preproceedings of UJNR Workshop on Soil-Structures Interaction, U.S. Geological Survey, Menlo Park, California.

Prakash, S. and Basavanna, B.M. (1969). "Earth Pressure Distribution behind Retaining Wall during Earthquakes," Proceedings of the Fourth World Conference on Earthquake Engineering, Santiago, Chile.

Prakash, S. (1981). "Dynamic Earth Pressures," State of the Art Report - International Conference on Recent Advances on Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. III, 993-1020.

Richards, R, and Elms, D.G. (1979). "Seismic Behavior of Gravity Retaining Walls," Journal of the Geotechnical Engineering Division, ASCE, (105) GT4: 449–64.

Richards, R., Huang, C. and Fishman, K.L. (1999). "Seismic Earth Pressure on Retaining Structures," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 125(9): 771–778.

Scott, R.F. (1973). "Earthquake-induced Earth Pressures on Retaining Walls," Proceedings, Fifth World Conference on Earthquake Engineering, Vol. 2, Rome, Italy.

Seed, H.B. and Whitman, R.V. (1970). "Design of Earth Retaining Structures for Dynamic Loads," ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell Univ., Ithaca, New York, 103–147.

Sherif, M.A. and Fang, Y.S. (1984). "Dynamic Earth Pressures on Walls Rotating about the Top," Soils and Foundations, Vol. 24, No. 4, 109-117.

Sherif, M.A., Ishibashi, I., and Lee, C.D. (1982). "Earth Pressure against Stiff Retaining Walls," Journal of Geotechnical Engineering, ASCE, 108, 679-695
.
Sitar, N., Mikola, R.G., and Candia, G. (2012). "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls," Geotechnical Engineering State of the Art Practice Keynote Lectures from GeoCongress 2012, ASCE Geotechnical Special Publication No. 226.

Stadler, A.T. (1996). "Dynamic Centrifuge Testing of Cantilever Retaining Walls," PhD Thesis, University of Colorado at Boulder.

Steedman, R.S. and Zeng, X. (1990). "The Seismic Response of Waterfront Retaining Walls," Design and Performance of Earth Retaining Structures, Conference Proceedings, Cornell University, Ithaca, New York, June 18-21, ASCE Geotechnical Special Publication No. 25.

Steedman, R.S. and Zeng, X. (1991). "Centrifuge Modeling of the Effects of Earthquakes on Free Cantilever Walls," Centrifuge '91, Ko (ed.), Balkema, Rotterdam.

Steedman, R. S. and X. Zeng. (1996). "Rotation of Large Gravity Retaining Walls on Rigid Foundations Under Seismic Loading," in Analysis and Design of Retaining Walls Against Earthquakes, ASCE/SEI Geotechnical Special Publication 60, edited by S. Prakash, pp. 3 8-56.

Tokida, K., Matsuo, O., and Nakamura, S.. (2001). "Damage of Earth Retaining Structures in the 1999 Chichi, Taiwan Earthquake." Proceedings of the Joint Meeting of the U.S.-Japan Cooperative Program in Natural Resources-Panel on Wind and Seismic Effects, (32) 495-512.

Veletsos, A.S. and Younan, A.H. (1997). "Dynamic Response of Cantilever Retaining Walls," Journal of Geotechnical and Geoenvironmental Engineering, (123) 2: 161-172.

Whitman, R.V. (1991). "Seismic Design of Earth Retaining Structures," Proceedings, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, ASCE, St. Louis, MO., 1767-1778.

Whitman, R.V. and Liao, S. (1984). "Seismic Design of Gravity Retaining Walls", Proc., Eight World Conference on Earthquake Engineering, San Francisco, USA.

Wood, J.H. (1973). "Earthquake Induced Soil Pressures on Structures," PhD Thesis, California Institute of Technology, Pasadena, CA.

Woodward, P.K. and Griffiths, D.V. (1992). "Dynamic Active Earth Pressure analysis," Proc. 4th Int. Symp. on Numerical Models in Geomechanics (NUMOG IV), Swansea, (eds. G.N. Pande and S. Pietruszczak), Pub. Balkema, Vol.1, pp.403-410.

Younan, A.H. and Veletsos, A.S. (2000). "Dynamic Response of Flexible Retaining Walls," Earth. Eng. Struct. Dyn., (29):1815-1844.

Zhang, J.M., Shamoto, Y. and Tokimatsu, K. (1998). "Evaluation of Earth Pressure under Lateral Deformation," Soils and Foundations, (38)1: 15-33.

Zeng, X. (1990). "Modelling the Behavior of Quay Walls in Earthquakes," PhD. Thesis, Cambridge University, Cambridge, England.

Zeng X. and Steedman, R.S. (2000). "Rotating block method for seismic displacement of gravity walls," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, (126) 8: 709–717.

Keywords
cantilever structures lateral earth
pressure retaining structures
retaining walls seismic earth
pressures
seismic increment

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 2019)

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

SEAOC Seismology Committee (2019). "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Reinforced Masonry Design Overview

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
Table 12.2-1 Table 12.12-1	1605 2106 2107 2108	TMS 402-16

Introduction

This article highlights seismic design provisions for reinforced masonry structures and presents SEAOC Seismology Committee recommendations.

Relevant Codes and Standards

Chapter 21 of the 2019 California Building Code (CBC) adopts and amends TMS 402-16 for masonry construction. The classes of reinforced masonry shear walls that can be used for resisting seismic forces are:

- Ordinary reinforced masonry TMS 402 Section 7.3.2.4
- Intermediate reinforced masonry TMS 402 Section 7.3.2.5
- Special reinforced masonry TMS 402 Section 7.3.2.6
- Intermediate prestressed masonry TMS 402 Section 7.3.2.11
- Special prestressed masonry TMS 402 Section 7.3.2.12

Prestressed and unreinforced masonry walls are only permitted as a part of the seismic force-resisting system in Seismic Design Categories A and B, in accordance with Table 12.2-1 of ASCE 7-16. Table 12.2-1 restricts the use of ordinary reinforced masonry and intermediate reinforced masonry as a part of the seismic force-resisting system to SDC C. Only special reinforced masonry walls are permitted in all Seismic Design Categories. Maximum height limits are also provided 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. TMS 402 provides prescriptive requirements for autoclaved aerated concrete (AAC) masonry shear walls in the higher seismic design categories. These also will not be discussed in this article, other than to state that it is the SEAOC Seismology Committee's 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 TMS 402 Commentary for discussion of out-of-plane loads and slender walls, as those are not directly addressed in this Blue Book article.

The seismic design coefficients in Table 12.2-1 of ASCE 7-16 imply that the detailing requirements in the TMS 402 Standard have a significant influence on expected ductility and element overstrength. The TMS 402 standards for reinforced masonry detailing specify quantity and spacing of vertical and horizontal reinforcement to classify the types of reinforced masonry. Shear walls with openings may have failure modes in piers, columns, spandrels, and joints similar to a pseudo-wall-frame. Even if designed using near-elastic seismic forces, the designer must consider shear stresses and development of reinforcement in joints between vertical and horizontal elements of the system.

The provisions for reinforced masonry shear wall systems in the TMS 402-16, ASCE 7-16, and 2018 IBC are similar, with the exception of prestressed masonry shear wall systems. Prestressed masonry shear wall systems are adopted as a single system (for both the Bearing Wall and Building Frame System Categories) in Table 12.2-1 of ASCE 7-16 with an R = 1½. Prestressed masonry shear wall systems are only allowed in Seismic Design Categories A and B. There are no special detailing requirements for prestressed masonry shear wall systems in ASCE 7-16, but it adopts seismic detailing requirements for masonry shear walls through the TMS 402-16 provisions. TMS 402-16 defines seismic design and detailing requirements for three classes of Prestressed Masonry Shear Walls: Ordinary

Plain (Unreinforced), Intermediate and Special. However, it is not clear what class of masonry shear wall detailing is required by ASCE 7-16 for the prestressed masonry shear walls listed in Table 12.2-1. For example, Prestressed Masonry Shear Walls designed and detailed to meet TMS 402-16 requirements for either Intermediate or Special walls are required by ASCE 7-16 to use the same values for R, Ωo, and Cd and the same Structural Design Limitations as a wall that is designed and detailed to meet the TMS 401-16 requirements for Ordinary Plain walls. The 2019 CBC also adopts the seismic detailing requirements for masonry shear walls through the TMS 402-16 provisions for the prestressed masonry wall systems in Section 2106.1. Unfortunately, it is not clear what design coefficients (R, Ωo, Cd) and system limitations apply to the prestressed masonry shear wall systems in the 2019 CBC. To provide additional guidance, the Commentary to TMS 402-16 recommends design coefficients (R, Ωo, Cd) and system limitations for prestressed masonry wall systems that are similar to, but slightly more conservative than, those for conventionally reinforced masonry systems in Table 12.2-1 of ASCE 7-16 with seismic detailing. However, to use these recommendations, the requirements in TMS 402-16 Section 1.3 on Approval of Special Systems of Design and Construction would have to be met.

Selection of Design Procedure

The 2019 CBC permits the use of Allowable Stress Design (ASD) procedures and Strength Design procedures. The load combinations for Strength Design are given in IBC Section 1605.2, while the load combinations in Section 1605.3 must be used while designing with ASD procedures. The TMS 402 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 (TMS 402 Section 10.4.3).

It is recommended that designers use strength design procedures for seismic design 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 more closely predict experimental results than ASD.

Partially Grouted vs Fully Grouted Reinforced Masonry Shear Walls

Ordinary reinforced masonry and intermediate reinforced masonry shear walls have traditionally been partially grouted masonry. TMS 402 Section 7.3.2.6.1.1, Design Shear Strength, discourages use of partially grouted masonry in high seismic regions; 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 forces. Since it would be difficult to provide sufficient shear strength in partially grouted walls, it is recommended that walls be grouted solid.

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 of masonry walls. The design of lines of reinforced masonry seismic force resistance involves the following steps:

- Determination of relative stiffness of types of walls, such as cantilever, coupled, and shear walls with openings.
- Determination of stiffness of lines of seismic resistance composed of types of walls.
- Distribution of seismic forces to lines of resistance and each wall segment, spandrel, pier or column in each line of seismic resistance.
- Calculation of nominal strengths for each wall, pier, spandrel, and column.
- Estimation of locations of plastic hinges and the sequence of formation of plastic hinges in the system.

The TCCMaR research program found that seismic design factors related to ductility, μ and element overstrength, $\Omega_{element}$, must be related to flexural failure modes. The ductility, μ, 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 force-resisting system should be steel yielding.

All elements in a line of seismic resistance have a forced common displacement. Occurrence of individual flexural failure modes in force-resisting elements are related to their aspect ratio, applied flexural moment and axial loading. TMS 402 Section 7.4.3.2.4 requires the stiffness (rigidity) of shear walls to be at least 80% of the total stiffness of each line of seismic resistance. Note that this limit is on stiffness, not in-plane strength. The base shear assigned to a line of seismic force 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.

TMS 402 Section 7.3.1 requires that "masonry elements that are not part of the seismic-force-resisting system...shall be isolated in their own plane from the seismic-force-resisting system except as required for gravity support," and that the isolation joint accommodate the design story drift. TMS 402 Section 7.3.1 implies that load-bearing frames or columns may not be considered as part of the seismic force-resisting system. It is recommended that columns and other non-participating elements that are 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 shear failures under design story drifts determined in accordance with ASCE 7 section 12.8.6.

Reinforced masonry walls may consist of the following:

- Solid cantilever shear walls
- Shear walls with openings composed of piers, columns, and spandrels
- Coupled cantilever shear walls

Masonry cantilever shear walls are specified in footnote ([d]) of Table 12.12-1 of ASCE 7-16, which defines them as masonry shear walls constructed so that "moment transfer between shear walls (coupling) is negligible." The allowable interstory drift is increased for cantilever shear walls compared to coupled shear walls, recognizing that reliance on a single nonlinear hinge at the base of the shear wall for ductility is desirable. The Commentary Section C12.12 compares the expected drift tolerance of masonry cantilevered shear walls as "similar to a prefabricated steel structure with a metal skin."

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 system is shear walls with openings, which are composed of piers, columns, and spandrels. The definition of piers in textbooks is that the stiffness of a spandrel that frames 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 limited flexure in the vertical length of the pier. A seismic force-resisting system with uniform pier length and story height tends to develop a soft-story mechanism in the lowest story and will typically have limited ductility (Paulay and Priestley 1992, 536-537).

The third 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 strengths and stiffnesses of the spandrels can degrade and the system behavior converts to cantilever shear walls with demand moments at their bases (Paulay and Priestley 1992, 538). The second behavioral mode is when the strengths of coupling beams framing into shear walls modify the desirable single curvature of the shear walls into double curvature (restrained ends) in low-rise structures.

The TMS Standard does not have commentary on coupled shear walls. Formulas for top displacements due to coupling beam moments, M_s, can be rewritten where M_s is a percentage of M_o., where M_o is the base moment due to a unit force applied at the top of the cantilever. The SEAOC Seismology Committee recommends 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 for each wall line.

When the shear wall has coupling beams from adjacent shear walls or shear walls with openings, the rigidity can be estimated by superposition of effects of coupling beams on the top displacement of the shear wall calculated as a cantilever from its base. 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 opposing wall curvature due to a unit force at the top of the 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 amplify this behavior. The top displacements due to the coupling beams are deducted from the Δ_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 the coupling beams. The ductility demands on coupled shear walls are partially controlled by the lower limit for allowable story drift of 0.007 h_{sx}. In contrast, the allowable story drift for cantilever shear walls is 0.010 h_{sx}.

This approximate procedure cannot be as accurate as the actual moment that accounts for member rotations between wall segments coupling beams, and cantilevers. 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 force-resisting system when estimated by these procedures will have unacceptable results for conditions such as the following:

- A solid shear wall that has openings at lower-story levels and is supported at those levels by columns and/or piers.
- A shear wall with randomly sized and distributed openings. These openings cannot be logically separated into vertical or horizontal segments to ensure reliable analysis.
- The flexural strength of coupling beams of coupled shear walls substantially exceeds the recommended strength limits of 10% and 20% cumulative of the wall base moment.

The SEAOC Seismology Committee recommends that the relative rigidities of the seismic force-resisting system having one or more of the above-stated conditions 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. Careful attention to computer modeling is required to capture all the rigidities, rotations and displacements of the different components. (Drysdale, Hamid, and Baker 1999, 459).

Requirements for Ductile Behavior of Reinforced Masonry Shear Walls

The TMS 402 Standard has three Sections with special requirements to ensure ductile behavior of shear walls:

Limit the Contribution of Columns to Seismic Resistance: TMS 402 Section 7.4.3.2.4 states that along each line of lateral resistance at each story, at least 80 percent of the lateral stiffness shall be provided by walls in Seismic Design Categories C and greater. This TMS Section has an exception: where seismic forces are determined based on a seismic response modification factor, *R,* not greater than 1.5, columns are permitted to be used to provide seismic force resistance.

Avoid Shear Failures: TMS 402 Section 7.3.2.6.1.1 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 strength to required shear demand is 2.5; φ is 0.8. The factored shear associated with development of nominal flexural strength is: $\varphi V_n / M_u * d_v$. This can be modified to this form: $\varphi V_u/(V_u * (h/d_v)$, where h is the 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 demand at that story level. This 2.5 factor is also applicable to squat shear walls that have an in-plane flexural strength in excess of the required flexural strength.

Limit Flexural Reinforcement: TMS 402 Section 9.3.3.2 describes a procedure for limiting the quantity of flexural reinforcement in elements that are a part of the seismic force-resisting system. This section applies to all members in the seismic force-resisting system, including spandrels, piers, columns, and joints. These members should have flexural reinforcement quantities that do not significantly exceed that required by the ASCE 7-16 load combinations for in-plane and out-of-plane loading. The SEAOC Seismology Committee recommends that Section 9.3.3.2 be used for determining combined axial loading and maximum reinforcement quantities.

TMS 402 Section 9.3.3.2.1 limits the general application of Section 9.3.3.2 to masonry members where $M_u/V_u * d_v \geq 1$. Section 9.3.3.2.4 modifies this limitation to: where $M_u/V_u*d_v \leq 1$ and the member or wall is designed using $R \leq 1.5$, there is no upper limit to the quantity of flexural reinforcement. The SEAOC Seismology Committee recommends that designers consider the effects of axial loading and quantity of flexural reinforcement to ensure a "tension controlled design" where possible using 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.

Compliance with Section 9.3.3.2 requires data including dimensions of members and walls, axial loading, required flexural strength, extent of grouting, seismic design coefficients, and similar information needed for the seismic design process. Compliance with Section 9.3.3.2 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. Even if some 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, although only steel confined by lateral ties can resist compression.

The requirements of Section 9.3.3.2 do not apply when designing per Section 9.3.6.6. Section 9.3.6.6 should be used when special boundary elements are not provided.

References

ASCE (2016). Minimum design loads and associated criteria for buildings and other structures, ASCE/SEI 7-16. American Society of Civil Engineers, Reston, VA.

BSSC (1994). NEHRP recommended provisions for seismic regulations for new buildings and other structures, 1994 Edition. Building Seismic Safety Council, Washington, DC.

BSSC (2001). NEHRP recommended provisions for seismic regulations for new buildings and other structures, 2000 edition, part 1: Provisions (FEMA 368). Building Seismic Safety Council, Washington, DC.

BSSC (2004). NEHRP recommended provisions for seismic regulations for new buildings and other structures, Part 1: Provisions (FEMA 450-1/2003 Edition). Building Seismic Safety Council, Washington, DC.

Drysdale, R. G., Hamid, A. A. and Baker, L. R. (1999). Masonry structures behavior and design, second edition. The Masonry Society, Boulder, CO.

He, L. and Priestley, M.J.N. (1988). TCCMaR Report No. 4.4-2, Seismic behavior of flanged masonry shear walls. The Masonry Society, Boulder CO.

Hsieh, S.S, Ting, E.C., W.F. Chen (1980). NFEAP, CE-STR-81-3. School of Civil Engineering, Purdue University, February 1981.

CBC (2019). California Building Code, California Building Standards Commission, International Building Code, International Code Council, Country Club Hills, IL.

Laursen, P.T. and Ingham, J.M. (2001). "Structural testing of single-story post-tensioned concrete masonry walls." The Professional Journal of The Masonry Society, 19 (1), September 2001.

TMS 402 (2016). Building code requirements and specification for masonry structures. Masonry Society, Longmont, CO.

Paulay, T. and Priestley, M.J.N. (1992). Seismic design of reinforced concrete and masonry buildings. John Wiley and Sons, New York, NY.

Priestley, M.J.N. and Limin, H. (1990). Seismic response of T-section masonry shear walls, Proceedings of the Fifth North American Masonry Conference, University of Illinois at Urbana-Champaign, June 1990. p. 359-372.

Seible, F., Hegemier, G., Priestley, M.J.N., Kurkchubasche, A., Igrashi, A. and Weeks, J.S. (1994). The U.S.-TCCMaR 5-Story full-scale masonry research building test; Part 1, Executive summary. Report No.9.4-1. Department of Applied Mechanics and Engineering sciences. University of California, San Diego.

Valles, R. E., Reinhorn, A. M., Kunnath, S. K., Li, C., and Madan, A. (2006). IDARC 2D, Version 6.1. University at Buffalo, State University of New York, Dept of Civil, Structural and Environmental Engineering, February 2006.

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 2019)

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

SEAOC Seismology Committee (2019). "Reinforced masonry design overview," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA

ASCE 7-16 reference section(s)		2019 CBC reference section(s)	Other standard reference section(s)
ASCE 7-02 9.5.2.6.2.2 9.5.2.6.3.1	ASCE 7-05 12.14.6.2 14.5.3.2	2305.1 2305.3 2306.3 2307.1	NEHRP 2000 12.4.2; 12.4.2.9; 12.4.3; C12.4.3 NEHRP 2003 ANSI/AWC SDPWS 2015: 4.3 SEAOC Seismic Design Manual, vol. II 2015: Pp. 40-46, 69-76

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 design 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. Also, the expanded use of 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 designed to the code limit. Collectively, these construction and seismic code trends have resulted in higher densities, taller buildings, and narrower shear wall segments, all of which intensify demands on the structural system. This intensity highlights the necessity of good engineering practice with regard to woodframe shear walls and the treatment of openings.

A prudent shear wall scheme is considered essential for good seismic performance of light wood framed construction (Breyer et al 2015). Shear wall placement will compete with nonstructural or architectural requirements, particularly at the perimeter of the building. Designers are encouraged to address these issues early in the design process when there is still some flexibility in the architectural layout. The proper distribution of shear walls is considered by many practitioners to be more of an art than a science, but in practice a prudent engineer will tend to minimize long drag (collector) elements and will consider the load sharing of the elements. No amount of additional calculations or detailing of openings will make up for a poorly planned shear wall scheme.

A new feature of the 2015 SDPWS is that cantilever diaphragms are explicitly recognized in multi-story wood structures, subject to several limitations indicated in SDPWS 4.2.5.2. Cantilever diaphragms eliminate the need for perimeter shear walls in certain situations, however the limitations are significant. One limitation is the maximum cantilevered diaphragm length of 35 feet. Another limitation is that SDPWS 4.2.5.2.3 requires the maximum story drift at each edge of the structure to not exceed the ASCE 7 allowable story drift for seismic forces, rather than merely evaluating drift at the center of mass of the story under consideration. The 2015 SDPWS does not provide a rational method for calculating deflection of cantilevered diaphragms. Until this issue is addressed, the SEAOC Seismology Committee does not recommend designs that incorporate cantilevered diaphragms.

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, known as "segmented design," is described in SDPWS 4.3.5.1.

When segmented design is assumed, the structural drawings generally do not show sheathing except at designated full-height shear wall segments. Thus drawings may differ from 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) Projects may have exteriors completely sheathed in wood structural panels for backing water proofing or other architectural concerns.

Accordingly, the opportunity exists to take advantage of the coupling action created by the sheathing above and below the windows by using a different method of calculation. Accounting for the coupling of shear walls can greatly reduce tiedown forces.

Review of Code Provisions, Intent and Application

The following SDPWS code sections are related to the design of shear walls:

1) Shear walls/piers in a line (SDPWS 4.3.3.4)
2) Aspect Ratios (SDPWS 4.3.4, Table 4.3.4)
    a. Perforated Shear Wall Segments (SDPWS 4.3.4.3)
    b. Force-transfer Shear Walls (SDPWS 4.3.4.4)
3) Shear wall types (SDPWS 4.3.5)
4) Aspect ratio definitions (SDPWS Figures 4C, 4D, and 4E)

Shear Wall/Pier Requirements

Since the inception of the codes, there have been limitations regarding wood shear wall aspect ratios. In addition to adding clarifications that forces be distributed according to the stiffness of individual elements, aspect ratios became more restrictive following the 1994 Northridge Earthquake. These changes resulted from observed issues with drift control and stiffness degradation in narrow shear walls. 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 in Seismic Zone 4. Similarly, the SDPWS incorporated a variation of the original UBC aspect ratio requirement, which was relocated from the IBC. Aspect ratio adjustments are now applied to shear walls regardless of seismic design category or wind speed, as opposed to just high seismic regions in previous codes. The SDPWS (and NEHRP Provisions) provisions state that wood structural panel shear walls with aspect ratios greater than 2:1 but less than 3.5:1 be adjusted by one of the following options:

1. SDPWS 4.3.4.2: The nominal shear capacity for wood structural panels shall be multiplied by the aspect ratio factor = $1.25 – 0.125h/b_s$, and the shear distribution to individual shear walls in a shear wall line shall provide the same calculated deflection in each shear wall (SDPWS 4.3.3.4.1).
2. SDPWS 4.3.3.4.1 Exception 1: The nominal shear capacities are multiplied by $2b_s/h$, and shear distribution to individual full-height wall segments are permitted to be taken as proportional to the shear capacities of individual full height wall segments used in design.

To apply aspect ratio limitations to a typical geometry, consider a shear wall with top plate height of 9 feet subject to the 2:1 aspect ratio. In this situation, the minimum full-height shear wall segments must have a width of 4 ft 6 in. Similarly, the 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 for the seismic force-resisting system and can be ignored with respect to direct shear transfer or can have its strength capacity reduced as discussed above.

The 2015 SDPWS Commentary provides a detailed comparison of the two approaches. In short, SDPWS 4.3.4.2 addresses the shear wall strength limit state while 4.3.3.4 addresses deflection compatibility. Both criteria need to be assessed in the design, as implied by option 1 above. Taking the exception in option 2 addresses both by applying a more restrictive penalty on the design. See examples in SDPWS C4.3.3.4.1-1 and C4.3.3.4.1-2 for more information.

Practitioners have been faced with many interpretations regarding the intent of these provisions and how to best apply the code requirements. For example:

1. To determine shear wall height, *h*, the code 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 pier length of 24 in SDPWS 4.3.5.2: The importance of this provision is that it promote good performance of piers consistent with available tests.
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.
5. Regarding perforated shear walls, the definition of element length, l_w, is not clear when calculating the redundancy factor, ρ.

To summarize, 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 openings (SDPWS 4.3.4.4) Once the shear segments are qualified, the designer must determine forces and design the segments.

Shear Wall Design with Force Transfer around Openings (FTAO)

For seismic and wind forces, the IBC stipulates that openings be designed by "rational analysis." Similarly, ASCE 7-16 Sec. 12.11.2.2.1 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 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, *Co*. The narrow wall reduction factors 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 ASCE 7-16 Commentary Section C12.10.

10. Only wood structural panels designated as a part of the seismic force resisting system are considered in the calculations.

One method of providing force transfer around window openings may be found in the SEAOC Seismic Design Manual (SEAOC 2015). 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 one 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 in a wall 9 ft tall, unless the stresses above the window are ignored. On the other hand, for structures where sufficient head room exists above the opening, analytical methods are often used to find stresses in the panels around the openings to justify force transfer around the openings, sometimes with holddowns occurring at the jamb locations and sometimes without, depending upon force levels.

To overcome the portal frame limitation that occurs with the typical limited sheathing thickness above the window, other methods can be employed. 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. 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 other lines 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 wood frame 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 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 Virginia Tech 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 PSW method found in the SDPWS shows a table of adjustment factors representing the loss of capacity caused by the un-reinforced openings (SDPWS Table 4.3.3.5). Accordingly, the Shear Resistance Adjustment Factor, *Co*, 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 SDPWS also specifies the following limitations:

1. Full-height segments are required at each end of the PSW
2. Maximum nominal unit shear capacity of 1,740 plf for seismic on the full-height segments
3. No out-of-plane offsets within the PSW
4. Continuous collector elements
5. Uniform height required
6. Maximum height of 20 ft.

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 the wall as a whole. With these limitations, the PSW method can be 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 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 with more intense seismic demands, the PSW method has some limitations. For example, 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. Commonly, 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 SDPWS 4.3.3.4.1e(1). 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.

The designer should be aware of several other items regarding the PSW method:

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. Judgment dictates that a well-centered opening will 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 sensitive to the distance between the shear segments. Arguably, once the walls are separated, the distance between them should have little effect 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 SDPWS 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. Note that to develop the Co values shown in the tables of the SDPWS 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, tension and compression are not equal, as some could presume. Compression forces, especially on multi-story construction, are usually greater due to the presence of a resisting moment and differing basic load combinations. 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. Also, 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 SDPWS Table 4.3.3.5 are applied. Stacked openings are common in walls between 10-ft and 20-ft tall where the PSW method is permitted.

Future Research

Wood shear walls with openings are complicated compared to the mathematical models used in most design offices. Some researchers have developed sophisticated modeling techniques (Li et al, 2011). White and Dolan (1995), for example, refined a finite element program that calculated forces and stresses of shear wall elements. However, typical field variations such as overnailing, issues with panel placement, and the presence of random splits or misalignments can challenge the assumptions of the analytical model. Additionally, nonstructural walls can contribute greatly to a building's strength and stiffness; these are not commonly integrated into analytical models. On larger buildings with many interior walls, there can be a significant capacity that is not considered as a part of the seismic force-resisting system, although these redundancies are typically brittle and may not be reliable seismic force-resisting elements. The SEAOC Seismology Committee supports ongoing efforts to rationalize shear wall design based on empirical data. To properly evaluate building stiffness, equations must be developed to determine the displacement of shear walls with openings. It is recommended that a testing program be developed for shear walls with openings to formulate the deflection equations.

References

AWC (2018). National Design Specification for Wood Construction. American Wood Council, Leesburg, VA.

ATC (1995). Cyclic testing of narrow plywood shear walls - ATC-R-1, Applied Technology Council, Redwood City, CA.

Breyer, Donald, Fridley, Kenneth J. Jr., Pollock, David G. and Cobeen, Kelly. (2015). Design of wood structures, ASD/LRFD, 7th Edition. McGraw-Hill, New York, NY.

BSSC (2015). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions (FEMA P-1050-1/2015 Edition), Building Seismic Safety Council, Washington, DC.

BSSC (2015). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2: Commentary (FEMA P-1050-2/2015 Edition), Building Seismic Safety Council, Washington, DC.

Cobeen, K., Russell, J. and Dolan, J.D. (2003). Recommendations for earthquake resistance in the design and construction of woodframe buildings, CUREE Publication W-30, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA

Douglas, B.K. (1994). "Perforated shearwall design approach," 1994 ASAE International Winter Meeting, Paper No. 944548.

Ge, Y.Z., Gopalaratam, V.S., and Lui, H. (1991). "Effect of openings on the stiffness of wood-frame walls," Forest Products Journal, 41 (1) 65-70.

Heine, C. (1997). The effect of tie-down anchorage on long shear walls with openings, master's thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Johnson, Andrew C. (1997). Monotonic and Cyclic Performance of Long Shear Walls with Openings, master's thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.

ICBO (1997). 1997 Uniform Building Code. International Conference of Building Officials, Whittier, CA.

Li. M., F. Lam, B. Yeh and T. Skaggs. (2011). Modeling force transfer around openings in perforated wood-frame shear walls. Submitted to Journal of Structural Engineering.

Mendes, Stanley B. (1995). "Lessons learned from four earthquake damaged multi-story Type V structures," Proc. 64th Annual SEAOC Convention, Indian Wells, CA, October 19-21 1995.

Patton-Mallory, M., Wolfe, R.W., Soltis, L.A., and Gutdowski, R.M. (1985). "Light frame shear wall length and opening effects," Journal of Structural Engineering, 111 (10) 2227-2239.

Rose, J.D. and Keith, E.L. (1996). Wood structural panel shear walls with gypsum wallboard and window/door openings, Research Report 157, APA - The Engineered Wood Association, Tacoma, WA.

SEAOC (2016). Seismic design manual, volume II: Building design examples - Light frame, masonry and tilt-up (2015 IBC), Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). Recommended lateral force requirements and commentary, seventh edition, Structural Engineers Association of California, Sacramento, CA.

Sugiyama, H. and Matsumoto, T. (1993a). "A simplified method of calculating the shear strength of a plywood-sheathed shear wall with openings; I. evaluation of the racking load of shear walls without openings," Mokuzai Gakkaishi 39 (1) 75-79.

Sugiyama, H. and Matsumoto, T. (1993b). A Simplified Method of Calculating the Shear Strength of a Plywood-Sheathed Shear Wall with Openings II. Analysis of the Shear Resistance and Deformation of a Shear Wall with Openings, Mokuzai Gakkaishi, 39 (8) 924-929.

Sugiyama, H., and Matsumoto, T. (1994). "Empirical equations for the estimation of racking strength of a plywood-sheathed shear wall with openings," Summaries of Technical Papers of Architectural Institute of Japan Annual Meeting.

Thurston, S.J. (1994). Racking resistance of long sheathed timber framed walls with openings. Building Research Association of New Zealand. Study Report SR 54, Porirua City, New Zealand.

Tissel, J.R. and Rose, J.D. (1994). Wood Structural Panel Sheathing for Narrow-Width Wall Bracing. Research, Report 156, APA - The Engineered Wood Association, Tacoma, WA.

Weichiang Pang, Tom VanDorpe, Mikhail Gershfeld, and Ershad Ziaei, Analytical Evaluation of the Contribution of Perimeter Shear Walls to the Seismic Performance of Five-Story Wood Frame Structures thru Collapse (Draft), Light Frame Performance Modeling Group (LFPMG), Orange, California; August 2017.
White, Maurice and J. Daniel Dolan (1995). "Non-linear shear-wall analysis." Journal of Structural Engineering, 121 (11), 1629-1635, November 1995.

Yasamura, M. (1986). Racking resistance of wooden frame walls with various openings, International Council for Building Research Studies and Documentation, Working Commission W18 - Timber Structures, CIB - W18, IFURO S5.02, vol. II Meeting Nineteen. Firenze, Italia, 1986.

Keywords
force transfer method around openings
holddowns
tiedowns
openings in wood shear walls
perforated shear wall
shear walls
wood shear walls
woodframe construction
wood framed construction

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Openings in Wood Framed Shear Walls," August 2019, The SEAOC Blue Book: Seismic Design Recommendations, Structural Engineers Association of California, Sacramento, CA.

Light-Frame Wall Hold-downs

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
12.3.3.3 12.4.2 12.4.3 12.12 and T12.12.-1	2305.1 2305.3	SDPWS (2015) Sections 4.3.2, 4.3.6.2 AISI S400 (2016)

Background

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." Although definition of hold-downs no longer appears in ASCE 7, AWC Special Provisions for Wind and Seismic (AWC 2015) defines a Tie-Down (Hold Down) as "a device used to resist uplift of chords of shear walls." Language used in the 2016 AISI Standard for Seismic Design, S400 (AISI 2016) also defines boundary elements as shear wall or steel strap 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, more often referred to as a Tie-Down system, consisting of continuous rods with couplers and bearing plates, 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 for 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 uplift at the ends of the panel.

Engineers initially designed hold-downs as standard details using 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 for plate washers in the 1950's to the 1990's.

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 failures of manufactured hold-downs in the Northridge Earthquake, installed without plate washers and exhibiting excessive vertical deformations, created some revisions to hold-down designs and installation instructions. Manufacturers later developed hold-downs that were sandwiched between 2 wood members to alleviate the induced bending moments in the connections.

Design Loads and Capacities

The design loads for hold-downs in wood construction are based on the load combinations of ASCE 7. The design loads for hold-downs in cold-formed steel construction are based on ASCE 7 and AISI 400-16, which requires boundary members of shear walls and their uplift anchorage to resist amplified seismic loads. Therefore, hold-downs in wood construction can be designed for the seismic load effects of ASCE 7 section 12.4.2, while hold-downs in cold-formed steel construction must be designed for the load combinations including over-strength factor in accordance with ASCE 7 section 12.4.3.

2019 CBC section 2305.1 references 2015 AWC NDS (AWC 2015) as its standard for wood design and the 2015 AWC *Special Design Provisions for Wind and Seismic* (AWC 2015), or SDPWS, as its standard for wind and seismic provisions for wood design. The design of hold-downs is addressed in SDPWS Section 4.3.6.4.2, which requires anchoring devices at ends of shear walls where the dead load is not sufficient to resist uplift due to overturning moments.

Overstrength requirements. The 2018 IBC adopts AISI S400-16, which, in Section E.1.4.1.2, requires that the available strength (e.g. nominal strength modified by the phi factor) of the hold-down be greater than the required strength. The required strength shall be based on the expected strength of the shear wall but need not exceed the load effects determined by using load combinations including seismic loads with overstrength for the shear wall.

This over-strength approach is not required for the design of wood frame shear walls, but it is the SEAOC Seismology Committee position that for wood frame buildings, a similar capacity-based approach should be considered for the design of both the hold-down and its anchorage, where the element design should consider the nominal strength of the structural panel sheathing. This should encourage the more predictable yielding behavior of the wood shear panels compared to their boundary elements and hold-down anchorage. Currently only the anchorage into the concrete is required to be designed for amplified forces or the capacity of the embedded rod, per ACI 318-14 Chapter 17 (ACI 2014). 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, 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 ($\Omega = 2.0$) or LRFD design ($\phi = 0.80$). For shear panels (wood or steel sheet sheathed) on cold-formed steel studs, AISI S400 lists safety factors and phi factors as $\Omega_v = 2.5$ for ASD design and $\phi_v = 0.60$ for LRFD design. For shear panels on cold-formed steel not listed in the tables, AISI S400 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.

Discontinuous walls. ASCE 7 section 12.3.3.3 requires special load combinations with Ω_0 for elements supporting discontinuous walls when specific irregularities exist. In these cases, that the Commentary C12.3.3.3 clarifies that the special load combinations need not be applied to the design of the shear wall nailing, the boundary members, or the hold-downs but should be applied to any non-ductile anchorage of the hold-downs. In lieu of using Ω_0 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 an element supporting a discontinuous wall.

Capacity. In Seismic Design Categories D, E, and F, the SEAOC Seismology Committee recommends 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 lag screws, not through-bolts, be used to connect the hold-down to the post or other wood framing chord. If through-bolts are used for this connection, additional deflection will occur due to slip between the bolt and the bolt hole and rotation of the bolt in the hole. This should be accounted for in the design or testing of the hold-down.

Hold-down Deformation

ASCE 7 section 12.12 and Table 12.12-1 set deflection limits for all building structures based on the type of seismic force-resisting system. CBC Section 2305.3 provides a four-term deflection 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 (SDPWS 2015) has a deflection equation, though it has only three terms rather than four (it incorporates the previous second and third term into one term).

The SDPWS Commentary section 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 analysis, design steps should be taken to minimize the deformation.

Design examples using the IBC deflection equation are given in *Structural/Seismic Design Manual* (SEAOC 2015) examples 1B, 2, and 3. For specific design values of d_a, 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. The International Code Council Evaluation Service's Acceptance Criteria for Hold-downs (Tie-downs) Attached to Wood Members (AC155) (ICC-ES 2005) provides a testing standard and evaluation standard for hold-downs. This standard includes acceptable design load value determination based on several criteria, 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.

Since the 1994 Northridge Earthquake, manufacturers have incorporated features to reduce hold-down deformation. Hold-downs are now commonly connected to posts with ¼ 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 wood posts with multiple ¼ inch lag screws significantly reduces the flexing and distortion of light-gauge hold-downs. Alternatively, hold-down devices using short steel tube sections sandwiched between and through-bolted to wood posts can also reduce the flexing and distortion of the hold-downs, thereby mitigating the potential effects of an eccentric connection.

For multi-story continuous tension rod hold-down applications, shrinkage-compensating devices are now frequently used to minimize the effects of slippage or looseness of the tie-down system associated with floor joist shrinkage occurring in the depths of the members. 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. Shrinkage-compensating devices are required by several building departments (City of San Diego, 2003) when such multi-story tie-down systems are used, which is consistent with recommendations by Nelson and Patel (2003).

Prior to innovations such as pre-deflected hold-downs, a team led by Ben Schmid tested 4 ft wide by 8 ft tall panels with ½ in plywood sheathing and doubled 2x4 edge studs. They found that proprietary light-gauge hold-downs with two 3/4 in diameter bolts to the edge member accounted for approximately half of the horizontal displacement at the top of the wall.

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 or wood screws as opposed to through bolts
- if through bolts are used, provide bearing plate washers
- 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 consider requiring lumber with a moisture content less than 12% at time of hold-down installation

Where lag screws are used, they should be of sufficient length to penetrate sufficiently into the farthest piece of a built-up boundary member so that the vertical loads (tension and compression), are distributed to each stud or post in a built-up boundary member. For example, if 1/4" diameter lag screws are used, they should penetrate a minimum of 1 inch (4 diameters) as required by NDS Table 12K into the "outer" stud.

In addition to continuing the lag screws through all of the vertical chord members at the hold-down, the chord members can be stitch nailed or screwed together to carry the uplift load from all of the chord members into the screws of the hold-down body. Distribution, not duplication, of shearwall edge nailing into the multiple chord members would be required. For instance, if the edge nailing requirement is nails spaced at 3 inches on center and double studs are used as the chord members, the nailing would be at 6 inches on center to each stud. Stitch nailing, if used, should be checked for capacity under the hold-down's design loading and, if deflection of the structure is being determined, the nail slip should be considered.

For multi-story buildings, the hold-downs should be placed directly above one another from one floor to the next, where feasible. Where hold-downs are not placed directly above one another from floor to floor, concentrated reactions would be supported by beams and/or headers that would then be supported at their ends by posts and hold-downs that carry the vertical force to the foundation. The beams or headers that support the discontinuous seismic overturning vertical forces (upward and downward) as well as the posts and hold-downs at the ends of these members, and all vertical elements that support these discontinuous forces including the hold-downs at the foundation and the foundation must be designed using the seismic loads including the overstrength factor as indicated in ASCE 7-16 section 12.3.3.3.

Drift limits and Performance. ASCE 7 requirements in Table 12.12-1 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 $0.01h$ (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. Plywood has a lower effective shear stiffness than does OSB, such that deflection of a plywood wall will be marginally larger. To obtain the anticipated performance, structural notes on design documents must specify which material is to be used, based upon what was assumed in the deflection calculations.

Shear walls have exhibited splitting of foundation sill plates in 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 is more pronounced for narrow shear walls with large hold-downs. In static load tests of narrow plywood shear walls, the sill plates split when the sheathing panel corners had vertical displacements of approximately 0.2 in (ATC 2007). Excessive hold-down deformation also contributes to failures 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 and uplift data for their products.

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 poorly-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 and deflection perpendicular to grain. Since the modulus of elasticity of the sill plate perpendicular to the grain is on the order of 10 to 30 percent of the parallel-to-grain modulus, hold-down deformation d_a may be significantly affected by the crushing of the sill plate. 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 transfer the compressive load to the foundation.

The foundation or structural base should be designed to resist the hold-down anchor force and provide equivalent ductility to the shear wall, or it should be designed for increased forces proportional to the ductility ratio.

Cold-formed steel framing

For cold-formed steel walls sheathed with wood structural panels, the design loads are similar to wood framed shear walls. Shear wall values for ASD and strength design are provided in 2016 AISI S400 for seismic design. The hold-downs are typically pre-fabricated 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 fastened together above the hold-down strap to transfer the sheathing forces at the panel edge to the hold-down anchor. The hold down studs should be checked for buckling as well as tension forces. The 2016 NIST publication: "Seismic Design of Cold-Formed Steel Lateral Load Resisting Systems: A Guide for Practicing Engineers," provides additional design and research information.

Buckling of studs in UC Irvine tests (SEAOSC-UCI 2001) was 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

Published testing results of hold-downs in cold-formed steel walls sheathed with wood structural panels are nearly non-existent. The SEAOC Seismology Committee recommends testing be performed on these types of hold-downs.

Improved design and reliable performance of hold-downs in wood sheathed shear walls, both for wood framed and cold-formed steel buildings, will require a standard for sub-assembly testing that still does not yet 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.

References

AISI (2016), North American Standard for Seismic Design of Cold-formed Steel Structural Systems, S400. American Iron and Steel Institute, Washington, D.C.

APA (2000). "Wood structural panel shear wall deflection formula" (Form No. TT-053). APA, Technical Services Division, August. Available via the World Wide Web at: www.apawood.org.

APA (2001). Diaphragms and shear walls: Design/construction guide (Form No. L350G). APA, September. Available via the World Wide Web at: www.apawood.org.

ASCE (2002). ASCE 7-02, Minimum design loads for buildings and other structures, American Society of Civil Engineers, Reston, VA.

ASCE (2006). ASCE/SEI 7-05, Minimum design loads for buildings and other structures, including supplement no. 1. American Society of Civil Engineers, Reston, VA.

ATC (2007). "Seismic testing of a full-scale woodframe building," pp. 239-249, 2007 SEAOC Convention Proceeding, Structural Engineers Association of California, Sacramento, CA.

AWC 2015. National design specification for wood construction, American Forest and Paper Association, Washington, DC.

AWC 2015. National design specification for wood, special design provisions for wind and seismic, American Forest and Paper Association, Washington, DC.BSSC (2001). NEHRP Recommended provisions for seismic regulations for new buildings and other structures, 2000 Edition, part 2—Commentary (FEMA 369), Building Seismic Safety Council, Washington, DC.

BSSC (2004). NEHRP recommended provisions for seismic regulations for new buildings and other structures, Part 1: Provisions, and Part II: Commentary (FEMA 450-1/2003 Edition), Building Seismic Safety Council, Washington, DC.

CBC (2019). California Building Code. California Building Standards Commission, published by ICC, Sacramento, CA.

California Seismic Safety Commission (1994). A compendium of background reports on the Northridge Earthquake (January 17, 1994) for Executive Order W-78-94. California Seismic Safety Commission, Sacramento, CA.

City of San Diego (2003)."Technical policy B-16-1, alternative shearwall uplift anchorage devices."

CUREE (1999). Proceedings of the Workshop on Seismic Testing, Analysis and Design of Wood Frame Construction, California Universities for Research in Earthquake Engineering, Richmond, CA.

EQE International (1999). Results of cyclic (reversed load testing for shear resistance of wood framing plywood shear walls with USP lumber connector hold-downs.

ICBO ES (1999). “Eccentricity in hold-down design and anchorage into concrete and/or masonry, subject AC155-R1-0999 (BCG/AT," International Conference of Building Officials, Whittier, CA.

ICC-ES (2005) “Acceptance criteria for hold-downs (tie-downs) attached to wood members,” International Code Council, Washington, DC.

ICC-ES (2006). "Proposed acceptance criteria for continuous tie-down systems, subject AC348-1006-R2 (BG/BNH)," International Code Council, Washington, DC.

Nelson, R. F. and Hamburger, R. O. (1999). “Hold-down eccentricity and the capacity of the vertical wood member.” Building Standards, November-December, p. 27-28.

Nelson, Rawn (2001). "Hold-down connectors and wood member end-post capacity’ Structure, September 2001.

Nelson, R. F. and Patel, S. T. (2003). “Continuous tie-down systems for wood panel shear walls in multi-story structures.” Structure, March, 2003.

2016 NIST publication: “Seismic design of cold-formed steel lateral load resisting systems: A guide for practicing engineers,”

Schmid, B. (2002). “Results and ramifications of cyclic tests of typical sheathing material used on one- and two-story wood framed residences,” in Proceedings of the 71st Annual Convention, Santa Barbara, California, Structural Engineers Association of California, Sacramento.

Schmid, B., Nielsen, R., and Linderman, R. (1994). “Narrow plywood shear panels,” Earthquake Spectra, vol. 10, no. 3, August.

SEAOC (2015). Structural/seismic design manual, volume II, building design examples for light frame, tilt-up and masonry, Structural Engineers Association of California, Sacramento, CA, re-published by the International Code Council.

SEAOSC-UCI (2001). Report of a testing program of light-framed walls with wood sheathed shear panels, Department of Civil and Environmental Engineering, University of California, Irvine, December, 2001.
Shepherd, R. and Allred, B. (1995). Cyclic testing of narrow plywood shear walls. Applied Technology Council, Redwood City, CA.

Stochlia, K. (1999). "Eccentricity in hold-down design and anchorage into concrete and/or masonry, subject AC155-R1-0999 (BCG/AT," (Subject of correspondence from ICBO Evaluation Service, Inc. to "Proponents of Hold-Down Anchorages and Other Interested Parties), November 8, 1999.

Keywords

hold-down
tie-down
light-frame metal
light-frame wood
woodframe

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "Light-Frame Wall Hold-Downs," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

ASCE 7-16 reference section(s)	2019 CBC reference section(s)	Other standard reference section(s)
Ch 12 Seismic Design Requirements	1613 Earthquake Loads 1905.1.8 Anchorage	ACI 318-05 Appendix D ACI 318-14 Chapter 17 ACI 355.2 AWC NDS®-2018 Table 12E

Introduction

Wood-frame shear walls have traditionally been connected to concrete foundations with cast-in anchor bolts or post-installed anchors in accordance with applicable building codes and standards. At shear wall locations, lateral forces are assumed to load anchor bolts in pure shear parallel to the concrete edge. This article assumes the following:

- Typical cast-in place “L-bolt,” minimum 7-inch embedment.
- Bolt diameter of nominal ½ inch through ¾ inch.
- 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.

In January 2007, the model code in California changed from the 1997 UBC to the 2006 IBC. This 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 referenced ACI 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 codes and ACI standards.

Two assumptions that affect ACI 318-14’s anchorage criteria in Chapter 17 (formerly Appendix D) are the ductility parameter and the cracked concrete parameter. The resultant low concrete capacity values indicate that a failure of the connection will occur in the concrete before it occurs in the anchor bolt or the wood sill plate, which is counter-intuitive and inconsistent with observed earthquake and wind performance. 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 the focus. 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 Fennel, et. al., “*Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances*,” dated March 29, 2009.

The SEAOC test data show that the yield strength of the wood sill plate connection governs over the strength of the concrete in typical connections. This component testing was necessary to determine the connection behavior, particularly the large amount of yielding the bolts achieve above the concrete surface and the beneficial clamping effect due to square plate washers.

In this article, we present additional commentary on the test report findings and review the underlying assumptions that may be appropriately considered by the designer. As a result of the laboratory testing, the subject anchors may be conservatively designed assuming a wood yield mode as predicted by the yield limit equations associated with Mode III_s and Mode IV behavior in the *ANSI/AWC NDS-2018 National Design Specification® (NDS) for Wood Construction*. These values are subject to the same limitations as NDS Table 12E and are included at the end of this article for reference. Based on the SEAOC test data, the IBC has been revised to reinstate the anchor bolt shear capacities shown in the NDS for most anchor bolts. These values do not apply to anchorage in light-weight concrete or post-installed anchors. Finally, recommendations for further testing are discussed.

Background

In California until 2007, the design procedure and capacity of anchor bolts were based on 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.

The scope and provisions of ACI 318 concrete anchorage provisions in Chapter 17 (formerly in Appendix D) are the result of many years of testing directed at providing designers more transparency into the limit states associated with concrete anchorage. Wood sill plate anchorage forms a small subset of possible anchorage conditions covered by Chapter 17. 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 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 Chapter 17 provisions are rooted in the very low capacity values that were implied relative to past practice. As described in the report, proper application of the ductility and cracked concrete parameters provide a rational, usable set of bolt values. Such a rational anchor bolt value should have 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 the ACI anchorage provisions that were one quarter to one fifth of the traditional value when assuming a non-ductile connection and cracked concrete. This led to a design solution that was inappropriate for the wood sill attachment of many 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, shear walls that would have traditionally required 2 anchors per stud bay would now require 8 anchors per stud bay, which are not constructible.

Another 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 of implementing dramatic changes to the anchor bolt design methodology. Since issues with the old values were not apparent from performance observed after past earthquakes, the need for substantial change was puzzling. In contrast, sill splitting has frequently occurred in past earthquakes.

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 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 5/8 inch diameter bolts since they are representative of most medium and heavy duty shear wall applications. While it was noted 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. The highest compressive test cylinder result was 2710 psi.

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. Secondly, 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 features, every effort was made to test materials representative of the most common shear wall connections.

The independent variables tested were:

Item	Configuration Tested
Sill plate sizes:	2x4, 3x4, 2x6 and 3x6
Anchor bolt edge distances:	1.75 inches and 2.75 inches, depending on sill plate widths
Testing protocols:	monotonic and pseudo-cyclic
Wood-concrete interface conditions:	friction and “frictionless” membranes

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

1. The monotonic tests were an accurate predictor of the elastic performance exhibited in the cyclic tests. Once the anchors were loaded to approximately 5000 pounds, they slowly started to exhibit plastic behavior. 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.
2. The tests 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.
3. 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 significant bolt bending, peak strengths from cyclic tests of the 1-3/4" 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 capacity beyond the yield limit state of the connection. This translates to the peak concrete strength being 6 to 8 times higher than the NDS allowable values. The peak value was generally accompanied by a complete, but shallow concrete delamination.

Figure 1: Concrete Delamination Failure

Use of the impact-echo measurements often signaled internal concrete delamination prior to any visual evidence, although no delamination 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 2). Significant ductile mechanisms were observed in the form of large deflections of the sill plate and bending of the anchor bolt. It should be mentioned that 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 of accurately predicting 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 coincided with initial delamination.

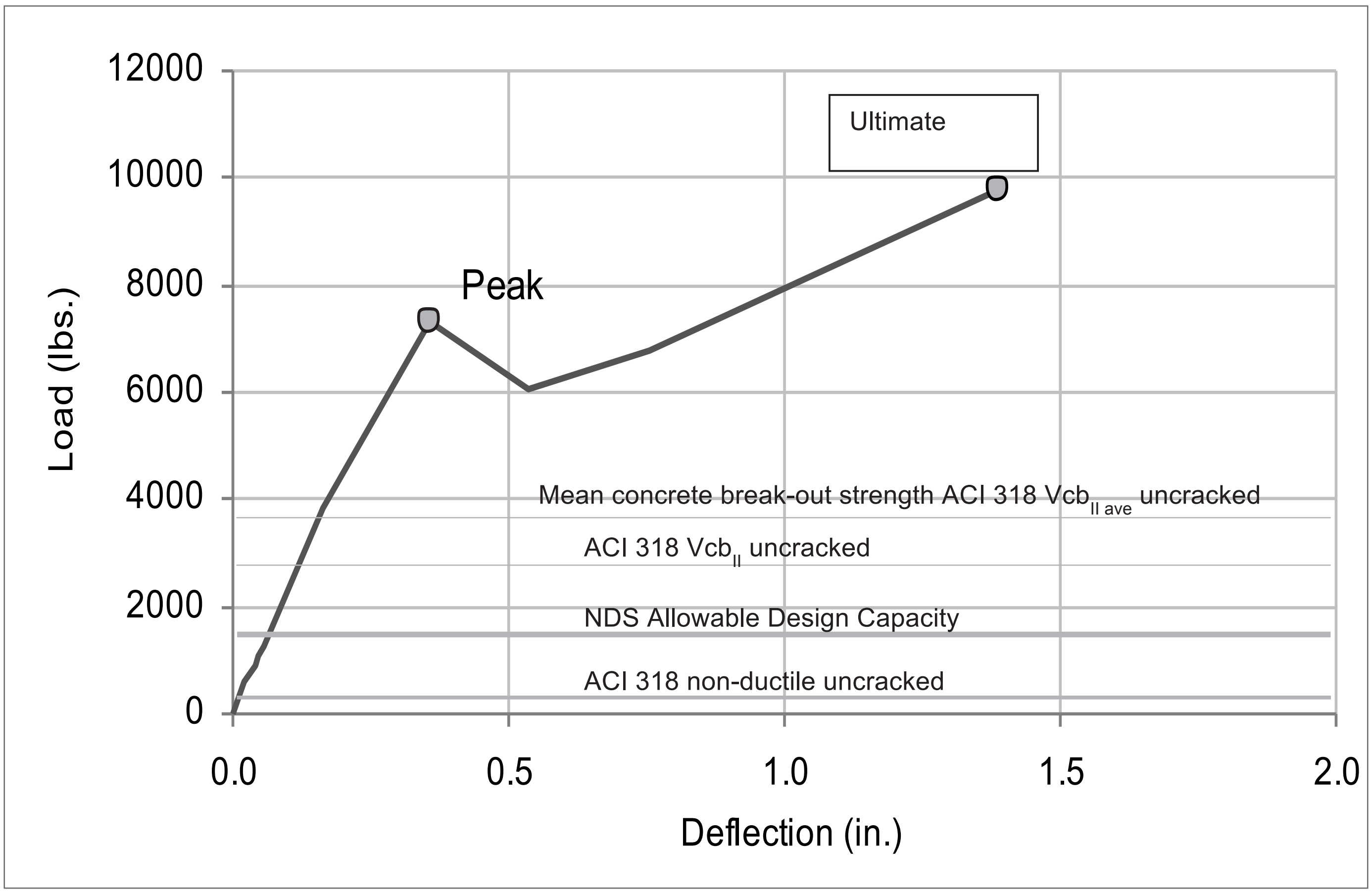


Figure 2: Typical inelastic behavior showing secondary peak.

4. The ACI concrete break-out strength taken from the estimated mean appeared overly conservative for the 1-3/4" 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 Chapter 17 calculated values adjusted to represent concrete break-out strengths. Taken on the whole (i.e. with and without the friction reducing membrane) the 2x4 and 3x4 sill cyclic tests averaged 1.9 times the ACI concrete break out calculated value adjusted for the mean strength.

5. Since the ACI 318 break-out equation approximated the 5% fractile strength as described in section 17.3.2, the SEAOC test report adjusted the break-out strength to the mean-based estimate in order to provide an appropriate 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 Anchor Bolt Test Report. However, whatever adjustment is made, the aggregate of testing has shown the connection exhibits good capacity and ductility that was previously unaccounted for.
6. Since the ultimate values corresponded to large drifts, the data 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 Assumption: As indicated above, ACI 318 Chapter 17 (formerly Appendix D) applies to "cast-in anchors" and "connected structural elements" that include anchor bolts. However, the ACI Commentary R17.1.1 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, since at least four connections are required (two hold downs and two anchor bolts), there are often other interior walls in wood-frame structures, and with the likelihood of substantial friction at sill plate connections, there may be multiple load paths present. Therefore, some engineers have suggested that the subject anchor bolts may not fall within the scope of Chapter 17 based upon the commentary. While this point may have certain merits, the IBC provides that if anchors are not to be regulated by Chapter 17, another "approved method" is necessary. Such an approved method should incorporate a similar level of sophistication as Chapter 17.

Supplementary Reinforcement: ACI 318 Section 17.3.3 provides for the use a strength reduction factor of Φ=0.75 (rather than Φ=0.70), "where supplementary reinforcement is present." Section R17.5.2.9 further clarifies that supplementary reinforcement need only be present and "explicit design is not required" in order to utilize the higher factor. "However, the arrangement of supplementary reinforcement should generally conform to that of the anchor reinforcement shown in Fig. R17.5.2.9a and R17.5.2.9b." 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. This reinforcement bar does not conform to the ACI detailing requirements and should not be considered supplemental reinforcement.

Cracked Concrete Assumption: The first UBC code reference regarding cracked concrete appeared in 1997 UBC section 1923.2 that 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 standard, ACI 318 section 17.5.2.7 stipulates "where analysis indicates cracking at service load levels", $\Psi_{c,V}$ shall be taken as 1.0 for anchors "without supplementary reinforcement or with 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 uncracked concrete is justified, cast-in anchors are allowed a 40% capacity increase since $\Psi_{c,V}$ can be taken as 1.4.

The uncracked assumption is generally justified in light frame construction based on the original testing in cracked concrete. A good review of available test information was recently published by Eligehausen, Mallée, and Silva in the publication, *Anchorage in Concrete Construction* (2006). In this publication (Pp. 157) 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 anchors 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...even anchors that exhibit inferior performance when loaded in tension in cracks are usually adequate to resist shear loads in cracked concrete*". It should be expected that the subject sill plate anchors will not be significantly compromised, since it would require cracks intersecting the anchor and running parallel to the concrete edge that 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 standard requires 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. 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 elastic capacity 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.

Test data indicate that it is rational to use the values obtained from ACI Chapter 17 assuming uncracked concrete and a ductile attachment. Test results also indicate concrete will not govern for the anchorage of 2x and 3x sill plates, and it is conservative to use the NDS design values for bolts up to ¾ inch in diameter. Note that 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_s failure for the ¾ inch anchors. Typical failure modes are illustrated in Figure 3, below.

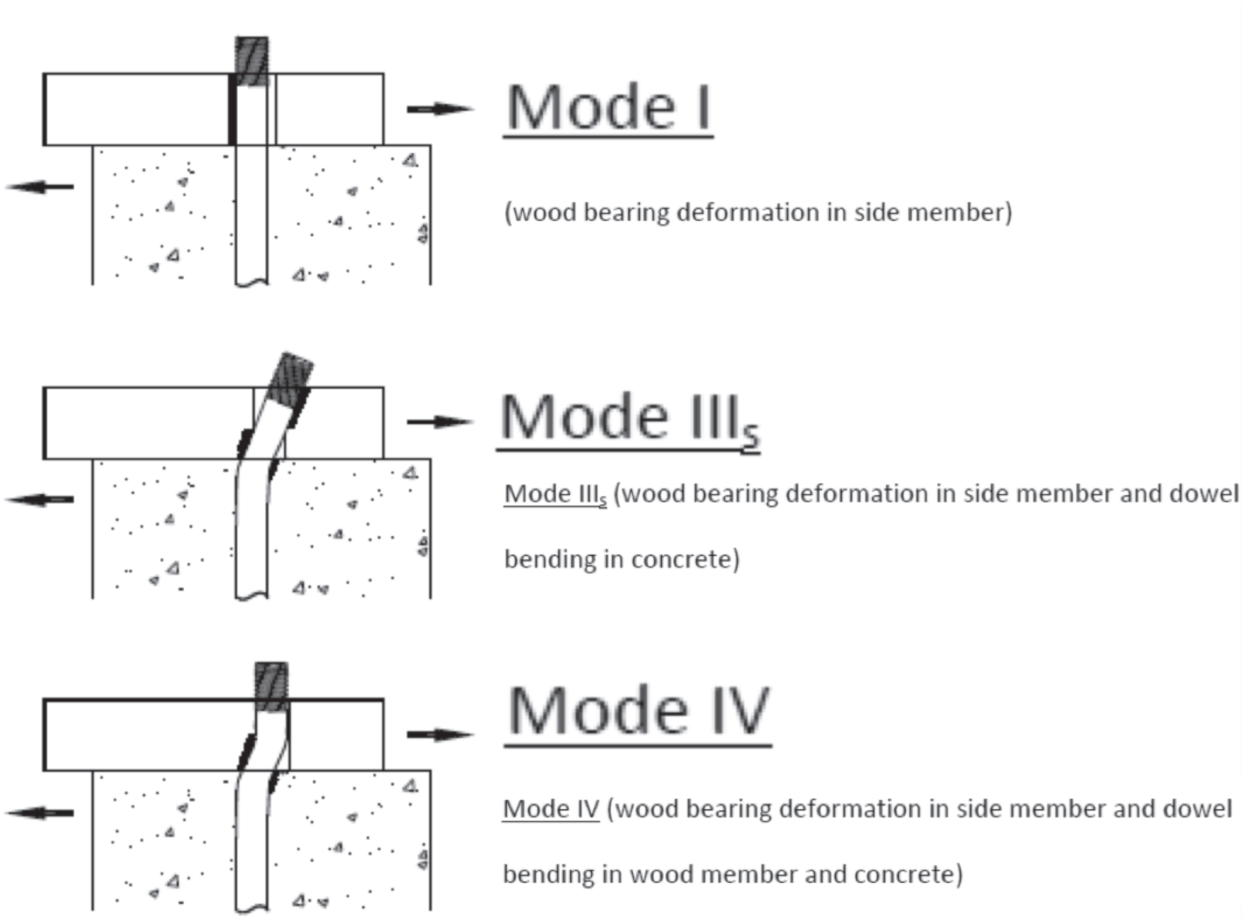


Figure 3: Anchor Bolt Failure Modes
(Fennel, 2009)

Table 1 shows representative anchor bolt shear values based upon the NDS-2018 (AWC 2018).

Table 1: Anchor Bolt Shear Values Based upon the NDS-18 (1.6 Z)

	Bolt Diameter		
Sill Plate	**1/2”**	**5/8"**	**3/4"[1]**
2x	1040	1488	2032
3x	1232	1888	2426

1. **3/4 anchor bolt limited to 6-inch nominal width sill plates.**
2. **Values shown are in lbs. (ASD)**

As a result of these test results, the 2019 CBC, section 1905.1.8 modifies ACI 318-14, section 17.2.3.5.2 to reinstate the former values in the code rather than the reduced values obtained using ACI 318 Chapter 17 equations. This applies only to anchor bolts with a maximum nominal diameter of 5/8” with the same close edge distances, spacing, and end distances used in the tests. Also, based upon these tests, the values for anchor bolts used in cold formed steel walls with wood sheathing were deemed to have similar ductile properties of the sill plate. The code allowable values for anchor bolts in these systems were also reinstated.

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 cyclic testing. Friction is significantly increased due to bending of the anchors and 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. This warrants further study.

Finally, designers are cautioned that damage occurring to these connections may not be readily apparent. Post event observers should be aware that severe damage can be masked by the top of the sill plate.

In addition to the efforts of the 2008-2009 SEAOC Seismology Committee, a number of firms donated time, materials and/or effort which included, Scientific Construction Laboratories, Inc., Structural Solutions, Inc., Certus Consulting, Inc., and VanDorpe Chou Associates, Inc. Phil Line of the American Forest and Paper Association also 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.

References

1991 International Timber Engineering Conference, London – United States Adaptation of European Yield Model to Large Diameter Dowel Fasteners Specifications. Soltis, L.A.; Wilkinson, T.L.

ACI 318-05 Appendix D. Building Code Requirements for Structural Concrete and Commentary. American Concrete Institute, 2005, Pages 379-402.

AF & PA (2001). National Design Specification for Wood Construction. American Forest and Paper Association, Washington, DC.

ASTM Standard D 5764 – 97a, 2007 – Standard Test Method for Evaluating Dowel-Bearing Strength of Wood and Wood-Based products. ASTM International, West Conshohocken, PA.

ASTM Standard E 2126, 2008 – Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings. ASTM International, West Conshohocken, PA.

AWC 2018. National Design Specification for Wood Construction. American Wood Council, Leesburg, VA.

BSSC (2004). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions (FEMA 450-part 1/2003 Edition), Building Seismic Safety Council, Washington, DC.

BSSC (2004). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2: Commentary (FEMA 450-part 2/2003 Edition), Building Seismic Safety Council, Washington, DC.

Canadian Journal of Civil Engineering, 2003 – Lateral Resistance of Bolted Wood-to-Concrete Connections Loaded Parallel or Perpendicular to Grain. Mohammad, M.; Karacabeyli, E.; and Quenneville, J.H.P.

CBC 2019. California Building Code. California Building Standards Commission, Sacramento, CA.

CBC (2007). California Building Code. California Building Standards Code, 2007.

Cook, Ronald A. Strength Design of Anchorage to Concrete. Portland Cement Association, 1999.

Cook, Ronald A. PhD, P.E., Wollmershauser, P.E. and Gerber, Brian S.E. Post-Installed Anchor Approvals for ACI318 Appendix D. Structure Magazine, January 2006, pg 58.

CUREE (2001). Publication W□02 – Development of a Testing Protocol for Wood□Frame Structures. CUREE, Richmond, CA.

Eligehausen, Rolf, Mallee, Ranier and Silva, John F. Anchorage in Concrete Construction. Berlin, Ernst & Sohn, 2006.

Fennell, W.A.; Mochizuki, G.L.; Moore, K.S.; Line, P.; and Van Dorpe, T, "Report on Laboratory Testing of Anchor Bolts Connecting Wood Sill Plates to Concrete with Minimum Edge Distances", February 2009.

Fogstad, Christian, P.E. and Gerber, Brian, P.E., S.E. Code Changes Affecting Post-Installed Concrete Anchor Design. Structure Magazine, December 2007, Pages 45-48.

FEMA 461 □ Interim Protocols for Determining Seismic Performance Characteristics of Structural and Nonstructural Components Through Laboratory Testing. May 2007.

Ghosh, S.K., Dowty, Susan, Dasgupta, P. Analysis of Revisions to the 2006 IBC Structural Provisions, pgs 97-106, 111-112. Structures and Codes Institute, 2006.

Hess, Richard. For What Planet is This Code Written? Structure Magazine, November 2008, page 73.

IBC (2003). 2003 International Building Code. International Code Council, Inc, 2003.

IBC (2006). 2006 International Building Code. International Code Council, Inc, 2006.

ICBO (1997). 1997 Uniform building code. International Conference of Building Officials, Whittier, CA.

ICC, IBC Final Draft Public Hearing "S-229/1912, " Proponent: James R. Cagley, Cagley & Associates and Joseph J. Messersmith, Portland Cement Association; representing the American Concrete Institute Committee 318 and Portland Cement Association respectively, March 1999, Pp. S246-249.

ICC-ES Proposed Acceptance Criteria for Cast-in-Place Proprietary Bolts in Concrete for Light-frame Construction, Subject AC399 0508-R1, Proposed April 2008.

Mendes, Stanley B. "Lessons Learned From Four Earthquake Damaged Multi-Story Type V Structures," Proc. 64th Annual SEAOC Convention, Indian Wells, CA, October 19-21 1995.

SEAOC (2002). Seismic Design Manual, Volume II: Building Design Examples - Light Frame, Masonry and Tilt-up (1997 UBC), Structural Engineers Association of California, Sacramento, CA.

SEAOC Seismology Committee (1999). Recommended Lateral Force Requirements and Commentary, Seventh Edition, Structural Engineers Association of California, Sacramento, CA.

SEAOC, Seismology and Structural Standards Committee, "Wood-Framed Design in Seismic areas: Re-tooling for the 2006 IBC in Light of Current Design Practice," October 2007.

VPI Research Report No. TE1994003 Determination of ShortTerm DurationofLoad Performance of Nailed and Bolted Connections Using Sequential Phased Displacement Tests. Virginia Polytechnic Institute and State University, Blacksburg, VA. Dolan, J.D.; Gutshall S.T.; and McLain T.E. 1996b.

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 2019)

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

SEAOC Seismology Committee (2019). "Anchor Bolts in Light-Frame Construction at Small Edge Distances," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Introduction

Seismic isolation is arguably the most effective way to help protect structures against damage from earthquakes. It is a relatively mature technology that has been used on thousands of buildings and bridges around the world over the last 30 years. To date, there are currently more than 6500 isolated buildings in Japan, more than 125 buildings in the United States and hundreds more throughout the world (Taylor and Aiken, 2012). At a recent conference[1] it was reported that more than 5000 buildings in China, more than 400 in Italy and 700 in Russia have now been isolated.

Applications of seismic isolation in buildings typically include government buildings, hospitals, and emergency centers where immediate occupancy and continued operation is essential to public health and safety. Seismic isolation also finds application in museums and data centers where protection of valuable or sensitive building content is a major concern, as well as manufacturing facilities where downtime following a seismic event could lead to significant economic impacts. The improved performance of seismic isolated buildings can also translate into reduced life-cycle costs (i.e., complete building costs through its functional life) because of the reduced likelihood of building damage resulting from an earthquake during the life of the building. In some regions of the world, most notably Japan, seismic isolation has reached a level of public awareness where it is used as a selling point to residents and building owners who are concerned with the seismic performance of their buildings.

Provisions addressing the analysis and design of seismically isolated structures first appeared as an appendix to Chapter 23 of the 1991 Uniform Building Code. These provisions were based on the work of the Structural Engineers Association of California (SEAOC). Provisions covering seismic isolation are now embodied in the California and International Building Code through reference to ASCE/SEI 7, and exist in a similar form for building evaluation and rehabilitation provisions (retrofit construction) in ASCE/SEI 41.

In light of the performance and potential life-cycle benefits that seismic isolation provides, the SEAOC Protective Systems Committee seeks to promote the use and expand the understanding of seismic isolation. The committee recognizes that a variety of economic, regulatory and technical issues affect the selection of seismic isolation and seeks to inform the reader of how these issues might affect a particular project. The primary focus of this article is the use of seismic isolation for building structures; however, isolation is also used on bridges, special structures and low mass non-structural components within non-isolated buildings.

This article is targeted to a broad audience, including engineers, architects, and project stakeholders. It introduces the main aspects of seismic isolation, discusses its benefits, outlines favorable conditions for its implementation, and discusses the circumstances where its use may be more challenging.

What is Seismic Isolation?

For building structures, seismic isolation refers to the addition of a special support system, often at the base of the structure, which supports the structure vertically while allowing movement in the horizontal direction. This is achieved by placing the structure on special isolation devices, commonly called ‘bearings’, which have low horizontal stiffness and high vertical stiffness.

Under service conditions and typical wind loading, a seismically isolated building performs like a traditional ‘fixed-base’ building. During an earthquake, however, the building experiences significantly reduced horizontal shaking because the isolation system decouples the building from the horizontal shaking of the ground through deformation of the isolation bearings.

[1] 14th World Conference on Seismic Isolation, Energy Dissipation and Active Vibration Control of Structures, Sept 9-11, 2015, San Diego, CA.

Since building structures are heavy, they have a tendency to stay in place. By disconnecting or "isolating" the building from the ground through a horizontally flexible layer, movement of the ground occurs while the building remains essentially stationary. If the building was "fully isolated," it would not move at all (in an absolute sense), effectively floating above the moving ground. In reality, full isolation is neither possible nor desired and therefore some motion is transmitted through the isolation bearings into the building.

To illustrate the effect of seismic isolation, consider a situation where you are standing next to a friend sitting on a swing. Now imagine a strong earthquake occurs. You would likely have trouble standing due to the violent ground shaking. Your friend on the swing, on the other hand, would not experience the horizontal ground shaking but instead would be gently rocking because the swing is 'isolating' them and preventing the ground motions from reaching them. Seismic isolation works in a similar manner.

In more technical terms, a seismic isolation system is used to increase the lateral (horizontal) flexibility of the base of the structure, resulting in the elongation or shift of its fundamental period beyond the predominant (and potentially most damaging) period of ground shaking. This can be visualized in Figure 1 (left) where the fundamental periods of an isolated and non-isolated building are plotted on an acceleration response spectrum. The period of 0.5 seconds represents the fixed-base period of a theoretical superstructure. Moving from this point to the right represents an increasing period shift accomplished through the addition of an isolation system. The use of an isolation system shifts the fundamental period of the building to the right (towards longer periods), thereby reducing the imposed acceleration, and hence the demands, on the building. Figure 1 (right) shows that increasing the period shift also results in higher displacement demands, however, note that this figure represents displacement in the isolation system and not in the superstructure (superstructure drifts reduce with increasing period shift because of the reduced accelerations).

A second but typically less significant effect of seismic isolation is an increase in damping through energy dissipation in the isolation system. This higher damping further reduces the structural response as seen in Figure 1 (left and right). Moving from the red to green lines represents increased damping in the isolation system. A further increase in damping can be achieved by adding supplemental dampers to the isolation system which act in parallel with the isolators.

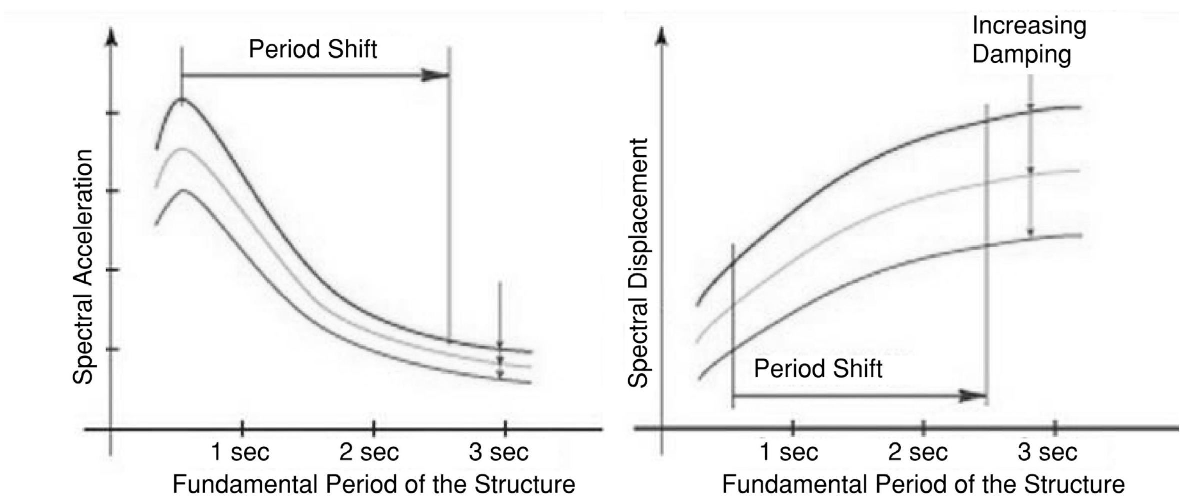


Figure 1. Illustration showing effect of period-shift (left) and increased damping (left and right) on acceleration and displacement response. Figure courtesy of Dynamic Isolation Systems, Inc.

A common misconception is that seismically isolated buildings are sitting on "rollers." In reality, devices specifically designed and manufactured for seismic isolation are used. These devices, often referred to as isolation bearings, provide the required low horizontal and high compression stiffness properties. Additionally, practical considerations require that the isolation system have calibrated stiffness and damping to limit displacements during extreme seismic events and prevent unintentional movements during windstorms. Isolation bearings are typically provided below each building support, thus requiring multiple bearings to seismically isolate a single building.

There are several general categories of seismic isolation bearings. Each uses different mechanical and material properties to achieve the seismic isolation system properties:

- **Elastomeric (Rubber) Bearings** - Elastomeric bearings typically consist of alternating layers of rubber and steel shims bonded together. Elastomeric bearings provide both stiffness and damping. Damping is typically provided through a lead core or special rubber additives. Typical bearings in this category include lead rubber, high damping, and natural rubber bearings.
- **Sliding Bearings** – Sliding bearings typically consist of a flat smooth steel surface on one side and a special bearing liner on the other which permits sliding on the low-friction interface with little damage or wear. Sliding bearings are typically used in combination with elastomeric bearing types to provide the necessary self-centering and restoring-force capability to avoid large residual horizontal offsets following an earthquake
- **Pendulum Bearings** – Pendulum bearings are an adaptation of sliding bearings. A curved steel surface is used in place of a flat one. As the building shifts horizontally – and therefore moves up the curved surface – the weight of the building tends to return the structure to its initial position. Pendulum bearings provide damping through friction and stiffness through their curved shapes and the effect of gravity.

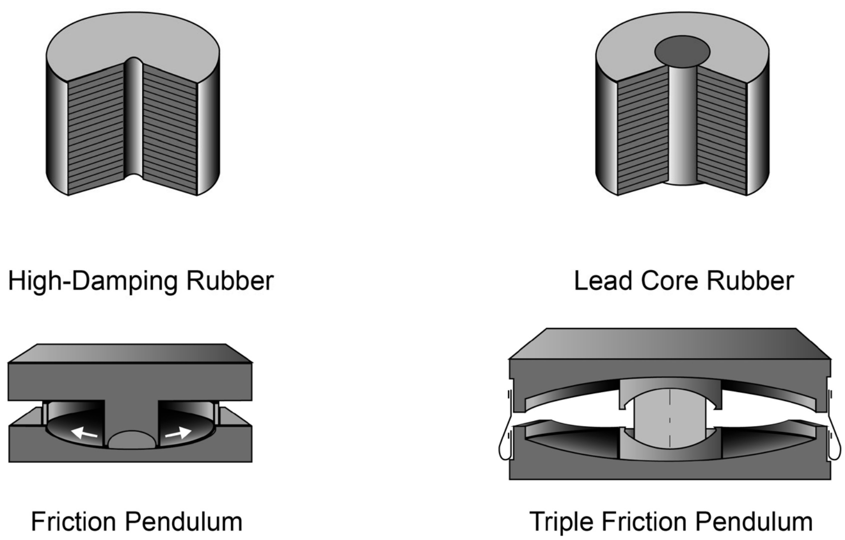


Figure 2. Elastomeric (rubber) bearings (top) and pendulum bearings (bottom). Reprinted from "*Seismic Isolation – The Gold Standard of Seismic Protection*," by M. Walters, *STRUCTURE*, July 2015. Reprinted with permission.

Why Isolate?

Isolation is a highly effective strategy to minimize structural and non-structural damage during an earthquake and to maintain functionality of the building during and after an earthquake. As outlined below, seismic isolation can be used to enhance structural and non-structural performance in new buildings as well as existing and seismically deficient structures.

The clearest illustration of this increased seismic performance is seen by comparing actual recorded videos between isolated and non-isolated buildings in past earthquakes. A striking example of an isolated building response can be seen in this video[2] taken inside of the Ishinomaki Red Cross Hospital during the 2011 Earthquake off the Pacific Coast of Tohoku. The first 90 seconds of the video show doctors and administrative staff inside the hospital during the earthquake. The blinds and hanging items are swinging gently as the building moves. It is remarkable that during this very large seismic event there is virtually no perceivable disruption to operations with the exception of a few papers falling to the floor. Occupants are able to stand without bracing themselves and doctors and staff begin discussing emergency response procedures *before* the earthquake finishes. This response is vastly different from a fixed base building that would show severe disruption to occupants and contents. Cabinets, bookshelves, furniture and equipment are toppled over, and broken glass, fallen ceiling tiles and other hazards would likely have been present immediately following the earthquake.

Reduced Earthquake Damage to the Building: It is a common misconception that buildings designed to the current building code are 'earthquake proof'. In fact, the intent of the building code is primarily to ensure the life safety of its occupants and the public. Section 1626.1 of the 2001 California Building Code noted, *"The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function."* The code does not intend to prevent building damage and implicitly accepts it in pursuit of more economical designs. Depending on the type of structural system, this damage may be repairable but in many cases it is not.

Typical engineering benefits that can be achieved with seismic isolation include reduced seismic base shear, reduced ductility demand (closely associated with building damage) on the superstructure lateral system, reduced inter-story drifts (relative horizontal deformation between two adjacent floors), and reduced horizontal floor accelerations (shaking). Base shear coefficients in the range of 0.15 to 0.20 (i.e., 15 to 20% of the building weight is acting as a lateral force on the structure during the earthquake), are not uncommon for fixed-base structures in California and other high seismicity regions. In contrast to traditional fixed base building designs, the superstructure of an isolated building would remain essentially elastic at this base shear level. In the committee's view, these benefits represent an avenue to achieving the closest thing to what a layperson would define as 'earthquake proof'.

It is also worth noting that seismic isolation fundamentally changes the behavior of the building lateral-force resisting system. The first mode shape (the shape most dominant during earthquake shaking) of an isolated building is typically a nearly straight vertical line with a large offset at the isolation system (Figure 3 left) whereas for a fixed base building the building profile is often an inverted triangle (Figure 3 right). Because of the period shift the building can be envisioned as rigid box on top of a flexible isolation system.

Traditional fixed base lateral force resisting systems can only limit either floor accelerations or interstory drift, and therefore a tradeoff must be made to select which one to reduce. For example, a moment frame system (which is relatively flexible) can reduce floor accelerations, but with the penalty of increased interstory drifts. This is in contrast to a braced frame system (a relatively stiffer structure) which can reduce interstory drifts with the penalty of increased floor accelerations. Seismic isolation is the only structural system that can simultaneously reduce both floor accelerations and interstory drifts.

Additionally, the reliability of an isolated building is increased because the ductility demand, and therefore the uncertainty in the superstructure response, of a fixed-base building during an earthquake (the R value) is traded for the well-defined and stable nonlinearity in the isolation system that has been extensively tested and characterized

[2] https://www.youtube.com/watch?v=Pc1ZO7YwcWc

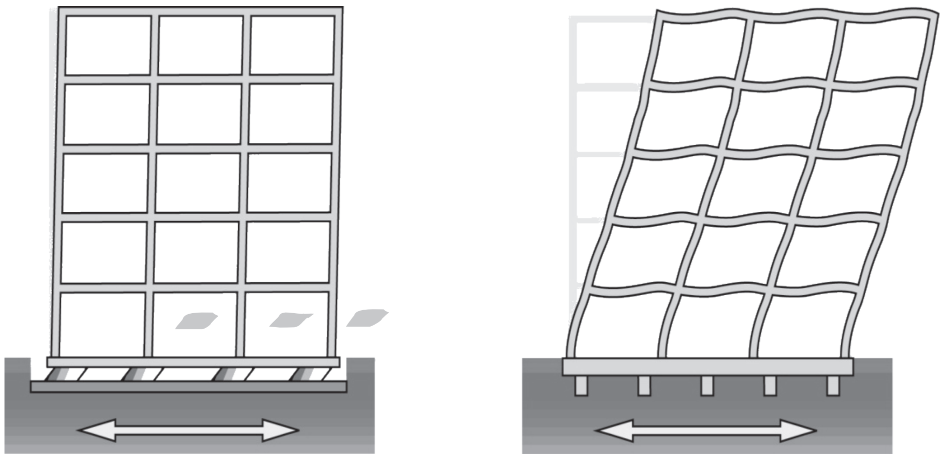


Figure 3: Comparison of superstructure response in an isolated structure (left) and fixed-base structure (right). Figure courtesy of Dynamic Isolation Systems, Inc.

Reduced Damage to Non-Structural Components: In addition to the structural performance benefits of reduced inter-story drifts and base shear, nonstructural components also benefit from the use of seismic isolation. These components can usually be separated into those where damage increases with drift (e.g. partition walls, cladding) or with acceleration (e.g. mechanical, medical, computer, or industrial equipment and ceiling tiles). As outlined in the videos above, seismic isolation can also reduce the disruption to occupants and contents by reducing floor accelerations.

Structural Rehabilitation: Conventional superstructure strengthening often involves the addition of new, stiff lateral force-resisting elements such as braced frames or shear walls. Placing these elements in an existing building can often be challenging and costly given the pre-established building configuration or the need for continued occupancy during retrofit. The use of seismic isolation can reduce, and possibly eliminate, this work in the superstructure through significant reduction in base shear on the structure.

The International Building Code makes no mention of the seismic performance benefits of an isolated building and essentially considers it to be equivalent to a traditional fixed base building when they are both designed to code minimum requirements. However, the National Earthquake Hazards Reduction Program (NEHRP) provisions note that seismically isolated buildings that meet such minimum requirements are expected to perform better than minimum code-complying fixed-base structures. There is nothing that prevents an engineer from utilizing seismic isolation primarily as a way to reduce the seismic base shear of a building. However, in practice, the increased seismic benefits offered by seismic isolation are typically acknowledged.

Where Seismic Isolation Works Best

Base isolation provides enhanced performance to nearly all building configurations; however, certain building and site characteristics maximize the performance benefits and/or minimize the costs for isolated building construction. The characteristics listed in this section provide the greatest benefit and enable construction with the lowest cost premium.

An ideal candidate for seismic isolation can be constructed at the same cost, or even at a cost savings, compared to a fixed base alternative due to the resulting savings in the superstructure. See the next section for a more detailed discussion of costs.

The following building structural and architectural characteristics are particularly suited to seismic isolation:

- **Stiff Superstructure**
  Seismic isolation relies on lengthening the period of a building as described in Section 2. Greater efficiency in the isolation system is achieved by a greater period shift. Generally, a superstructure may be considered "stiff" if it has a fixed-base fundamental period shorter than 1 second.
- **Sufficient Superstructure Strength**
  Isolated structures benefit from lower design forces compared to their fixed base counterparts; however, minimum superstructure strength is required to resist the acceleration of the superstructure. This minimum strength ensures that the superstructure will remain essentially elastic, thereby preserving its seismic response and period shift. It is not uncommon for isolated superstructure design base shear to be on the order of 10 percent of the building weight in high seismic regions. This lower required design strength threshold may be particularly useful for brittle structures or unconventional structural systems.
- **Heavy Buildings**
  High seismic weight is typically a detriment in seismic design; however, it is much less of a concern in a seismically isolated building as isolators can accommodate large axial loads. Elastomeric systems are better able to achieve a significant period shift under high loads, while friction pendulum system displacements are largely unaffected by the seismic mass of the superstructure, as long as a minimum axial compression is maintained on the bearing and significant resistance to uplift is not required for stability.
- **Ability to accommodate an Above-Grade Isolation Plane (No Retaining Walls or Moat)**
  The location of the isolation plane may be the single most important cost consideration in the design of a seismically isolated building or retrofit application. Locating the isolation system below grade typically triggers several costly requirements: (1) a basement must be excavated and constructed, (2) retaining walls must be constructed around the basement, and (3) moat details must be provided. A seismic moat is the space around a building reserved to allow unimpeded movement of the isolation system. An above-grade isolation plane configuration typically places the building on small pedestals to provide room for the isolation system. This configuration allows the entire occupied space to move over the top of the existing grade and eliminates or reduces the costly addition of retaining walls and moats. Despite this, most seismically isolated buildings in the U.S. have used a below ground isolation plane. This is often due to architectural requirements for new buildings and existing conditions for retrofits. Nevertheless, seismic isolation is typically most economically competitive with a traditional fixed base building (on a first-cost basis) when an above grade isolation plane is used.

In addition to the structural requirements outlined above, some stakeholder and location characteristics are more suited to isolated buildings:

- **Long Term Owner**
  Additional costs for the increased performance with seismic isolation are more easily justified by institutional owners with a long-term interest in the seismic performance of a building.
- **Enhanced Seismic Performance Criteria**
  Stakeholders and owners of buildings with critical contents, components, or functions will likely require seismic isolation or other advanced seismic protective systems to achieve their performance objectives. Often, the design of the isolation system is tailored to meet a specific design goal of increased performance (i.e., reduce base shear to prevent damage to the superstructure or to reduce horizontal floor accelerations to a threshold below which a specific piece of equipment is undamaged).

- **Market Demand**
  As noted in the introduction, some markets in Japan have a demand for buildings with higher seismic performance. Historically this has not been seen in the US; however, this may change after a major earthquake.
- **Architectural Demands**
  In addition to increased seismic performance, the reduced base shear, accelerations, drifts, or superstructure ductility demands provided by seismic isolation can also be used to achieve architecture that would not otherwise be possible with a fixed base building. Examples of this include reducing the size and number of lateral force resisting elements, omitting or reducing ductility detailing, and allowing the use of irregular or unconventional structures that would otherwise not be feasible in a fixed base building.

When Seismic Isolation May Be a Challenge

Modern seismic isolation systems, and the analysis tools available to evaluate them, have made significant advancements since their inception. These advancements have opened a broad range of applications where seismic isolation can be used. In technical terms, almost any building can be seismically isolated; however, several factors can make seismic isolation less effective or more difficult to implement. The following is a list of factors identified by the Protective Systems Committee to be examined when considering seismic isolation:

- **Site Conditions**
  - **Sloping Site** – A sloping site that requires a stepped isolation plane (multiple levels of seismic isolation within the same superstructure) requires careful consideration of the forces at the different isolation levels and the nonstructural systems crossing these levels.
  - **Adjacent Structures** – Sufficient clearance is required between the isolated building and all other fixed objects to accommodate the large seismic movements of the isolation system. This is even more crucial in a seismic retrofit where adjacent structures, utilities or site boundaries may be especially restrictive.
  - **Soil Conditions** – Although uncommon, soft soil conditions that produce long period excitation can reduce the effectiveness of seismic isolation because the long period seismic motions are more likely to resonate with the period of the isolation system. Buildings located on stiff soil will realize a greater benefit from isolation than those located on soft soil.
  - **Ground Motion Characteristics** – Similar to the point above, sites with long period pulse motion require special attention to ensure sufficient deformation capacity in the isolation system.

- **Superstructure Configuration**
  - **Long Period Structures** – Flexible structures including tall and slender buildings with a relatively long primary structural mode (greater than about 2-3 seconds) can be challenging to isolate because the period shift required to achieve worthwhile gains in seismic performance requires a very long isolation system period and a very large displacement capacity of the isolation bearings. Practical limits on isolation bearings restrict the length of the period that can be achieved. This, nonetheless, does not preclude the use of seismic isolation for these structures.
  - **Overturning** – Seismic isolation bearings perform best when uplift forces are reduced or eliminated (with the notable exception of several highly specific bearings designed to resist uplift). Single, Double, and Triple Friction Pendulum bearings cannot resist uplift forces, but can accommodate limited uplift displacements. Lead rubber and high damping rubber bearings can accommodate limited tension deformations. Because of this, slender superstructure lateral systems that result in uplift during an earthquake should be avoided, or appropriate measures must be taken to limit or accommodate the uplift. Possible solutions to reduce uplift forces include reducing or stepping the aspect ratio of the lateral system up the building height and adding a stiff element at

the base of the building to distribute the overturning forces across a greater distance. Outriggers may be used to engage additional dead load to resist seismic overturning.

- **Other notable challenges particular to isolated structures**
  - **Cost** – Costs from related elements can be larger than that of the isolation bearings themselves. The following is a list of possible items that should be considered in the cost analysis when considering isolation:
    - An additional elevated floor and/or isolator pedestals. Note that depending on the isolation system used, this floor may require the strength to resist some portion of the isolator P-delta moments.
    - Excavation and retaining wall (unless the isolation plane is above grade level)
    - Accommodations for utilities
    - Moat covers and expansion joints (unless the isolation plane is above grade level)
    - Testing of isolator units
    - Code-required Peer Review

    These initial costs should be weighed against the life cycle cost benefits achieved by the increased seismic performance of the isolation system. Current research shows that the expected post-earthquake repair costs over the life span of a fixed base building can be significant, especially when drift-sensitive partitions or acceleration-sensitive equipment are present. Conversely, the post-earthquake repair costs for an isolated building can be considerably less. At the writing of this document, ongoing research is trying to quantify this reduction in life cycle costs.
  - **Utility Connections** – Utilities (power, water, sewer, etc.) crossing the isolation plane must be able to move freely with the building without being damaged or affecting the performance of the building. Particular attention should be paid to utilities entering the building and the space required to accommodate the seismic joints. Common solutions include coiling excess utility line so that binding will not occur and providing specialized flexible couplings.
  - **Vertical Circulation (Clearance)** – Special attention should be paid to the seismic clearances required around stairs and elevators that cross an isolation plane.
  - **Moat Covers** – Special sliding covers are typically used to keep the moat covered during an earthquake; this can result in seismic moat covers with displacement capacity more than twice that of the movements of the isolation system (because the moat must open and close to accommodate the isolation system displacement capacity)

Where Seismic Isolation Has Been Used in the United States

Examples of isolated buildings in the United States include:

Project	City, State	New/Retrofit
Hospitals		
San Francisco General Hospital	San Francisco, CA	New
Mills Peninsula	Burlingame, CA	New
LAC+USC Medical Center	Los Angeles, CA	New
Veteran Administration (Long Beach Medical Center)	Long Beach, CA	Retrofit
Stanford University, New Hospital	Palo Alto, CA	New
Arrowhead Regional Medical Center	Colton, CA	New
Saint John's Health Center	Santa Monica, CA	New

Hoag Medical Center	Newport Beach, CA	New
Channing House	Palo Alto, CA	New
Martin Luther King/Drew Medical Center	Los Angeles, CA	New
Federal/Local Government		
Los Angeles City Hall	Los Angeles, CA	Retrofit
Oakland City Hall	Oakland, CA	Retrofit
San Francisco City Hall	San Francisco, CA	Retrofit
Hayward City Hall	Hayward, CA	New
Utah State Capitol	Salt Lake City, UT	Retrofit
Pasadena City Hall	Pasadena, CA	Retrofit
Salt Lake City and County Building	Sal Lake City, UT	Retrofit
Berkeley Civic Center	Berkeley, CA	Retrofit
San Francisco International Airport – International Terminal	San Francisco, CA	New
San Francisco Main Library	San Francisco, CA	Retrofit
South Carolina Statehouse	Columbia, South Carolina	Retrofit
Emergency Operation Centers/Critical Facilities		
Public Safety Building	Berkeley, CA	New
Caltrans/CHP Traffic Management Center	San Diego, CA	New
Los Angeles Emergency Operations Center	Los Angeles, CA	New
Washington State Emergency Operations Center	Camp Murray, WA	New
San Quentin Health Services	San Quentin, CA	Retrofit
LAPD Metro Communication Center	Los Angeles, CA	New
San Francisco 911	San Francisco, CA	New
Los Angeles Regional Transportation Management Center	Los Angeles, CA	New
Long Beach 911	Long Beach, CA	New
San Diego EOC	San Diego, CA	New
Portland Water Bureau	Portland, OR	New
Fire Command and Control Center	East Los Angeles, CA	New
Museums		
de Young Museum	San Francisco, CA	New
Asian Art Museum	San Francisco, CA	Retrofit
Data Centers/Fabrication		
Liberty Mutual Data Center	Redmond, WA	New
Conexant Mexicali Semiconductor Facility	Mexicali, Mexico	New

Kaiser Data Center	Corona, CA	New
Higher Education		
Hearst Memorial Mining Building, UC Berkeley	Berkeley, CA	Retrofit
Kerkoff Hall, UCLA	Los Angeles, CA	Retrofit
Campbell Hall, W. Oregon State College	Monmouth, OR	Retrofit
Mackay School of Mines	Reno, NV	Retrofit
Missouri Botanical Garden Research Center	St. Louis, MO	New
UCSF Broad Center of Regeneration Medicine	San Francisco, CA	New
Corporate Headquarters		
Pixar	Emeryville, CA	New
Autozone HQ	Memphis, TN	New
Rockwell International Corporate Headquarters (Building 80)	Newport Beach, CA	Retrofit
Williams Mullen Headquarters	Richmond, VA	New
Courthouses		
Pioneer Courthouse	Portland, OR	Retrofit
San Bernardino Justice Center	San Bernardino, CA	New
Foothill Communities Law & Justice Center	Rancho Cucamonga, CA	New
US Court of Appeals	San Francisco, CA	Retrofit
Residential		
Marina Apartment	San Francisco, CA	
Religious		
Cathedral of Christ the Light	Oakland, CA	New
Cathedral of Our Lady of the Angels,	Los Angeles, California	New
BAPS Hindu Temple		
Commercial/Office Space		
185 Berry St	San Francisco, CA	New/Retrofit
Amgen/Immunex Building J	Seattle, WA	New
Mountain Fuel/Questar	Salt Lake City	New
Hughes Building S-12	El Segundo, CA	Retrofit

Conclusions

SEAOC's Protective Systems Committee recognizes the seismic performance benefits of seismic isolation and seeks to promote the understanding and implementation of it. This article presents a general introduction to seismic isolation, an examination of its benefits, and conditions where its application is likely to be more or less favorable.

References

Technical References

ASCE Task Committee on Seismic Isolation, Primer on Seismic Isolation, A.W. Taylor and T. Igusa, Editors. 2004, American Society of Civil Engineers, ASCE: Reston, VA.

Christopoulos, C., A. Filiatrault, and V.V. Bertero, Principles of passive supplemental damping and seismic isolation, 2006: Iuss press.
Constantinou, M.C., et al., Performance of seismic isolation hardware under service and seismic loading, in Technical Report MCEER-07-00122007, Multidisciplinary Center for Earthquake Engineering Research.

International Council for Research and Innovation in Building and Construction-CIB, Response control and seismic isolation of buildings, ed. M. Higashino and S. Okamoto, 2006, New York, NY: Taylor & Francis.

JSSI, How to Plan and Implement Seismic Isolation for Buildings, ed. The Japan Society of Seismic Isolation (JSSI), 2013: Ohmsha, Ltd.

Naeim, F. and J.M. Kelly, Design of Seismic Isolated Structures: From Theory to Practice, 1999, John Wiley & Sons: New York, NY.

Seismology Committee-Structural Engineers Association of California (SEAOC), Recommended lateral force requirements and commentary. 5th ed, 1975, Sacramento, CA.

Skinner, R.I., W.H. Robinson, and G.H. McVerry, An introduction to seismic isolation, 1993, John Wiley & Sons: Chichester, NY.

Structural Engineers Association of Northern California (SEAONC), Tentative Seismic Isolation Design Requirements. Yellow Book, 1986, San Francisco, CA.

Taylor, A. and I. Aiken, What's Happened to Seismic Isolation of Buildings in the U.S.?, in STRUCTURE, March 2012, A Joint Publication of NCSEA | CASE | SEI. p. 10-13.

SEAOC, 2012 IBC SEAOC Structural/Seismic Design Manual Volume 5: Examples for Seismically Isolated Buildings and Buildings with Supplemental Damping, 2014, Structural Engineers Association of California: Sacramento, CA.

Walters, M., Seismic Isolation – The Gold Standard of Seismic Protection., in STRUCTURE, July 2015, A Joint Publication of NCSEA | CASE | SEI. p. 11-14.

Online Resources for more information about Seismic Isolation

A collection of nontechnical background documents by NEES (Network for Earthquake Engineering Simulation): http://nees.org/calacademy

Base isolation featured in Wired Magazine:
http://www.wired.com/wiredscience/2009/11/worlds-largest-earthquake-safe-building/

A brief nontechnical overview of base isolation by MCEER (Multidisciplinary Center for Earthquake Engineering Research):
http://mceer.buffalo.edu/infoservice/Reference_Services/advEQdesign.asp

A selected list of base isolated structures by MCEER:
http://mceer.buffalo.edu/infoservice/reference_services/baseIsolation1.asp

Key Words

Seismic Isolation
Protective Systems
Elastomeric Bearing
Friction Pendulum Bearing

How to Cite This Publication

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

(SEAOC Seismology Committee 2019)

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

SEAOC Seismology Committee (2019). "When to Consider Seismic Isolation," August 2019, *The SEAOC Blue Book: Seismic Design Recommendations*, Structural Engineers Association of California, Sacramento, CA.

Structural Engineers Association of California

Since 1959, the Recommended Lateral Force Requirements "Blue Book," published by the Structural Engineers Association of California (SEAOC), has heavily influenced the seismic design of structures. This edition is SEAOC's ninth—and a lot has changed in 60 years.

In its first decades, this collection of interpretations and recommendations from SEAOC's Seismology Committee served as the basis for the seismic provisions of the Uniform Building Code (UBC) used in the Western US. Since the move in the 1990s to a national code-development process, the Blue Book's tenor has shifted from writing the code to interpreting the code for practicing structural engineers and putting it context. Importantly, the 2019 edition captures how that context is evolving—from a historic focus on "life-safety" to the modern pursuits of performance-based design and resiliency.