Seismic Design Manual

Volume I
Code Application Examples

Copyright

Publishe

Structural Engineers Association of California (SEAOC)
555 University Avenue, Suite 126
Sacramento, California 95825-6510
Telephone: (916) 427-3647; Fax: (916) 568-0677
E-mail: seaoc@aol.com; Web address: www.seaint.org

The Structural Engineers Association of California (SEAOC) is a professional association of four regional member organizations (Central California, Northern California, San Diego, and Southern California). SEAOC represents the structural engineering community in California. This document is published in keeping with SEAOC's stated mission: "to advance the structural engineering profession; to provide the public with structures of dependable performance through the application of state-of-the-art structural engineering principles; to assist the public in obtaining professional structural engineering services; to promote natural hazard mitigation; to provide continuing education and encourage research; to provide structural engineers with the most current information and tools to improve their practice; and to maintain the honor and dignity of the profession."

Editor

Gail Hynes Shea, Albany, California, shea@slip.net

Disclaime

Practice documents produced by the Structural Engineers Association of California (SEAOC) and/or its member organizations are published as part of our association's educational program. While the information presented in this document is believed to be correct, neither SEAOC nor its member organizations, committees, writers, editors, or individuals who have contributed to this publication make any warranty, expressed or implied, or assume any legal liability or responsibility for the use, application of, and/or reference to opinions, findings, conclusions, or recommendations included in this publication. The material presented in this publication should not be used for any specific application without competent examination and verification of its accuracy, suitability, and applicability by qualified professionals. Users of information from this publication assume all liability arising from such use.

Table of Contents

Table of Contents (continued)

Preface

This document is the initial volume in the three-volume SEAOC *Seismic Design Manual*. It has been developed by the Structural Engineers Association of Californi (SEAOC) with funding provided by SEAOC. Its purpose is to provide guidance on the interpretation and use of the seismic requirements in the 1997 *Uniform Building Code* (UBC), published by the International Conference of Building Official (ICBO), and SEAOC's 1999 *Recommended Lateral Force Requirements and Commentary* (also called the Blue Book).

The *Seismic Design Manual* was developed to fill a void that exists between the Commentary of the Blue Book, which explains the basis for the UBC seismic provisions, and everyday structural engineering design practice. The *Seismic Design Manual* illustrates how the provisions of the code are used. *Volume I: Code Application Examples,* provides step-by-step examples of how to use individual code provisions, such as how to compute base shear or building period. *Volumes II and III: Building Design Examples,* furnish examples of the seismic design of common types of buildings. In Volumes II and III, important aspects of whole buildings are designed to show, calculation-by-calculation, how the various seismic requirements of the code are implemented in a realistic design.

SEAOC intends to update the *Seismic Design Manual* with each edition of the building code used in California.

Ronald P. Gallagher
Project Manager

Acknowledgments

Authors

The *Seismic Design Manual* was written by a group of highly qualified structural engineers. These individuals are both California registered structural engineers and SEAOC members. They were selected by a Steering Committee set up by the SEAOC Board of Directors and were chosen for their knowledge and experience with structural engineering practice and seismic design. The Consultants for Volumes I, II and III are

Ronald P. Gallagher, *Project Manager*
David A. Hutchinson
Jon P. Kiland
John W. Lawson
Joseph R. Maffei
Douglas S. Thompson
Theodore C. Zsutty

Volume I was written principally by Theodore C. Zsutty and Ronald P. Gallagher. Many useful ideas and helpful suggestions were offered by the other Consultants. Consultant work on Volumes II and III is currently underway.

Steering Committee

Overseeing the development of the *Seismic Design Manual* and the work of the Consultants was the Project Steering Committee. The Steering Committee was made up of senior members of SEAOC who are both practicing structural engineers and have been active in Association leadership. Members of the Steering Committee attended meetings and took an active role in shaping and reviewing the document. The Steering Committee consisted of

John G. Shipp, Chair
Robert N. Chittenden
Stephen K. Harris
Maryann T. Phipps
Scott A. Stedman

Reviewers

A number of SEAOC members and other structural engineers helped check the examples in this volume. During its development, drafts of the examples were sent to these individuals. Their help was sought in both review of code interpretations as well as detailed checking of the numerical computations. The assistance of the following individuals is gratefully acknowledged

Saeed R. Amirazizi
Jefferson W. Asher
Brent Berensen
Donald A. Cushing
Vincent DeVita
Richard M. Drake
Todd W. Erickson
Daniel Fisher
Kenneth Gebhar
Edward R. Haninger
Thomas Hunt
Mark S. Jokerst
Isao M. Kawasaki
John W. Lawson
Ronald Lugue
Robert Lyons
Peter Maranian
Brian McDonal
Rory M. McGruer
Brian Montes
Manuel Morden
Farzad Naeim
David A. Napoleon
Josh Plummer
Mehran Pourzanjani
Ian Robertson
John G. Shipp
Donald R. Strand

Seismology Committee

Close collaboration with the SEAOC Seismology Committee was maintained during the development of the document. The 1997-1998 and 1998-1999 Committees reviewed the document and provided many helpful comments and suggestions. Their assistance is gratefully acknowledged.

1998-1999

Saif M. Hussain, *Chair*
Tom H. Hale, *Past Chair*
Robert N. Chittenden
Stephen K. Harris
Douglas Hohbach
Y. Henry Huang
Saiful Islam
Martin W. Johnson
Jaiteerth B. Kinha
Eric T. Lehmkuhl
Simin Naaseh
Hassan Sassi, *Assistant to the Chair*

1997-1998

Tom H. Hale, *Chair*
Ali M. Sadre, *Past Chair*
Robert N. Chittenden
Stephen K. Harris
Saif M. Hussain
Saiful Islam
Martin W. Johnson
Eric T. Lehmkuhl
Roumen V. Mladjov
Simin Naaseh
Carl B. Schulze
Chris V. Tokas
Joyce Copelan, *Assistant to the Chair*

Production and Art

Special thanks are due Lenore Henry of R.P. Gallagher Associates, Inc. who input the entire text from handwritten copy, did all the subsequent word processing, drew al the figures, and formatted the entire document. Without her expertise, this project would never have come to fruition.

Suggestions for Improvement

In keeping with two of its Mission Statements: (1) “to advance the structura engineering profession” and (2) “to provide structural engineers with the most current information and tools to improve their practice”, SEAOC plans to update this document as seismic requirements change and new research and better understanding of building performance in earthquakes becomes available.

Comments and suggestions for improvements are welcome and should be sent to the following:

Structural Engineers Association of California (SEAOC)
Attention: Executive Director
555 University Avenue, Suite 126
Sacramento, California 95825-6510
Telephone: (916) 427-3647; Fax: (916) 568-0677
E-mail: seaoc@aol.com; Web address: www.seaint.org

Errata Notification

SEAOC has made a substantial effort to ensure that the information in this document is accurate. In the event that corrections or clarifications are needed, these will be posted on the SEAOC web site at http://www.seaint.org or on the ICBO website at http://ww.icbo.org. SEAOC, at its sole discretion, may or may not issue written errata.

Seismic Design Manual

Volume I
Code Application Examples

Introduction

Volume I of the SEAOC *Seismic Design Manual: Code Application Examples* deals with interpretation and use of the seismic provisions of the 1997 Uniform Building Code (UBC). The *Seismic Design Manual* is intended to help the reader understand and correctly use the UBC seismic provisions and to provide clear, concise, and graphic guidance on the application of specific provisions of the code. It primaril addresses the major seismic provisions of Chapter 16 of the UBC, with interpretation of specific provisions and examples highlighting their proper application.

Volume I presents 55 examples that illustrate the application of specific seismic provisions of the UBC. Each example is a separate problem, or group of problems, and deals primarily with a single code provision. Each example begins with a description of the problem to be solved and a statement of given information. The problem is solved through the normal sequence of steps, each of which are illustrated in full. Appropriate code references for each step are identified in the right-hand margin of the page.

The complete *Seismic Design Manual* will have three volumes. Volumes II and III will provide a series of seismic design examples for buildings illustrating the seismic design of key parts of common building types such as a large three-story wood frame building, a tilt-up warehouse, a braced steel frame building, and a concrete shear wal building.

While the *Seismic Design Manual* is based on the 1997 UBC, there are some provision of SEAOC's *1999 Recommended Lateral Force Provisions and Commentary* (Blue Book) that are applicable. When differences between the UBC and Blue Book are significant, these are brought to the attention of the reader.

The *Seismic Design Manual* is applicable in regions of moderate and high seismicity (e.g., Zones 3 and 4), including California, Nevada, Oregon, and Washington. It is intended for use by practicing structural engineers and structural designers, building departments, other plan review agencies, and structural engineering students.

How to Use This Document

The various code application examples of Volume I are organized in numerical order by 1997 UBC section number. To find an example for a particular provision of the code, look at the upper, outer corner of each page, or in the table of contents.

Generally, the UBC notation is used throughout. Some other notation is also defined in the following pages, or in the examples.

Reference to UBC sections and formulas is abbreviated. For example, "1997 UBC Section 1630.2.2" is given as §1630.2.2 with 1997 UBC being understood. "Formula (32-2)" is designated Equation (32-2) or just (32-2) in the right-hand margins. Throughout the document, reference to specific code provisions and equations (the UBC calls the latter formulas) is given in the right-hand margin under the category Code Reference. Similarly, the phrase "Table 16-O" is understood to be 1997 UBC Table 16-O.

Generally, the examples are presented in the following format. First, there is a statement of the example to be solved, including given information, diagrams, and sketches. This is followed by the "Calculations and Discussion" section, which provides the solution to the example and appropriate discussion to assist the reader. Finally, many of the examples have a third section designated "Commentary." In this latter section, comments and discussion on the example and related material are made. Commentary is intended to provide a better understanding of the example and/or to offer guidance to the reader on use of the information generated in the example.

In general, the Volume I examples focus entirely on use of specific provisions of the code. No design is illustrated. Design examples are given in Volumes II and III.

The *Seismic Design Manual* is based on the 1997 UBC, unless otherwise indicated. Occasionally, reference is made to other codes and standards (e.g., ACI 318-95 or 1997 NDS). When this is done, these documents are clearly identified.

Notation

The following notations are used in this document. These are generally consistent with that used in the UBC. However, some additional notations have also been added.

A_B = ground floor area of structure in square feet to include area covered by all overhangs and projections.

A_c = the combined effective area, in square feet, of the shear walls in the first story of the structure.

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

A_x = the torsional amplification factor at Leve x.

a_p = numerical coefficient specified in §1632 and set forth in Table 16-O of UBC.

C_a = seismic coefficient, as set forth in Table 16-Q of UBC.

C_t = numerical coefficient given in §1630.2.2 of U BC.

C_v = seismic coefficient, as set forth in Table 16-R of UBC.

D = dead load on a structural element.

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

E, E_h, E_m, E_v, F_i, F_n = earthquake loads set forth in §1630.1 of UBC.

F_x = design seismic force applied to Leve i, n or x, respectively.

F_p = design seismic force on a part of the structure.

F_{px} = design seismic force on a diaphragm.

F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n.

F_a = axial stress.

F_y = specified yield strength of structural steel.

f_c' = specified compressive strength of concrete.

f_i = lateral force at Level i for use in Formula (30-10) of UBC.

f_m' = specified compressive strength of masonry.

f_p = equivalent uniform load.

f_y = specified yield strength of reinforcing steel

g = acceleration due to gravity.

h_i, h_n,h_x = height in feet above the base to Leve *i*, *n* or *x*, respectively.

I = importance factor given in Table 16-K of UBC.

I_p = importance factor specified in Table 16-K of UBC.

L = live load on a structural element.

Level *i* = level of the structure referred to by the subscript *i*. "$i = 1$" designates the first level above the base.

Level *n* = that level that is uppermost in the main portion of the structure.

Level *x* = that level that is under design consideration. "$x = 1$" designates the first level above the base.

N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16-S and 16-U of UBC.

N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16-T and 16-U of UBC.

R = numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral-force-resisting systems, as set forth in Table 16-N or 16-P of UBC.

r = a ratio used in determining ρ. See §1630.1 of UBC.

S_A, S_B, S_C, S_D, S_E, S_F = soil profile types as set forth in Table 16-J of UBC.

T = elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration.

V = the total design lateral force or shear at the base given by Formula (30-5), (30-6), (30-7) or (30-11) of UBC.

V_x = the design story shear in Story x.

W = the total seismic dead load defined in §1620.1.1 of UBC.

w_i, w_x = that portion of W located at or assigned to Level i or x, respectively.

W_p = the weight of an element of component.

w_{px} = the weight of the diaphragm and the element tributary thereto at Level x, including applicable portions of other loads defined in §1630.1.1 of UBC.

Z = seismic zone factor as given in Table 16-I of UBC.

Δ_M = Maximum inelastic response displacement, which is the tota drift or total story drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in §1630.9 of UBC.

Δ_S = Design level response displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.

δ_i = horizontal displacement at Level i relative to the base due to applied lateral forces, f, for use in Formula (30-10) of UBC.

ϕ = capacity-reduction or strength-reduction factor.

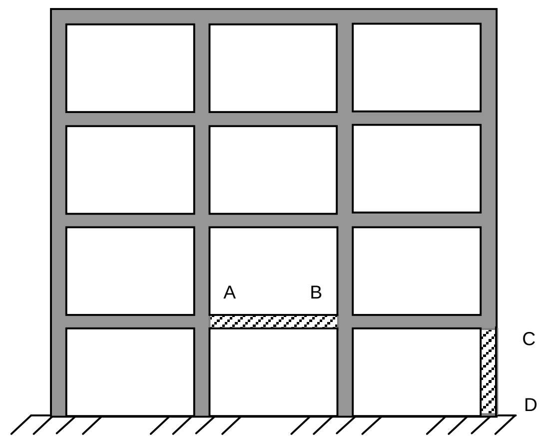
ρ = Redundancy/reliability factor given by Formula (30-3) of UBC.

Ω_o = Seismic force amplification factor, which is required to account for structural overstrength and set forth in Table 16-N of UBC.

Example 1
Earthquake Load Combinations: Strength Design §1612.2

This example demonstrates the application of the strength design load combinations that involve the seismic load E given in §1630.1.1. This will be done for the moment-resisting frame structure shown below:

Zone 4
$C_a = 0.44$
$I = 1.0$
$\rho = 1.1$
$f_1 = 0.5$
Snow load $S = 0$



Beam A-B and Column C-D are elements of the special moment-resisting frame. Structural analysis has provided the following individual beam moments at A, and the column axial loads and moments at C due to dead load, office building live load, and lateral seismic forces.

	Dead Load D	*Live Load L*	*Lateral Seismic Load E_h*
Beam Moment at A	100 kip-ft	50 kip-ft	120 kip-ft
Column C-D Axial Load	90 kips	40 kips	110 kips
Column Moment at C	40 kip-ft	20 kip-ft	160 kip-ft

Find the following:

1. **Strength design moment at beam end A.**

2. **Strength design axial load and moment at column top C.**

Calculations and Discussion	Code Reference

1. Strength design moment at beam end A.

To determine strength design moments for design, the earthquake component E must be combined with the dead and live load components D and L. This process is illustrated below.

a. Determine earthquake load E: **§1630.1.1**

The earthquake load E consists of two components as shown below in Equation (30-1). E_h is due to horizontal forces, and E_v is due to vertical forces.

$$E = \rho E_h + E_v \quad (30\text{-}1)$$

The moment due to vertical earthquake forces is calculated as

$$E_v = 0.5 C_a I D = 0.5(0.44)(1.0)(100) = 22 \text{ k-ft}$$ §1630.1.1

The moment due to horizontal earthquake forces is given as

$$E_h = 120 \text{ k-ft}$$

Therefore

$$E = \rho E_h + E_v = 1.1(120) + 22 = \underline{\underline{154 \text{ k-ft}}}$$

b. Apply earthquake load combinations: **§1612.2.1**

The basic load combinations for strength design (or LRFD) are given in §1612.2.1. For this example, the applicable equations are:

$$1.2D + 1.0E + f_1 L \quad (12\text{-}5)$$

$$0.9D \pm 1.0E \quad (12\text{-}6)$$

Using Equation (12-5) and Equation (12-6), the strength design moment at A for combined dead, live, and seismic forces are determined.

$$M_A = 1.2M_D + 1.0M_E + f_1 M_L = 1.2(100) + 1.0(154) + 0.5(50) = 299 \text{ k-ft}$$

$$M_A = 0.9M_D \pm 1.0M_E = 0.9(100) \pm 1.0(154) = 244 \text{ k-ft} \;\; or \;\; -64 \text{ k-ft}$$

$$\therefore M_A = \underline{\underline{299 \text{ k-ft}}} \;\; or \;\; \underline{\underline{-64 \text{ k-ft}}}$$

c. Specific material requirements:

There are different requirements for concrete (and masonry) frames than for steel as follows.

Structural Steel: Section 2210 specifies use of the load combinations of §1612.2.1 as given above without modification.

Reinforced Concrete: Section 1909.2.3 specifies use of the load combinations of §1612.2.1, where Exception 2 requires the factor load combinations of Equation (12-5) and Equation (12-6) to be multiplied by 1.1 for concrete and masonry elements. (*Note:* At the time of publication, April 1999, the 1.1 factor is under consideration for change to 1.0.) Therefore, for a reinforced concrete frame, the combinations are:

$$1.1\left(1.2D + 1.0E + f_1 L\right) = 1.32D + 1.1E + 1.1 f_1 L \quad (12\text{-}5)$$

$$1.1\left(0.9D \pm 1.0E\right) = 0.99D \pm 1.1E \quad (12\text{-}6)$$

$$M_A = 1.1\left(299 \text{ k-ft}\right) = 328.9 \text{ k-ft}$$

$$M_A = 1.1\left(244 \text{ k-ft} \;\; or \;\; -64 \text{ k-ft}\right) = 268.4 \text{ k-ft} \;\; or \;\; -70.4 \text{ k-ft}$$

$$\therefore M_A = \underline{\underline{328.9 \text{ k-ft}}} \;\; or \;\; \underline{\underline{-70.4 \text{ k-ft}}} \text{ for a concrete frame.}$$

2. Strength design axial load and moment at column top C.

a. Determine earthquake load E: **§1630.1.1**

$$E = \rho E_h + E_v \quad (30\text{-}1)$$

where

$$E_v = 0.5 C_a I D = 0.22D \quad \text{§1630.1.1}$$

For axial load

$$E = E_h + E_v = 1.1\left(110 \text{ kips}\right) + 0.22\left(90 \text{ kips}\right) = \underline{\underline{140.8 \text{ kips}}}$$

For moment

$$E = E_h + E_v = 1.1\left(160\text{k-ft}\right) + 0.22\left(40\text{k-ft}\right) = \underline{\underline{184.8 \text{ k-ft}}}$$

b. **Apply earthquake load combinations:** **§1630.1.1**

$$1.2D + 1.0E + f_1 L \tag{12-5}$$

$$0.9D \pm 1.0E \tag{12-6}$$

Design axial force P_C at point C is calculated as

$$P_C = 1.2D + 1.0E + f_1 L = 1.2\,(90) + 1.0\,(140.8) + 0.5\,(40) = 268.8 \text{ kips}$$

$$P_C = 0.9D \pm 1.0E = 0.9\,(90) \pm 1.0\,(140.8) = 221.8 \textit{ and } -59.8 \text{ kips}$$

$\therefore P_C = \underline{\underline{268.8 \text{ kips}}}$ compression, or $\underline{\underline{59.8 \text{ kips}}}$ tension

Design moment M_C at point C is calculated as

$$M_C = 1.2D + 1.0E + f_1 L = 1.2\,(40 \text{ k-ft}) + 1.0\,(184.8 \text{ k-ft}) + 0.5\,(20 \text{ k-ft}) = 242.8 \text{ k-ft}$$

$$M_C = 0.9\,D \pm 1.0E = 0.9\,(40 \text{ k-ft}) \pm 1.0(184.8 \text{ k-ft}) = 220.8 \text{ k-ft} \textit{ or } -148.8 \text{ k-ft}$$

$\therefore M_C = 242.8$ k-ft *or* –148.8 k-ft

Note that the column section capacity must be designed for the interaction of $P_C = 268.8$ kips compression and $M_C = 242.8$ k-ft (for dead, live and earthquake), and the interaction of $P_C = 59.8$ kips tension and $M_C = -148.8$ k-ft (for dead and earthquake).

c. **Specific material requirements** **§1630.1.1**

Structural Steel: Section 2210 specifies the use of the load combinations of §1612.2.1 as given above without modification.

Reinforced Concrete: The axial force P_C and the moment M_C must be multiplied by 1.1 per §1612.2.1.

Commentary

Use of strength design requires consideration of vertical seismic load E_v. When allowable stress design is used, the vertical seismic load E_v is not required under §1630.1.1.

The incorporation of E_v in the load combinations for strength design has the effect of increasing the load factor on the dead load action D. For example, consider the load combination of Equation (12-5)

$$1.2D + 1.0E + (f_1 L + f_2 S) \tag{12-5}$$

where $E = \rho E_h + E_v$

and $E_v = 0.5 C_a I D$

this becomes

$$1.2D + 1.0(0.5 C_a I D + \rho E_h) + (f_1 L + f_2 S)$$

$$(1.2 + 0.5 C_a I) D + 1.0 \rho E_h + (f_1 L + f_2 S)$$

in the numerical example

$$0.5 C_a I = 0.22$$

Thus, the total factor on D is $1.2 + 0.22 = 1.42$

For the allowable stress design load combinations of §1612.3, E_v may be taken as zero. When these combinations are converted to an equivalent strength design basis, the resulting factor on dead load D is comparable to $(1.2 + 0.5 C_a I)$ in §1612.2.

For example, consider the following:

The basic load combinations of §1612.3.1, without increase in allowable stresses, have a 1.70 factor on D (using the procedure permitted in §1630.8.2.1 for conversion to design strength).

The alternate basic load combinations of §1612.3.2 with a permitted one-third increase in allowable stress has a $\dfrac{1.70}{1.33} = 1.28$ factor on D.

Example 2
Combinations of Loads §1612.3

The code requires the use of allowable stress design for the design of wood members and their fastenings (see §2301 and §2305). Section 1612.3 permits two different combinations of load methods. These are:

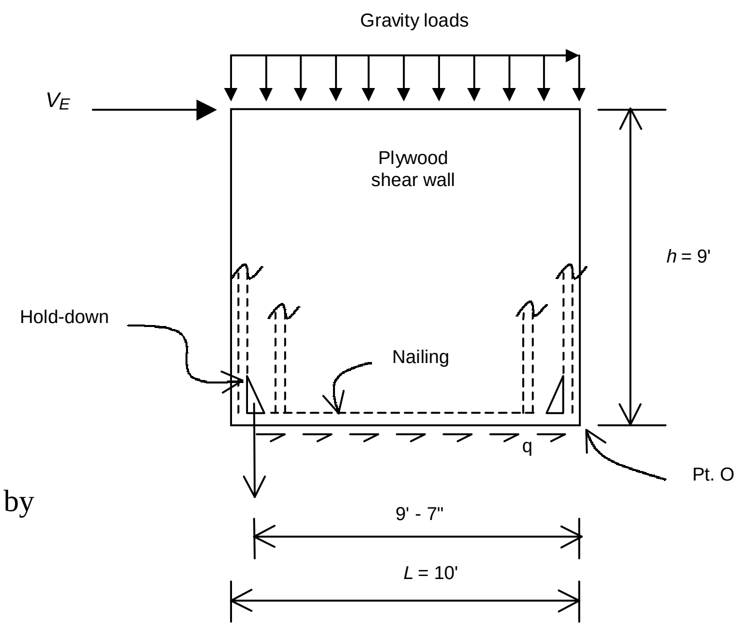
1. Allowable stress design (ASD) of §1612.3.1
2. Alternate allowable stress design of §1612.3.2

This example illustrates the application of each of these methods. This is done for the plywood shear wall shown below. The wall is a bearing wall in a light wood framed building.

The following information is given:

Zone 4
$I = 1.0$
$\rho = 1.0$
$C_a = 0.40$
$V_E = 4.0$ kips (seismic force determined from §1630.2)

Gravity loads:
Dead $w_D = 0.3$ klf (tributary dead load, including weight of wall)
Live $w_L = 0$ (roof load supported by other elements)



Determine the required design loads for shear capacity q and hold-down capacity T for the following load combinations:

1. **Basic allowable stress design.**

2. **Alternate allowable stress design.**

Calculations and Discussion — *Code Reference*

1. Basic allowable stress design. **§1612.3.1**

The governing load combinations for basic allowable stress design are Equations (12-9), (12-10) and (12-11). These are used without the usua one-third stress increase except as permitted by 1809.2 for soil pressure. For wood design, however, the allowable stresses for short-time loads due to wind or earthquake may be used.

$$D + \frac{E}{1.4} \qquad (12\text{-}9)$$

$$0.9D \pm \frac{E}{1.4} \qquad (12\text{-}10)$$

$$D + 0.75L + 0.75\frac{E}{1.4} \qquad (12\text{-}11)$$

where

$$E = \rho E_h + E_v = (1.0)E_h + O = E_h \qquad (30\text{-}1)$$

Note that under the provisions of §1630.1.1, E_v is taken as zero for ASD.

Dead and live load are not involved when checking shear, and both the governing Equations (12-10) and (12-11) reduce to $1.0E$. In this example, E reduces to E_h. For checking tension (hold-down capacity), Equation (12-10) governs. Whenever compression is checked, then Equations (12-9) and (12-11) must be checked.

a. Required unit shear capacity *q*.

Base shear and the resulting element seismic forces determined under §1630.2 are on a strength design basis. For allowable stress design, these must be divided by 1.4 as indicated above in Equations (12-9), (12-10) and (12-11). Thus

$$\frac{E}{1.4} = \frac{E_h}{1.4} = \frac{V_e}{1.4} = V_{ASD} = \frac{4{,}000}{1.4} = 2{,}857 \text{ lbs}$$

The unit shear is

$$q = \frac{V_{ASD}}{L} = \frac{2{,}857}{10'} = \underline{\underline{286 \text{ plf}}}$$

This unit shear is used to determine the plywood thickness and nailing requirements from Table 23-II-I-1. Footnote 1 of that Table states that the allowable shear values are for short-time loads due to wind or earthquake.

b. **Required hold-down capacity *T*.**

Taking moments about point O at the right edge of wall and using $V_E = 2{,}875 \text{ lbs}$, the value of the hold-down force T_E due to horizontal seismic forces is computed

$$9.58T_E = 9V_E$$

$$T_E = \frac{9V}{9.58} = \frac{9' \times 2.857}{9.58'} = \underline{\underline{2.68 \text{ kips}}}$$

Using Equation (12-10) the effect of dead load and seismic forces are combined to determine the required ASD hold-down capacity. In this example

$$D = \frac{1}{2}(w_D)(10') = \frac{1}{2}(0.3)(10) = 1.5 \text{ kips}$$

$$T = 0.9D - \frac{E}{1.4} = 0.9D - T_E = 0.9\,(1.5) - 2.68 = \underline{\underline{-1.33 \text{ kips}}} \text{ tension} \tag{12-10}$$

This value is used for the selection of the premanufactured hold-down elements. Manufacturer's catalogs commonly list hold-down sizes with their "$1.33 \times$allowable" capacity values. Here the 1.33 value represents the allowed Load Duration factor, C_D, given in Table 2.3.2 of §2316.2 for resisting seismic loads. This is not considered a stress increase (although it has the same effect). Therefore, the "$1.33 \times$allowable" capacity values may be used to select the appropriate hold-down element.

2. **Alternate allowable stress design.** **§1612.3.2**

Under this method of load combination, the customary one-third increase in allowable stresses is allowed. However, Item 5 of §2316.2 states that the one-third increase shall not be used concurrently with the load duration factor C_D. The governing load combinations, in the absence of snow load, are the following:

$$D + L + \frac{E}{1.4} \tag{12-13}$$

$$0.9D \pm \frac{E}{1.4} \tag{12-16-1}$$

where $E = \rho E_h + E_v = (1.0)\,E_h + O = E_h$ (30-1)

Note: Equation (12-16-1) is a May 1998 errata for the first printing of the code.

Note that E_v is taken as zero for ASD per §1630.1.1.

a. **Required unit shear capacity *q*.**

$$\frac{E}{1.4} = \frac{E_h}{1.4} = \frac{V_e}{1.4} = V_{ASD} = \frac{4{,}000}{1.4} = 2{,}857 \text{ lbs}$$

$$q = \frac{V_{ASD}}{L} = \frac{2{,}856}{10} = \underline{\underline{286 \text{ plf}}}$$

This value may be used directly to select the plywood thickness and nailing requirements from Table 23-II-I-1. This method recognizes that Table 23-II-I-1 already includes a 1.33 allowable stress increase for seismic loading, and the one-third increase cannot be used again with the tabulated values.

b. **Required hold-down capacity *T*.**

Taking moments about point O at the right edge of wall for only seismic forces

$$9.58T_E = 9V_E$$

$$T_E = \frac{9\,(2.857 \text{ kips})}{9.58} = 2.68 \text{ kips}$$

The dead load effect on the hold-down is one-half the dead load. Thus,

$$D = \frac{1}{2}(w_D)(10') = \frac{1}{2}(0.3)(10) = 1.5 \text{ kips}$$

The governing tension is determined from Equation (12-16-1)

$$T = 0.9D - \frac{E}{1.4} = 0.9D - T_E = 0.9\,(1.5) - 2.68 = \underline{\underline{-1.33 \text{ kips}}} \text{ tension} \qquad (12\text{-}10)$$

This value may be used directly, without modification, to select the "1.33×allowable" capacity of the hold-down elements. Note that the "1.33×allowable" value can be considered either as the one-third increase permitted by §1612.3.1, or the use of a load-duration factor of $C_D = 1.33$.

Commentary

For wood design, the use of the load duration factor C_D is not considered as an increase in allowable stress. Together with the other factors employed in establishing the allowable resistance of wood elements, it is the means of representing the extra strength of wood when subject to short duration loads and provides the allowable stress for wind or earthquake load conditions. The allowable shear values given in the Chapter 23 Tables 23-II-H, 23-II-I-1, and 23-II-1-2 are based on this use of this load duration factor. Therefore, the use of the C_D factor or the aforementioned table values is permitted for the wind and earthquake load combinations of §1612.3. However, both §1622.3.1 and §2316.2, Item 5, prohibit the *concurrent* use of a one-third increase in the normal loading allowable stress with the load duration factor C_D.

It is important to note that, for other than the wind or earthquake load combinations, and for other materials such as masonry where there is no load duration factor, the equivalency of the capacity requirements for §1612.3.1 and §1612.3.2 does not apply mainly because of the prohibited use of a stress increase in §1612.3.1. In this case, the minimum required allowable stress design capacity requirements are best given by the alternate basic load combinations in §1612.3.2.

Example 3
Seismic Zone 4 Near-Source Factor §1629.4.2

The 1997 UBC introduced the concept of near-source factors. Structures built in close proximity to an active fault are to be designed for an increased base shear over similar structures located at greater distances. This example illustrates the determination of the near-source factors N_a and N_v. These are used to determine the seismic coefficients C_a and C_v used in §1630.2.1 to calculate design base shear.

1. **Determine the near-source factors N_a and N_v for a site near Lancaster, California.**

Calculations and Discussion — Code Reference

First locate the City of Lancaster in the book *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.* This is published by the International Conference of Building Officials and is intended to be used with the 1997 *Uniform Building Code.* Lancaster is shown on map M-30. Locate the site on this map (see figure), and then determine the following:

The shaded area on map M-30 indicates the source is a type A fault. Therefore

Seismic source type: A

The distance from the site to the beginning of the fault **zone** is 6 km. Another 2 km must be added to reach the **source** (discussed on page vii of the UBC *Maps of Known Active Faults*). Thus, the distance from the site to the source is 6 km + 2 km = 8 km.

Distance from site to source: 8 km.

Values of N_a and N_v are given in Tables 16-S and 16-T for distances of 2, 5, 10, and 15 km. For other distances, interpolation must be done. N_a and N_v have been plotted below. For this site, N_a and N_v can be determined by entering the figures at a distance 8 km. and using the source type A curves. From this

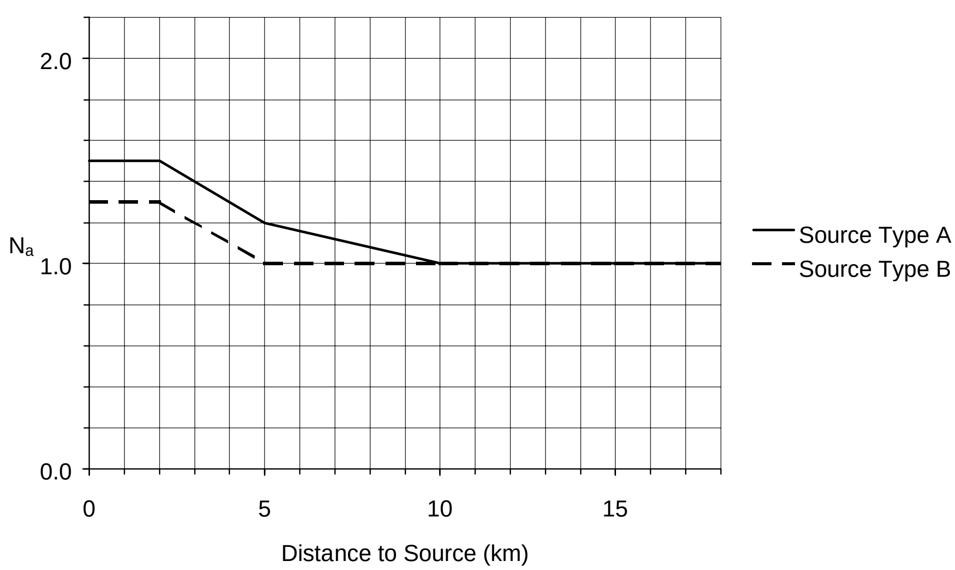
$$N_a = \underline{\underline{1.08}}$$

$$N_v = \underline{\underline{1.36}}$$

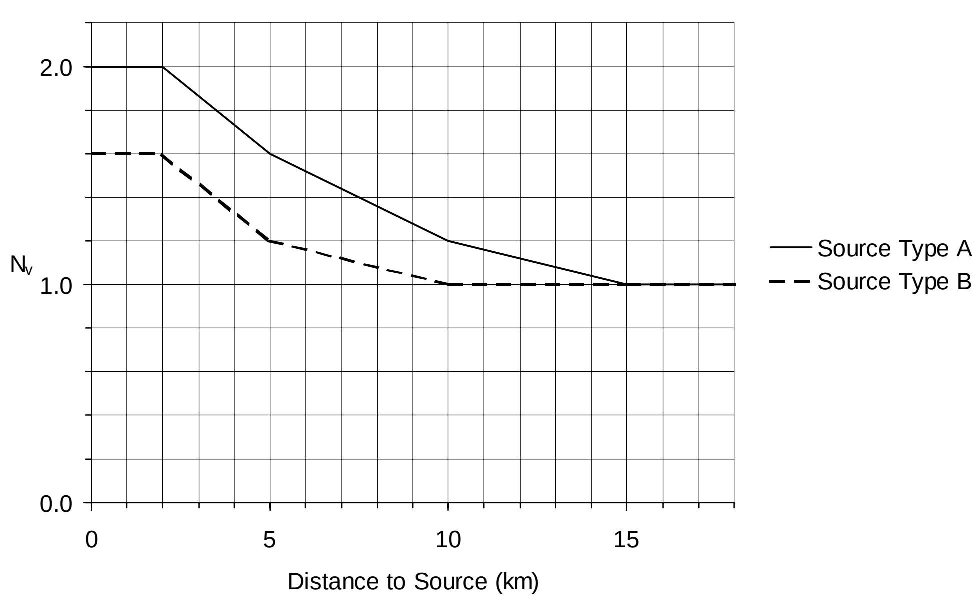
Commentary

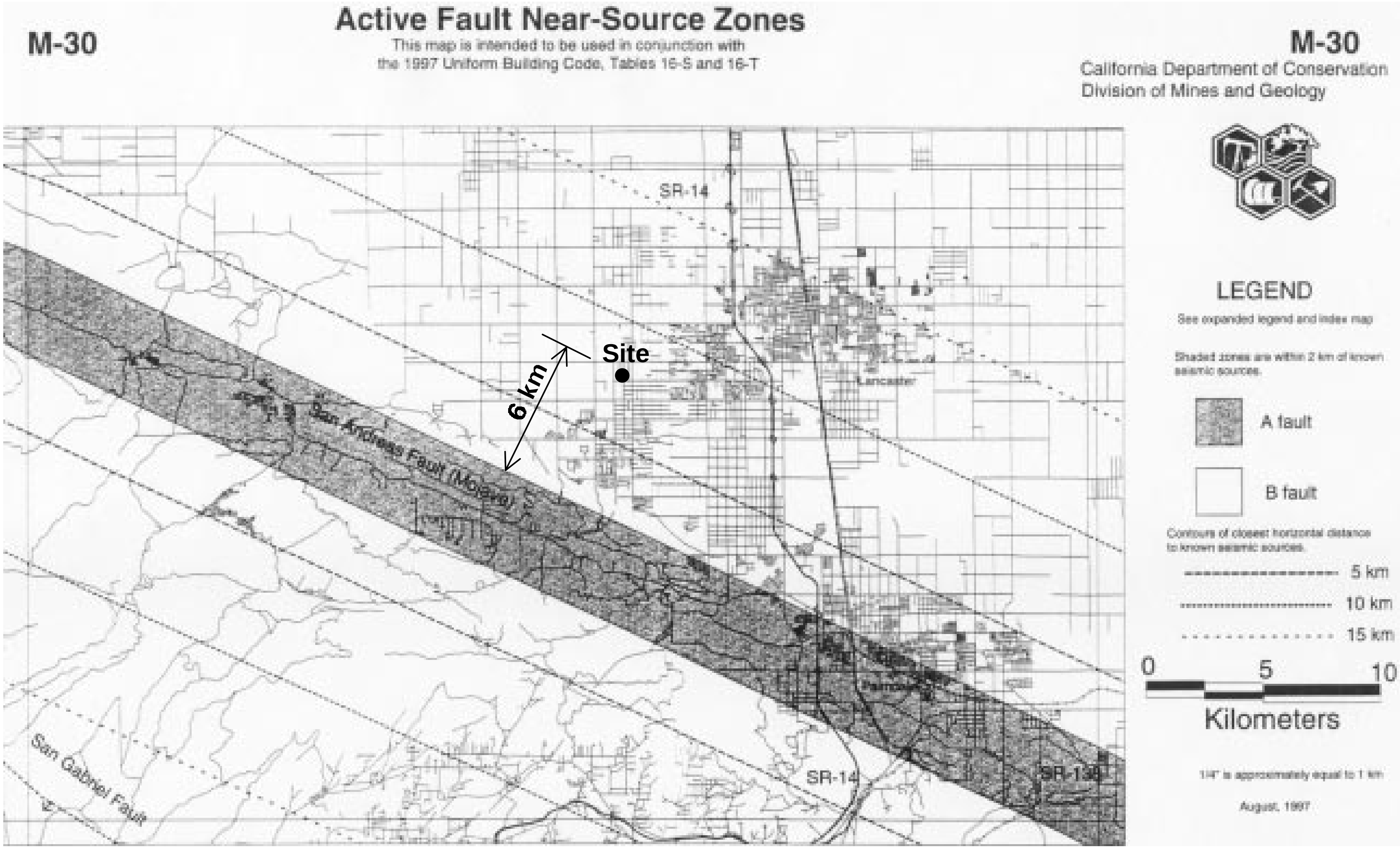
The values of N_a and N_v given above are for the site irrespective of the type of structure to be built on the site. Had N_a exceeded 1.1, it would have been possible to use a value of 1.1 when determining C_a, provided that all of the conditions listed in §1629.4.2 were met.

Ref. Table 16-S



Ref. Table 16-T




M-30
Active Fault Near-Source Zones
This map is intended to be used in conjunction with
the 1997 Uniform Building Code, Tables 16-S and 16-T
M-30
California Department of Conservation
Division of Mines and Geology
SR-14
Site
6 km
Lancaster
San Andreas Fault (Mojave)
Palmdale
SR-14
SR-138
San Gabriel Fault
LEGEND
See expanded legend and index map
Shaded zones are within 2 km of known seismic sources.
A fault
B fault
Contours of closest horizontal distance to known seismic sources.
5 km
10 km
15 km
0
5
10
Kilometers
1/4" is approximately equal to 1 km
August, 1997

Introduction to Vertical Irregularities §1629.5.3

Vertical irregularities are identified in Table 16-L. These can be divided into two categories. The first are dynamic force distribution irregularities. These are irregularity Types 1, 2, and 3. The second category is irregularities in load path or force transfer, and these are Types 4 and 5. The five vertical irregularities are as follows:

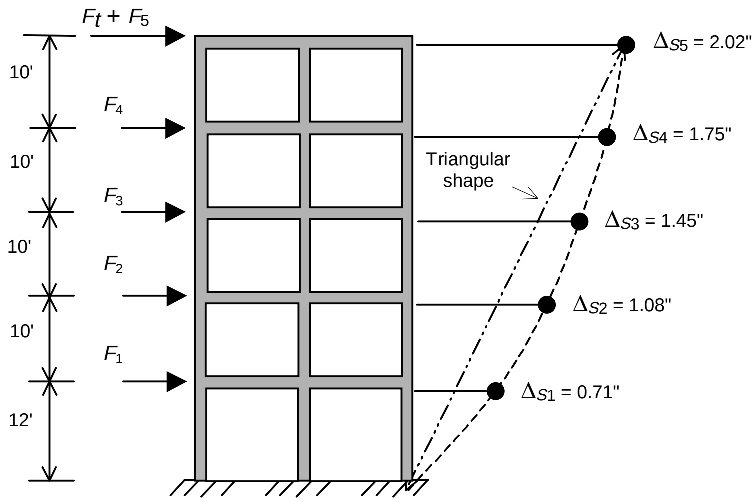
1. Stiffness irregularity-soft story
2. Weight (mass) irregularity
3. Vertical geometric irregularity
4. In-plane discontinuity in vertical lateral-force resisting element
5. Discontinuity in capacity-weak story

The first category, dynamic force distribution irregularities, requires that the distribution of lateral forces be determined by combined dynamic modes of vibration. For regular structures without abrupt changes in stiffness or mass (i.e., structures without “vertical structural irregularities”), this shape can be assumed to be linearly-varying or a triangular shape as represented by the code force distribution pattern. However, for irregular structures, the pattern can be significantly different and must be determined by the combined mode shapes from the dynamic analysis procedure of §1631. The designer may opt to go directly to the dynamic analysis procedure and thereby bypass the checks for vertical irregularity Types 1, 2, and 3.

Regular structures are assumed to have a reasonably uniform distribution of inelastic behavior in elements throughout the lateral force resisting system. When vertical irregularity Types 4 and 5 exist, there is the possibility of having localized concentrations of excessive inelastic deformations due to the irregular load path or weak story. In this case, the code prescribes additional strengthening to correct the deficiencies.

Example 4
Vertical Irregularity Type 1 §1629.5.3

A five-story concrete special moment-resisting frame is shown below. The specified lateral forces F_x from Equations (30-14) and (30-15) have been applied and the corresponding floor level displacements Δ_x at the floor center of mass have been found and are shown below.



1. **Determine if a Type 1 vertical irregularity—stiffness irregularity-soft story—exists in the first story.**

Calculations and Discussion *Code Reference*

1. **To determine if this is a Type 1 vertical irregularity—stiffness irregularity-soft story—here are two tests:**

1. The story stiffness is less than 70 percent of that of the story above.
2. The story stiffness is less than 80 percent of the average stiffness of the three stories above.

If the stiffness of the story meets at least one of the above two criteria, the structure is considered to have a soft story, and a dynamic analysis is generally required under §1629.8.4 Item 2, unless the irregular structure is not more than five stories or 65-feet in height (see §1629.8.3 Item 3).

The definition of soft story in the code compares values of the lateral stiffness of individual stories. Generally, it is not practical to use stiffness properties unless these can be easily determined. There are many structural configurations where the evaluation of story stiffness is complex and is often not an available output from computer programs. Recognizing that the basic intent of this irregularity check is to determine if the lateral force distribution will differ significantly from the linear pattern prescribed by Equation (30-15), which assumes a triangular shape for the first dynamic mode of response, this type of irregularity can also be determined by comparing values of lateral story displacements or drift ratios due to the prescribed lateral forces. This deformation comparison may even be more effective than the stiffness comparison because the shape of the first mode shape is often closely approximated by the structure displacements due to the specified triangular load pattern. Floor level displacements and corresponding story drift ratios are directly available from computer programs. To compare displacements rather than stiffness, it is necessary to use the reciprocal of the limiting percentage ratios of 70 and 80 percent as they apply to story stiffness, or reverse their applicability to the story or stories above. The following example shows this equivalent use of the displacement properties.

From the given displacements, story drifts and the story drift ratio values are determined. The story drift ratio is the story drift divided by the story height. These will be used for the required comparisons, since these better represent the changes in the slope of the mode shape when there are significant differences in interstory heights. (Note: story displacements can be used if the story heights are nearly equal.)

In terms of the calculated story drift ratios, the soft story occurs when one of the following conditions exists:

1. When 70 percent of $\dfrac{\Delta_{S1}}{h_1}$ exceeds $\dfrac{\Delta_{S2}-\Delta_{S1}}{h_2}$

   or

2. When 80 percent of $\dfrac{\Delta_{S1}}{h_1}$ exceeds

$$\frac{1}{3}\left[\frac{(\Delta_{S2}-\Delta_{S1})}{h_2}+\frac{(\Delta_{S3}-\Delta_{S2})}{h_3}+\frac{(\Delta_{S4}-\Delta_{S3})}{h_4}\right]$$

The story drift ratios are determined as follows:

$$\frac{\Delta_{S1}}{h_1}=\frac{(0.71-0)}{144}=0.00493$$

$$\frac{\Delta_{S2}-\Delta_{S1}}{h_2}=\frac{(1.08-0.71)}{120}=0.00308$$

$$\frac{\Delta_{S3}-\Delta_{S2}}{h_3}=\frac{(1.45-1.08)}{120}=0.00308$$

$$\frac{\Delta_{S4} - \Delta_{S3}}{h_4} = \frac{(1.75 - 1.45)}{120} = 0.00250$$

$$\frac{1}{3}(0.00308 + 0.00308 + 0.00250) = 0.00289$$

Checking the 70 percent requirement:

$$0.70\left(\frac{\Delta_{S1}}{h_1}\right) = 0.70\,(0.00493) = 0.00345 > 0.00308$$

$\therefore$ Soft story exists

Checking the 80 percent requirement:

$$0.80\left(\frac{\Delta_{S1}}{h_1}\right) = 0.80\,(0.00493) = 0.00394 > 0.00289$$

$\therefore$ Soft story exists

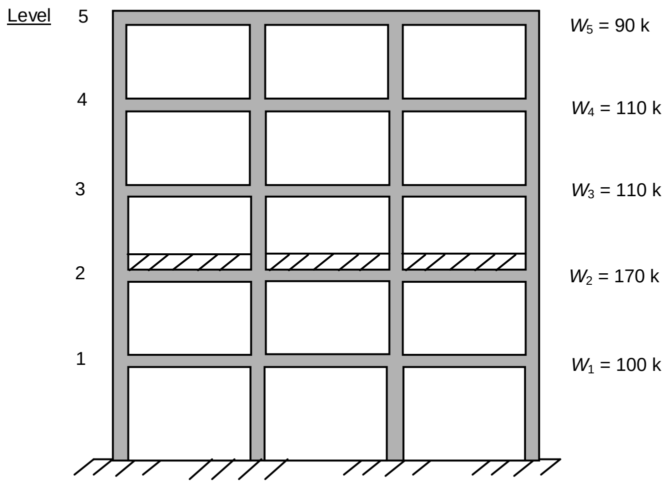
Commentary

Section 1630.10.1 requires that story drifts be computed using the maximum inelastic response displacements Δ_M. However, for the purpose of the story drift, or story drift ratio, comparisons needed for soft story determination, the displacement Δ_S due to the design seismic forces can be used as done in this example. In the example above, only the first story was checked for possible soft story vertical irregularity. In practice, all stories must be checked, unless a dynamic analysis is performed. It is often convenient to create a table as shown below to facilitate this exercise.

Level	Story Displacement	Story Drift	Story Drift Ratio	.7x (Story Drift Ratio)	.8x (Story Drift Ratio)	Avg. of Story Drift Ratio of Next 3 Stories	Soft Story Status
5	2.02 in.	0.27 in.	0.00225	0.00158	0.00180	—	No
4	1.75	0.30	0.00250	0.00175	0.00200	—	No
3	1.45	0.37	0.00308	0.00216	0.00246	—	No
2	1.08	0.37	0.00308	0.00216	0.00246	0.00261	No
1	0.71	0.71	0.00493	0.00345	0.00394	0.00289	Yes

Example 5
Vertical Irregularity Type 2 §1629.5.3

The five-story special moment frame office building has a heavy utility equipment installation at Level 2. This results in the floor weight distribution shown below:



1. **Determine if there is a Type 2 vertical weight (mass) irregularity.**

Calculations and Discussion — Code Reference

A weight, or mass, vertical irregularity is considered to exist when the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. However, this requirement does not apply to the roof if the roof is lighter than the floor below.

Checking the effective mass of Level 2 against the effective mass of Levels 1 and 3

At Level 1

$$1.5 \times W_1 = 1.5(100\text{ k}) = 150\text{ k}$$

At Level 3

$$1.5 \times W_3 = 1.5(110\text{ k}) = 165\text{ k}$$

$$W_2 = 170\text{ k} > 150\text{ k}$$

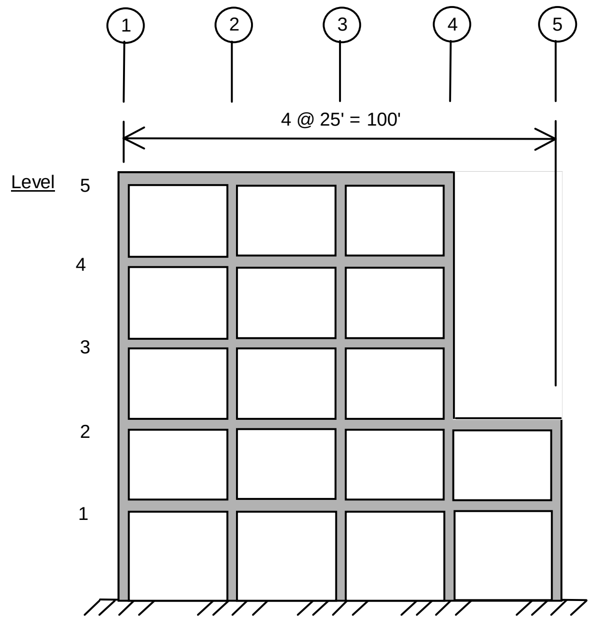
$\therefore$ $\underline{\underline{\text{Weight irregularity exists}}}$

Commentary

As in the case of vertical irregularity Type 1, this type of irregularity also results in a primary mode shape that can be substantially different from the triangular shape and lateral load distribution given by Equation (30-15). Consequently, the appropriate load distribution must be determined by the dynamic analysis procedure of §1631, unless the irregular structure is not more than five stories or 65 feet in height (see §1629.8.3 Item 3).

Example 6
Vertical Irregularity Type 3 §1629.5.3

The lateral force-resisting system of the five-story special moment frame building shown below has a 25-foot setback at the third, fourth and fifth stories.



1. **Determine if a Type 3 vertical irregularity, vertical geometric irregularity, exists.**

Calculations and Discussion *Code Reference*

A vertical geometric irregularity is considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in the adjacent story. One-story penthouses are not subject to this requirement.

In this example, the setback of Level 3 must be checked. The ratios of the two levels is

$$\frac{\text{Width of Level 2}}{\text{Width of Level 3}} = \frac{(100')}{(75')} = 1.33$$

133 percent > 130 percent

$\therefore$ <u>Vertical geometric irregularity exists</u>

Commentary

The more than 130 percent change in width of the lateral force-resisting system between adjacent stories could result in a primary mode shape that is substantially different from the triangular shape assumed for Equation (30-15). If the change is a decrease in width of the upper adjacent story (the usual situation), the mode shape difference can be mitigated by designing for an increased stiffness in the story with a reduced width.

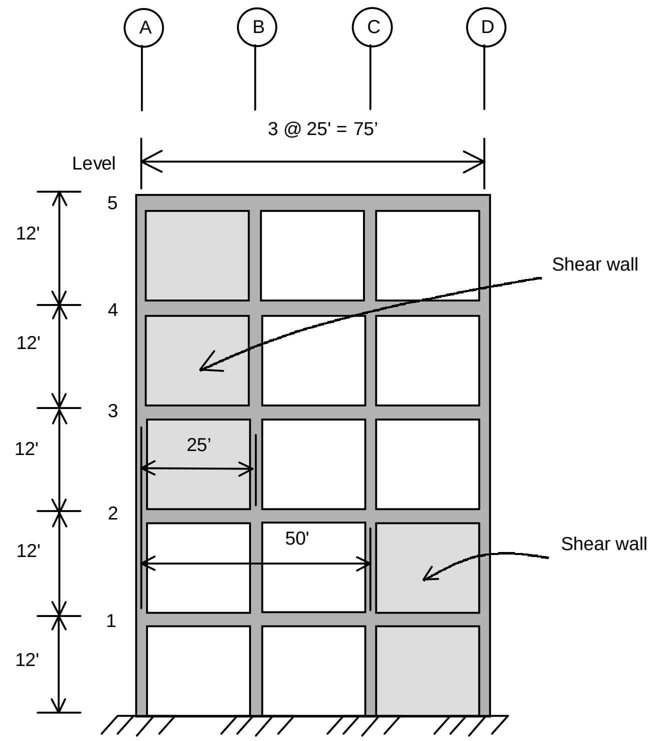
Similarly, if the width decrease is in the lower adjacent story (the unusual situation), the Type 1 soft story irregularity can be avoided by a proportional increase in the stiffness of the lower story. However, when the width decrease is in the lower story, there could be an overturning moment load transfer discontinuity that would require the application of §1630.8.2.

When there is a large decrease in the width of the structure above the first story along with a corresponding large change in story stiffness that creates a flexible tower, then §1629.8.3, Item 4 and §1630.4.2, Item 2 may apply.

Note that if the frame elements in the bay between lines 4 and 5 were not included as a part of the designated lateral force resisting system, then the vertical geometric irregularity would not exist. However, the effects of this adjoining frame would have to be considered under the adjoining rigid elements requirements of §1633.2.4.1.

Example 7
Vertical Irregularity Type 4 §1629.5.3

A concrete building has the building frame system shown below. The shear wall between Lines A and B has an in-plane offset from the shear wall between Lines C and D.



1. **Determine if there is a Type 4 vertical irregularity, in-plane discontinuity in the vertical lateral force-resisting element.**

Calculations and Discussion Code Reference

A Type 4 vertical irregularity exists when there is an in-plane offset of the lateral load resisting elements greater than the length of those elements. In this example, the left side of the upper shear wall (between lines A and B) is offset 50 feet from the left side of the lower shear wall (between lines C and D). This 50-foot offset is greater than the 25-foot length of the offset wall elements.

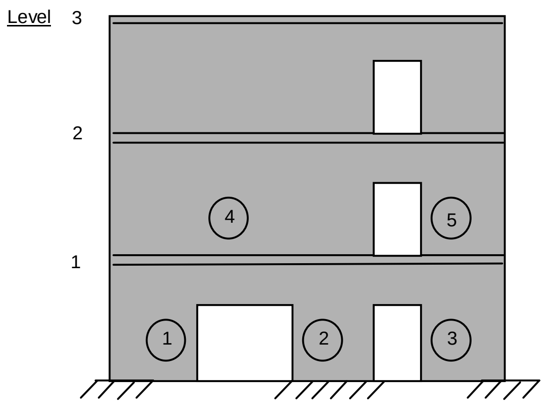
$$\therefore \underline{\underline{\text{In-plane discontinuity exists}}}$$

Commentary

The intent of this irregularity check is to provide correction of force transfer or load path deficiencies. It should be noted that any in-plane offset, even those less or equal to the length or bay width of the resisting element, can result in an overturning moment load transfer discontinuity that requires the application of §1630.8.2. When the offset exceeds the length of the resisting element, there is also a shear transfer discontinuity that requires application of §1633.2.6 for the strength of collector elements along the offset. In this example, the columns under wall A-B are subject to the provisions of §1630.8.2 and §1921.4.4.5, and the collector element between Lines B and C at Level 2 is subject to the provisions of §1633.2.6.

Example 8
Vertical Irregularity Type 5 §1629.5.3

A concrete bearing wall building has the typical transverse shear wall configuration shown below. All walls in this direction are identical, and the individual piers have the shear contribution given below. V_n is the nominal shear strength calculated in accordance with §1921.6.5, and V_m is the shear corresponding to the development of the nominal flexure strength calculated in accordance with §1921.6.6.



Pier	V_n	V_m
1	20 k	30 k
2	30	40
3	15	10
4	80	120
5	15	10

1. **Determine if a Type 5 vertical irregularity, discontinuity in capacity – weak story, condition exists.**

Calculations and Discussion — Code Reference

A Type 5 weak story discontinuity in capacity exists when the story strength is less than 80 percent of that in the story above. The story strength is considered to be the total strength of all seismic force-resisting elements sharing the story shear for the direction under consideration.

Using the smaller values of V_n and V_m given for each pier, the story strengths are

First story strength = $20 + 30 + 10 = 60$ k

Second story strength = $80 + 10 = 90$ k

Check if first story strength is less than 80 percent of that of the second story:

$$60\text{k} < 0.8(90) = 72 \text{ k}$$

$\therefore$ Weak story condition exists

Commentary

This irregularity check is to detect any concentration of inelastic behavior in one supporting story that can lead to the loss of vertical load capacity. Elements subject to this check are the shear wall piers (where the shear contribution is the lower of either the shear at development of the flexural strength, or the shear strength), bracing members and their connections, and frame columns. Frame columns with weak column-strong beam conditions have a shear contribution equal to that developed when the top and bottom of the column are at flexural capacity. Where there is a strong column-weak beam condition, the column shear resistance contribution should be the shear corresponding to the development of the adjoining beam yield hinges and the column base connection capacity. In any case, the column shear contribution shall not exceed the column shear capacity.

Because a weak story is prohibited (under §1629.9.1) for structures greater than two stories or 30 feet in height, the first story piers in this example must either be strengthened by a factor of 72/60 = 1.2, or designed for Ω_o times the forces prescribed in §1630.

Example 9
Vertical Irregularity Type 5 §1629.5.3

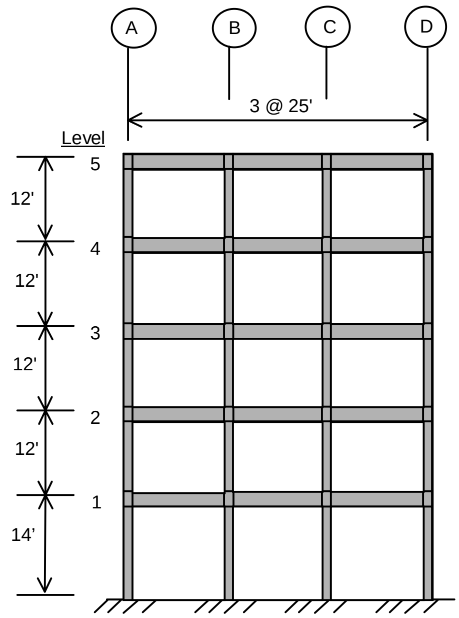
A four-story building has a steel special moment resisting frame (SMRF). The frame consists of W24 beams and W14 columns with the following member strength properties (determined under 2213.4.2 and 2213.7.5):

Beams at Levels 1 and 2:
$M_b = ZF_y = 250$ kip-ft

Columns on lines A, B, C, and D at both levels:
$M_c = Z(F_y - f_a) = 200$ kip-ft at axial loading of $1.2P_D + 0.5P_L$.

Column base connections at grade:
$M_f = 100$ kip-ft

In addition, the columns meet the exception of §2213.7.5 such that a strong beam-weak column condition is permitted.



Determine if a Type 5 vertical irregularity—discontinuity in capacity-weak story—condition exists in the first story:

1. **Determine first story strength.**

2. **Determine second story strength.**

3. **Determine if weak story exists at first story.**

Calculations and Discussion — Code Reference

A Type 5 weak story discontinuity in capacity exists when the story strength is less than 80 percent of that of the story above. The story strength is considered to be the total strength of all seismic force-resisting elements that share the story shear for the direction under consideration.

To determine if a weak story exists in the first story, the sums of the column shears in the first and second stories—when the member moment capacities are developed by lateral loading—must be determined and compared.

In this example, it is assumed that the beam moments at a beam-column joint are distributed equally to the sections of the columns directly above and below the joint. Given below is the calculations for first and second stories.

1. Determine first story strength.

Columns A and D must be checked for strong column-weak beam considerations.

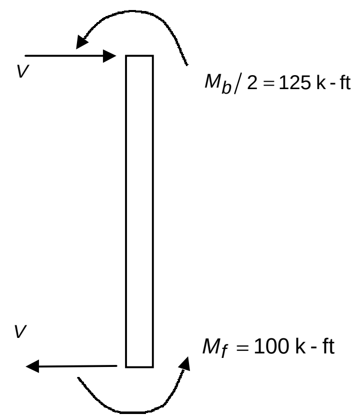
$$2M_c = 400 > M_b = 250$$

$\therefore$ strong column-weak beam condition exists.

Next, the shear in each column must be determined.

Clear height = 14 ft − 2 ft = 12 ft

$$V_A = V_D = \frac{125 + 100}{12} = 18.75 \text{ k}$$



Checking columns B and C for strong column-weak beam considerations.

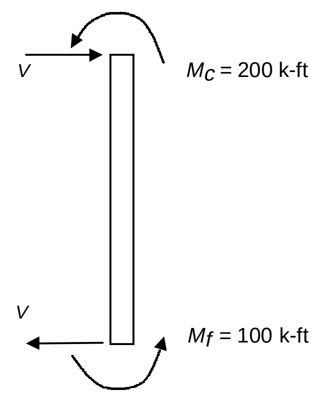
$$2M_c = 400 < 2M_b = 500$$

$\therefore$ Strong beam-weak column condition exists.

Next, the shear in each column must be determined.

Clear height = 14 ft − 2 ft = 12 ft

$$V_B = V_C = \frac{200 + 100}{12} = 25.0 \text{ k}$$



First story strength = $V_A + V_B + V_C + V_D = 2(18.75) + 2(25.0) = \underline{\underline{87.5 \text{ k}}}$

2. Determine second story strength.

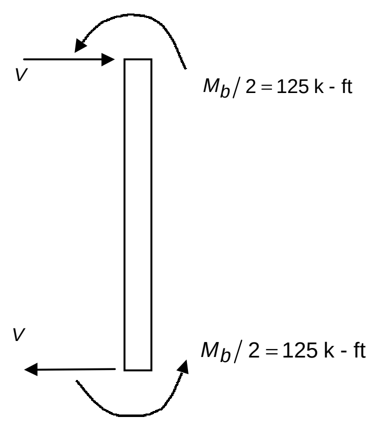
Columns A and D must be checked for strong column-weak beam at Level 2.

$$2M_c = 400 > M_b = 250$$

$\therefore$ strong column-weak beam condition exists.

Clear height = $12\text{ ft} - 2\text{ ft} = 10\text{ ft}$

$$V_A = V_D = \frac{125 + 125}{10} = 25.0\text{ k}$$



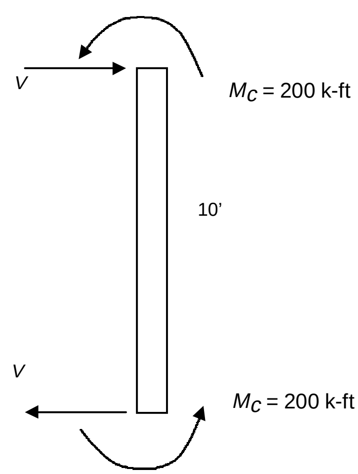
Checking columns B and C for strong column-weak beam considerations.

$$2M_c = 400 < 2M_b = 500$$

$\therefore$ Strong beam-weak column condition exists.

Clear height = $12\text{ ft} - 2\text{ ft} = 10\text{ ft}$

$$V_B = V_C = \frac{200 + 200}{10} = 40.0\text{ k}$$



Second story strength = $V_A + V_B + V_C + V_D = 2(25.0) + 2(40.0) = \underline{\underline{130.0\text{ k}}}$

3. **Determine if weak story exists at first story.**

First story strength = 87.5 k

Second story strength = 130.0 k

$$87.5 < 0.80(130) = 104$$ Table 16-L Item 5

$\therefore$ $\underline{\underline{\text{Weak story condition in first story exists}}}$

Introduction to Plan Irregularities §1629.5.3

Plan structural irregularities are identified in Table 16-M. There are five types of plan irregularities:

1. Torsional irregularity—to be considered when diaphragms are not flexible
2. Re-entrant corners
3. Diaphragm discontinuity
4. Out-of-plane offsets
5. Nonparallel systems

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1, 2, 3, and 5 are special response conditions:

Type 1. When the ratio of maximum drift to average drift exceeds the given limit, there is the potential for an unbalance in the inelastic deformation demands at the two extreme sides of a story. As a consequence, the equivalent stiffness of the side having maximum deformation will be reduced, and the eccentricity between the centers of mass and rigidity will be increased along with the corresponding torsions. An amplification factor A_x is to be applied to the accidental eccentricity to represent the effects of this unbalanced stiffness.

Type 2. The opening and closing deformation response or flapping action of the projecting legs of the building plan adjacent to re-entrant corners can result in concentrated forces at the corner point. Elements must be provided to transfer these forces into the diaphragms.

Type 3. Excessive openings in a diaphragm can result in a flexible diaphragm response along with force concentrations and load path deficiencies at the boundaries of the openings. Elements must be provided to transfer the forces into the diaphragm and the structural system.

Type 4. The Type 4 plan irregularity, out-of-plane offset, represents the irregular load path category. In this case, shears and overturning moments must be transferred from the level above the offset to the level below the offset, and there is a horizontal "offset" in the load path for the shears.

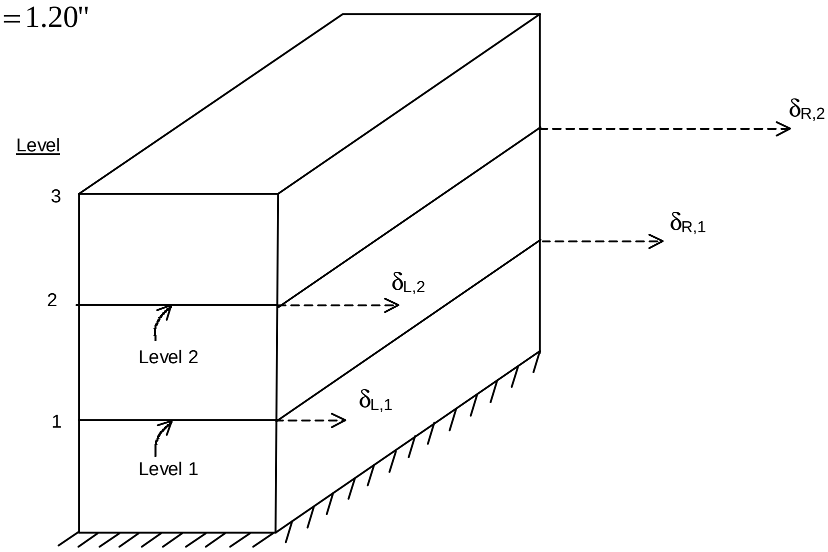
Type 5. The response deformations and load patterns on a system with nonparallel lateral force-resisting elements can have significant differences from that of a regular system. Further analysis of deformation and load behavior may be necessary.

Example 10
Plan Irregularity Type 1 §1629.5.3

A three-story special moment resisting frame building has rigid floor diaphragms. Under specified seismic forces, including the effects of accidental torsion, it has the following displacements at Levels 1 and 2:

$\delta_{L,2} = 1.30"$ $\delta_{R,2} = 1.90"$

$\delta_{L,1} = 1.00"$ $\delta_{R,1} = 1.20"$



1. **Determine if a Type 1 torsional irregularity exists at the second story.**

If it does:

2. **Compute the torsional amplification factor A_x for Level 2.**

Calculations and Discussion — Code Reference

A Type 1 torsional plan irregularity is considered to exist when the maximum story drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.

1. **Determine if a Type 1 torsional irregularity exists at the second story.** **Table 16-M**

Referring to the above figure showing the displacements δ due to the prescribed lateral forces, this irregularity check is defined in terms of story drift $\Delta\delta_X = (\delta_X - \delta_{X-1})$ at ends R (right) and L (left) of the structure. Torsional irregularity exists at level x when

$$\Delta_{max} = \Delta_{R,X} > \frac{1.2\left(\Delta_{R,x} + \Delta_{L,x}\right)}{2} = 1.2\left(\Delta_{avg}\right)$$

where

$$\Delta\delta_{L,2} = \delta_{L,2} - \delta_{L,1}$$

$$\Delta\delta_{R,2} = \delta_{R,2} - \delta_{R,1}$$

$$\Delta\delta_{max} = \Delta\delta_{R,X}\,, \quad \Delta\delta_{avg} = \frac{\Delta\delta_{L,X} + \Delta\delta_{R,X}}{2}$$

Determining story drifts at Level 2

$$\Delta_{L,2} = 1.30 - 1.00 = 0.30 \text{ in.}$$

$$\Delta_{R,2} = 1.90 - 1.20 = 0.70 \text{ in.}$$

$$\Delta_{avg} = \frac{0.30 + 0.70}{2} = 0.50 \text{ in.}$$

Checking 1.2 criteria

$$\frac{\Delta_{max}}{\Delta_{avg}} = \frac{\Delta_{R,2}}{\Delta_{avg}} = \frac{0.7}{0.5} = 1.4 > 1.2$$

$$\therefore \underline{\underline{\text{Torsional irregularity exists}}}$$

2. Compute amplification factor A_x for Level 2. **§1630.7**

When torsional irregularity exists at a level x, the accidental eccentricity, equal to 5 percent of the building dimension, must be increased by an amplification factor A_x. This must be done for each level, and each level may have a different A_x value. In this example, A_x is computed for Level 2.

$$A_x = \left(\frac{\delta_{max}}{1.2\ {}_{avg}}\right)^2 \qquad \text{(30-16)}$$

$$\delta_{max} = \delta_{R,2} = 1.90 \text{ in.}$$

$$\delta_{avg} = \frac{\delta_{L,2} + \delta_{R,2}}{2} = \frac{1.30 + 1.90}{2} = 1.60 \text{ in.}$$

$$A_2 = \left(\frac{1.90}{1.2\,(1.60)} \right)^2 = 0.98 < 1.0$$

$$\therefore \text{ use } \underline{\underline{A_x = 1.0}}$$

Commentary

In §1630.7, there is the provision that "the most severe load combination must be considered." The interpretation of this for the case of the story drift and displacements to be used for the average values $\Delta\delta_{avg}$ and δ_{avg} is as follows. The most severe condition is when both $\delta_{R,X}$ and $\delta_{L,X}$ are computed for the same accidental center of mass displacement that causes the maximum displacement δ_{max}. For the condition shown in this example where $\delta_{R,X} = \delta_{max}$, the centers of mass at all levels should be displaced by the accidental eccentricity to the right side R, and both $\delta_{R,X}$ and $\delta_{L,X}$ should be evaluated for this load condition.

While Table 16-M calls only for §1633.2.9, Item 6 (regarding diaphragm connections) to apply if this irregularity exists, there is also §1630.7, which requires the accidental torsion amplification factor A_x given by Equation (30-16). It is important to recognize that torsional irregularity is defined in terms of story drift $\Delta\delta_X$ while the evaluation of A_x by Equation (30-16) is in terms of displacements δ_X. There can be instances where the story drift values indicate torsional irregularity and where the related displacement values produce an A_x value less than one. This result is not the intent of the provision, and the value of A_x used to determine accidental torsion should not be less than 1.0.

The displacement and story drift values should be obtained by the equivalent lateral force method with the specified lateral forces. Theoretically, if the dynamic analysis procedure were to be used, the values of $\Delta\delta_{max}$ and $\Delta\delta_{avg}$ would have to be found for each dynamic mode, then combined by the appropriate SRSS or CQC procedures, and then scaled to the specified base shear. However, in view of the complexity of this determination and the judgmental nature of the 1.2 factor, it is reasoned that the equivalent static force method is sufficiently accurate to detect torsional irregularity and evaluate the A_x factor.

If the dynamic analysis procedure is either elected or required, then §1631.3 requires the use of a three-dimensional model if there are any of the plan irregularities listed in Table 16-M.

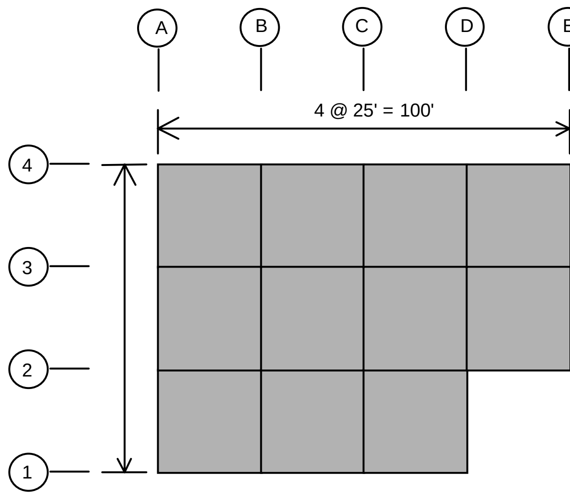
For cases of large eccentricity and low torsional rigidity, the static force procedure can result in a negative displacement on one side and a positive on the other. For example, this occurs if $\delta_{L,3} = -0.40''$ and $\delta_{R,3} = 1.80''$. The value of δ_{avg} in Equation (30-16) should be calculated as the algebraic average:

$$\delta_{avg} = \frac{\delta_{L,3} + \delta_{R,3}}{2} = \frac{(-0.40) + 1.80}{2} = \frac{1.40}{2} = 0.70 \text{ in.}$$

When dynamic analysis is used, the algebraic average value δ_{avg} should be found for each mode, and the individual modal results must be properly combined to determine the total response value for δ_{avg}.

Example 11
Plan Irregularity Type 2 §1629.5.3

The plan configuration of a ten-story special moment frame building is as shown below:



1. **Determine if there is a Type 2 re-entrant corner irregularity.**

Calculations and Discussion — Code Reference

A Type 2 re-entrant corner plan irregularity exists when the plan configuration of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the direction considered.

The plan configuration of this building, and its lateral force-resisting system, have identical re-entrant corner dimensions. For the sides on Lines 1 and 4, the projection beyond the re-entrant corner is

$$100\,\text{ft} - 75\,\text{ft} = 25\,\text{ft}$$

This is $\frac{25}{100}$ or 25 percent of the 100 ft plan dimension.

For the sides on Lines A and E, the projection is

$$60\,\text{ft} - 40\,\text{ft} = 20\,\text{ft}$$

This is $\frac{20}{60}$ or 33.3 percent of the 60 ft plan dimension.

Since both projections exceed 15 percent, there is a re-entrant corner irregularity.

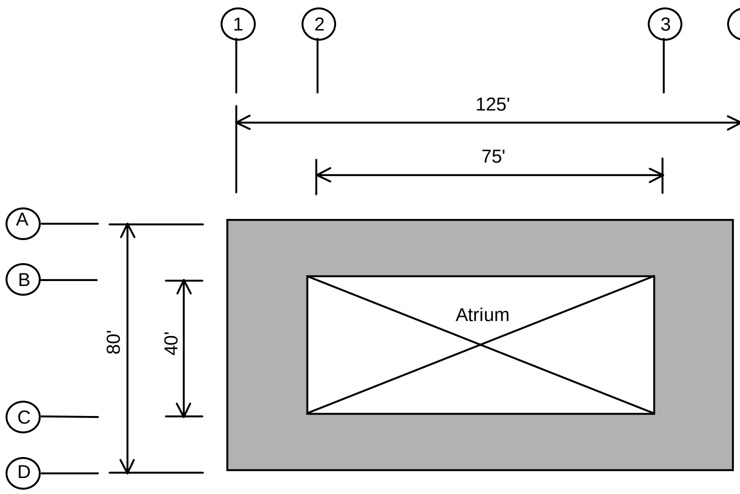
$$\therefore \underline{\underline{\text{Re - entrant corner irregularity exists}}}$$

Commentary

Whenever the Type 2 re-entrant corner plan irregularity exists, see the diaphragm requirements of §1633.2.9 Items 6 and 7.

Example 12
Plan Irregularity Type 3 *§1629.5.3*

A five-story concrete building has a bearing wall system located around the perimeter of the building. Lateral forces are resisted by the bearing walls acting as shear walls. The floor plan of the second floor of the building is shown below. The symmetrically placed open area in the diaphragm is for an atrium, and has dimensions of 40 ft x 75 ft. All diaphragms above the second floor are without significant openings.



Second floor pla

1. **Determine if a Type 3 diaphragm discontinuity exists at the second floor level.**

Calculations and Discussion *Code Reference*

A Type 3 diaphragm discontinuity irregularity exists when diaphragms have abrupt discontinuities or variations in stiffness, including cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.

Gross enclosed area of the diaphragm is $80\text{ ft} \times 125\text{ ft} = 10{,}000$ sq ft

Area of opening is $40' \times 75' = 3{,}000$ sq ft

50 percent of gross area = $0.5(10{,}000) = 5{,}000$ sq ft

$3{,}000 < 5{,}000$ sq ft

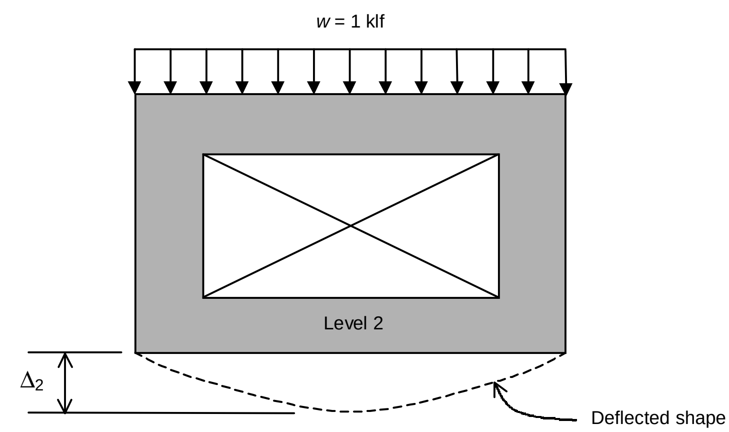
$\therefore$ No diaphragm discontinuity irregularity exists

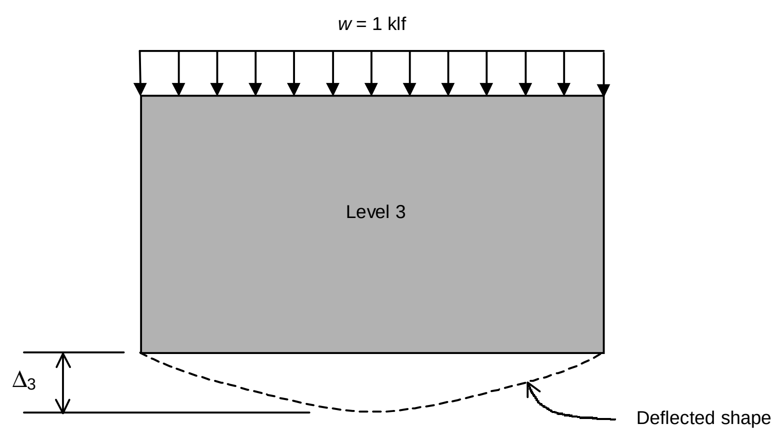
Commentary

The stiffness of the second floor diaphragm with its opening must be compared with the stiffness of the solid diaphragm at the third floor. If the change in stiffness exceeds 50 percent, then a diaphragm discontinuity irregularity exists for the structure.

This comparison can be performed as follows:

Find the simple beam mid-span deflections Δ_2 and Δ_3 for the diaphragms at Levels 2 and 3, respectively, due to a common distributed load w, such as 1 klf.

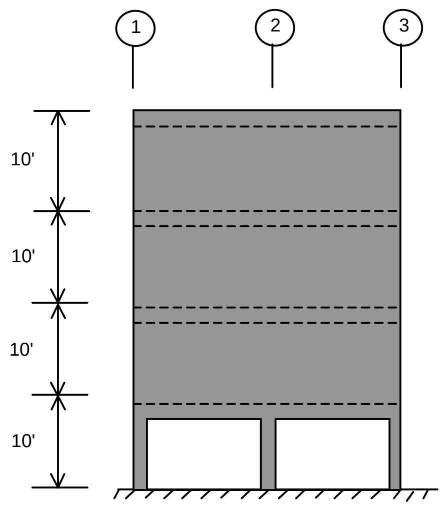




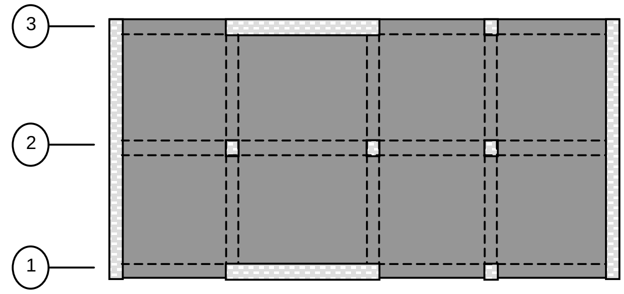
If $\Delta_2 > 1.5\Delta_3$, then there is diaphragm discontinuity.

Example 13
Plan Irregularity Type 4 §1629.5.3

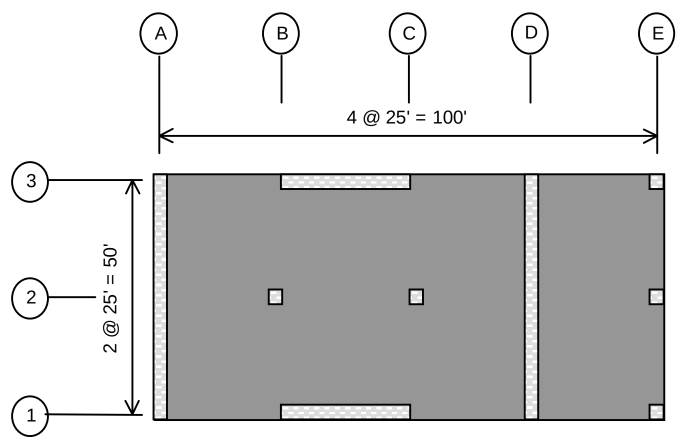
A four-story building has a concrete shear wall lateral force-resisting system in a building frame system configuration. The plan configuration of the shear walls is shown below.



Elevation Line E



Typical floor plan



Ground (first) floor pla

1. **Determine if there is a Type 4 out-of-plane offset plan irregularity between the first and second stories.**

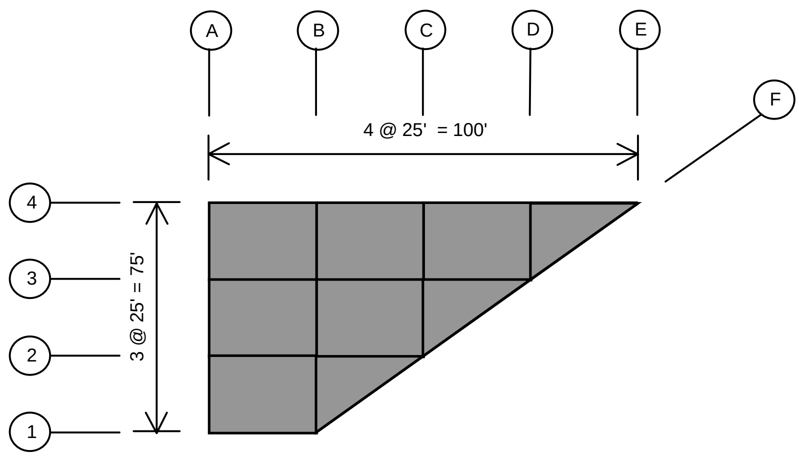
Calculations and Discussion *Code Reference*

An out-of-plane offset plan irregularity exists when there are discontinuities in a lateral force path, for example: out-of-plane offsets of vertical resisting elements such as shear walls. The first story shear wall on Line D has 25 ft out-of-plane offset to the shear wall on Line E at the second story and above. This constitutes an out-of-plane offset irregularity, and the referenced sections in Table 16-M apply to the design.

$\therefore$ Offset irregularity exists

Example 14
Plan Irregularity Type 5 §1629.5.3

A ten-story building has the floor plan shown below at all levels. Special moment resisting-frames are located on the perimeter of the building on Lines 1, 4, A, and F.



Typical floor plan

1. **Determine if a Type 5 nonparallel system irregularity exists.**

Calculations and Discussion — Code Reference

A Type 5 nonparallel system irregularity is considered to exist when the vertical lateral load resisting elements are not parallel to or symmetric about the major orthogonal axes of the building's lateral force-resisting system.

The vertical lateral force-resisting frame elements located on Line F are not parallel to the major orthogonal axes of the building (i.e., Lines 4 and A). Therefore a nonparallel system irregularity exists, and the referenced section in Table 16-M applies to the design.

∴ A nonparallel system irregularity exists

Example 15
Reliability/Redundancy Factor ρ §1630.1.1

Evaluate the reliability/redundancy factor, ρ, for the three structural systems shown below. Given information for each system includes the story shears V_i due to the design base shear V, and the corresponding element forces E_h. The ρ factor is defined as

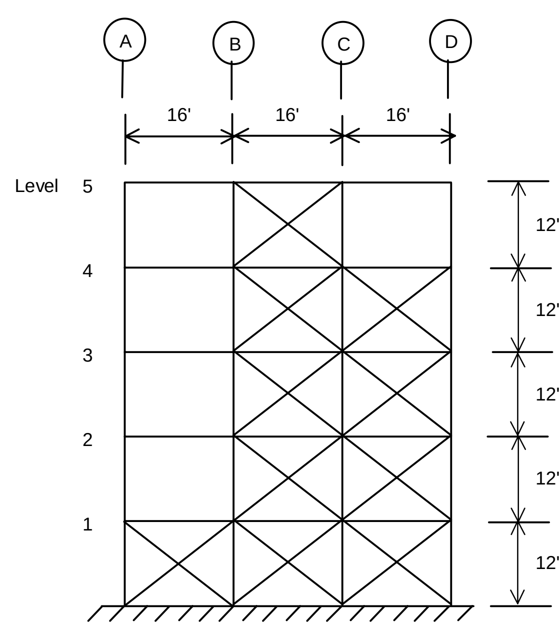
$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \tag{30-3}$$

where $r_{\max}$ is the largest of the element-story shear ratios, r_i, that occurs in any of the story levels at or below the two-thirds height level of the building; and A_B is the ground floor area of the structure in square feet. Once ρ has been determined, it is to be used in Equation (30-1) to establish the earthquake load E for each element of the lateral force-resisting system.

For purposes of this example, only the frame line with maximum seismic force is shown. In actual applications, all frame lines in a story require evaluation. The E_h forces given include any torsional effects. Note that the story shear V_i is the total of the shears in all of the frame lines in the direction considered.

Calculations and Discussion — Code Reference

1. **Braced frame structure.**



The following information is given:

Story i	Total Story Shear V_i	Brace Force E_h	Horizontal Component F_x	$r_i = F_x / V_i$
1	952 kips	273 kips	218.4 kips	0.229
2	731	292	233.6	0.320
3	517	112	89.6	0.173
4	320	91.4	73.1	0.229
5	Not required above 2/3 height level (see definition of r_i)			

$A_B = 48\text{ ft} \times 100\text{ ft} = 4{,}800\text{ sq ft}$, where 100 ft is the building width.

Horizontal component in each brace is

$$F_x = \frac{4}{5} E_h$$

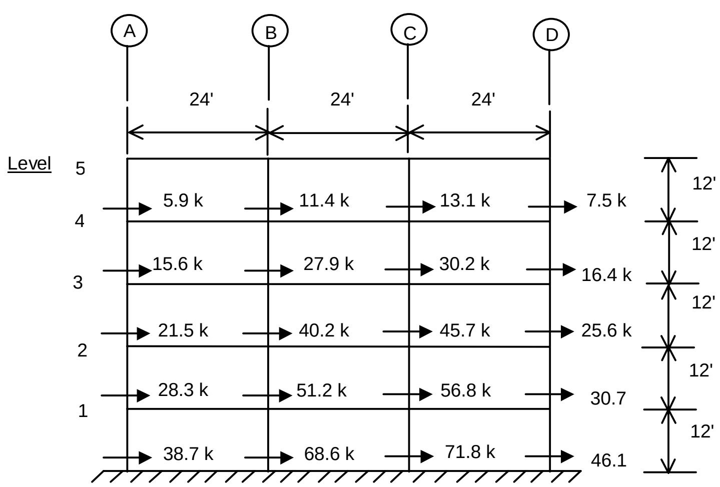
where E_h is the maximum force in a single brace element in story i.

For braced frames, the value of r_i is equal to the maximum horizontal force component F_x in a single brace element divided by the total story shear V_i.

$$r_{max} = 0.320$$

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} = 2 - \frac{20}{(0.320)\sqrt{4800}} = \underline{\underline{1.10}} \qquad (30\text{-}3)$$

2. **Moment frame structure.**



$A_B = 72' \times 120' = 8{,}640$ sq ft, where $120'$ is the building width

Column shears are given above.

$E_h = V_A, V_B, V_C, V_D$ in column lines A, B, C, D, respectively.

Column Lines B and C are common to bays on opposite sides.

For moment frames, r_i is maximum of the sum of
$V_A + 0.7V_B$, *or* $0.7\left(V_B + V_C\right)$, *or* $0.7V_C + V_D$ divided by the story shear V_i. §1630.1.1

Section 1630.1.1 requires that special moment-resisting frames have redundancy such that the calculated value of ρ does not exceed 1.25.

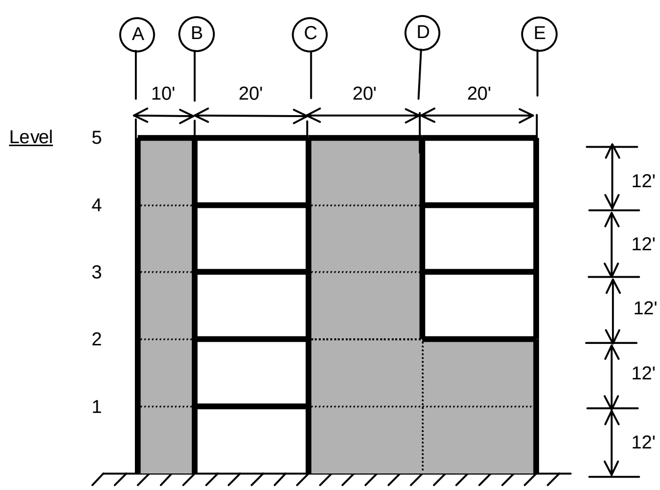
The story shears and r_i evaluations are:

Story i	V_i	$V_A + 0.7V_B$	$0.7(V_B + V_C)$	$0.7V_C + V_D$	r_i
1	388 kips	86.7 kips	98.3 kips	96.4 kips	0.253
2	306	64.1	75.6	70.5	0.247
3	228	49.6	60.1	57.6	0.264
4	151	35.1	40.7	37.5	0.270
5	Not required above 2/3 height level				

$r_{max} = r_4 = 0.270$

$$\rho = 2 - \frac{20}{(0.270)\sqrt{8640}} = \underline{\underline{1.20}} < 1.25 \quad o.k. \qquad (30\text{-}3)$$

3. **Building frame system with shear walls.**



$A_B = 70' \times 120' = 8{,}400$ sq ft., where $120'$ is the building width

E_h is the wall shear V_w

For shear walls, r_i is the maximum of $\frac{V_{wi}}{V_i}\left(\frac{10}{l_w}\right)$. The following information is given for the walls.

		Wall A-B		*Wall C-D-E and C-D*	
Story i	V_i	V_{wi}	l_{wi}	V_{wi}	l_{wi}
1	363 kips	34.1 kips	10 ft	92.4 kips	40 ft
2	288	26.9	10	75.2	40
3	208	36.3	10	69.3	20
4	105	19.7	10	39.8	20
5	Above 2/3 height level				

		A-B	*C-D-E and C-D*	
i	V_I	$\frac{V_{wi}}{V_i}\left(\frac{10}{l_w}\right)$	$\frac{V_{wi}}{V_i}\left(\frac{10}{l_w}\right)$	r_i
1	363 kips	0.094	0.064	0.094
2	288	0.093	0.065	0.093
3	208	0.175	0.167	0.175
4	105	0.188	0.190	0.190
5	Not required above 2/3 height level			

$r_{max} = r_4 = 0.190$

$$\rho = 2 - \frac{20}{(0.190)\sqrt{6000}} = 0.641 < 1.0 \qquad (30\text{-}3)$$

$\therefore$ use $\rho = \underline{\underline{1.0}}$

Commentary

A separate value of ρ must be determined for *each* principal building direction. Each value of ρ is applied to the elements of the vertical lateral force-resisting system for that direction. Note that the redundancy factor does *not* apply to horizontal diaphragms, except in the case of transfer diaphragms.

The following code provisions require the designer to provide sufficient redundancy such that ρ is less than or equal to specified values:

1. Section 1630.1.1 requires that the number of bays of special moment resisting frames be such that the value of ρ is less than or equal to 1.25.
2. Section 1629.4.2 allows that the near-source factor N_a need not exceed 1.1, if along with other stated conditions, the redundancy is such that the calculated ρ value is less than or equal to 1.00.

Example 16
Reliability/Redundancy Factor Applications §1630.1.1

The 1997 UBC introduced the concept of the reliability/redundancy factor. The intent of this provision is to penalize those lateral force-resisting systems without adequate redundancy by requiring that they be more conservatively designed. The purpose of this example is to develop approximate relationships that will enable the engineer to estimate the number of lateral force-resisting elements required to qualify for given values of the redundancy factor ρ. These relationships are particularly useful in the conceptual design phase. Note that a redundancy factor is computed for *each* principal direction and that these are *not* applied to diaphragms, with the exception of transfer diaphragms at discontinuous vertical lateral force-resisting elements.

For the following structural systems, find the approximate relation for ρ in terms of the number N of resisting elements (e.g., braces, frames, and walls).

1. **Braced frames.**

2. **Moment-resisting frames.**

3. **Shear walls.**

Calculations and Discussion Code Reference

Before developing the approximate relationships for the three structural systems, a brief discussion of methodology is presented.

For a given story level *i* with story shear V_i, the approximate number of lateral force-resisting elements N required a given value of ρ can be found as follows. The basic reliability/redundancy relationship given in §1630.1.1 is

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \qquad (30\text{-}3)$$

The term r_{max} is the maximum element-story shear ratio. This is the fraction of the total seismic shear at a given floor level that is carried by the most highly loaded element. A_B is the ground floor area of the structure in square feet.

The value of r_{max} can be approximated in terms of the story shear V_i and the number of elements *N* in the story. This is done for each system below to provide the approximate relationship for ρ.

1. Braced frames.

For a braced frame system with N_{braces} having a maximum force component H_{max} (this is the horizontal component of the maximum brace force), assume that the maximum component is 125 percent of the average. Thus

$$H_{max} = (1.25)H_{average} = (1.25)\frac{V_i}{N_{braces}}$$

$$r_{max} = \frac{H_{max}}{V_i} = \frac{1.25V_i}{N_{braces}(V_i)} = \frac{1.25}{N_{braces}}$$

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}}$$

$$\therefore \rho = 2 - \frac{20N_{braces}}{1.25\sqrt{A_B}}$$, where N_{braces} = number of braces.

2. Moment-resisting frames.

For a moment-resisting frame system with N_{bays} having a maximum shear per bay of $V_{bay,max}$, assume that the maximum component is 125 percent of the average component. Thus,

$$V_{bay,max} = (1.25)\frac{V_i}{N_{bays}}$$

$$r_{max} = \frac{V_{bay,max}}{V_i} = \frac{1.25}{N_{bays}}$$

$$\therefore \rho = 2 - \frac{20N_{bays}}{1.25\sqrt{A_B}}$$, where N_{bays} = number of bays

Note that for a SMRF, ρ shall not exceed 1.25. Thus, the number of bays of special moment-resisting frames must be increased to reduce r_{max}, such that ρ is less than or equal to 1.25. §1630.1.1

3. Shear walls.

Section 1630.1.1 requires that r_{max} be based on the number of 10-foot lengths of shear wall. For a shear wall system, let N_{10} = number of 10-foot-long wall segments in story i, and let the maximum shear per 10-foot length be $10\left(\frac{V_w}{l_w}\right)_{max}$. V_w and l_w are the shear and length for a wall pier. Assuming the maximum component is 125 percent of the average.

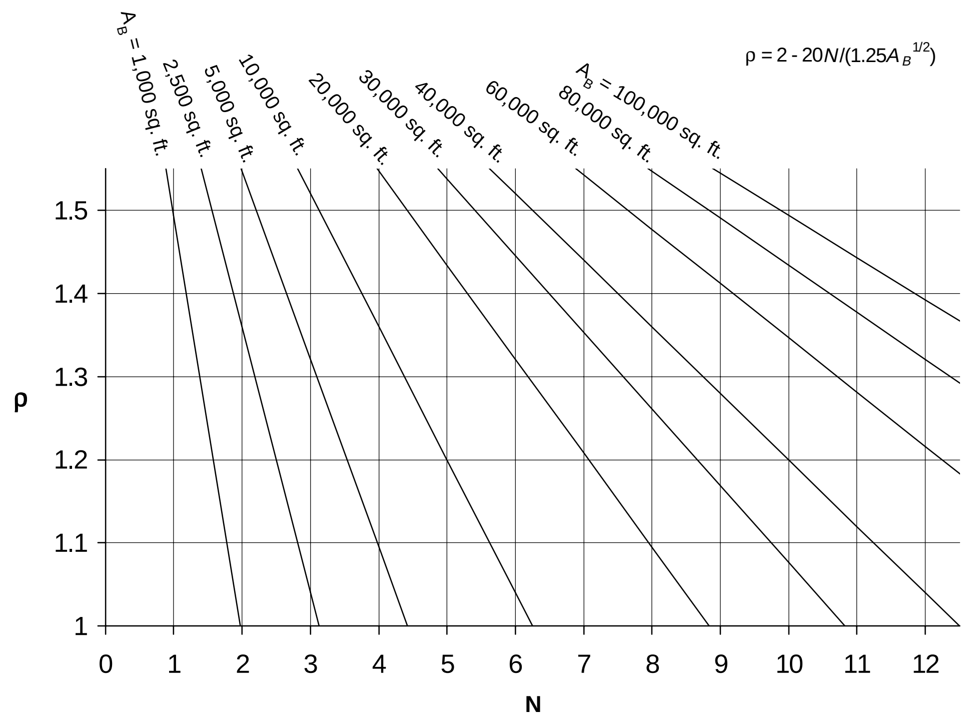
$$10\left(\frac{V_w}{l_w}\right)_{max} = (1.25)\frac{V_i}{N_{10}}$$

$$r_{max} = \frac{10\left(\frac{V_w}{l_w}\right)_{max}}{V_i} = \frac{1.25}{N_{10}}$$

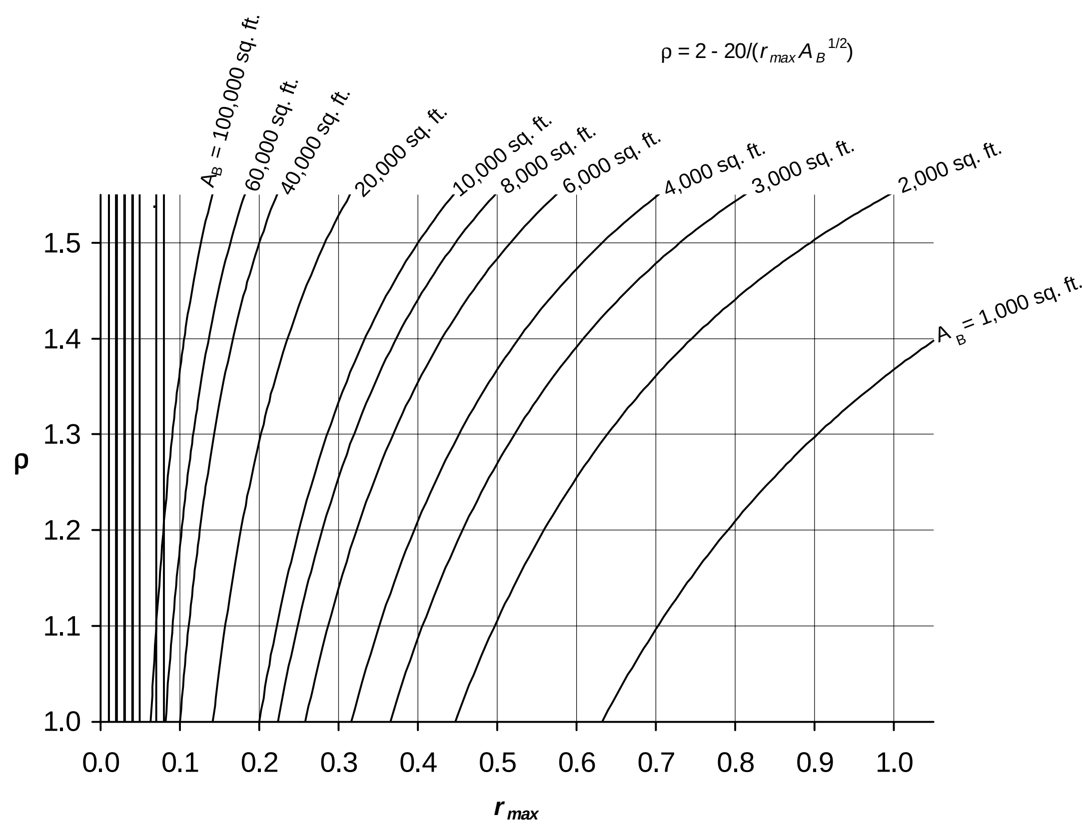
$\therefore \rho = \underline{\underline{2 - \frac{20N_{10}}{1.25\sqrt{A_B}}}}$, where N_{10} = number of 10-foot long segments of shear walls.

Commentary

Following this page is a plot of ρ versus N for the equation $\rho = 2 - \frac{20N}{1.25\sqrt{A_B}}$. This approximate relationship can be used to estimate ρ for conceptual design. Following this is a plot of $\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}}$. This is Equation (30-3) and can be used for final design.



Approximate relationship of ρ for various values of N and A_B



Reliability/redundancy factor ρ for various values of r_{max} and A_B

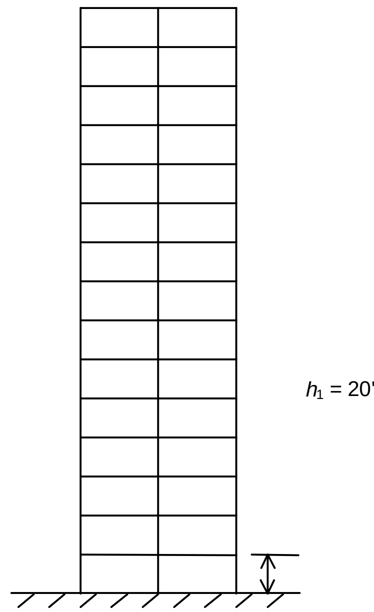
Example 17
$P\Delta$ Effects §1630.1.3

In highrise building design, important secondary moments and additional story drifts can be developed in the lateral force-resisting system by $P\Delta$ effects. $P\Delta$ effects are the result of the axial load P in a column being "moved" laterally by horizontal displacements, thereby causing additional "secondary" column and girder moments. The purpose of this example is to illustrate the procedure that must be used to check the overall stability of the frame system for such effects.

A 15-story building has a steel special moment-resisting frame (SMRF). The following information is given:

Zone 4
$R = 8.5$

At the first story,
$\Sigma D = W = 8{,}643$ kips
$\Sigma L = 3{,}850$ kips
$V_1 = V = 0.042W = 363.0$ kips
$h_1 = 20$ ft
Story drift = $\Delta_{S1} = 0.003h_1 = 0.72$ in.



Determine the following:

1. **$P\Delta$ criteria for the building.**

2. **Check the first story for $P\Delta$ requirements.**

Calculations and Discussion — Code Reference

1. $P\Delta$ criteria for the building. §1630.1.3

$P\Delta$ effects must be considered whenever the ratio of secondary moments to primary moments exceed 10 percent. As discussed in Section C105.1.3 of the 1999 SEAOC Blue Book Commentary, this ratio is defined as a stability coefficient θ:

$$\theta_x = \frac{P_x \Delta_{sx}}{V_x h_x}$$

where

θ_x = stability coefficient for story x

P_x = total vertical load (unfactored) on all columns in story x

Δ_{sx} = story drift due to the design base shear

V_x = design shear in story x

h_x = height of story x

$P\Delta$ effects must be considered when $\theta > 0.10$

An alternative approach is to check story drift.

In Seismic Zones 3 and 4, $P\Delta$ effects need not be considered for SMRF buildings whenever the story drifts satisfy the following criterion:

$$\frac{\Delta_s}{h} \le \frac{0.02}{R} = \frac{0.02}{8.5} = .00235$$ §1630.1.3

Therefore, when the story drift in a given story of an SMRF is less than or equal to .00235, $P\Delta$ effects need not be considered for that story.

2. **Check $P\Delta$ requirements for the first story.**

The first story drift ratio is

$$\frac{\Delta_{S1}}{h_1} = \frac{0.003h_1}{h_1} = 0.003$$

Check drift criteria

$$.003 > .00235$$

Section 1630.1.3 requires that the total vertical load P_1 at the first story be considered as the total dead (ΣD) plus floor live (ΣL) and snow (S) load above the first story. These loads are unfactored for determination of $P\Delta$ effects.

$$P_1 = \Sigma D + \Sigma L + S$$

using $S = 0$ for the building site

$$P_1 = 8{,}643 + 3{,}850 = 12{,}493 \text{ kips}$$

$$\theta_1 = \frac{P_1\Delta_{S1}}{V_1 h_1} = \frac{(12{,}493)(0.003h_1)}{(363.0)h_1} = 0.103 > 0.100$$

$\therefore$ $\underline{\underline{P\Delta \text{ effects must be considered}}}$

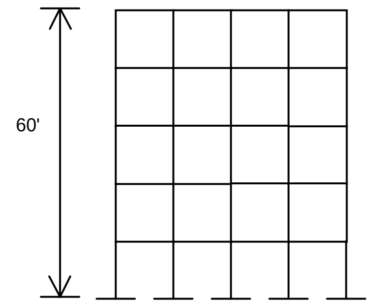
Commentary

The 1999 SEAOC Blue Book Commentary, in Section C105.1.3, provides an acceptable $P\Delta$ analysis: for any story x where $P\Delta$ effects must be considered, the story shear V_x must be multiplied by a factor $(1+a_d)$, where $a_d = \dfrac{\theta}{1-\theta}$, and the structure is to be re-analyzed for the seismic force effects corresponding to the augmented story shears. Also, some computer programs include the option to include $P\Delta$ effects. The user should verify that the particular method is consistent with the requirements of this §1630.1.3.

Example 18
Design Base Shear §1630.2.1

Find the design base shear for a 5-story steel special moment-resisting frame building shown below, given the following information:

Z = 0.4
Seismic source type = B
Distance to seismic source = 5 km
Soil profile type = S_C
I = 1.0
R = 8.5
W = 1,626 kips



In solving this example, the following steps are followed:

1. **Determine the structure period.**

2. **Determine the seismic coefficients C_a and C_v.**

3. **Determine base shear.**

Calculations and Discussion — Code Reference

1. **Determine the structure period.** **§1630.2.2**

Method A to be used. C_t for steel moment-resisting frames is 0.035.

$$T = C_t (h_n)^{3/4} = .035 (60)^{3/4} = \underline{\underline{.75 \text{ sec}}}. \qquad (30\text{-}8)$$

2. **Determine the seismic coefficients C_a and C_v** **§1628**

From Table 16-Q for soil profile type S_C and Z = .4

$$C_a = .40 N_a$$

From Table 16-R for soil profile type S_C and Z = .4

$$C_v = .56 N_v$$

Find N_a and N_v from Tables 16-S and 16-T, respectively, knowing that the seismic source type is B and the distance 5 km.

$$N_a = 1.0$$

$$N_v = 1.2$$

Therefore

$$C_a = .40\,(1.0) = \underline{\underline{.40}}$$

$$C_v = .56\,(1.2) = \underline{\underline{.672}}$$

3. **Determine base shear.** **§1630.2.1**

The total base shear in a given direction is determined from:

$$V = \frac{C_v I}{RT} W = \frac{.672 \times 1.0}{8.5 \times .75} \times 1{,}626 = 171.4 \text{ kips} \quad (30\text{-}4)$$

However, the code indicates that the total design base shear need not exceed:

$$V = \frac{2.5 C_a I}{R} W = \frac{2.5 \times 0.4 \times 1.0}{8.5} \times 1{,}626 = 191.3 \text{ kips} \quad (30\text{-}5)$$

Another requirement is that total design base shear cannot be less than:

$$V = 0.11 C_a I W = 0.11 \times .40 \times 1.0 \times 1{,}626 = 71.5 \text{ kips} \quad (30\text{-}6)$$

In Zone 4, total base shear also cannot be less than:

$$V = \frac{.8 Z N_v I}{R} W = \frac{0.8 \times 0.4 \times 1.2 \times 1.0}{8.5} \times 1{,}626 = 73.5 \text{ kips} \quad (30\text{-}7)$$

In this example, design base shear is controlled by Equation 30-4.

$$\therefore V = \underline{\underline{171.4 \text{ kips}}}$$

Commentary

The near source factor N_a used to determine C_a need not exceed 1.1 if the conditions of §1629.4.2 are met.

Example 19
Structure Period Using Method A §1630.2.2

Determine the period for each of the structures shown below using Method A. Method A uses the following expression to determine period:

$$T = C_t (h_n)^{3/4} \tag{30-8}$$

The coefficient C_t is dependent on the type of structural system used. The code also allows use of Method B for the analytical evaluation of the fundamental period. It should be noted that the computation of the fundamental period using Equation 30-10 of this method can be cumbersome and time consuming. With widespread use of personal computers and structural analysis software in practice, a computer can determine periods much more easily than through use of Equation 30-10.

1. **Steel special moment-resisting frame (SMRF) structure.**

2. **Concrete special moment-resisting frame (SMRF) structure.**

3. **Steel eccentric braced frame (EBF).**

4. **Masonry shear wall building.**

5. **Tilt-up building.**

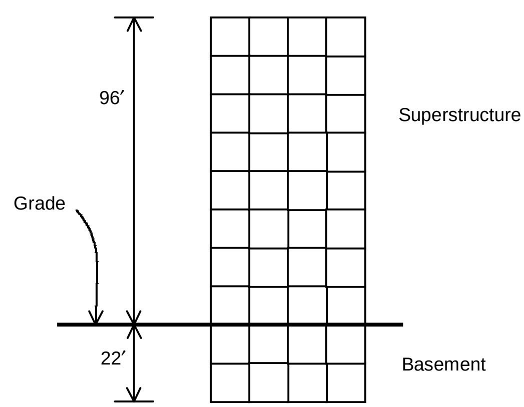
Calculations and Discussion — Code Reference

1. **Steel special moment-resisting frame (SMRF) structure.** **§1630.2.2**

Height of the structure above its base is 96 feet. The additional 22-foot depth of the basement is not considered in determining h_n for period calculation.

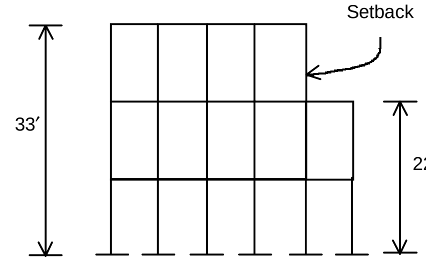
$$C_t = 0.035$$

$$T = C_t (h_n)^{3/4} = 0.035 (96)^{3/4} = \underline{\underline{1.07 \text{ sec.}}}$$



2. Concrete special moment-resisting frame (SMRF) structure. §1630.2.2

Height of tallest part of the building is 33 feet, and this is used to determine period. Roof penthouses are generally not considered in determining h_n, but heights of setbacks are included. However, if the setback represents more than a 130 percent change in the lateral force system dimension, then there is a vertical geometric irregularity (Table 16-L). For taller structures, more than five stories or 65 feet in height, dynamic analysis is required for this type of irregularity.

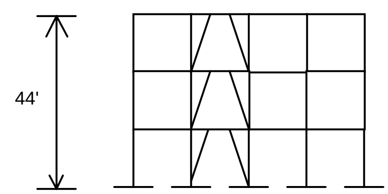


$$C_t = 0.030$$

$$T = C_t (h_n)^{3/4} = 0.030\,(33)^{3/4} = \underline{\underline{0.41 \text{ sec}}}.$$

3. Steel eccentric braced frame (EBF). §1630.2.2

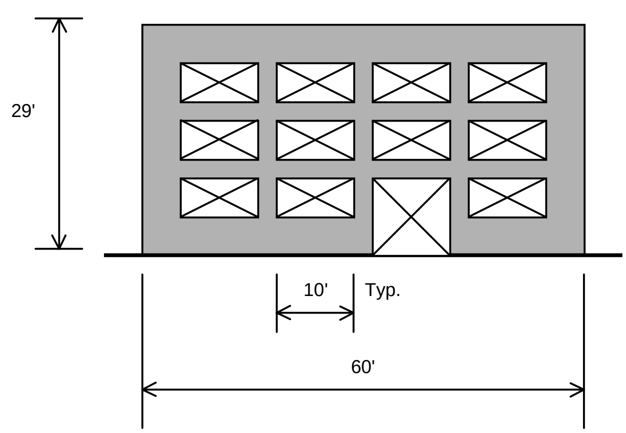
EBF structures use the C_t for the category "all other buildings."



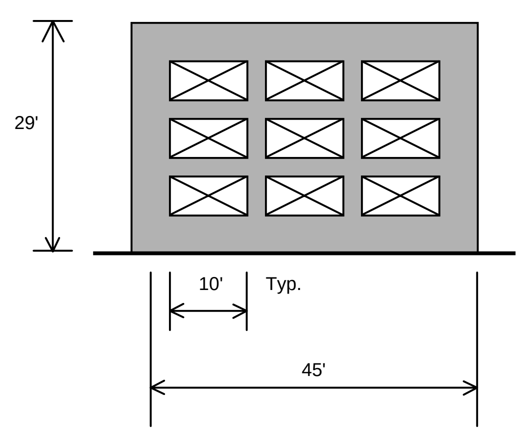
$$C_t = 0.030$$

$$T = C_t (h_n)^{3/4} = 0.030\,(44)^{3/4} = \underline{\underline{0.51 \text{ sec}}}.$$

4. Masonry shear wall building. §1630.2.2



Front wall elevation



Back wall elevatio

For this structure, C_t may be taken as 0.020, the value for "all other buildings," or its value may be computed from the following formula:

$$C_t = \frac{0.1}{\sqrt{A_c}} \qquad \text{§1630.2.2}$$

where

$$A_c = \sum A_e\left[0.2 + \left(\frac{D_e}{h_n}\right)^2\right] \qquad (30\text{-}9)$$

Solving for D_e and A_e for front and back walls, respectively, the value of A_c can be determined.

Front Wall

Nominal CMU wall thickness = 8"

Actual CMU wall thickness = 7.625"

$h_n = 29\text{ ft}$

$D_e = 60\text{ ft}$

$$A_e = (60' - 4x10')\,x\,\frac{7.63}{12} = 12.7\text{ sq ft}$$

$$\frac{D_e}{h_n} = 2.07$$

Back Wall

$D_e = 45\text{ ft}$

$$A_e = (45' - 3x10')\,x\,\frac{7.63}{12} = 9.5\text{ sq ft}$$

$$\frac{D_e}{h_n} = 1.55$$

Using Equation 30-9, the value of A_c is determined. Note that the maximum value of D_e/h_n that can be used is 0.9.

$$A_c = \left[12.7\left(0.2 + 0.9^2\right)\right] + \left[9.5\left(0.2 + 0.9^2\right)\right] = 22.4\text{ sq ft}$$

$$C_t = \frac{0.1}{\sqrt{22.4}} = 0.021$$

$$T = C_t (h_n)^{3/4} = 0.021 (29)^{3/4} = \underline{\underline{0.26 \text{ sec}}}.$$

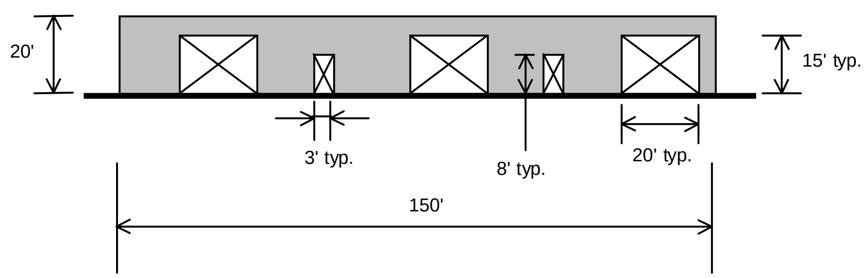
Alternately, the period can be determined using C_t = .020 for "all other buildings"

$$T = C_t (h_n)^{3/4} = 0.020 (29)^{3/4} = \underline{\underline{0.25 \text{ sec}}}.$$

Under current code provisions, either period can be used to determine base shear.

5. **Tilt-up building.**

Consider a tilt-up building 150 ft x 200 ft in plan that has a panelized wood roof and the typical wall elevation shown below.



Typical wall elevatio

$$C_t = 0.020$$

$$T = C_t (h_n)^{3/4} = 0.02 (20)^{3/4} = \underline{\underline{0.19 \text{ sec}}}.$$

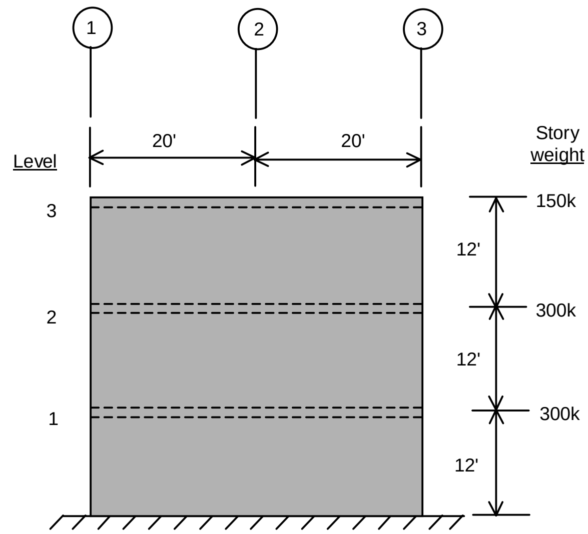
This type of structural system has relatively rigid walls and a flexible roof diaphragm. The code formula for period does not take into consideration the fact that the real period of the building is highly dependent on the roof diaphragm construction. Thus, the period computed above is not a good estimate of the real fundamental period of this type of building. It is acceptable, however, for use in determining design base shear.

It should be noted that the actual diaphragm response is approximately taken into account in the design process by increased seismic force provisions on wall anchors and by the limit of R = 4 for calculation of diaphragm loads as required under §1633.2.9.3.

Example 20
Simplified Design Base Shear §1630.2.3

Determine the design base shear and the design lateral forces for a three-story wood structural panel wall building using the simplified design base shear. The soil profile type for the site is unknown. The following information is known:

$Z = 0.4$
Seismic source type B
Distance to seismic source = 5 km
$R = 5.5$
$W = 750\text{k}$



In solving this example, the following steps are followed:

1. **Check applicability of simplified method.**

2. **Determine base shear.**

3. **Determine lateral forces at each level.**

Calculations and Discussion — Code Reference

1. **Check applicability of simplified method.** **§1629.8.2**

Light frame construction not more than three stories, or other buildings not more than two stories can use the simplified method.

$\therefore$ *o.k.*

2. **Determine base shear.** **§1630.2.3**

Because soil properties for the site are not known, a default/prescribed soil profile must be used. Section 1630.2.3.2 requires that a Type S_D soil profile be used in seismic Zones 3 and 4.

$N_a = 1.0$ Table 16-S

$$C_a = 0.44N_a = 0.44(1.0) = 0.44 \qquad \text{Table 16-Q}$$

$$V = \frac{3.0C_a}{R}W = \frac{3.0(0.44)750}{5.5} = (0.24)750 = \underline{\underline{180\text{ k}}} \qquad (30\text{-}11)$$

3. **Determine lateral forces at each level.** **§1630.2.3.3**

$$F_x = \frac{3.0C_a}{R}w_x = 0.24w_x \qquad (30\text{-}12)$$

$$F_1 = 0.24(300) = \underline{\underline{72\text{ k}}}$$

$$F_2 = 0.24(300) = \underline{\underline{72\text{ k}}}$$

$$F_3 = 0.24(150) = \underline{\underline{36\text{ k}}}$$

Commentary

The following is a comparison of simplified base shear with standard design base shear. The standard method of determining the design base shear is as follows:

$$V = \frac{2.5C_a I}{R}W = \frac{2.5(0.44)(1.0)}{5.5}W = 0.2W = 0.2(750) = 150\text{ kips} \qquad (30\text{-}12)$$

The distribution of seismic forces over the height of the structure is

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (30\text{-}15)$$

where

$V - F_t = 150$ kips since $F_t = 0$ in this example.

Level x	h_x	w_x	$w_x h_x$	$\frac{w_x h_x}{\Sigma w_i h_i}$	F_x	F_x / w_x
3	36 ft	150 kips	5,400 k-ft	0.333	50.0 kips	0.33
2	24	300	7,200	0.444	66.7	0.22
1	12	300	3,600	0.222	33.3	0.11
			$\Sigma w_i h_i = 16{,}200$		$\Sigma v = 150.0$	

The design base shear V and the lateral force values F_x at each level are all less than those determined by the simplified method. The principal advantage of the simplified method is that there is no need to conform to the provisions listed in §1630.2.3.4, which are otherwise applicable.

Another advantage is that the value of the near-source factor N_a used to determine C_a need not exceed:

> 1.3 if irregularities listed in §1630.2.3.2 are not present

and

> 1.1 if the conditions of §1629.4.2 are complied with

It should be noted that Section 104.8.2 of the 1999 SEAOC Blue Book has different requirements for applicability of the simplified method:

> Single family two stories or less
>
> Light frame up to three stories
>
> Regular buildings up to two stories

Blue Book §105.2.3 allows the near source factor $N_a = 1.0$ for evaluation of C_a. The Blue Book equation $V = 0.8 C_a W$ does not contain the *R* factor, which eliminates the sometimes difficult problem of selecting the appropriate *R* value for small buildings that have complex and/or mixed lateral load resisting systems.

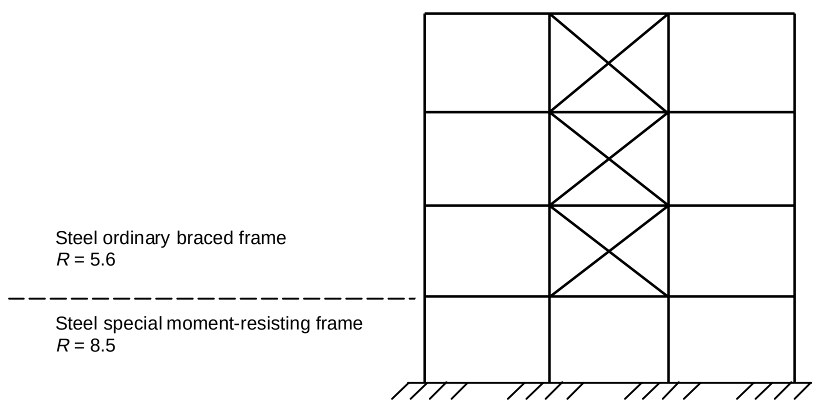
Example 21
Combination of Structural Systems: Vertical §1630.4.2

In structural engineering practice, it is sometimes necessary to design buildings that have a vertical combination of different lateral force-resisting systems. For example, the bottom part of the structure may be a rigid frame and top part a braced frame or shear wall. This example illustrates use of the requirements of §1630.4.2 to determine the applicable R values for combined vertical systems.

For the three systems shown below, determine the required R factor and related design base shear requirements.

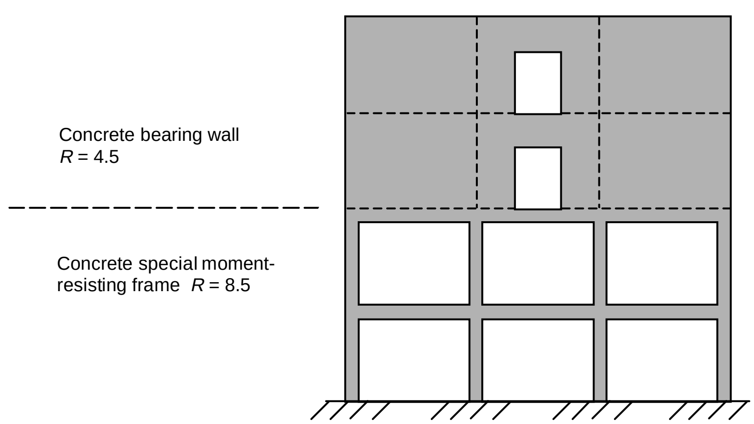
Calculations and Discussion | Code Reference

1. **Steel ordinary braced frame over steel SMRF.**



This combined system falls under vertical combinations of §1630.4.2. Because the rigid system is above the flexible system, Item 2 of §1630.4.2 cannot be used. Therefore, under Item 1 of §1630.4.2, the entire structure must use $R = \underline{\underline{5.6}}$.

2. **Concrete bearing wall over concrete SMRF.**

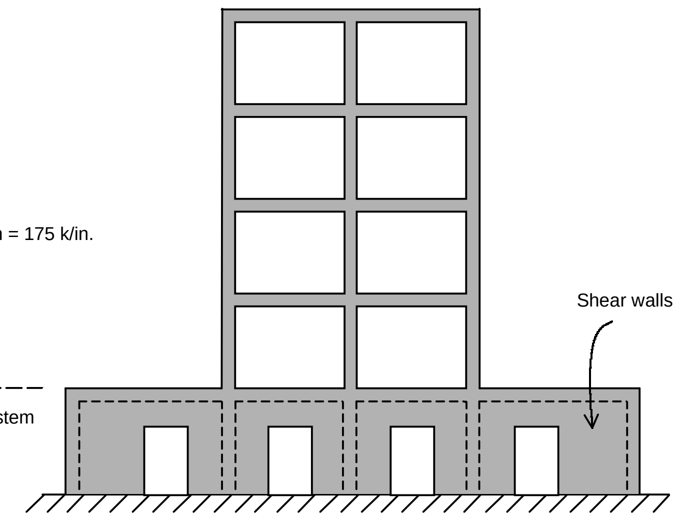


This combined system falls under vertical combinations of §1630.4.2. Because the rigid portion is above the flexible portion, Item 2 of §1630.4.2 cannot be used. Therefore, under Item 1 of §1630.4.2, the entire structure must use $R = \underline{\underline{4.5}}$.

3. Concrete SMRF over a concrete building frame system.

a. Applicable criteria.

This is a vertical combination of a flexible system over a more rigid system. Under §1630.4.2, Item 2, the two stage static analysis may be used, provided the structures conform to §1629.8.3, Item 4.



Check requirements of §1629.8.3, Item 4:

1. Flexible upper portion supported on rigid lower portion. *o.k.*
2. Average story stiffness of lower portion is at least 10 times average story stiffness of upper portion.

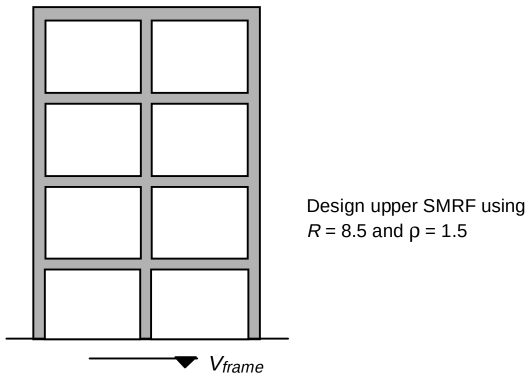
$$10{,}000 \text{ k/in.} > 10\,(175) = 1{,}750 \text{ k/in. } \; o.k.$$

3. Period of entire structure is not greater than 1.1 times period of upper portion.

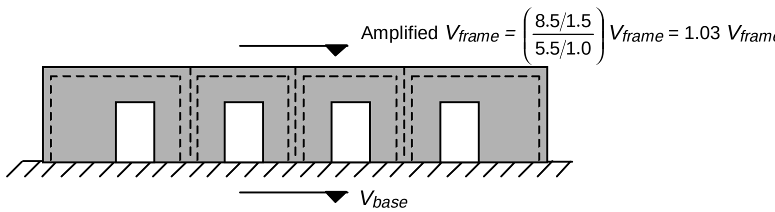
$$0.56\,\text{sec} < 1.1\,(.55) = 0.61\,\text{sec} \;\; o.k.$$

$$\therefore \underline{\underline{\text{Provisions of } \S1630.4.2, \text{Item 2 can be used}}}$$

b. **Design procedures for upper and lower structures.**



Design the lower portion of the building frame system for the combined effects of the amplified V_{frame} force and the lateral forces due to the base shear for the lower portion of the structure (using $R = 5.5$ and $\rho = 1.0$ for the lower portion).

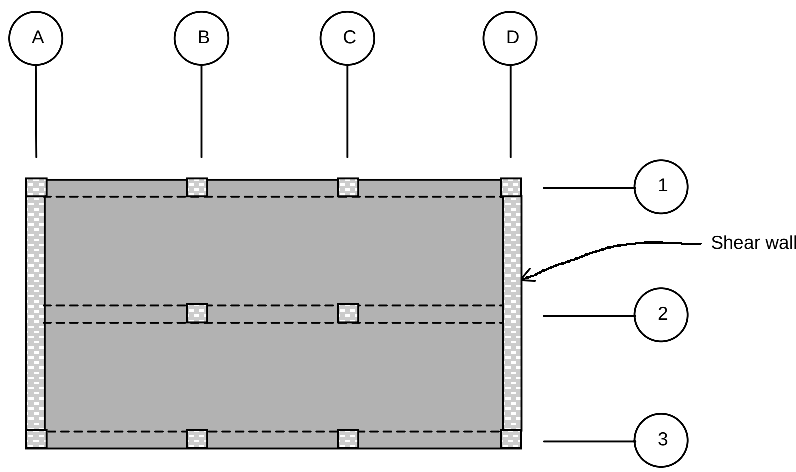


$$\therefore V_{base} = (\text{Amplified } V_{frame}) + (V_{lower})$$

Example 22
Combination of Structural Systems: Along Different Axes §1630.4.3

This example illustrates determination of R values for a building that has different structural systems along different axes (i.e., directions) of the building.

In this example, a 3-story building has concrete shear walls in one direction and concrete moment frames in the other. Floors are concrete slab, and the building is located in Zone 4. Determine the R value for each direction.



Typical floor plan

Lines A and D are reinforced concrete bearing walls: $R = 4.5$

Lines 1, 2 and 3 are concrete special moment-resisting frames: $R = 8.5$

1. **Determine th R value for each direction.**

Calculations and Discussion — Code Reference

In Zones 3 and 4, the provisions of §1630.4.3 require that when a structure has bearing walls in one direction, the R value used for the orthogonal direction cannot be greater than that for the bearing wall system.

$\therefore$ Use $R = \underline{\underline{4.5}}$ in both directions.

Commentary

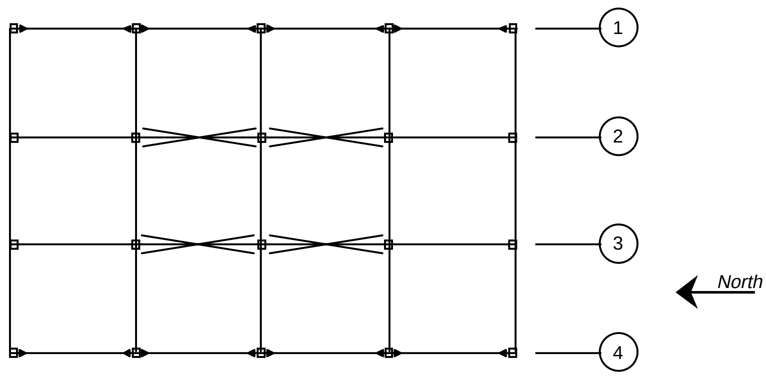
The reason for this orthogonal system requirement is to provide sufficient strength and stiffness to limit the amount of out-of-plane deformation of the bearing wall system. A more direct approach would be to design the orthogonal system such that the Δ_M value is below the value that would result in the loss of bearing wall capacity.

The design loads for the special moment-resisting frames are calculated using $R = 4.5$. However, the frame details must comply with the requirements for the $R = 8.5$ system.

Example 23
Combination of Structural Systems: Along the Same Axis §1630.4.4

Occasionally, it is necessary to have different structural systems in the same direction. This example shows how the *R* value is determined in such a situation.

A one-story steel frame structure has the roof plan shown below. The structure is located in Zone 4. Determine the *R* value for the N/S direction.



Roof plan

Lines 1 and 4 are steel ordinary moment-resisting frames: $R = 4.5$.

Lines 2 and 3 are steel ordinary braced frames: $R = 5.6$.

1. **Determine th *R* value for the N/S direction.**

Calculations and Discussion — Code Reference

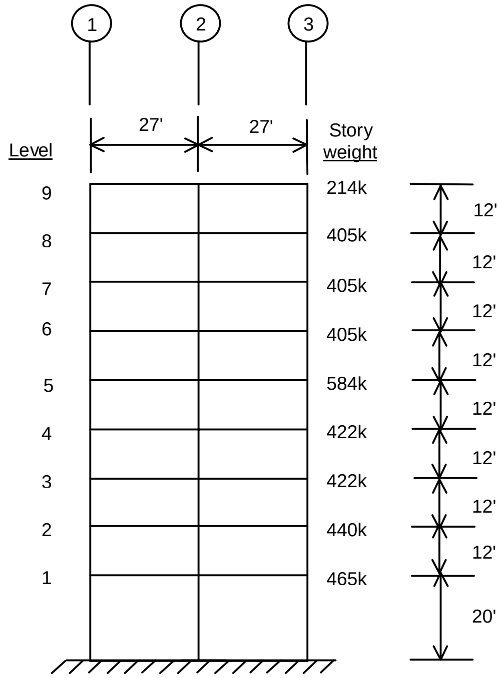
In Zones 2, 3, and 4, when a combination of structural systems is used in the same direction, §1630.4.4 requires that the value of *R* used be not greater than the least value of the system utilized.

$\therefore$ Use $R = \underline{\underline{4.5}}$ for entire structure.

Example 24
Vertical Distribution of Force §1630.5

A 9-story building has a moment resisting steel frame for a lateral force-resisting system. Find the vertical distribution of lateral forces F_x. The following information is given:

Zone 4
$W = 3{,}762$ k
$C_v = 0.56$
$R = 8.5$
$I = 1.0$
$T = 1.06$ sec.
$V = 233.8$ k



In solving this example, the following steps are followed:

1. **Determine F_t.**
2. **Find F_x at each level.**

Calculations and Discussion — Code Reference

1. Determine F_t. §1630.5

This is the concentrated force applied at the top of the structure. It is determined as follows. First, check that the F_t is not zero.

$$T = 1.06\,\text{sec.} > 0.7\,\text{sec} \quad \therefore F_t > 0$$

$$F_t = 0.07TV = 0.07(1.06)(233.8) = 17.3\text{ k} \qquad (30\text{-}14)$$

2. **Find F_x at each level.**

The vertical distribution of seismic forces is determined from Equation 30-15.

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h} \tag{30-15}$$

where

$$(V - F_t) = (233.8 - 17.3) = 216.5\text{k}$$

Since there are nine levels above the ground, $n = 9$. Therefore

$$F_x = \frac{216.5\, w_x h_x}{\sum_{i=1}^{9} w_i h_i}$$

This equation is solved in the table below.

Level x	h_x	w_x	$w_x h_x$	$\frac{w_x h_x}{\Sigma w_i h_i}$	F_x	F_x / w_x
9	116 ft	214 kips	24,824 k-ft	0.103	22.3 + 17.3 = 39.6 kips	0.185
8	104	405	42,120	0.174	37.7	0.093
7	92	405	37,260	0.154	33.3	0.082
6	80	405	32,400	0.134	29.0	0.072
5	68	584	39,712	0.164	35.5	0.061
4	56	422	23,632	0.098	21.2	0.050
3	44	422	18,568	0.077	16.7	0.039
2	32	440	14,080	0.058	12.6	0.028
1	20	465	9,300	0.038	8.2	0.018
		Σ =3,762	241,896		233.8	

Commentary

Note that certain types of vertical irregularity can result in a dynamic response having a load distribution significantly different from that given in this section. If the structural system has any of the stiffness, weight, or geometric vertical irregularities of Type 1, 2, or 3 of Table 16-L, then Item 2 of §1629.8.4 requires that the dynamic lateral force procedure be used unless the structure is less than five stories or 65 feet in height. The configuration and final design of this structure must be checked for these irregularities. Most structural analysis programs used in practice today perform this calculation, and it is generally not necessary to manually perform the calculations shown above. However, it is recommended that these calculations be performed to check the computer analysis and to gain insight to structural behavior.

Example 25
Horizontal Distribution of Shear §1630.6

A single story building has a rigid roof diaphragm. Lateral forces in both directions are resisted by shear walls. The mass of the roof can be considered to be uniformly distributed, and in this example, the weight of the walls is neglected. In actual practice, particularly with concrete shear walls, the weight of the walls should be included in the determination of the Center of Mass (CM). The following information is given:

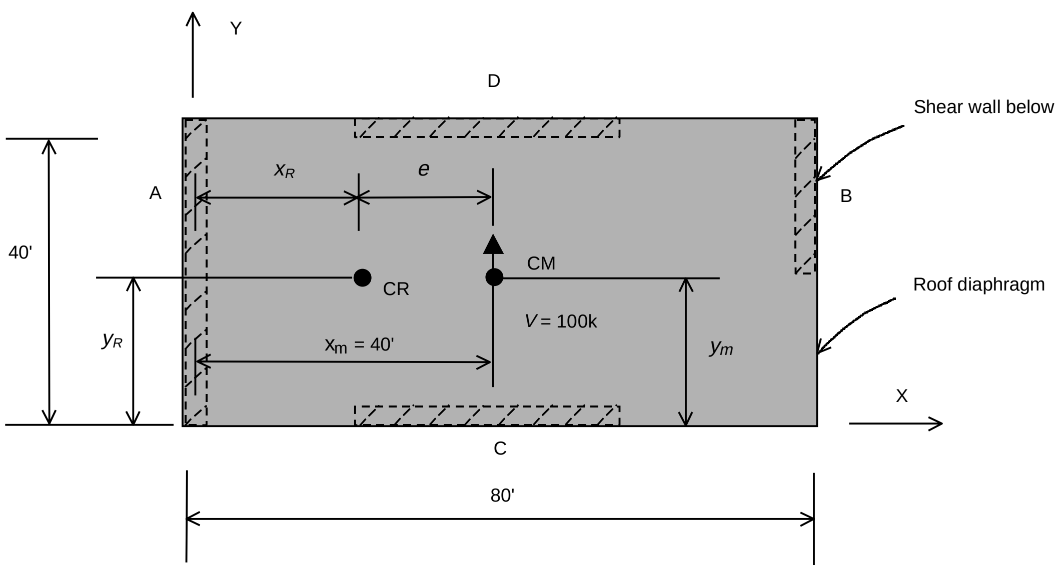
Design base shear: $V = 100$ k

Wall rigidities: $R_A = 300$ k/in.

$R_B = 100$ k/in.

$R_C = R_D = 200$ k/in.

Center of mass: $x_m = 40$ ft $y_m = 20$ ft



Roof plan

Determine the following:

1. **Eccentricity and rigidity properties.**

2. **Direct shear in walls A and B.**

3. **Plan irregularity requirements.**

4. **Torsional shear in walls A and B.**

5. **Total shear in walls A and B.**

Calculations and Discussion — Code Reference

1. Eccentricity and rigidity properties. **§1630.6**

The rigidity of the structure in the direction of applied force is the sum of the rigidities of walls parallel to this force.

$$R = R_A + R_B = 300 + 100 = 400 \text{ k/in.}$$

The center of rigidity (CR) along the x and y axes are

$$x_R = \frac{R_B(80')}{R_A + R_B} = 20 \text{ ft.}$$

$$y_R = \frac{R_D(40')}{R_D + R_C} = 20 \text{ ft}$$

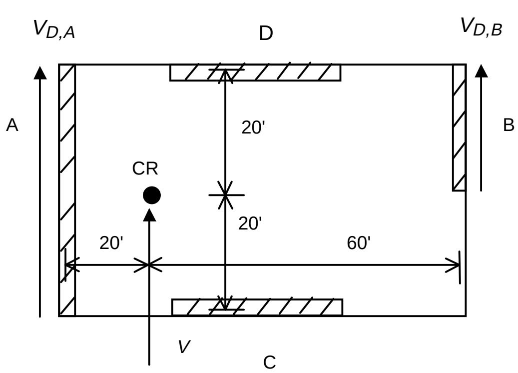
eccentricity $e = x_m - x_R = 40 - 20 = 20 \text{ ft}$

Torsional rigidity about the center of rigidity is determined as

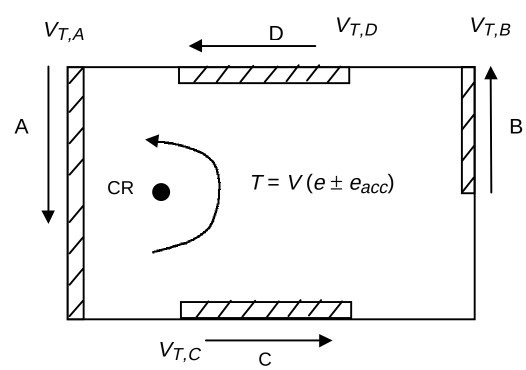
$$J = R_A(20)^2 + R_B(60)^2 + R_C(20)^2 + R_D(20)^2$$

$$= 300(20)^2 + 100(60)^2 + 200(20)^2 + 200(20)^2 = 64 \times 10^4 (\text{k/in.})\,\text{ft}^2$$

The seismic force V applied at the CM is equivalent to having V applied at the CR together with a counter-clockwise torsion T. With the requirements for accidental eccentricity e_{acc}, the total shear on walls A and B can be found by the addition of the direct and torsional load cases:



Direct shear contribution



Torsional shear contributio

2. **Direct shear in walls A and B.**

$$V_{D,A} = \frac{R_A}{R_A + R_B} \times (V) = \frac{300}{300 + 100} \times 100 = \underline{\underline{75.0 \text{ kips}}}$$

$$V_{D,B} = \frac{R_B}{R_A + R_B} \times (V) = \frac{100}{300 + 100} \times 100 = \underline{\underline{25.0 \text{ kips}}}$$

3. **Plan irregularity requirements.**

The determination of torsional irregularity, Item 1 in Table 16-M, requires the evaluation of the story drifts in walls A and B. This evaluation must include accidental torsion due to an eccentricity of 5 percent of the building dimension.

$$e_{acc} = 0.05\,(80') = 4.0 \text{ ft}$$

The corresponding initial most severe torsional shears V' using $e_{acc} = 4.0$ ft are:

$$V'_{T,A} = \frac{V(e - e_{acc})\,(x_R)\,(R_A)}{J} = \frac{100\,(20-4)\,(20)\,(300)}{64 \times 10^4} = 15.0 \text{ kips}$$

$$V'_{T,B} = \frac{V(e + e_{acc})\,(80 - x_R)\,(R_B)}{J} = \frac{100\,(20+4)\,(60)\,(100)}{64 \times 10^4} = 22.5 \text{ kips}$$

Note: these initial shears may need to be modified if torsional irregularity exists and the amplification factor $A_x > 1.0$.

The initial total shears are:

$$V'_A = V'_{D,A} - V'_{T,A} = 75.0 - 15.0 = 60.0 \text{ kips}$$

(Torsional shears may be subtracted if they are due to the reduced eccentricity $e - e_{acc}$)

$$V'_B = V'_{D,B} + V'_{T,B} = 25.0 + 22.5 = 47.5 \text{ kips}$$

The resulting displacements δ', which for this single story building are also the story drift values, are:

$$\delta'_A = \frac{V'_A}{R_A} = \frac{60.0}{300} = 0.20 \text{ in.}$$

$$\delta'_B = \frac{V'_B}{R_B} = \frac{47.5}{100} = 0.48 \text{ in.}$$

$$\delta_{avg} = \frac{0.20 + 0.48}{2} = 0.34 \text{ in.}$$

$$\delta_{max} = \delta'_B = 0.48 \text{ in.}$$

$$\frac{\delta_{max}}{\delta_{avg}} = \frac{0.48}{0.34} = 1.41 > 1.2$$

$\therefore$ Torsional irregularity exists.

Section 1630.7 requires the accidental torsion amplification factor,

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 = \left(\frac{0.48}{1.2\,(0.34)}\right)^2 = 1.38 < 3.0 \qquad (30\text{-}16)$$

4. Torsional shears in walls A and B.

The final most severe torsional shears are determined by calculating the new accidental eccentricity and using this to determine the torsional shears

$$e_{acc} = A_x\,(4.0) = (1.38)\,(4.0) = 5.54'$$

$$V_{T,A} = \frac{100\,(20 - 5.54)\,(20)\,(300)}{64 \times 10^4} = \underline{\underline{13.6 \text{ kips}}}$$

$$V_{T,B} = \frac{100\,(20 + 5.54)\,(60)\,(100)}{64 \times 10^4} = \underline{\underline{23.9 \text{ kips}}}$$

5. Total shear in walls A and B.

Total shear in each wall is the algebraic sum of the direct and torsional shear components.

$$V_A = V_{D,A} - V_{T,A} = 75.0 - 13.6 = \underline{\underline{61.4 \text{ kips}}}$$

$$V_B = V_{D,B} + V_{T,B} = 25.0 + 23.9 = \underline{\underline{48.9 \text{ kips}}}$$

Commentary

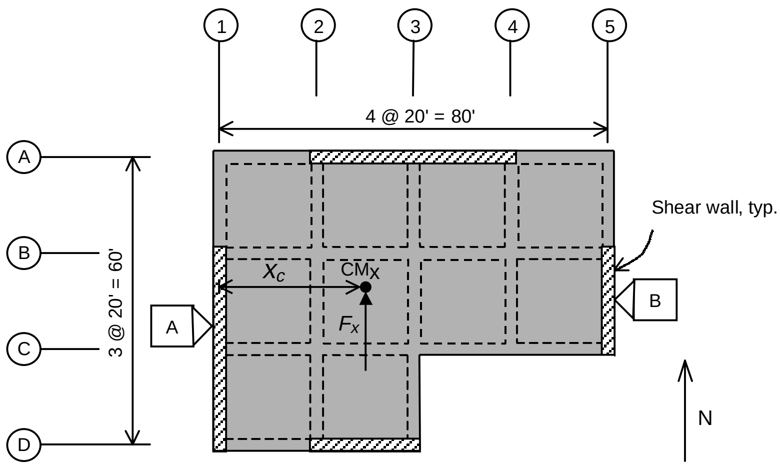
Section 1630.7 requires that "the most severe load combination for each element shall be considered for design." This load combination involves the direct and torsional shears, and the "most severe" condition is as follows:

1. For the case where the torsional shear has the same sense, and is therefore added to the direct shear, the torsional shear shall be calculated using actual eccentricity plus the accidental eccentricity so as to give the largest additive torsional shear.

2. For the case where the torsional shear has the opposite sense to that of the direct shear and is to be subtracted, the torsional shear shall be based on the actual eccentricity minus the accidental eccentricity so as to give the smallest subtractive shear.

Example 26
Horizontal Torsional Moments §1630.7

This example illustrates how to include the effects of accidental eccentricity in the lateral force analysis of a multi-story building. The structure is a five-story reinforced concrete building frame system. A three-dimensional rigid diaphragm model has been formulated per §1630.1.2 for the evaluation of element actions and deformations due to prescribed loading conditions. Shear walls resist lateral forces in both directions.



Floor plan at Level x

The lateral seismic forces F_x in the north-south direction, structure dimensions, and accidental eccentricity e_{acc} for each level x are given below:

Level x	F_x	L_x	$\bar{x}_{cx}$	$e_{acc} = 0.05L_x$
5	110.0 kips	80.0 ft	24.2 ft	± 4.0 ft
4	82.8	80.0	25.1	± 4.0
3	65.1	80.0	27.8	± 4.0
2	42.1	80.0	30.3	± 4.0
1	23.0	80.0	31.5	± 4.0

In addition, for the given lateral seismic forces F_x a computer analysis provides the following results for the second story. Separate values are given for the application of the forces F_x at the centers of mass and the $\pm 0.05L_x$ displacements as required by §1630.6.

	Force F_x Position		
	$\bar{x}_{c2}$	$\bar{x}_{c2} - e_{acc}$	$\bar{x}_{c2} + e_{acc}$
Wall shear V_A	185.0 k	196.0 k	174.0 k
Wall shear V_B	115.0 k	104.0 k	126.0 k
Story drift $\Delta\delta_A$	0.35"	0.37"	0.33"
Story drift $\Delta\delta_B$	0.62"	0.56"	0.68"
Level 2 displacement δ_A	0.80"	0.85"	0.75"
Level 2 displacement δ_B	1.31"	1.18"	1.44"

For the second story find the following:

1. **Maximum force in shear walls A and B.**

2. **Check if torsional irregularity exists.**

3. **Determine the amplification factor A_x.**

4. **New accidental torsion eccentricity.**

Calculations and Discussion — Code Reference

1. **Maximum force in shear walls A and B.**

The maximum force in each shear wall is a result of direct shear and the contribution due to accidental torsion. From the above table, it is determined that

$$V_A = \underline{\underline{196.0 \text{ k}}}$$

$$V_B = \underline{\underline{126.0 \text{ k}}}$$

2. **Check if torsional irregularity exists.**

The building is L-shaped in plan. This suggests that it may have a torsion irregularity Type 1 of Table 16-M. The following is a check of the story drifts.

$$\Delta\delta_{max} = 0.68 \text{ in.}$$

$$\Delta\delta_{avg} = \frac{0.68 + 0.33}{2} = 0.51 \text{ in.}$$

$$\frac{\Delta\delta_{max}}{1.2\Delta\delta_{avg}} = \frac{0.68}{0.51} = 1.33 > 1.2$$

$$\therefore \underline{\underline{\text{Torsional irregularity exists}}}$$

3. Determine the amplification factor A_x.

Because a torsional irregularity exists, §1630.7 requires that the second story accidental eccentricity be amplified by the following factor.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \qquad (30\text{-}16)$$

where $\delta_{max} = \delta_B = 1.44$ in.

The average story displacement is computed as

$$\delta_{avg} = \frac{1.44 + 0.75}{2} = 1.10 \text{ in.}$$

$$A_2 = \left(\frac{1.44}{(1.2)(1.10)} \right)^2 = \underline{\underline{1.19}}$$

4. New accidental torsion eccentricity.

Since A_2 (i.e., A_x for the second story) is greater than unity, a second analysis for torsion must be done using the new accidental eccentricity.

$$e_{acc} = (1.19)(4.0') = \underline{\underline{4.76 \text{ ft}}}$$

Commentary

Example calculations were given for the second story. In practice, each story requires an evaluation of the most severe element actions and a check for the torsional irregularity condition.

If torsional irregularity exists and A_x is greater than one at any level (or levels), then a second torsional analysis must be done using the new accidental eccentricities. However, it is *not* necessary to find the resulting new A_x values and repeat the process a second or third time (until the A_x iterates to a constant or reaches the limit of 3.0). The results of the first analysis with the use of A_x are sufficient for design purposes.

While this example involved the case of wall shear evaluation, the same procedure applies to the determination of the most severe element actions for any other lateral force-resisting system having rigid diaphragms.

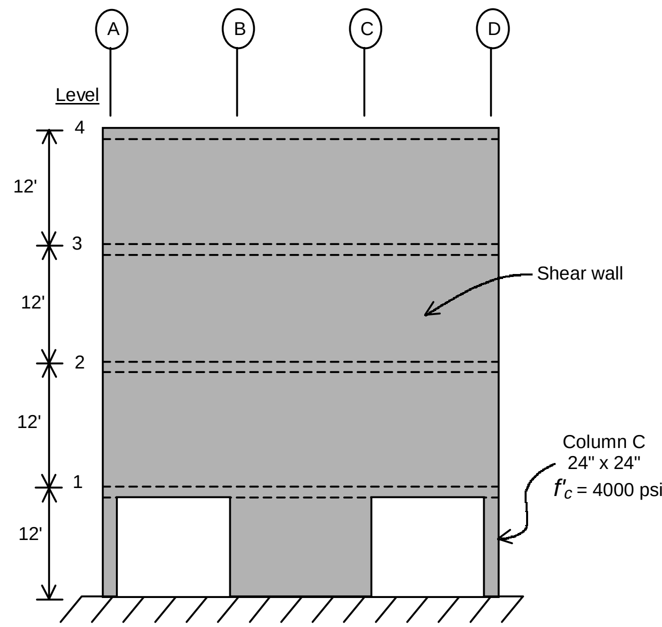
When the dynamic analysis method of §1631.5 is used, rather than static force procedure of §1630.2, the following equivalent static force option may be used in lieu of performing the two extra dynamic analyses for mass positions at $\bar{x}_{cx} \pm (0.05L_x)$ as per §1631.5.6:

1. Perform the dynamic analysis with masses at the center of mass, and reduce results to those corresponding to the required design base shear.
2. Determine the F_x forces for the required design base shear, and apply pure torsion couple loads $F_x(0.05L_x)$ at each level x. Then add the absolute value of these couple load results to those of the reduced dynamic analysis.

Example 27
Elements Supporting Discontinuous Systems §1630.8.2

A reinforced concrete building has the lateral force-resisting system shown below. Shear walls at the first floor level are discontinuous between Lines A and B and Lines C and D. The following information is given:

Zone 4
Concrete shear wall building frame system: R = 5.5 and $\Omega_o = 2.8$ Table 16-N
Office building live load: $f_1 = 0.5$ §1612.4
Axial loads on column C: $D = 40$ kips $L = 20$ kips $E_h = 100$ kips



Determine the following for column C:

1. **Required strength.**

2. **Detailing requirements.**

Calculations and Discussion *Code Reference*

This example demonstrates the loading criteria and detailing required for elements supporting discontinued or offset elements of a lateral force-resisting system.

1. **Required strength.** **§1630.8.2.1**

Because of the discontinuous configuration of the shear wall at the first story, the first story columns on Lines A and D must support the wall elements above this level. Column "C" on Line D is treated in this example. Because of symmetry, the column on Line A would have identical requirements.

Section 1630.8.2 requires that the column strength be equal to or greater than

$$P_u = 1.2D + f_1 L + 1.0E_m \quad (12\text{-}17)$$

$$P_u = 0.9D \pm 1.0E_m \quad (12\text{-}18)$$

where

$$E_m = \Omega_o E_h = 2.8(100) = 280 \text{ kips} \quad (30\text{-}2)$$

Substituting the values of dead, live and seismic loads

$$P_u = 1.2\,(40) + 0.5\,(20) + 280 = \underline{\underline{338 \text{ kips}}} \text{ compression, and}$$

$$P_u = 0.9(40) - 1.0(280) = \underline{\underline{-244 \text{ kips}}} \text{ tension}$$

2. **Detailing requirements.** **§1630.8.2.2**

The concrete column must meet the requirements of §1921.4.4.5. This section requires transverse confinement tie reinforcement over the full column height if

$$P_u > \frac{A_g f'_c}{10} = \frac{(24)^2(4 \text{ ksi})}{10} = 230 \text{ kips}$$

$$P_u = 338 > 230 \text{ kips}$$

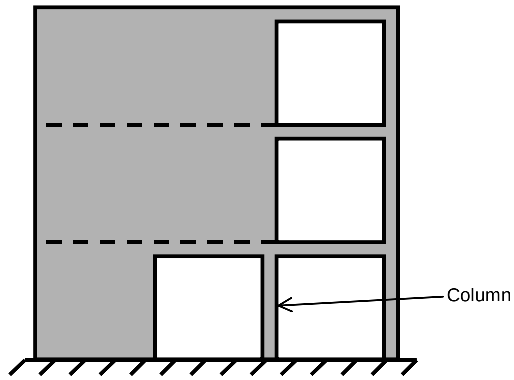
$$\therefore \underline{\underline{\text{Confinement is required over the full height}}}$$

Commentary

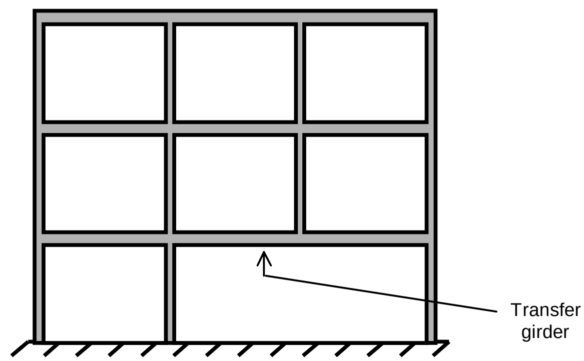
To transfer the shears from walls A-B and C-D to the first story wall B-C, collector beams A-B and C-D are required at Level 1. These would have to be designed according to the requirements of §1633.2.6.

The load and detailing requirements of §1630.8.2, Elements Supporting Discontinuous Systems, apply to the following vertical irregularities and vertical elements:

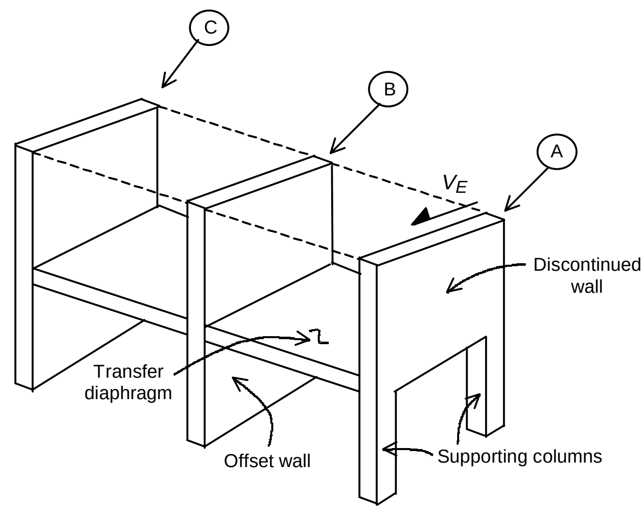
1. **Discontinuous shear wall.** The wall at left has a Type 4 vertical structural irregularity.



2. **Discontinuous column.** This frame has a Type 4 vertical structural irregularity.



3. **Out-of-plane offset.** The wall on Line A at the first story is discontinuous. This structure has a Type 4 plan structural irregularity, and §1620.8.2 applies to the supporting columns. The portion of the diaphragm transferring shear (i.e., transfer diaphragm) to the offset wall must be designed for shear wall detailing requirements, and the transfer loads must use the reliability/redundancy factor ρ for the vertical-lateral-force-resisting system.



It should be noted that for any of the supporting elements shown above, the load demand E_m of Equation (30-2) need not exceed the maximum force that can be transferred to the element by the lateral force-resisting system.

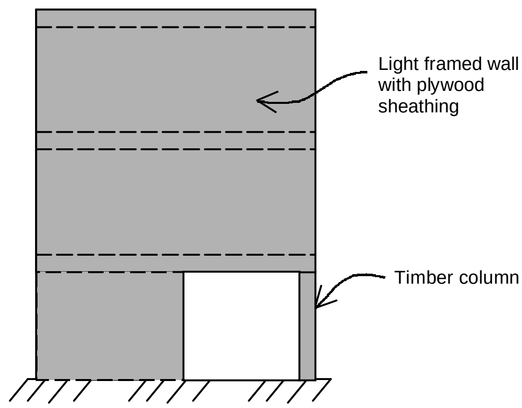
Example 28
Elements Supporting Discontinuous Systems §1630.8.2

This example illustrates the application of the requirements of §1630.8.2 for the allowable stress design of elements that support a discontinuous lateral force-resisting system.

In this example, a light-framed wood bearing wall building with plywood shear panels has a Type 4 vertical irregularity in one of its shear walls, as shown below.

The following information is given:

Zone 4
$R = 5.5$
$\Omega_o = 2.8$
$f_1 = 0.5$



Axial loads on the timber column under the discontinuous portion of the shear wall are:

Dead $D = 6.0$ kips
Live $L = 3.0$ kips
Seismic $E_h = \pm 7.0$ kips

Determine the following:

1. **Applicable load combinations.**

2. **Required column design strength.**

Calculations and Discussion — Code Reference

1. **Applicable load combinations.**

For vertical irregularity Type 4, §1630.8.2.1 requires that the timber column have the "design strength" to resist the special seismic load combinations of §1612.4. This is required for both allowable stress design and strength design. These load combinations are:

$$1.2D + f_1 L + 1.0E_m \quad (12\text{-}17)$$

$$0.9D \pm 1.0E_m \quad (12\text{-}18)$$

2. **Required column design strength.**

In this shear wall, the timber column carries only axial loads. The appropriate dead, live and seismic loads are determined as:

$$D = 6.0 \text{ kips}$$

$$L = 3.0 \text{ kips}$$

$$E_m = \Omega_o E_h = 2.8\,(7.0) = 19.6 \text{ kips} \tag{30-2}$$

For the required "design strength" check, both Equations (12-17) and (12-18) must be checked.

$$P = 1.2D + f_1 L + E_m \tag{12-17}$$

$$P = 1.2\,(6.0) + 0.5\,(3.0) + 19.6 = \underline{\underline{28.3 \text{ kips}}}$$

$$P = 0.9D \pm 1.0E_m \tag{12-18}$$

$$P = 0.9\,(6.0) \pm 1.0\,(19.6) = 25.0 \text{ kips or } \underline{\underline{-14.2 \text{ kips}}}$$

Commentary

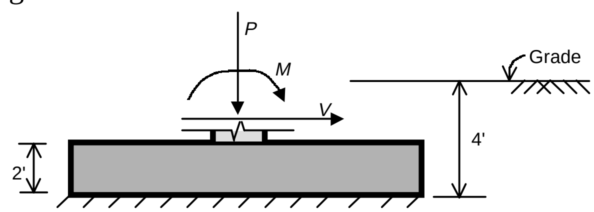
For allowable stress design, the timber column must be checked for a compression load of 28.3 kips and a tension load of 14.2 kips. In making this "design strength" check, §1630.8.2.1 permits use of an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. The 1.7 increase is not to be combined with the one-third increase permitted by §1612.3.2, but may be combined with the duration of load increase $C_D = 1.33$ given in Table 2.3.2 of Chapter 23, Division III. The resulting "design strength" $= (1.7)(1.0)(1.33)$ (allowable stress). This also applies to the mechanical hold-down element required to resist the tension load.

The purpose of the "design strength" check is to check the column for higher and, hopefully, more realistic loads that it will be required to carry because of the discontinuity in the shear wall at the first floor. This is done by increasing the normal seismic load in the column, E_h, by the factor $\Omega_o = 2.8$.

Example 29
At Foundation §1630.8.3

Foundation reports usually provide soil bearing pressures on an allowable stress design basis while seismic forces in the 1997 UBC, and most concrete design, are on a strength design basis. The purpose of this example is to illustrate footing design under this situation.

A spread footing supports a reinforced concrete column. The soil classification at the site is sand (SW). The following information is given:



Zone 4
$\rho = 1.0$ for structural system
$P_D = 80 \text{ k} \quad M_D = 15 \text{ k-ft}$
$P_L = 30 \text{ k} \quad M_L = 6 \text{ k-ft}$
$P_E = \pm 40 \text{ k} \quad V_E = 30 \text{ k} \quad M_E = \pm 210 \text{ k-ft}$
Snow load $S = 0$

Find the following:

1. **Determine the design criteria and allowable bearing pressure.**
2. **Determine footing size.**
3. **Check resistance to sliding.**
4. **Determine soil pressures for strength design of the footing section.**

Calculations and Discussion — Code Reference

1. Determine the design criteria and allowable bearing pressure. §1630.8.3

The seismic force reactions on the footing are based on strength design. However, §1629.1 states that allowable stress design may be used for sizing the foundation using the load combinations given in §1612.3. Here it is elected to use the alternate basic load combinations of §1612.3.2.

$$D + L + S \tag{12-12}$$

$$D + L + \frac{E}{1.4} \tag{12-13}$$

$$0.9D \pm \frac{E}{1.4} \tag{12-16-1}$$

Because foundation investigation reports for buildings typically specify bearing pressures on an allowable stress design basis, criteria for determining footing size are also on this basis.

The earthquake loads to be resisted are specified in §1630.1.1 by Equation 30-1.

$$E = \rho E_h + E_v \tag{30-1}$$

Since $E_v = 0$ for allowable stress design, Equation 30-1 reduces to

$$E = \rho E_h = (1.0) E_h$$

Table 18-1-A of §1805 gives the allowable foundation pressure, lateral bearing pressure, and the lateral sliding friction coefficient. These are default values to be used in lieu of site-specific recommendations given in a foundation report for the building. They will be used in this example.

For the sand (SW) class of material and footing depth of 4 feet, the allowable foundation pressure p_a is

$$p_a = 1.50 + (4\text{ ft} - 1\text{ ft})(0.2)(1.50) = \underline{\underline{2.40\text{ ksf}}}$$ Table 18-1-A and Footnote 2

A one-third increase in p_a is permitted for the load combinations that include earthquake load.

2. Determine footing size.

The trial design axial load and moment will be determined for load combination of Equation (12-13) and then checked for the other combinations.

$$P_a = D + L + \frac{E}{1.4} = P_D + P_L + \frac{P_E}{1.4} = 80 + 30 + \frac{40}{1.4} = 138.6\text{ kips} \tag{12-13}$$

$$M_a = D + L + \frac{E}{1.4} = M_D + M_L + \frac{M_E}{1.4} = 15 + 6 + \frac{210}{1.4} = 171.0\text{ k - ft} \tag{12-13}$$

Select trial footing size.

Try 9 ft x 9 ft footing size, B = L = 9 ft

$$A = BL = 81\text{ ft}^2, \quad S = \frac{BL^2}{6} = \frac{9^3}{6} = 121.5\text{ ft}^3$$

Calculated soil pressures due to axial load and moment

$$p = \frac{P_a}{A} + \frac{M_a}{S} = \frac{138.6}{81} + \frac{171.0}{121.5} = 1.71 + 1.41 = 3.12\text{ ksf}$$

Check bearing pressure against allowable with one-third increase,

$$3.12\text{ ksf} < 1.33p_a = 1.33(2.40) = 3.20\text{ ksf},\ o.k.$$

Check for the load combination of Equation (12-16-1).

$$P_a = 0.9D \pm \frac{E}{1.4} = 0.9P_D \pm \frac{P_E}{1.4} = 0.9(80) \pm \frac{40}{1.4} = 100.6\text{ kips or } 43.4\text{ kips} \tag{12-16-1}$$

$$M_a = 0.9D \pm \frac{E}{1.4} = 0.9M_D \pm \frac{M_E}{1.4} = 0.9(15) \pm \frac{210}{1.4} = 163.5\text{ k-ft or } 136.5\text{ k-ft}$$
(12-16-1)

Eccentricity $e = \frac{M_a}{P_a} = \frac{163.5\text{ k-ft}}{100.6} = 1.63\text{ ft, or } \frac{136.5\text{ k-ft}}{43.4} = .15\text{ ft},\ \therefore e = 3.15\text{ ft}$ governs.

Check for uplift.

$$e > \frac{L}{6} = \frac{9}{6} = 1.5\text{ ft (where } \frac{L}{6} \text{ is the limit of the kern area)}$$

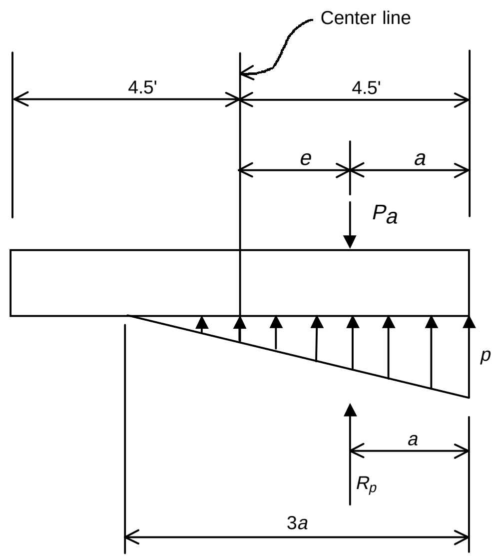
Since $e = 3.15 > 1.5$, there is partial uplift, and a triangular pressure distribution is assumed to occur.

For the footing free-body:

$$P_a = R_p = \frac{p}{2}(3a)B$$

R_p must be co-linear with P_a such that the length of the triangular pressure distribution is equal to $3a$.

$R_p =$ Pressure resultant



The load combination $0.9D - \frac{E}{1.4}$, with $P_a = 43.4\text{ kips}$ and $M_a = 136.5\text{ k-ft}$ (12-10)

governs bearing pressure

$$a = \frac{B}{2} - e = 4.5 - 3.15 = 1.35\text{ ft}$$

$$P_a = \frac{p}{2}(3a)B$$

or

$$p = \frac{2}{3}P_a\left(\frac{1}{aB}\right) = \frac{2}{3}(43.4)\left[\frac{1}{(1.35)(9.0)}\right] = 2.38\,\text{ksf} < 1.33p_a = 3.20\,\text{ksf} \quad o.k.$$

If p had been greater than $1.33p_a$, the footing size would have to be increased.

Finally, check the gravity load combination (12-12) for $p < p_a = 3.2\,\text{ksf}$.

$$P_a = D + L = P_D + P_L = 80 + 30 = 110\,\text{kips} \tag{12-12}$$

$$M_a = D + L = M_D + M_L = 15 + 6 = 21\,\text{k-ft} \tag{12-12}$$

$$p = \frac{P_a}{A} + \frac{M_a}{S} = \frac{110}{81} + \frac{21}{121.5} = 1.53\,\text{ksf} < 3.2\,\text{ksf},\ o.k.$$

All applicable load combinations are satisfied, therefore a 9ft x 9ft footing is adequate.

3. Check resistance to sliding.

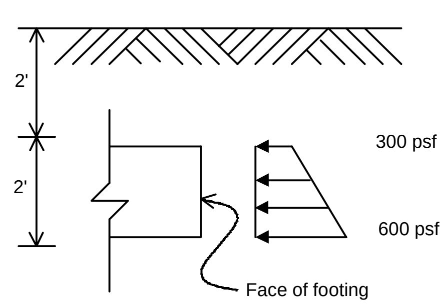
Unless specified in the foundation report for the building, the friction coefficient and lateral bearing pressure for resistance to sliding can be determined from Table 18-1-A. These values are:

Friction coefficient	$\mu = 0.25$	Table 18-1-A
Lateral bearing resistance	$p_L = 150\,\text{psf} \times$ depth below grade	Table 18-1-A

Assume the footing is 2 feet thick with its base 4 feet below grade. Average resistance on the 2 feet deep by 9 feet wide footing face is $\frac{300+600}{2} = 450\,\text{psf}$.

$$p_L = 450\,\text{psf} = 0.45\,\text{ksf}$$

Load combination of Equation (12-16-1) will be used because it has the lowest value of vertical load $0.9D = 0.9P_D$). The vertical and lateral loads to be used in the sliding resistance calculations are:



$$P = 0.9P_D = 0.9(80) = 72\,\text{kips}$$

Lateral load = $\dfrac{V_E}{1.4} = \dfrac{30}{1.4} = 21.4$ kips

The resistance due to friction is

$$P(\mu) = 72(0.25) = 18.0 \text{ kips}$$

The resistance from lateral bearing is

$$p_L \text{ (face area)} = 0.45\,(2' \times 9') = 8.1 \text{ kips}$$

The total resistance is then the sum of the resistance due to friction and the resistance due to lateral bearing pressure.

Total resistance = $18.0 + 8.1 = 26.1 > 21.4$ kips, *o.k.*

$$\therefore \underline{\underline{\text{No sliding occurs}}}$$

4. **Determine soil pressures for strength design of footing section.**

To obtain the direct shear, punching shear, and moments for the strength design of the reinforced concrete footing section, it is necessary to compute the upward design soil pressure on the footing due to factored strength loads:

$$1.2D + 1.0E + f_1 L \qquad (12\text{-}5)$$

$$0.9D \pm 1.0E \qquad (12\text{-}6)$$

The section design must have the capacity to resist the largest moments and forces resulting from these load combinations.

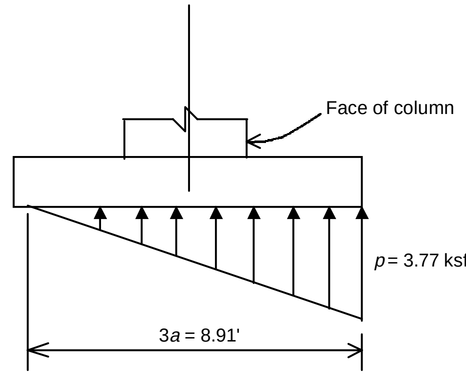
a. **Soil pressure due to load combination** $1.2D + 1.0E + f_1 L$ **.**

$$f_1 = 0.5 \qquad \text{§1612.2.1}$$

$$P_u = 1.2P_D + 1.0P_E + 0.5P_L = 1.2\,(80) + 1.0\,(40) + 0.5\,(30) = 151 \text{ kips} \qquad (12\text{-}5)$$

$$M_u = 1.2M_D + 1.0M_E + 0.5M_L = 1.2\,(15) + 1.0\,(210) + 0.5\,(6) = 231 \text{ k-ft} \qquad (12\text{-}5)$$

Eccentricity $e = \dfrac{M_u}{P_u} = \dfrac{231}{151} = 1.53\,\text{ft}$



$$e > \frac{L}{6} = \frac{9}{6} = 1.5\,\text{ft}$$

Therefore partial uplift occurs.

$$a = 4.5 - e = 4.5 = 1.53 = 2.97\,\text{ft}$$

$$p = \frac{2}{3}P_u\left(\frac{1}{aB}\right) = \frac{2}{3}(151)\left(\frac{1}{(2.97)(9.0)}\right) = \underline{\underline{3.77\,\text{ksf}}}$$

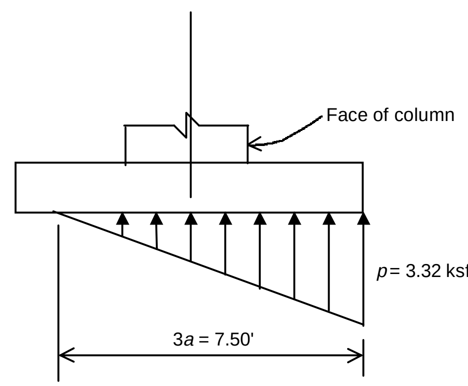
b. **Soil pressure due to load combination** $0.9D \pm 1.0E$ **:**

$$P_u = 0.9P_D \pm 1.0P_E = 0.9(80) \pm 1.0(40) = 112\,\text{kips or }32\,\text{kips} \qquad (12\text{-}6)$$

$$M_u = 0.9M_D \pm 1.0M_E = 0.9(15) \pm 1.0(210) = 223.5\,\text{k-ft or }196.5\,\text{k-ft} \qquad (12\text{-}6)$$

Compute pressure load due to $P_u = 112\,\text{kips}$ and $M_u = 223.5\,\text{k-ft}$

Eccentricity $e = \dfrac{M_u}{P_u} = \dfrac{223.5}{112} = 2.00\,\text{ft}$



$$e > \frac{L}{6} = 1.5\,\text{ft}$$

therefore partial uplift occurs.

$$a = 4.5 - e = 4.5 - 2.0 = 2.50\,\text{ft}$$

$$p = \frac{2}{3}P_u\left(\frac{1}{aB}\right) = \frac{2}{3}(112)\left(\frac{1}{(2.50)(9.0)}\right) = \underline{\underline{3.32\,\text{ksf}}}$$

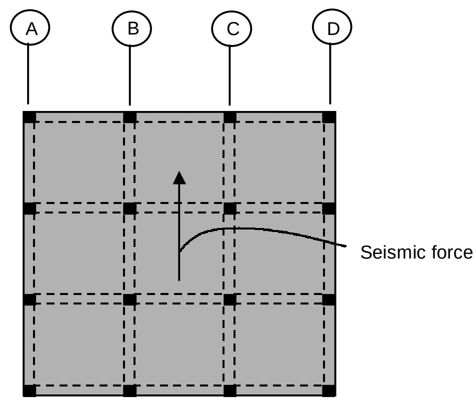
The footing pressure is less than that for the combination of $1.2D + 1.0E + f_1L$. Therefore the $1.2D + 1.0E + f_1L$ combination governs. Note that the resulting direct shear, punching shear, and moments must be multiplied by 1.1 per Exception 1 of §1612.2.1. (*Note:* At the time of publication, the 1.1 factor is under consideration for change to 1.0).

Note also that the value of p due to the strength design factored loads need not be less than $1.33p_a = 3.20\,\text{ksf}$, since it is used as a load for concrete section design rather than for determining footing size.

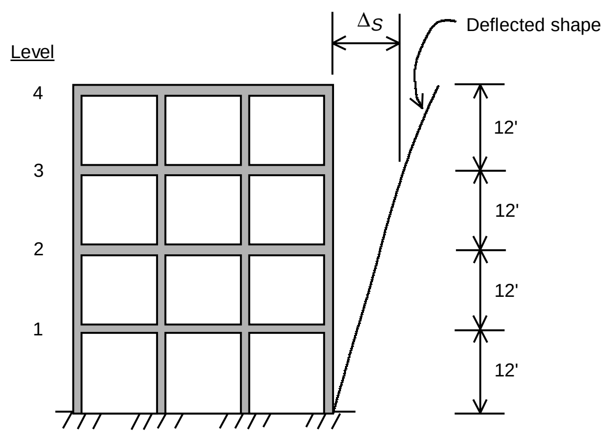
Example 30
Drift §1630.9

A four-story special moment-resisting frame (SMRF) building has the typical floor plan as shown below. The elevation of Line D is also shown, and the following information is given:

Zone 4
$I = 1.0$
$R = 8.5$
$\Omega_o = 2.8$
$T = 0.60\,\text{sec}$



Typical floor plan



Elevation of Line D

The following are the design level response displacements Δ_S (total drift) for the frame along Line D. These values include both translational and torsional (with accidental eccentricity) effects. As permitted by §1630.10.3, Δ_S has been determined due to design forces based on the unreduced period calculated using Method B.

Level	Δ_S
4	1.51 in
3	1.03
2	.63
1	.30

For the frame on Line D, determine the following:

1. **Maximum inelastic response displacements Δ_M.**

2. **Story drift in story 3 due to Δ_M.**

3. **Check story 3 for story drift limit.**

Calculations and Discussion *Code Reference*

1. **Maximum inelastic response displacements Δ_M.** **§1630.9.2**

These are determined using the Δ_S values and the R-factor

$$\Delta_M = 0.7R\Delta_S = 0.7(8.5)(\Delta_S) = 5.95\Delta_S \tag{30-17}$$

Therefore

Level	Δ_S	Δ_M
4	1.51 in	8.98 in
3	1.03	6.12
2	0.63	3.75
1	0.30	1.79

2. **Story drift in story 3 due to Δ_M.** **§1630.10**

Story 3 is located between Levels 2 and 3. Thus

$$\Delta_M \text{ drift} = 6.12 - 3.75 = \underline{\underline{2.37 \text{ in}}}.$$

3. **Check story 3 for story drift limit.** **§1630.10.2**

For structures with a fundamental period less than 0.7 seconds, §1630.10.2 requires that the Δ_M story drift not exceed 0.025 times the story height.

For story 3

Story drift using $\Delta_M = 2.37$ in.

Story drift limit $= .025(144) = 3.60 \text{ in} > 2.37$ in.

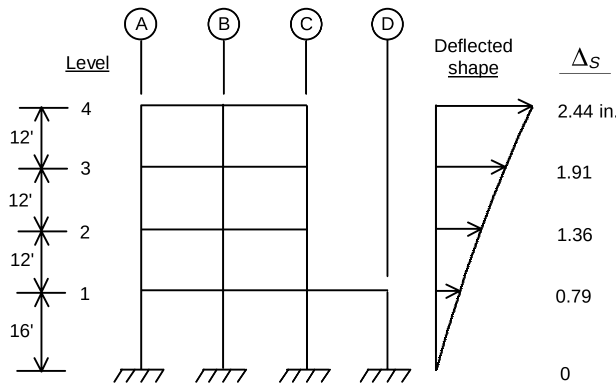
$\therefore$ $\underline{\underline{\text{Story drift is within limits}}}$

Example 31
Story Drift Limitations §1630.10

For the design of new buildings, the code places limits on story drifts. The limits are based on the maximum inelastic response displacements and not the design level response displacements determined from the design base shear of §1630.2.

In the example given below, a four-story steel special moment-resisting frame (SMRF) structure has the design level response displacements Δ_S shown. These have been determined according to §1630.9.1 using a static, elastic analysis.

Zone 4
T = 0.60 sec.
R = 8.5



Determine the following:

1. **Maximum inelastic response displacements.**

2. **Compare story drifts with the limit value.**

Calculations and Discussion — Code Reference

1. **Maximum inelastic response displacements.** **§1630.9.1**

Maximum inelastic response displacements, Δ_M, are determined from the following:

$$\Delta_M = 0.7R\Delta_S \qquad (30\text{-}17)$$

$$\therefore \ \Delta_M = 0.7(8.5)\Delta_S = \underline{\underline{5.95\Delta_S}}$$

2. **Compare story drifts ith the limit value.** **§1630.10.2**

Using Δ_M story displacements, the calculated story drift cannot exceed 0.025 times the story height for structures having a period less than 0.7 seconds.

Check building period.

$$T = .60\,\text{sec} < .70\,\text{sec}$$

Therefore, limiting story drift is 0.025 story height.

Determine drift limit at each level.

Levels 4, 3, and 2

$$\Delta_M \text{ drift} \leq .025h = .025\,(12\text{ ft} \times 12\text{ in./ft}) = 3.60\text{ in.} \qquad \text{§1630.10.2}$$

Level 1

$$\Delta_M \text{ drift} \leq .025h = .025\,(16\text{ ft} \times 12\text{ in./ft}) = 4.80\text{ in.}$$

For Δ_M drift $= \Delta_{Mi} - \Delta_{Mi-1}$, check actual story drifts against limits:

Level i	Δ_S	Δ_M	Δ_M *drift*	*Limit*	*Status*
4	2.44 in.	14.52 in.	3.16 in.	3.60 in.	*o.k.*
3	1.91	11.36	3.27	3.60	*o.k.*
2	1.36	8.09	3.39	3.60	*o.k.*
1	0.79	4.70	4.70	4.80	*o.k.*

Therefore, the story drift limits of §1630.10 are satisfied.

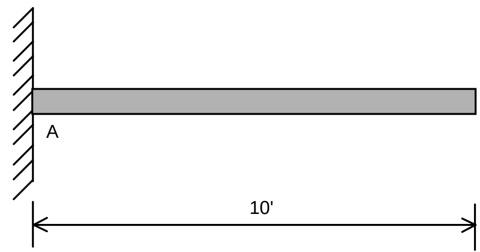
Commentary

Whenever the dynamic analysis procedure of §1631 is used, story drift should be determined as the modal combination of the story drift for each mode. Determination of story drift from the difference of the combined mode displacements may produce erroneous results because maximum displacement at a given level may not occur simultaneously with those of the level above or below. Differences in the combined mode displacements can be less than the combined mode story drift.

Example 32
Vertical Component §1630.11

Find the vertical seismic forces on the non-prestressed cantilever beam shown below. The following information is given:

Beam unit weight = 200 plf
$C_a = 0.40$
$I = 1.0$
$Z = .4$



Find the following:

1. **Upward seismic forces on beam.**

2. **Beam end reactions.**

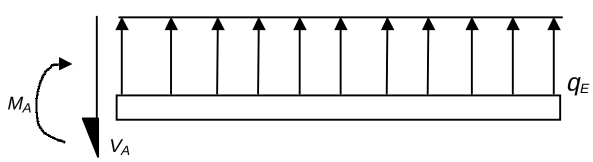
Calculations and Discussion — *Code Reference*

1. **Upward seismic forces on beam.** **§1630.11**

In Seismic Zones 3 and 4, the design of horizontal cantilever beams must consider a net upward seismic force. The terminology of "net upward seismic force" is intended to specify that gravity load effects cannot be considered to reduce the effects of the vertical seismic forces and that the beam must have the strength to resist the actions due to this net upward force without consideration of any dead loads. This force is computed as

$$q_E = 0.7C_a IW_p = 0.7\,(0.40)(1.0)(200\text{ plf}) = \underline{\underline{56\text{ plf}}}$$ §1630.11

2. **Beam end reactions.**



$$V_A = q_E l = 56\text{ plf }(10\text{ ft}) = \underline{\underline{560\text{ lbs}}}$$

$$M_A = q_E\frac{l^2}{2} = \frac{56\,(10)^2}{2} = \underline{\underline{2{,}800\text{ lb/ft}}}$$

The beam must have strengths ϕV_n and ϕM_n to resist these actions.

Example 33
Design Response Spectrum §1631.2

Determine the elastic design response spectrum for a site in Zone 4 with the following characteristics:

Soil Profile Type S_D
Seismic source type C
Distance to nearest seismic source = 23 km

1. **Determine design response spectrum.**

Calculations and Discussion — Code Reference

The design response spectrum can be determined, under §1631.2, using Figure 16-3 of the code and the coefficients C_a and C_v. The values of C_a and C_v are determined from the soil profile type, seismic source type, and distance to nearest source. In Zone 4, the values of C_a and C_v are dependent upon the near field factors N_a and N_v, respectively, as given in Tables 16-Q and 16-R.

Determine N_a and N_v §1629.4.2

From Table 16-S with seismic source type C and distance of 23 km.

$$N_v = 1.0$$

From Table 16-T with seismic source type C and distance of 23 km.

$$N_v = 1.0$$

Determine C_a and C_v §1629.4.3

From Table 16-Q with Soil Profile Type S_D and Z = 0.4

$$C_a = 0.44N_a = (0.44)(1.0) = 0.44$$

From Table 16-R with Soil Profile Type S_D and Z = 0.4

$$C_v = 0.64N_v = (0.64)(1.0) = 0.64$$

Once the values of C_a and C_v for the site are established, the response spectrum can be constructed using Figure 16-3. The peak ground acceleration (PGA) is the value of spectral acceleration at the zero period of the spectrum (T = 0). In this case it is 0.44g.

PGA is designated as the coefficient C_a by the code. This is also called the zero period acceleration (ZPA).

The peak of the response spectrum for 5 percent damping is 2.5 times C_a. In this example, it is

$$2.5C_a = (2.5)(0.44) = 1.1g$$

The control periods T_o and T_s are

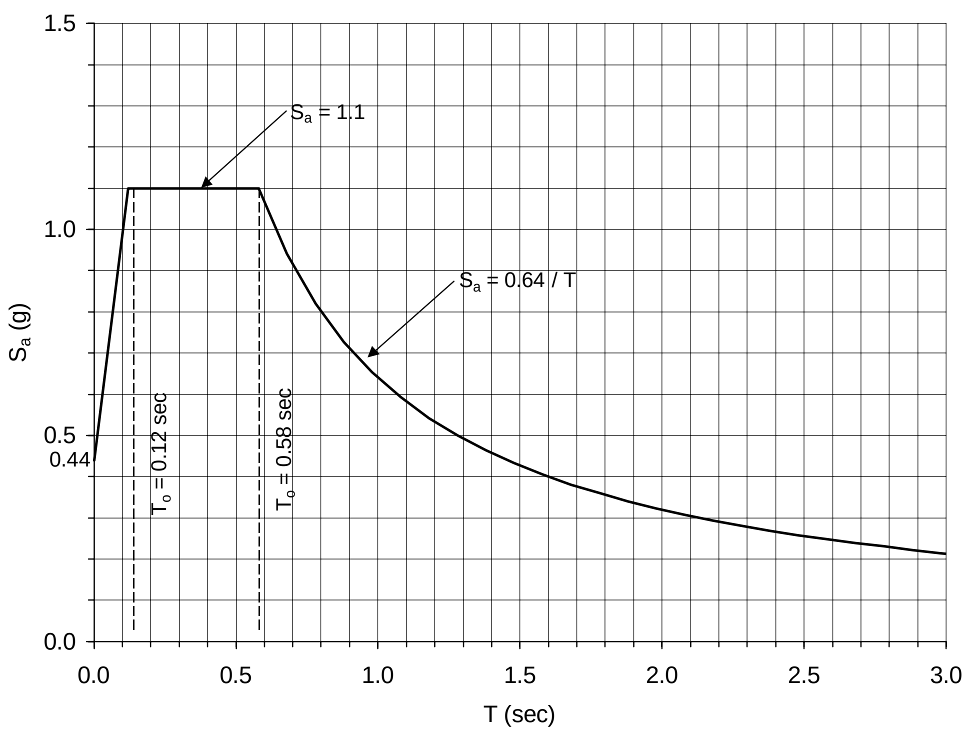
$$T_s = \frac{C_v}{2.5C_a} = \frac{0.64}{(2.5 \times .44)} = 0.58 \sec \qquad \text{Figure 16-3}$$

$$T_o = 0.2T_s = (0.2)(0.58) = 0.12 \sec$$

The long period portion of the spectrum is defined as

$$\frac{C_v}{T} = \frac{0.64}{T} \qquad \text{Figure 16-3}$$

From this information the elastic design response spectrum for the site can be drawn as shown below.



Commentary

The spectrum shown above is for 5 percent damping. If a different damping is used, the spectral accelerations of the control periods T_o and T_s and values of C_v / T must be scaled. However, the value of C_a is *not* scaled.

Example 34
Dual Systems §1631.5.7

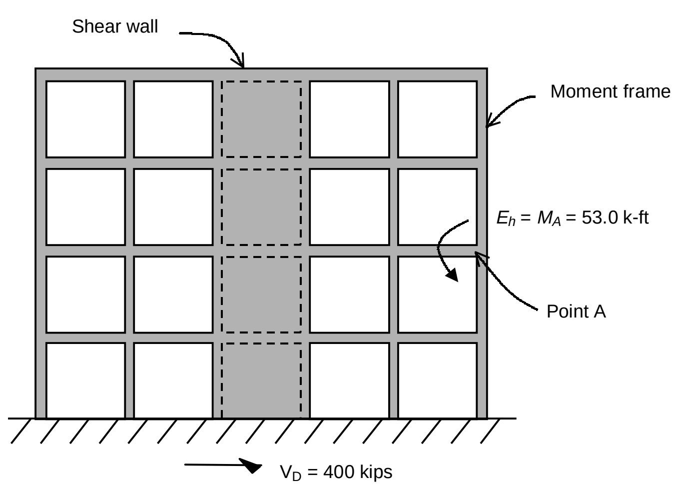
This example illustrates the determination of design lateral forces for the two basic elements of a dual system. Section 1629.6.5 prescribes the following features for a dual system:

1. An essentially complete space frame for gravity loads.
2. Resistance to lateral load is provided primarily by shear walls or braced frames, but moment-resisting frames must be provided to 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.

In present practice, the frame element design loads for a dual system are usually a result of a computer analysis of the combined frame-shear wall system.

In this example, a dynamic analysis using the response spectrum procedure of §1631.5 has been used to evaluate the seismic load E_h at point A in the dual system of the building shown below. This is the beam moment M_A. The building is classified as regular and the E_h values have been scaled to correspond to 90 percent of the design base shear determined under the requirements of §1630.2. The following information is given:

Zone 4
$I = 1.0$
Reduced dynamic base shear
$V_D = 0.9V = 400$ kips
$E_h = M_A = 53.0$ k-ft
$T = 0.50$ sec



Determine the following for the moment frame system:

1. **Design criteria.**

2. **Required design lateral seismic forces F_x.**

3. **Moment at A**

Calculations and Discussion — Code Reference

1. **Design criteria.**

Section 1629.6.5 Item 2 requires that the moment-resisting frame be designed to independently resist at least 25 percent of the design base shear, which in this case would be $0.25V_D$.

Section 1631.5.7 allows the use of either the static force method of §1630.5 or the response spectrum analysis of §1631.5, scaled to the $0.25V_D$ base shear.

Since the independent frame, without shear wall interaction, is an idealization that never really exists, the use of the response spectrum analysis is not particularly appropriate since the true dynamic characteristics would be those of the combined frame and wall system. The purpose of a response spectrum analysis is to better define the lateral load distribution, and this would not be achieved by an analysis of the independent frame. Therefore, the use of the static force option is judged to be more consistent with the simple requirement that the frame strength should meet or exceed $0.25V_D$.

$$\therefore V_D \text{ of frame} = 0.25V_D = 0.25\,(400) = \underline{\underline{100 \text{ kips}}}$$

2. **Required design lateral seismic forces F_x.**

Design base shear on the frame due to $0.25V_D = 100$ kips

This base shear must be distributed over the height of the structure, and the design lateral seismic forces at each level are determined from

$$F_x = \frac{(V - F_t)\,w_x h_x}{\Sigma w_i h_i} \qquad (30\text{-}15)$$

where

$$(V - F_t) = 0.25V_D = 100 \text{ kips}$$

In this example, $F_t = 0$ because the building period of 0.50 seconds is less than 0.7 seconds.

$$\therefore \; \underline{\underline{F_x = \frac{100 w_x h_x}{\Sigma w_i h_i}}}$$

3. Moment at A

Apply the F_x forces to the frame structure and find the resulting seismic moments, denoted M'_A. At point A,

$$E'_h = M'_A = 75.2 \text{ k-ft} > M_A = 53.0 \text{ k-ft}$$

The seismic moment at A must be the larger of the two values.

$$\therefore \underline{\underline{M'_A = 75.2 \text{ k} - \text{ft}}}$$

In actual application, each frame element load E_h due to V_D in the dual system must be compared with the E'_h value due to $0.25V_D$ in the independent frame, and the element must be designed for the larger of E_h or E'_h.

Commentary

Use of a dual system has the advantage of providing the structure with an independent vertical load carrying system capable of resisting 25 percent of the design base shear while at the same time the primary system, either shear wall or braced frame, carries its proportional share of the design base shear. For this configuration, the code permits use of a larger *R* value for the primary system than would be permitted without the 25 percent frame system.

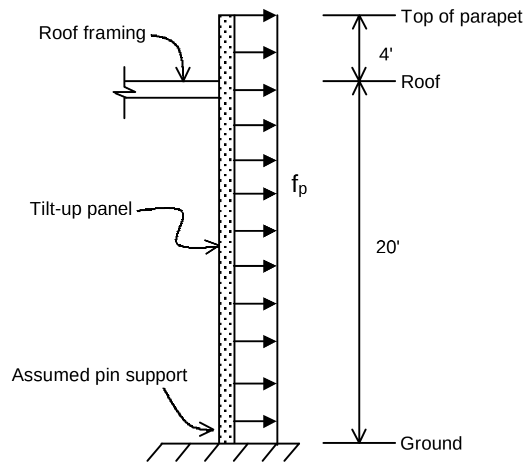
The dual system has been in the code for many years. The widespread use of computers in structural analysis revealed that the interaction between the frame and the shear wall (or braced frame) system produced results quite different than those obtained by the often cumbersome approximate methods used with hand calculations. For example, a shear wall system in a highrise building was found to be “loading” the frame system at the upper stories. Consequently, a dual system should be carefully analyzed as a combined system to detect critical interaction effects.

Example 35
Lateral Forces for One-Story Wall Panels §1632.2

This example illustrates the determination of the total design lateral seismic force on a tilt-up wall panel supported at its base and at the roof diaphragm level.

For the tilt-up wall panel shown below, determine the out-of-plane seismic forces required for the design of the wall section. This is usually done for a representative one-foot width of the wall length, assuming a uniformly distributed out-of-plane loading. The following information is given:

Zone 4
$I_p = 1.0$
$C_a = 0.4$
Panel thickness = 8 inches
Normal weight concrete (150 pcf)



Determine the following:

1. **Out-of-plane forces for wall panel design.**

2. **Shear and moment diagrams for wall panel design.**

3. **Loading, shear and moment diagrams for parapet design.**

Calculations and Discussion — Code Reference

1. **Out-of-plane forces for wall panel design.** **§1632.2**

Under §1632.2, design lateral seismic forces can be determined using either: a.) Equation (32-1), or b.) Equation (32-2) with the limits of Equation (32-3).

$$F_p = 4.0 C_a I_p W_p \tag{32-1}$$

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{32-2}$$

$$0.7 C_a I_p W_p \le F_p \le 4.0 C_a I_p W_p \tag{32-3}$$

Generally, it is more advantageous to use Equation (32-2) with the Equation (32-3) limits, and this will be used in this example.

The wall panel is laterally supported at its base and at the roof. The value of F_p to be used must represent the average of the acceleration inputs from these two attachment locations. Thus, the out-of-plane seismic forces on the wall panel are determined from the "average" of the seismic coefficients at the roof and the base. As will be shown below, the minimum force level from Equation (32-3) controls the seismic coefficient at the base.

Using the coefficient method, a general expression for the force F_p applied midway between the base and the top of the parapet is derived below.

$a_p = 1.0$ Table 16-O

$R_p = 3.0$ Table 16-O

At roof level, $h_x = h_r$, and the effective seismic coefficient from Equation (32-2) is

$$\frac{(1.0)C_a I_p}{3.0}\left(1+3\frac{h_r}{h_r}\right) = 1.33 C_a I_p < 4.0 C_a I_p$$

$\therefore$ use $1.33 C_a I_p$

At base level, $h_x = 0$, and the effective seismic coefficient from Equation (32-2) is

$$\frac{(1.0)C_a I_p}{3.0}\left(1+3\frac{0}{h_r}\right) = 0.33 C_a I_p < 0.7 C_a I_p$$

$\therefore$ use $0.7 C_a I_p$

The average coefficient over the entire height of the wall may be taken as

$$\frac{(1.33+0.70)}{2} C_a I_p = 1.02 C_a I_p$$

The force F_p is considered to be applied at the mid-height (centroid) of the panel, but this must be uniformly distributed between base and top of parapet.

For the given $C_a = 0.4$ and $I_p = 1.0$, the wall panel seismic force is

$$F_p = 1.02\,(0.4)(1.0)W_p = 0.408 W_p$$

The weight of the panel between base and the top of the parapet is

$$W_p = \left(\frac{8}{12}\right)(150)\ (24) = 2{,}400 \text{ lbs} \ \text{ per foot of width}$$

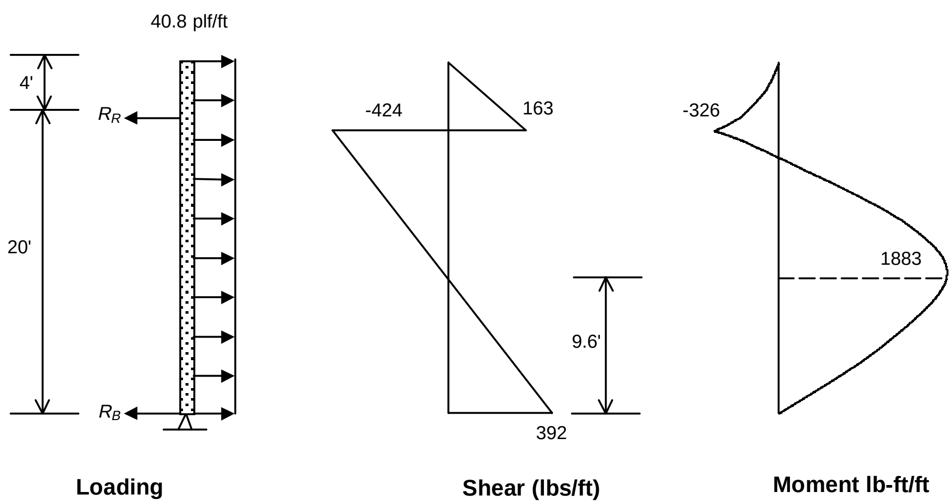
$$F_p = 0.408\,(2{,}400) = \underline{\underline{979 \text{ lbs/ft}}}$$

The force F_p is the total force on the panel. It acts at the centroid. For design of the panel for out-of-plane forces, F_p must be expressed as a distributed load f_p:

$$f_p = \frac{979 \text{ lbs/ft}}{24\,ft} = \underline{\underline{40.8 \text{ plf/ft}}}$$

2. **Shear and moment diagrams for wall panel design.**

Using the uniformly distributed load f_p, the loading, shear and moment diagrams are determined for a unit width of panel. The 40.8 plf/ft uniform loading is also applied to the parapet. See step 3, below, for the parapet design load.



When the uniform load is also applied to the parapet, the total force on the panel is

$$40.8 \text{ plf/ft } (24\text{ft}) = 979 \text{ plf}$$

$$R_R = \frac{979\ (12)}{20} = 587 \text{ lb/ft}$$

$$R_B = 979 - 587 = 392 \text{ lb/ft}$$

The shears and moments are the E_h load actions for strength design. However, the reaction at the roof, R_R, is not the force used for the wall-roof anchorage design.

This anchorage force must be determined under §1633.2.8.1 when the roof is a flexible diaphragm.

3. Loading, shear and moment diagrams for parapet design.

Table 16-O requires $a_p = 2.5$, and $R_p = 3.0$ for unbraced (cantilevered) parapets.

The parapet is considered as an element with an attachment elevation at the roof level.

$$h_x = h_r$$

The weight of the parapet is

$$W_p = \left(\frac{8}{12}\right)(150)(4) = 400 \text{ lbs} \text{ per foot of width}$$

The concentrated force applied at the mid-height (centroid) of the parapet is determined from Equation (32-2).

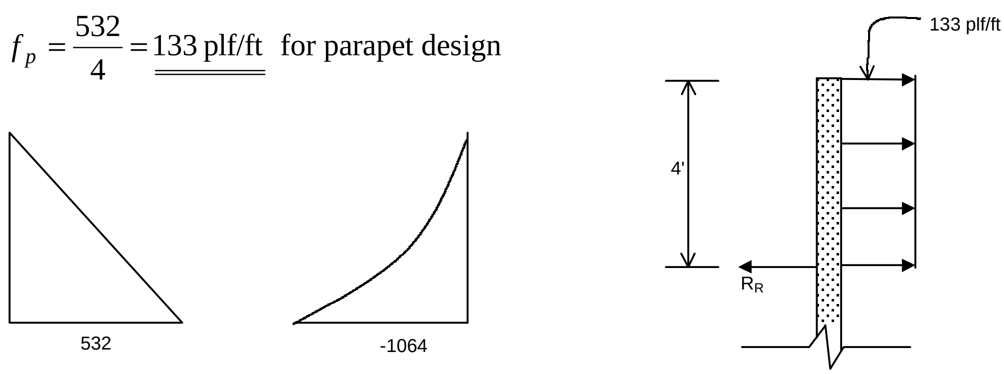
$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

$$F_p = \frac{2.5(0.4)(1.0)}{3.0}\left(1 + 3\frac{20}{20}\right)W_p$$

$$F_p = 1.33W_p = 1.33(400) = 532 \text{ lbs/ft} < 4.0C_a I_p W_p = 1.6W_p \quad o.k.$$

The equivalent uniform seismic force is

$$f_p = \frac{532}{4} = \underline{\underline{133 \text{ plf/ft}}} \text{ for parapet design}$$

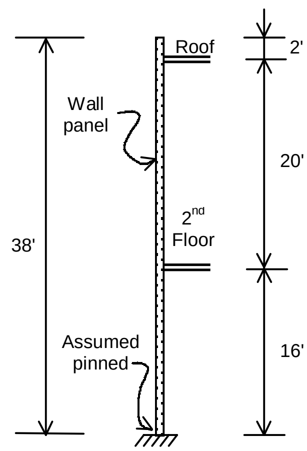


Shear (lbs/ft) **Moment (lb-ft/ft)** **Loading**

Example 36
Lateral Forces for Two-Story Wall Panel §1632.2

This example illustrates determination of out-of-plane seismic forces for the design of the two-story tilt-up wall panel shown below. In this example, a typical solid pane (no door or window openings) is assumed. Walls span from floor to floor to roof. The typical wall panel in this building has no pilasters and the tilt-up walls are bearing walls. The roof consists of 1½-inch, 20 gauge metal decking on open web steel joists and is considered a flexible diaphragm. The second floor consists of 1½-inch, 18 gauge composite decking with a 2½-inch lightweight concrete topping. This i considered a rigid diaphragm. The following information is given:

Zone 4
$I_p = 1.0$
$C_a = 0.4$
Wall weight = 113 psf



Wall sectio

Determine the following:

1. **Out-of-plane forces for wall panel design.**

2. **Out-of-plane forces for wall anchorage design.**

Calculations and Discussion — Code Reference

1. **Out-of-plane forces for wall panel design.** **§1632.2**

Requirements for out-of-plane seismic forces are specified in §1632.2 for Zones 3 and 4. Either Equations (32-1) or (32-2) and (32-3) are used to determine the forces on the wall.

$$F_p = 4.0 C_a I_p W_p \qquad (32\text{-}1)$$

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3 \times \frac{h_x}{h_r}\right) W_p \qquad (32\text{-}2)$$

$$0.7 C_a I_p W_p \le F_p \le 4.0 C_a I_p W_p \qquad (32\text{-}3)$$

$R_p = 3.0$ and $a_p = 1.0$ Table 16-O, Item 1.A.(2)

To determine out-of-plane forces over the height of the wall, seismic coefficients at the roof, second floor, and first floor are determined. An out-of-plane force, F_p, is determined for each story from the average of the seismic coefficients at the support points for that story. The required coefficients are evaluated as follows.

Seismic coefficient at roof:

$$\frac{a_p C_a I_p}{R_p}\left(1+3\times\frac{h_x}{h_r}\right)=\frac{1.0\,(0.4)\,(1.0)}{3.0}\left(1+3\times\frac{36}{36}\right)=0.533$$

$$4.0C_a I_p = 4.0\,(0.4)\,(1.0) = 1.60 > 0.533$$

$$\therefore \text{ use } 0.533$$

Seismic coefficient at second floor:

$$\frac{a_p C_a I_p}{R_p}\left(1+3\times\frac{h_x}{h_r}\right)=\frac{1.0\,(0.4)\,(1.0)}{3.0}\left(1+3\times\frac{16}{36}\right)=0.311$$

Seismic coefficient at first floor:

$$\frac{a_p C_a I_p}{R_p}\left(1+3\times\frac{h_x}{h_r}\right)=\frac{1.0\,(0.4)\,(1.0)}{3.0}\left(1+3\times\frac{0}{36}\right)=0.133$$

$$0.7C_a I_p = 0.7\,(0.4)\,(1.0) = 0.28 > 0.133$$

$$\therefore \text{ use } 0.28$$

Using the average of the coefficient for the given story, the out-of-plane seismic forces are determined as follows:

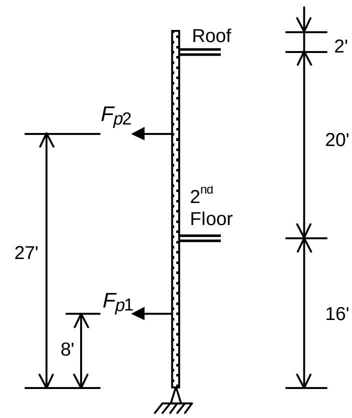
$$W_{p2} = 113\,(20+2) = 2{,}486 \text{ plf}$$

$$W_{p1} = (113)\,(16) = 1{,}808 \text{ plf}$$

$$F_{p2} = \frac{(0.533+0.311)}{2}W_{p2} = 0.422W_{p2} = 0.422\,(2{,}486) = \underline{\underline{1{,}049 \text{ plf}}}$$

$$F_{p1} = \frac{(0.311+0.280)}{2}W_{p1} = 0.296W_{p1} = 0.296\,(1{,}808) = \underline{\underline{535 \text{ plf}}}$$

F_{p2} and F_{p1} are the out-of-plane forces acting on the centroids of the second and first level portions, respectively, of the tilt-up wall panel. For design of the wall these forces must be uniformly distributed over their tributary height. Panel desi forces are given below.



Out-of-plane forces at centroids

$$f_{p2} = \frac{F_{p2}}{(20+2)} = \frac{1{,}049}{22} = \underline{\underline{47.7 \text{ plf}}}$$

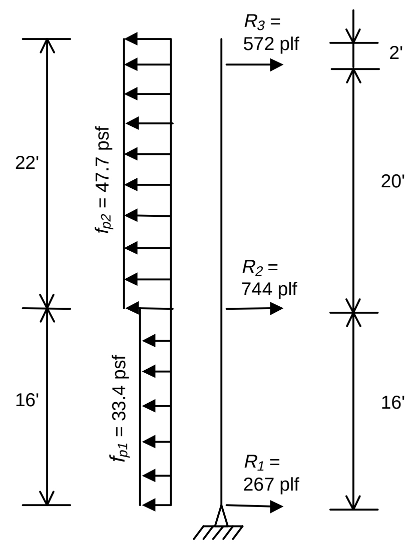
$$f_{p1} = \frac{F_{p1}}{16} = \frac{535}{16} = \underline{\underline{33.4 \text{ plf}}}$$

Alternatively, panel design forces can be determined using seismic coefficients as shown below.

$$f_{p2} = .422\,(113) = \underline{\underline{47.7 \text{ psf}}}$$

$$f_{p1} = .296\,(113) = \underline{\underline{33.4 \text{ psf}}}$$

Note that the 2-foot high parapet must be designed for seismic forces determined from Equations (32-2) and (32-3) with $R_p = 3.0$ and $a_p = 2.5$. This calculation is not shown.



Out-of-plane wall forces

2. Out-of-plane forces for wall anchorage design. §1633.2.8.1

For design of wall anchorage, §1633.2.8.1 requires use of higher design forces than those used for panel design. Anchorage forces are determined using Equations (32-1), or (32-2) and (32-3), where W_p is the weight of the panel tributary to each anchorage level. Values of R_p and a_p to be used at the second floor and roof are:

$R_p = 3.0$ and $a_p = 1.5$ §1633.2.8.1, Item 1

The building of this example has a flexible diaphragm at the roof and a rigid diaphragm at the second floor. Because the code is not clear about wall anchorage requirements for buildings with both rigid and flexible diaphragms, the requirements for flexible diaphragms will be used for determination of anchorage forces at *both* levels. Equation (32-3), with the limits of Equation (32-3), will be used with h_x equal to the attachment height of the anchorage.

Seismic anchorage force at roof:

$$W_3 = (113)\left(\frac{20}{2} + 2\right) = \underline{\underline{1{,}356 \text{ plf}}}$$

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

$$F_3 = \frac{1.5\,(0.4)\,(1.0)}{3.0}\left(1 + 3\frac{36}{36}\right)W_3 = 0.8\,(1{,}356) = 1{,}085 \text{ plf}$$

Check limit of Equation (32-3)

$$4.0 C_a I_p W_p = 4.0\,(0.4)\,(1.0)\,W_3 = 1.6W_3 > 0.8W_3 \quad o.k. \qquad (32\text{-}3)$$

$1{,}085 \text{ plf} > 420 \text{ plf}$ §1633.2.8.1, Item 1

$$\therefore\ F_3 = \underline{\underline{1{,}085 \text{ plf}}}$$

Seismic anchorage force at second floor:

$$W_2 = (113)\left(\frac{20 + 16}{2}\right) = 2{,}034 \text{ plf}$$

$$F_2 = \frac{1.5\,(0.4)\,(1.0)}{3.0}\left(1 + 3\frac{16}{36}\right)W_2 = .467\,(2{,}034) = \underline{\underline{950 \text{ plf}}} \qquad (32\text{-}2)$$

Seismic anchorage force at first floor:

At the first floor, $a_p = 1.0$ because there is no diaphragm.

$$W_1 = (113)\left(\frac{16}{2}\right) = 904 \text{ plf}$$

$$F_1 = \frac{1.0\,(0.4)\,(1.0)}{3.0}\left(1 + 3\frac{0}{36}\right)W_1 = 0.133W_1 \qquad (32\text{-}2)$$

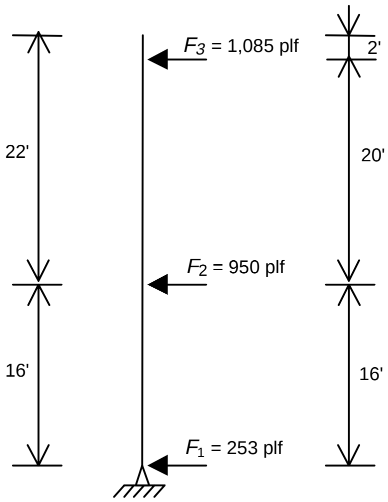
Check limit of Equation (32-3)

$$0.7C_a I_p W_p = 0.7(0.4)(1.0)W_1 = 0.28W_1 \text{ controls} \qquad (32\text{-}3)$$

$$\therefore\ F_1 = 0.28W_1 = 0.28(904) = \underline{\underline{253 \text{ plf}}}$$

Note that the 420 plf minimum anchorage force of §1633.2.8.1, Item 1 does not apply at the first floor.

Wall reactions for anchorage design are shown at right.



Wall anchorage forces

Commentary

Anchorage forces have been determined on the basis of the weight tributary to each level using Equation (32-2), with limits of Equation (32-3) and §1633.2.8.1, Item 1. Panel forces, on the other hand, have been determined using seismic coefficients for each floor level. If reactions are determined from the uniform out-of-plane forces used for panel design, these will be different than those determined for anchorage requirements. This inconsistency is rooted in the fact that the code does not call for determination of both panel design forces and anchorage design forces from the same method. To be consistent, forces would have to first be determined at the panel centroids (between floors) and then anchorage reactions determined from statics equilibrium.

In all significant California earthquakes, beginning with the 1971 San Fernando event, wall-roof anchorage for flexible diaphragms has failed repeatedly. After the 1994 Northridge earthquake, when over 200 tilt-up buildings in the city of Los Angeles experienced collapse or partial collapse of roofs and/or walls, wall-roof anchorage forces were increased significantly in the 1996 Supplement to the 1994 UBC. The 1997 UBC requirements reflect this change. It is extremely important that bearing wall tilt-up buildings maintain wall-roof (and wall-floor) connections under seismic motions. This is the principal reason that anchorage forces are 50-percent higher than those used for out-of-plane wall panel design.

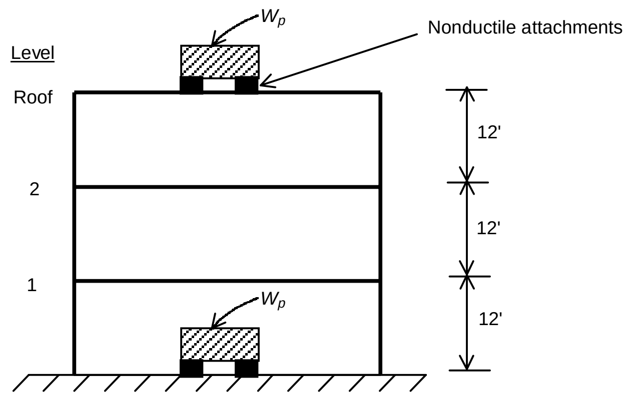
See §1633.2.8.1 for the special material load factors used for the design of steel and wood elements of the wall anchorage system (i.e., 1.4 for steel and 0.85 for wood).

Example 37
Rigid Equipment §1632.2

This example illustrates determination of the design seismic force for the attachments of rigid equipment. Attachment as used in the code means those components, including anchorage, bracing, and support mountings, that "attach" the equipment to the structure.

The three-story building structure shown below has rigid electrical equipment supported on nonductile porcelain insulators that provide anchorage to the structure. Identical equipment is located at the base and at the roof of the building.

Zone 4
$C_a = 0.4$
$I_p = 1.0$
$W_p = 10$ k



Find the following:

1. **Design criteria.**
2. **Design lateral seismic force at base.**
3. **Design lateral seismic force at roof.**

Calculations and Discussion *Code Reference*

1. Design criteria. §1632.2

The total design lateral seismic force is determined from

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{32-2}$$

Values of a_p and R_p are given in Table 16-O. Since the equipment is rigid and has nonductile attachments

$a_p = 1.0, R_p = 1.5$ Table 16-O, Item 4B

2. **Design lateral seismic force at base.** **§1632.2**

$$h_x = 0$$

$$F_p = \frac{(1.0)(0.4)(1.0)}{(1.5)}\left(1 + 3\frac{0}{36}\right)(10) = 2.67 \text{ k}$$

Section 1632.2 has a requirement that F_p be not less than $0.7 C_a I_p W_p$ (32-3)

Check $F_p \geq 0.7 C_a I_p W_p = 0.7\,(0.4)\,(1.0)\,10 = 2.8 \text{ k}$

$$\therefore F_p = \underline{\underline{2.8 \text{ k}}}$$

3. **Design lateral seismic force at roof.**

$$h_x = h_r = 36 \text{ ft}$$

$$F_p = \frac{(1.0)(0.4)(1.0)}{(1.5)}\left(1 + 3\frac{36}{36}\right)(10) = 10.7 \text{ k}$$

Section 1632.2 states that F_p need not exceed $4 C_a I_p W_p$ (32-3)

Check $F_p \leq 4 C_a I_p W_p = 4\,(0.4)\,(1.0)\,10 = 16 \text{ k}$

$$\therefore F_p = \underline{\underline{10.7 \text{ k}}}$$

Commentary

The definition of a rigid component (e.g., item of equipment) is given in §1627. Rigid equipment is equipment, including its attachments (anchorages, bracing, and support mountings), that has a period less than or equal to 0.06 seconds.

The anchorage design force F_p is a function of $1/R_p$, where $R_p = 1.0, 1.5,$ and 3.0 for nonductile, shallow, and ductile anchors, respectively.

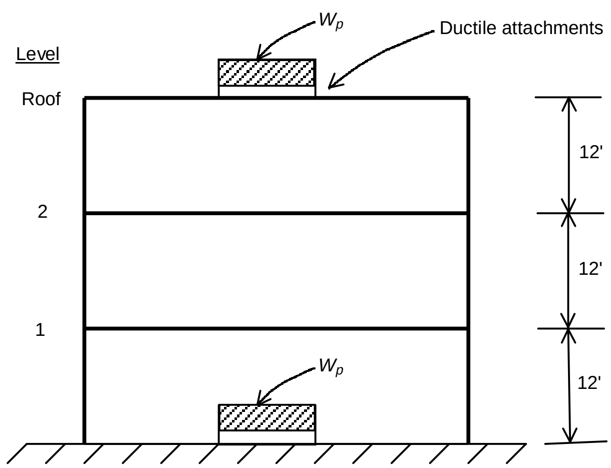
Generally, only equipment anchorage or restraints need be designed for seismic forces. This is discussed in Footnote 5 of Table 16-O. Item 3.C, also in Table 16-O states that this applies to “Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass.” For the case where equipment, which can be either flexible or rigid, comes mounted on a supporting frame that is part of the manufactured unit, then the supporting frame must also meet the seismic design requirements of §1632.2.

Example 38
Flexible Equipment §1632.2

This example illustrates determination of the design seismic force for the attachments of flexible equipment. Attachment as used in the code means those components, including anchorage, bracing, and support mountings, that "attach" the equipment to the structure.

The three-story building structure shown below has flexible air-handling equipment supported by a ductile anchorage system. Anchor bolts in the floor slab meet the embedment length requirements. Identical equipment is located at the base and at the roof of the building.

Zone 4
$C_a = 0.4$
$I_p = 1.0$
$W_p = 10$ k



Find the following:

1. **Design criteria.**

2. **Design lateral seismic force at base.**

3. **Design lateral seismic force at roof.**

Calculations and Discussion — Code Reference

1. **Design criteria.** **§1632.2**

The total design lateral seismic force is determined from

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

Values of a_p and R_p are given in Table 16-O. Since the equipment is flexible and has ductile supports

$a_p = 2.5, R_p = 3.0$ Table 16-O, Item 3C

2. Design lateral seismic force at base.

$h_x = 0$

$$F_p = \frac{(2.5)(0.4)(1.0)}{(3.0)}\left(1 + 3\frac{0}{36}\right)(10) = 3.33\text{ k}$$

Section 1632.2 has a requirement that F_p be not less than $0.7C_a I_p W_p$ (32-3)

Check $F_p \geq 0.7C_a I_p W_p = 0.7\,(0.4)\,(1.0)\,10 = 2.8\text{ k}$

$\therefore F_p = \underline{\underline{3.33\text{ k}}}$

3. Design lateral seismic force at roof.

$h_x = h_r = 36\text{ ft}$

$$F_p = \frac{(2.5)(0.4)(1.0)}{(3.0)}\left(1 + 3\frac{36}{36}\right)(10) = 13.33\text{ k}$$

Section 1632.2 states that F_p need not exceed $4C_a I_p W_p$ (32-3)

Check $F_p \leq 4C_a I_p W_p = 4\,(0.4)\,(1.0)\,10 = 16\text{ k}$

$\therefore F_p = \underline{\underline{13.33\text{ k}}}$

Commentary

The definition of flexible equipment is given in §1627. Flexible equipment is equipment, including its attachments (anchorages, bracing, and support mountings), that has a period greater than 0.06 seconds.

It should be noted that the anchorage design force F_p is a function of $1/R_p$, where $R_p = 1.0, 1.5,$ and 3.0 for nonductile, shallow, and ductile anchors, respectively.

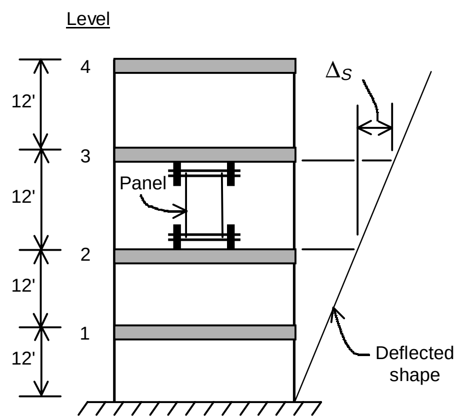
Generally, only equipment anchorage or restraints need be designed for seismic forces. This is discussed in Footnote 5 of Table 16-O. Item 3.C of that table states that this applies to "Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass." For the case where the equipment, which can be either flexible or rigid, comes mounted on a supporting frame that is part of the manufactured unit, then the supporting frame must also meet the seismic design requirements of §1632.2.

Example 39
Relative Motion of Equipment Attachments §1632.4

Section 1632.4 of the UBC requires that the design of equipment attachments in buildings having occupancy categories 1 and 2 of Table 16-K, essential facilities and hazardous facilities, respectively, have the effects of the relative motion of attachment points considered in the lateral force design. This example illustrates application of this requirement.

A unique control panel frame is attached to the floor framing at Levels 2 and 3 of the building shown below. The following information is given.

Zone 4
Occupancy Category 1,
(essential facility)
Story drift: $\Delta_S = 0.34$ in.
$R = 8.5$
Panel frame: $EI = 10 \times 10^4$ k/in.^{2}



Determine the following:

1. **Story drift to be considered.**

2. **Induced moment and shear in frame.**

Calculations and Discussion — Code Reference

1. **Story drift to be considered.**

Section 1632.4 requires that equipment attachments be designed for effects induced by Δ_M (maximum inelastic story drift). This is determined as follows:

$$\Delta_M = 0.7R\Delta_S = 0.7(8.5)0.34 = \underline{\underline{2.02 \text{ in}}}. \qquad (30\text{-}17)$$

2. **Induced moment and shear in frame.** **§1632.4**

$$M = \frac{6EI\Delta_M}{H^2} = \frac{6\left(10 \times 10^4\right)(2.02)}{(144)^2} = \underline{\underline{58.45 \text{ k - in}}}.$$

$$V = \frac{M}{(H/2)} = \frac{58.45}{72} = \underline{\underline{0.81 \text{ k}}}$$

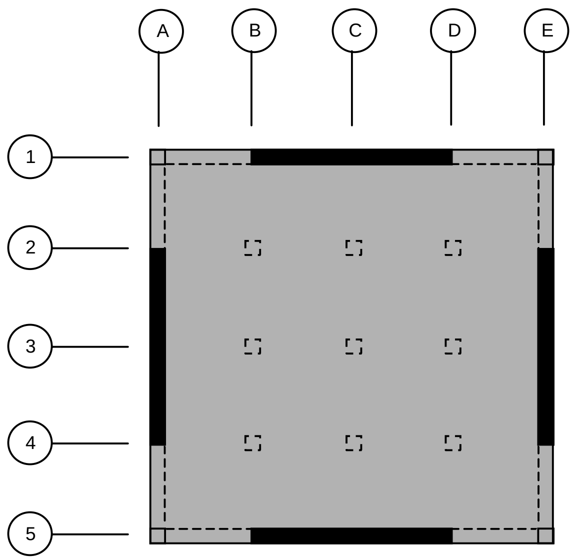
Commentary

The attachment details, including the body and anchorage of connectors, should follow the applicable requirements of §1632.2. For example, if the body of the attachment is ductile, then the induced forces can be reduced by $R_p = 3.0$. However, if the anchorage is provided by shallow anchor bolts, then $R_p = 1.5$.

When anchorage is constructed of nonductile materials, $R_p = 1.0$. One example of a nonductile anchorage is the use of adhesive. Adhesive is a "glued" attachment (e.g., attachment of pedestal legs for a raised computer floor). It should be noted that attachment by adhesive is not the same as anchor bolts set in a drilled hole with epoxy.

Example 40
Deformation Compatibility §1633.2.4

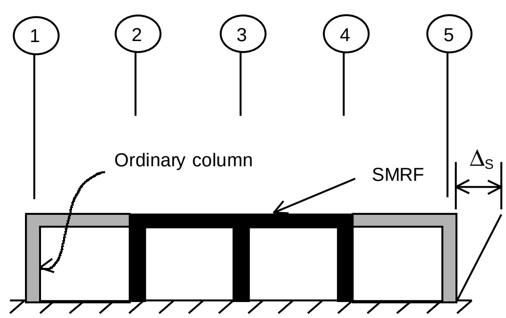
A two-level concrete parking structure has the space frame shown below. The designated lateral force-resisting system consists of a two bay special moment-resisting frame (SMRF) located on each side of the structure. The second level gravity load bearing system is a post-tensioned flat plate slab supported on ordinary reinforced concrete columns,



Plan at second level

The following information is given:

Zone 4
$\Delta_S = 0.42$ in.
$R = 8.5$
Column section = 12 in. x 12 in.
Column clear height = 12 ft
Concrete $E_c = 3 \times 10^3$ ksi



Elevation Line E

Find the following:

1. **Moment in ordinary column.**

2. **Detailing requirements for ordinary column.**

Calculations and Discussion — Code Reference

1. Moment in ordinary column. **§1633.2.4**

Section 1921.7 specifies requirements for frame members that are not part of the designated lateral force-resisting system. The ordinary columns located in the perimeter frames, and the interior flat plate/column system, fall under these requirements and must be checked for the moments induced by the maximum inelastic response displacement. For this example, the columns on Line E will be evaluated.

$$\Delta_M = 0.7R\Delta_S = 0.7(8.5)0.42 = 2.50 \text{ in.} \tag{30-17}$$

Section 1633.2.4 requires that the value of Δ_S used for this determination of Δ_M be computed by neglecting the stiffening effect of the ordinary concrete frame.

The moment induced in the ordinary column due to the maximum inelastic response displacement Δ_M on Line E must be determined.

For purposes of this example, a fixed-fixed condition is used for simplicity. In actual applications, column moment is usually determined from a frame analysis.

$$M_{col} = \frac{6E_c I_c \Delta_M}{h^2}$$

$$h = 12 \times 12 = 144 \text{ in.}$$

$$I_g = \frac{bd^3}{12} = 12\frac{(12)^3}{12} = 1728 \text{ in.}^4$$

The cracked section moment of inertia I_c can be approximated as 50 percent of the gross section I_g. Section 1633.2.4 requires that the stiffness of elements that are part of the lateral force-resisting system shall not exceed one half of the gross section properties. This requirement also applies to elements that are not part of the lateral force-resisting system.

$$I_c = \frac{I_g}{2} = 864 \text{ in.}^4$$

$$M_{col} = \frac{6\left(3 \times 10^3\right)(864)(2.5)}{(144)^2} = \underline{\underline{1875 \text{ k} - \text{in.}}}$$

2. **Detailing requirements for ordinary column.**

Section 1921.7 requires that frame members, such as the column, that are assumed not to be part of the lateral force-resisting system must be detailed according to §1921.7.2 or §1921.7.3, depending on the magnitude of the moments induced by Δ_M .

Commentary

In actual applications, the flat plate slab must be checked for flexure and punching shear due to gravity loads and the frame analysis actions induced by Δ_M .

Section 1633.2.4 requires that the stiffening effect of those elements not part of the lateral force-resisting system shall be neglected in the structural model used for the evaluation of Δ_M . To evaluate the force induced by Δ_M in the elements not part of the lateral force-resisting system when using frame analysis, it is necessary to formulate an additional structural model that includes the stiffening effect of these elements. This model should be loaded by the same lateral forces used for the evaluation of Δ_M to obtain the corresponding element forces F'_M and displacement Δ'_M . The required element forces F_M induced by Δ_M can then be found by:

$$F_M = \frac{\Delta_M}{\Delta'_M}\left(F'_M\right)$$

The values used for the displacements Δ_M and Δ'_M should be those corresponding to the frame line in which the element is located.

Section 1633.2.4 also requires the consideration of foundation flexibility and diaphragm deflections in the evaluation of displacement. The following criteria and procedures may be used for this consideration:

1. **Foundation Flexibility**
   If the design strength capacity at the foundation-soil interface is less than the combined loads resulting from the special load combinations of §1612.4, then the lateral stiffness of the supported shear wall, braced frame, or column shall be reduced by a factor of .5.

2. **Diaphragm Deflection**
   For a given diaphragm span between two lateral force-resisting elements, compare the mid-span diaphragm deflection for a given uniform load with the average of the story drifts of the two lateral force-resisting elements due to the reactions from the diaphragm load. If the diaphragm deflection exceeds 20 percent of the average story drift, then include diaphragm deflection in Δ_M .

Otherwise, for cases where the effects are critical for design, a soil-spring model of the foundation and/or a finite element model of the diaphragm may be required.

Example 41
Adjoining Rigid Elements §1633.2.4.1

During the 1994 Northridge earthquake in southern California, nonductile concrete and masonry elements in frame structures with ductile lateral force-resisting systems experienced failure because they lacked deformation compatibility. Deformation compatibility refers to the capacity of nonstructural elements, or structural elements not part of the lateral force system, to undergo seismic displacements without failure. It also implies that structural elements of the lateral force system will not be adversely affected by the behavior of nonstructural or nonseismic structural elements.

The 1997 UBC has new requirements for deformation compatibility. These are given in §1633.2.4.1. The purpose of this example is to illustrate use of these requirements.

The concrete special moment-resisting frame shown below is restrained by the partial height infill wall. The infill is solid masonry and has no provision for an expansion joint at the column faces. The maximum deflection Δ_M was computed neglecting the stiffness of the nonstructural infill wall, as required by §1633.2.4.

Zone 4
$\Delta_M = 2.5''$

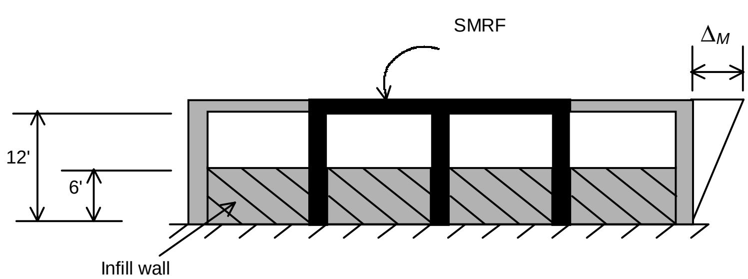
Column properties:

$f'_c = 3{,}000$ psi

$E_c = 3 \times 10^3$ ksi

$A_c = 144$ in.^{4}

$I_c = 854$ in.^{4}



Typical elevatio

Determine the following:

1. **Deformation compatibility criteria.**

2. **Approximate column shear.**

Calculations and Discussion — Code Reference

1. Deformation compatibility criteria. §1633.2.4.1

The infill wall, which is not required by the design to be part of the lateral force-resisting system, is an adjoining rigid element. Under §1633.2.4.1, it must be shown that the adjoining rigid element, in this case the masonry infill wall, must not impair the vertical or lateral load-resisting ability of the SMRF columns. Thus, the columns must be checked for ability to withstand the Δ_M displacement of 2.5 inches while being simultaneously restrained by the 6-foot-high infill walls.

2. **Approximate column shear.**

Column shear will be determined from the frame inelastic displacement Δ_M. For purposes of the example, the expression for the fixed-fixed condition will be used for simplicity.

$$V_{col} = \frac{12E_c I_c \Delta_M}{h^3} = \frac{12\left(3\times10^3\right)(854)(2.5)}{(72)^3} = \underline{\underline{205.9 \text{ kips}}}$$

Column clear height $= 72 \text{ in}$

Because the SMRF is the primary lateral force-resisting system, Δ_M is to be determined by neglecting the stiffness of the ordinary columns and the rigid masonry infill per §1633.2.4.

The induced column shear stress is $\dfrac{V_{col}}{A_c} = 1{,}447 \text{ psi}$. This is approximately $26\sqrt{f'_c}$ and would result in column shear failure. Therefore, a gap must be provided between the column faces and the infill walls. Alternately, it would be necessary to either design the column for the induced shears and moments caused by the infill wall, or demonstrate that the wall will fail before the column is damaged. Generally, it is far easier (and more reliable) to provide a gap sufficiently wide to accommodate Δ_M.

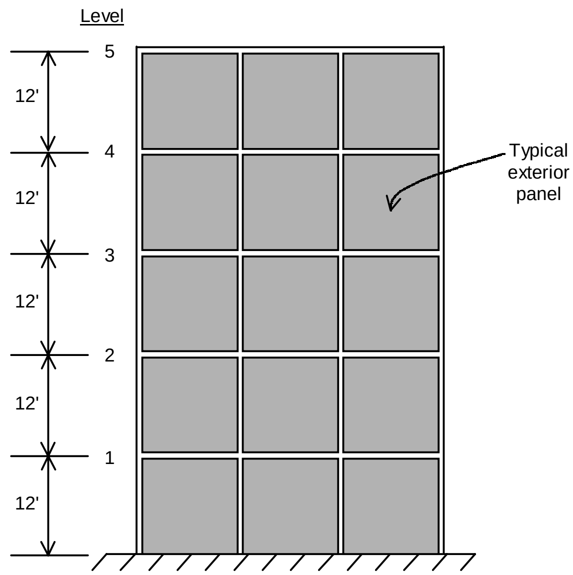
For this example, with the restraining wall height equal to one half the column height, the gap should be greater than or equal to $\dfrac{\Delta_M}{2} = 1.25 \text{ in}$. If this were provided, the column clear height would be 144 inches, with resulting column shear $V'_{col} = \dfrac{12\left(3\times10^3\right)(854)(2.5)}{(144)^3} = 25.7 \text{ kips}$. This is $\dfrac{1}{8}$ of the restrained column shear of 205.9 kips.

Example 42
Exterior Elements: Wall Panel §1633.2.4.2

This example illustrates the determination of the design lateral seismic force, F_p, on an exterior element of a building, in this case an exterior wall panel.

A five-story moment frame building is shown below. The cladding on the exterior of the building consists of precast reinforced concrete wall panels. The following information is known:

Zone 4
$I_p = 1.0$
$C_a = 0.4$
Panel size : 11'-11" x 19'-11"
Panel thickness: 6 in.
Panel weight: $W_p = 14.4$ kips



Find the following:

1. **Design criteria.**

2. **Design lateral seismic force on a panel at the fourth story.**

3. **Design lateral seismic force on a panel at the first story.**

Calculations and Discussion — Code Reference

1. **Design criteria.** **§1632.2**

For design of exterior elements, such as the wall panels on a building, that are attached to the building at two levels, design lateral seismic forces are determined from Equation (32-2). The panels are attached at the two elevations h_L and h_U. The intent of the code is to provide a value of F_p that represents the average of the acceleration inputs from the two attachment locations. This can be taken as the average of the two F_p values at h_x equal to h_L and h_U.

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \geq 0.7 C_a I_p W_p \qquad (32\text{-}2)$$

$a_p = 1.0, R_p = 3.0$ Table 16-O

2. **Design lateral seismic force on a panel at the fourth story.**

Assuming connections are 1 foot above and below the nominal 12-foot panel height

$h_U = 47$ ft

$h_L = 37$ ft

$h_r = 60$ ft

$$F_{pU} = \frac{(1.0)(0.4)(1.0)}{(3.0)}\left[1 + 3\left(\frac{47}{60}\right)\right]W_p = 0.447W_p$$

$$F_{pL} = \frac{(1.0)(0.4)(1.0)}{(3.0)}\left[1 + 3\left(\frac{37}{60}\right)\right]W_p = 0.380W_p$$

$$F_{p4} = \frac{F_{pU} + F_{pL}}{2} = \frac{(0.447 + 0.380)}{2}W_p$$

$$F_{p4} = 0.414W_p = (0.414)(14.4) = \underline{\underline{5.96 \text{ kips}}}$$

Check: $F_{p4} > 0.7 C_a I_p W_p = 0.7\,(0.4)(1.0)W_p = 0.2W_p$ *o.k.* (32-3)

3. **Design lateral seismic force on a panel at the first story.**

The following are known

$h_U = 11$ft

$h_L = 0$

$h_r = 60$ft

$$F_{pU} = \frac{(1.0)(0.4)(1.0)}{(3.0)}\left[1 + 3\left(\frac{11}{60}\right)\right]W_p = 0.207W_p$$

Check that F_{pU} is greater than $0.7 C_a I_p W_p$

$$F_{pU} = 0.7(0.4)(1.0)W_p = 0.28W_p \quad \textbf{not } o.k.$$

Also $F_{pL} < F_{pU} < 0.28W_p$

$\therefore$ use $F_{pL} = F_{pU} = 0.28W_p$

$$F_{p1} = \frac{F_{pU} + F_{pL}}{2} = 0.28W_p = (0.28)(14.4) = \underline{\underline{4.03\text{k}}}$$

Commentary

The design lateral seismic force F_p is to be used for the design of the panel for out-of-plane seismic forces. This can be represented by a distributed load equal to F_p divided by the panel area.

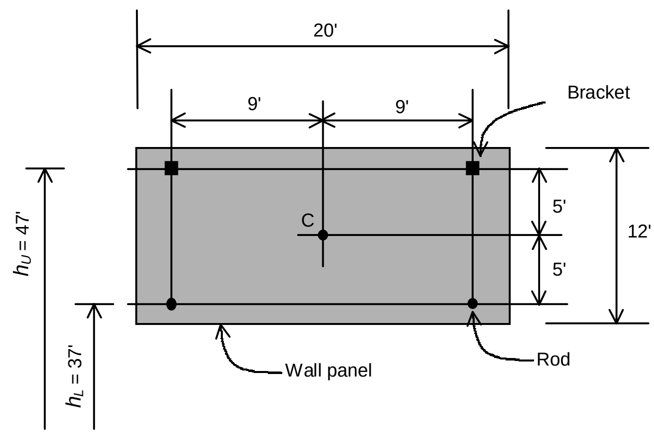
Note that the §163.2.4.2 Item 1 requirement to accommodate the relative movement of Δ_M is about twice the equivalent value of the previous code.

Example 43
Exterior Elements: Precast Panel §1633.2.4.2

This example illustrates the determination of the total design seismic lateral force for the design of the connections of an exterior wall panel to a building. Design of the body of the panel is often controlled by the non-seismic load conditions of the fabrication, transport, and erection.

An exterior nonbearing panel is located at the fourth story of a five-story moment frame building. The panel support system is shown below, where the pair of upper brackets must provide resistance to out-of-plane wind and seismic forces and in-plane vertical and horizontal forces. The panel is supported vertically from these brackets. The lower pair of rod connections provide resistance to only the out-of-plane forces.

Zone 4
$C_a = 0.4$
$I_p = 1.0$
Height to roof $h_r = 60$ ft
Panel weight = 14.4 k
$\rho = 1.0$ per 1632.2



Find the following:

1. **Strength design load combinations.**

2. **Lateral seismic forces on connections and panel.**

3. **Vertical seismic forces on panel.**

4. **Combined dead and seismic forces on panel and connections.**

5. **Design forces for the brackets.**

6. **Design forces for the rods.**

Calculations and Discussion *Code eference*

1. **Strength design load combinations.**

For design of the panel connections to the building, the strength design load combinations are:

$$1.2D + 1.0E + f_1 L \qquad (12\text{-}5)$$

$$0.9D \pm 1.0E \qquad (12\text{-}6)$$

where

$$E = \rho E_h + E_v \tag{30-1}$$

$\rho = 1.0$ §1632.2

$E_h =$ load due to application of Equations (32-2) and (32-3) §1630.1.1

$E_v = 0.5 C_a I_p D$ §1630.1.1

2. Lateral seismic forces on connections and panel.

Out-of-plane panel seismic forces on the connections are determined from Equations (32-2) and (32-3) for the particular elevation of the connections. Forces at the upper level connections will be different than those at the lower level.

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{32-2}$$

$$0.7 C_a I_p W_p \le F_p \le 4 C_a I_p W_p \tag{32-3}$$

$a_p = 1.0$ and $R_p = 3.0$ §1633.2.4.2, Item 4

$W_p =$ weight of portion of panel tributary to the connection

Upper bracket connections

$$h_x = h_U = 47 \text{ ft}$$

Tributary W_p for the two brackets $= \dfrac{14.4}{2} = 7.2$ kips

$$F_{pU} = \frac{(1.0)(0.4)(1.0)}{(3.0)}\left[1 + 3\left(\frac{47}{60}\right)\right]W_p = 0.447 W_p \tag{32-2}$$

Check minimum force requirements of Equation (32-3).

$$0.7 C_a I_p W_p = 0.7\,(0.4)(1.0) W_p = 0.28 W_p$$

The force on each bracket is:

$$P_B = \frac{1}{2} \times F_{pU}$$

$$\therefore\ P_B = \frac{0.447(7.2)}{2} = \underline{\underline{1.61\ \text{kips/bracket}}}$$

Lower rod connections

$$h_x = h_L = 37\text{ft}$$

Tributary W_p for the two rods $= \dfrac{14.4}{2} = 7.2$ kips

$$F_{pL} = \frac{(1.0)(0.4)(1.0)}{(3.0)}\left[1 + 3\left(\frac{37}{60}\right)\right]W_p = 0.38W_p > 0.28W_p \qquad (32\text{-}2)$$

The axial force on each rod is:

$$P_R = \frac{1}{2} \times F_{pL}$$

$$\therefore\ P_R = \frac{0.38\,(7.2)}{2} = \underline{\underline{1.39\ \text{kips/rod}}}$$

Body of panel

The body of the panel is also designed using $a_p = 1.0$ and $R_p = 3.0$ as indicated in Table 16-O, Item 1.A(2). Thus, the seismic force on the body of the panel is the sum of the forces on the upper and lower levels. Alternatively, as shown below, an equivalent coefficient for the panel body can be determined by using the average of the coefficients for the upper and the lower levels.

Upper coefficient $= 0.447 @ h_U$

Lower coefficient $= 0.380 @ h_L$

Average coefficient $= \dfrac{(0.447 + 0.380)}{2} = 0.413 > 0.28$ *o.k.*

The panel seismic force is the average coefficient times the weight of the entire panel:

$$F_P = 0.413\left(W_p\right) = 0.413\left(14.4\right) = \underline{\underline{5.95\text{ kips}}}$$

This force is applied at the panel centroid C and acts horizontally in either the out-of-plane or the in-plane direction.

For panel design for out-of-plane forces, this force can be made into an equivalent uniform loading:

$$f_P = \frac{5{,}950}{12 \times 20} = \underline{\underline{24.8\text{ psf}}}$$

3. **Vertical seismic forces on panel.** **§1630.1.1**

The code requires consideration of vertical seismic forces when strength design is used. Vertical forces are determined from the equation

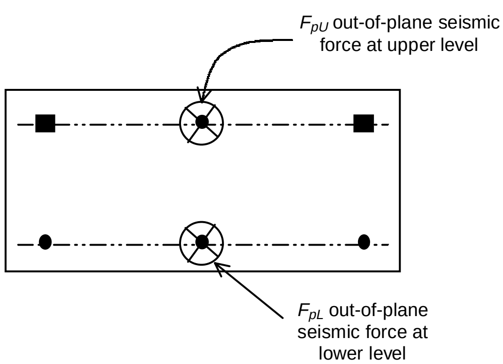
$$E_v = 0.5 C_a I_p D \qquad \S 1630.1.1$$

$D =$ dead load effect (or weight W_p of panel)

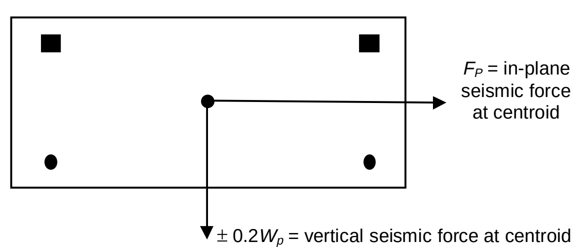
$$E_v = 0.5\left(.4\right)\left(1\right)W_p = 0.2W_p = 0.2\left(14.4\right) = \underline{\underline{2.88\text{ kips}}}$$

4. **Combined dead and seismic forces on panel and connections.** **§1630.1.1**

There are two seismic load conditions to be considered: out-of-plane and in-plane. These are shown below as concentrated forces. In this example, Equation (12-5) is considered the controlling load case. Because there is no live load on the panel, the term $f_1 L$ of this equation is zero.



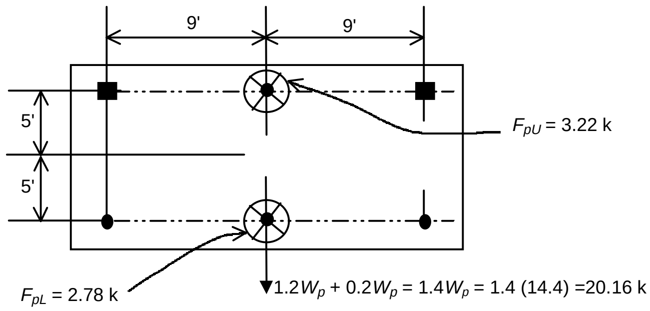
Out-of-plane seismic forces



In-plane seismic forces

a. **Dead load and seismic out-of-plane and vertical forces.**

Panel connection reactions due to dead load, out-of-plane seismic forces, and vertical seismic forces are calculated as follows:



Each bracket connection takes the following out-of-plane force due to lateral loads:

$$P_B = \frac{F_{pU}}{2} = \frac{3.22}{2} = \underline{\underline{1.61 \text{ kips}}}$$

Each bracket takes the following downward in-plane force due to vertical loads:

$$V_B = \frac{1.4W_p}{2} = \frac{20.16}{2} = \underline{\underline{10.08 \text{ kips}}}$$

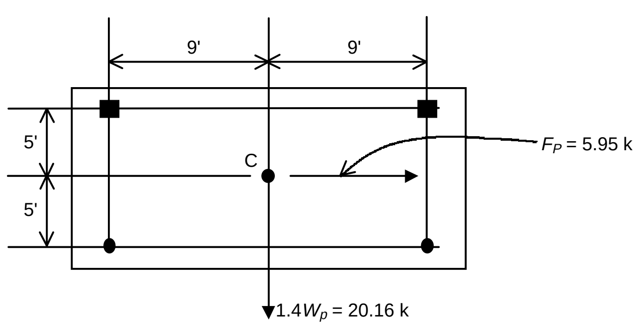
Each rod connection takes the following out-of-plane force due to lateral loads:

$$P_R = \frac{F_{pL}}{2} = \frac{2.78}{2} = \underline{\underline{1.39 \text{ kips}}}$$

Note that each rod, because it carries only axial forces, has no in-plane seismic loading.

b. **Dead load and seismic in-plane and vertical forces:**

Panel connection reactions due to dead load, in-plane seismic forces, and vertical seismic forces are calculated as follows:



Each bracket takes the following in-plane horizontal force due to lateral seismic load:

$$H_B = \frac{F_P}{2} = \frac{5.95}{2} = \underline{\underline{2.98 \text{ kips}}}$$

Each bracket takes the following upward or downward force due to lateral seismic load:

$$F_B = \frac{5(F_P)}{18} = \frac{5(5.95)}{18} = \underline{\underline{\pm 1.65 \text{ kips}}}$$

Each bracket takes the following downward force due to vertical loads:

$$R_B = \frac{1.4W_p}{2} = \frac{20.16}{2} = \underline{\underline{10.08 \text{ kips}}}$$

Under the in-plane seismic loading, each rod carries no force.

5. Design forces for the brackets.

a. Body of connection.

Under §1633.2.4.2, Item 4, the body of the connection must be designed for $a_p = 1.0$ and $R_p = 3.0$. These are the same values as used for the determination of F_{pU}, F_{pL} and F_P. Therefore there is no need to change these forces. The bracket must be designed to resist the following sets of forces:

$P_B = \pm\underline{\underline{1.61 \text{ k}}}$ out-of-plane together with

$V_B = \underline{\underline{10.08 \text{ k}}}$ downward shear

and

$H_B = \pm\underline{\underline{2.98 \text{ k}}}$ horizontal shear together with

$F_B + R_B = 1.65 + 10.08 = \underline{\underline{11.73 \text{ k}}}$ downward shear

b. Fasteners.

Under §1633.2.4.2, Item 5, fasteners must be designed for $a_p = 1.0$ and $R_p = 1.0$. Thus, it is necessary to multiply the F_{pU}, F_{pL} and F_P reactions by 3.0 since these values were based on $R_p = 3.0$. Fasteners must be designed to resist

$3P_B = 3(1.61) = \underline{\underline{4.83 \text{ k}}}$ out-of-plane together with

$$V_B = \underline{\underline{10.08\text{ k}}} \text{ downward shear}$$

and

$$3H_B = 3(2.98) = \underline{\underline{8.94\text{ k}}} \text{ horizontal shear together with}$$

$$3F_B + R_B = 3(1.5) + 10.08 = \underline{\underline{15.03\text{ k}}} \text{ downward shear}$$

6. **Design forces for the rods.**

a. **Body of connection.**

Under §1633.2.4.2, Item 4, the body of the connection must be designed to resist

$$P_R = \pm\underline{\underline{1.39\text{ k}}} \text{ out-of-plane}$$

b. **Fasteners.**

Under §1633.2.4, Item 5, all fasteners in the connecting system must be designed to resist a force based on $R_p = 1.0$:

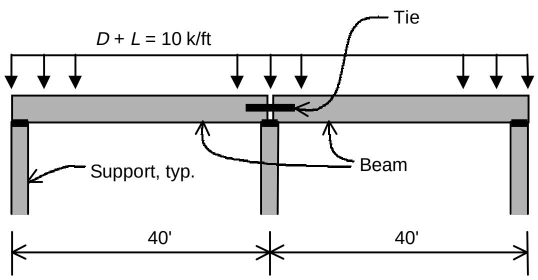
$$3P_R = 3(1.39) = \underline{\underline{4.17\text{ k}}} \text{ out-of-plane}$$

Example 44
Beam Horizontal Tie Force §1633.2.5

This example illustrates use of the beam tie requirement of §1633.2.5. This requirement derives from ATC-3 and is to ensure that important parts of a structure are "tied together."

Find the minimum required tie capacity for the connection between the two simple beams shown in the example below. The following information is given:

Zone 4
$C_a = 0.44$
$I = 1.0$



1. **Determine tie force.**

Calculations and Discussion — Code Reference

Requirements for ties and continuity are specified in §1633.2.5. For this particular example, it is required to determine the "tie force" for design of the horizontal tie interconnecting the two simply supported beams. This force is designated as E_h, where E_h is the horizontal earthquake load to be used in Equation (30-1). The minimum value of E_h is $0.5C_aI$ times the dead plus live load supported on the beam.

$$\text{Dead plus live load supported} = (10\text{ kpf})(40\text{ ft}) = 400\text{ kips}$$

$$E_h = 0.5(0.44)(1.0)(400) = \underline{\underline{88\text{ kips}}}$$

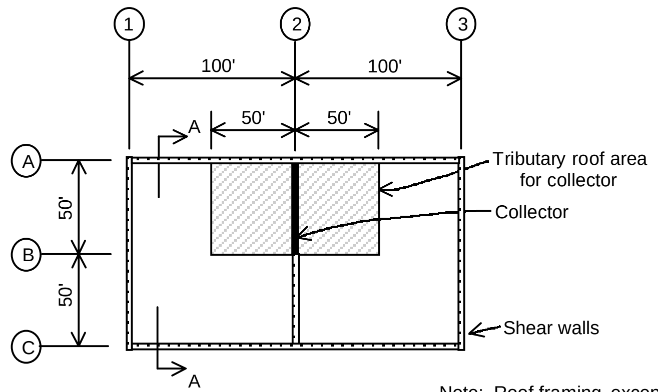
Commentary

The tie force calculated above for 1997 UBC requirements is .22 times dead plus live load. This is on a strength design basis and is about twice the load factored value given in the 1994 UBC. The 1994 UBC value is $Z/5$ times dead plus live load, or .112 times dead plus live load using a 1.4 load factor.

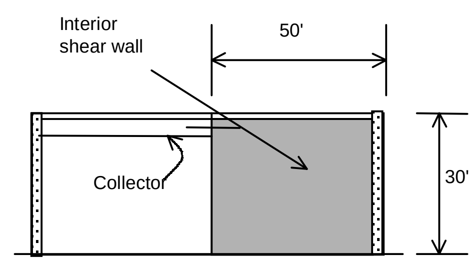
Example 45
Collector Elements §1633.2.6

Collectors “collect” forces and carry (i.e., drag) them to vertical shear-resisting elements. Collectors are sometimes called “drag struts.” The purpose of this example is to show the determination of the maximum seismic force for design of collector elements. In the example below, a tilt-up building with a panelized wood roof has a partial interior shear wall on Line 2. A collector is necessary to “collect” the diaphragm loads tributary to Line 2 and bring them to the shear wall. The following information is given:

Zone 4
$R = 4.5$
$\Omega_o = 2.8$
$I = 1.0$
$C_a = .44$
Roof dead load = 15 psf
Wall height = 30 ft, no parapet
Wall weight = 113 psf
Base shear = $V = .244W$



Roof plan



Elevation Section A-A

Determine the following:

1. **Collector design force at tie to wall.**

2. **Special seismic load of §1612.4 at tie to wall.**

Calculations and Discussion *Code Reference*

1. Collector design force at tie to wall. §1633.2.6

The seismic force in the collector is made up of two parts: (1) the tributary out-of-plane wall forces, and (2) the tributary roof diaphragm force. Because the roof is considered flexible, the tributary roof area is taken as the 100ft by 50ft area shown on the roof plan above. Seismic forces for collector design are determined from Equation

(33-1) used for diaphragm design. This equation reduces to the following for a single story structure:

$$F_{px} = \frac{F_{roof}}{W_{roof}} W_{px}$$

where

F_{px} = collector design force

W_{px} = weight tributary to collector

The term $\frac{F_{roof}}{W_{roof}}$ is the base shear coefficient adjusted for the diaphragm R value of 4 required by §1633.2.9.

$$\frac{F_{roof}}{W_{roof}} = \frac{V}{W}\left(\frac{R_{building}}{R_{diaphragm}}\right) = .244\left(\frac{4.5}{4}\right) = .275$$

$$F_{px} = .275 W_{px}$$

The tributary roof weight and out-of-plane wall weight is

$$W_{px} = 15 \text{ psf } (100)(50) + 113 \text{ psf } \left(\frac{30}{2}\right)(100) = 75{,}000 + 169{,}500 = 244.5 \text{ kips}$$

$$\therefore F_{px} = .275(244.5) = \underline{\underline{67.2 \text{ kips}}}$$

2. Special seismic load of §1612.4 at tie to wall. **§1633.2.6**

In addition to the forces specified by Equation (33-1), collectors must resist special seismic loads specified in §1612.4.

Collector load $E_h = 67.2$ kips

Required collector strength = $E_m = \Omega_o E_h = 2.8(67.2) = \underline{\underline{188.2 \text{ kips}}}$ (30-2)

This load is to be resisted on a strength design basis using a resistance factor of $\phi = 1.0$, and 1.7 times the allowable values for allowable stress design. The connection must have the capacity to deliver this collector load to the shear wall on Line 2.

Commentary

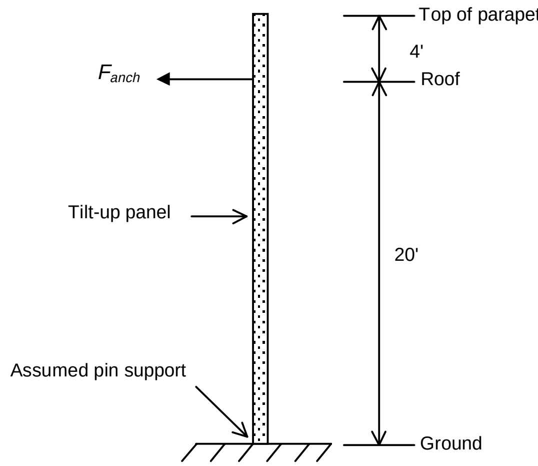
Note that the UBC in §1633.2.6 specifies that E_m need not exceed the maximum force that can be delivered by the diaphragm to the collector or other elements of the lateral force-resisting system. For example, the overturning moment capacity of the shear wall can limit the required strength of the collector and its connection to the shear wall.

Example 46
Out-of-Plane Wall Anchorage to Flexible Diaphragm §1633.2.8.1

For the tilt-up wall panel shown below, the seismic force required for the design of the wall anchorage to the flexible roof diaphragm will be determined. This will be done for a representative one foot width of wall.

The following information is given:

Zone 4
$I_p = 1.0$
$C_a = 0.4$
Panel thickness = 8 in.
Normal weight concrete (150 pcf)



Determine the following:

1. **Design criteria.**

2. **Wall anchorage force.**

Calculations and Discussion — Code Reference

1. **Design criteria.** **§1633.2**

Because of the frequent failure of wall/roof ties in past earthquakes, the code requires that the force used to design wall anchorage to flexible diaphragms be greater than that used to design the panel sections. Either Equation (32-1) or Equations (32-2) and (32-3) can be used to determine anchor design forces. Normally, Equations (32-2) and (32-3) are used.

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + \frac{3h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

$$0.7C_a I_p W_p \le F_p \le 4C_a I_p W_p \qquad (32\text{-}3)$$

The wall panel is supported at its base and at the roof level. The value of F_p to be used in wall/roof anchorage design is determined from Equation (32-2) using $h_x = h_r$, and W_p is the tributary weight.

For design of elements of wall anchorage system:

$R_p = 3.0, \quad a_p = 1.5$ §1633.2.8.1, Item 1

Also, the value of F_{anch} must not be less than 420 plf §1633.2.8.1, Item 1

2. **Wall anchorage force.**

The tributary wall weight is one-half of the weight between the roof and base *plus* all of the weight above the roof.

$$W_p = 150\left(\frac{8}{12}\right)(4' + 10')(1') = 1{,}400 \text{ lbs/ft}$$

Since

$h_x = h_r = 20 \text{ ft}$

$R_p = 3.0$

$a_p = 1.5$

The minimum force is

$$0.7C_a I_p W_p = 0.7\,(0.4)(1.0)W_p = 0.28W_p = 0.28\,(1{,}400) = 392 \text{ plf} \qquad (32\text{-}3)$$

Check Equation (32-2)

$$F_{anch} = \frac{1.5\,(0.4)(1.0)}{3.0}\left(1 + \frac{3\,(20)}{20}\right)W_p = 0.80W_p \qquad (32\text{-}2)$$

$$F_{anch} = 0.80W_p = 0.8\,(1{,}400) = 1{,}120 \text{ plf} > 420 \text{ plf} \;\; o.k., \text{ and} < 4.0C_a I_p W_p = 1.6W_p \;\; o.k.$$

$$\therefore\; F_{anch} = \underline{\underline{1{,}120 \text{ plf}}}$$

Commentary

Design of wall anchorage is crucial for successful earthquake performance of tilt-up buildings in Zones 3 and 4. Generally, it is desirable that the connections of walls to the diaphragm develop the strength of the steel. The following code sections apply to the anchorage design:

1. Sections 1605.2.3 and 1633.2.8 call for a positive direct connection. Embedded straps must be attached to, or hooked around, the wall reinforcing steel, or otherwise effectively terminated to transfer forces.

2. Section 1633.2.9, Item 4 states that F_{anch} may be carried by a subdiaphragm.
3. Section 1633.2.8.1 has the following additional anchorage requirements.

   Item 4: Steel elements of anchorage must be designed to take $1.4F_{anch}$.

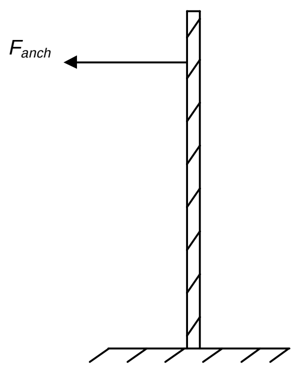
   Item 5: Wood elements of anchorage must have strength to take $0.85F_{anch}$, and wood elements must have minimum net thickness of $2\frac{1}{2}$" (i.e., be at least 3x members).
4. Section 1633.2.8 and §1633.2.9, Item 1 require that details of anchors tolerate Δ_M of the diaphragm.
5. When allowable stress design is used, the minimum anchorage force is not $420\,\text{plf}$ as specified in §1633.2.8.1, Item 1 but $300\,\text{plf}$. This is determined by substituting $E = 420\,\text{plf}$ in the load combinations of §1612.3. This gives:

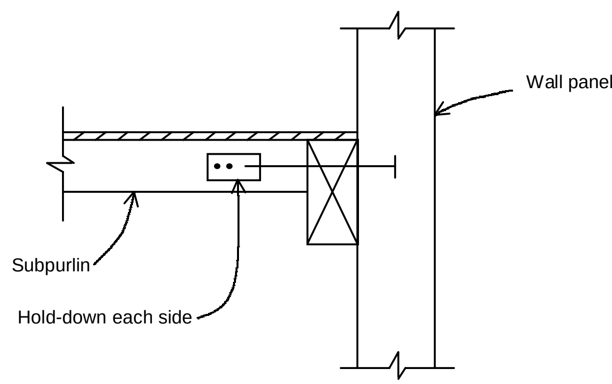
   $$\frac{E}{1.4} = \frac{420}{1.4} = 300\,\text{plf}\ .$$

Example 47
Wall Anchorage to Flexible Diaphragms §1633.2.8.1

This example illustrates use of the allowable stress design procedure for the design of steel and wood elements of the wall anchorage system in a building with a flexibl roof diaphragm.

In the example below, a tilt-up wall panel is shown. It is connected near its top to a flexible roof diaphragm. The anchorage force has been calculated per §1633.2.8.1 as $F_{anch} = 1{,}120$ plf. The wall anchorage connections to the roof are to be provided at 4 feet on center.





Wall-roof tie detail

Determine the strength design requirements for the following:

1. **Design force for premanufactured steel anchorage element.**

2. **Design force for wood subpurlin tie element.**

Calculations and Discussion — Code Reference

1. **Design force for premanufactured steel anchorage element.**

The basic task is to design the steel anchorage elements (i.e., hold-downs) that connect the tilt-up wall panel to the wood subpurlins of the roof diaphragm. The anchorage consists of two hold-down elements, one on each side of the subpurlin. The manufacturer's catalog provides allowable capacity values for earthquake loading for a given type and size of hold-down element. These include the allowabl stress increase and are typically listed under a heading that indicates a "1.33 x allowable" capacity.

For the steel hold-down elements of the anchorage system, the code requires that the anchorage force P_E used in strength design be 1.4 times the force otherwise required.

$$P_E = 1.4F_{anch}$$ §1633.2.8.1, Item 4

$$P_E = 1.4\,(1{,}120 \text{ plf})(4 \text{ ft}) = 6{,}272 \text{ lbs}$$

Since P_E is determined on a strength design basis, it is the earthquake load E to be used in the design load combinations. In this example, it is elected to use the alternate basic load combinations of §1612.3.2, where the applicable combinations of Equations (12-13), (12-16) and (12-16-1) permit $\frac{E}{1.4}$ to be resisted with a one-third increase in allowable stress.

The allowable stress design requirement for each pair of hold-down elements is:

$$\frac{E}{1.4} = \frac{P_E}{1.4} = \frac{6{,}272}{1.4} = 4{,}480 \text{ lbs}$$

From the manufacturer's catalog, select a hold-down element having a $(1.33 \times \text{allowable})$ capacity of at least

$$\frac{4{,}480}{2} = 2{,}240 \text{ lbs}$$

Whenever hold-downs are used in pairs, as shown in the wall-roof tie detail above, the through-bolts in the subpurlin must be checked for double shear bearing. Also, the paired anchorage embedment in the wall is likely to involve an overlapping pull-out cone condition in the concrete: refer to §1923 for design requirements. When single-sided hold-downs are used, these must comply with the requirements of Item 2 of §1633.2.8.1. Generally, double hold-downs are preferred, but single-sided hold-downs are often used with all eccentricities fully considered.

2. Design force for wood subpurlin tie element.

The strength design forces on the wood elements of the wall anchorage system can be 0.85 times the force otherwise required:

$$P_E = 0.85 F_{anch}$$ §1633.2.8.1, Item 5

$$P_E = 0.85\,(1120 \text{ plf})(4 \text{ ft}) = \underline{\underline{3{,}808 \text{ lbs}}}$$

Select the wood element such that 1.33 times the allowable capacity of the element, including dead load effects, is at least equal to

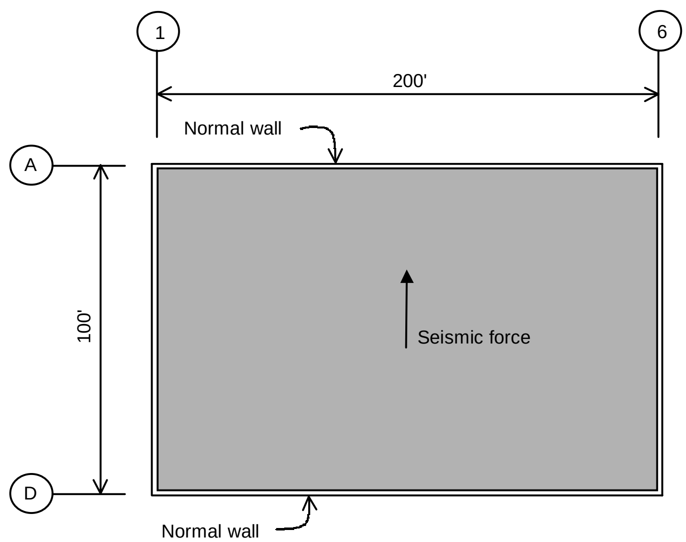
$$\frac{P_E}{1.4} = \frac{3{,}808}{1.4} = 2{,}720 \text{ lbs}$$

Note that tie elements, such as the subpurlin, are required to be 3x or larger. §1633.2.8.1, Item 5

Example 48
Determination of Diaphragm Force F_{px}: Lowrise §1633.2.9

This example illustrates determination of the diaphragm design force F_{px} of Equation (33-1), for the design of the roof diaphragm of a single story building.

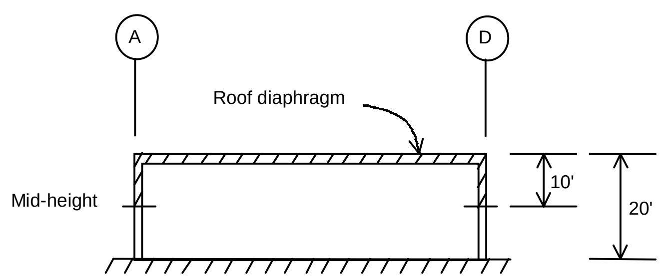
A single-story tilt-up building with a panelized wood roof is shown below. This type of roof construction is generally considered to have a flexible diaphragm.



Roof plan

Given:

Zone 4
$I = 1.0$
$C_a = 0.4$
$R = 4.5$ (bearing wall system)
$\rho = 1.2$
Diaphragm weight = 15 psf
Wall weight = 80 psf



Elevation through building

Find the following:

1. **Diaphragm force at the roof.**

Calculations and Discussion | Code Reference

1. **Diaphragm force at the roof.**

For buildings with tilt-up concrete walls, §1633.2.9, Item 3, requires that the flexible diaphragm design force be based on the design base shear and forces F_{px} using an R value not exceeding 4, even though the tilt-up wall-frame system uses $R = 4.5$.

For a short period single story building, the diaphragm force, using $R = 4$, becomes:

w_{px} = weight of diaphragm + weight of ½ height of normal walls =
$100(15) + 2(10)(80) = 3{,}100 \text{ plf}$

$$F_{px} = \frac{2.5 C_a I}{R} w_{px} = \frac{2.5(0.4)(1.0)}{4}(3{,}100) = \underline{\underline{775 \text{ plf}}} \qquad (33\text{-}1)$$

Note that the redundancy factor of $\rho = 1.2$ is *not* applied to the E_h loads due to F_{px} (such as chord forces and diaphragm shear loads in the diaphragm).

Commentary

1. The weight, w_{px}, includes the weight of the diaphragm plus the tributary weight of elements normal to the diaphragm that are one-half story height below and above the diaphragm level. Walls parallel to the direction of the seismic forces are usually not considered in the determination of the tributary roof weight because these walls do not obtain support, in the direction of the force, from the roof diaphragm.

2. The single story building version of Equation (33-1) is derived as follows:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (33\text{-}1)$$

$$F_i = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (30\text{-}15)$$

For a single story building,

$$i = 1,\ x = 1 \text{ and } n = 1$$

$$F_t = 0, \text{ since } T < 0.7 \sec$$

$$\sum_{i=1}^{1} w_i = W$$

and Equation (30-15) gives

$$F_1 = \frac{Vw_1h_1}{w_1h_1} = V$$

where

$$V = \frac{2.5C_a IW}{R} \tag{30-5}$$

Finally, for the single story building, Equation (33-1) is

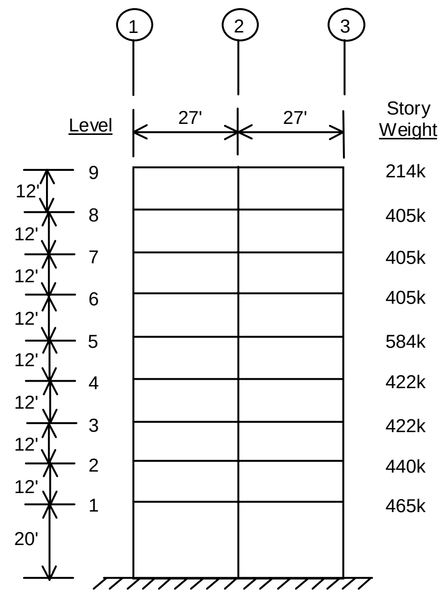
$$F_{p1} = F_1 w_{p1} = \frac{2.5C_a I}{R} w_{p1}$$

Example 49
Determination of Diaphragm Force F_{px}: Highrise §1633.2.9

This example illustrates determination of the diaphragm design force F_{px} of Equation (33-1) for a representative floor of a multi-story building.

The nine-story moment frame building shown below has the tabulated design seismic forces F_x. These were determined from Equations (30-14) and (30-15) and the design base shear. The following information is given:

Zone 4
$W = 3{,}762$ k
$C_a = 0.40$
$C_v = 0.56$
$R = 8.5$
$\rho = 1.2$
$I = 1.0$
$T = 1.06$ sec
$V = 233.8$ k
$F_t = 17.3$ k



Level x	h(ft)	w(k)	wh	$\frac{wh}{\Sigma wh}$	F_x (k)
9	116	214	24,824	0.103	22.3 + 17.3 = 39.6
8	104	405	42,120	0.174	37.7
7	92	405	37,260	0.154	33.3
6	80	405	32,400	0.134	29.0
5	68	584	39,712	0.164	35.5
4	56	422	23,632	0.098	21.2
3	44	422	18,568	0.077	16.7
2	32	440	14,080	0.058	12.6
1	20	465	9,300	0.038	8.2
		Σ =3,762	241,896		233.8

1. **Find the diaphragm force at Level 7.**

Calculations and Discussion — *Code Reference*

1. **Diaphragm force at Level 7.**

Seismic forces on floor and roof diaphragm are specified in §1633.2.9. The following expression is used to determine the diaphragm force F_{px} at level x:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \tag{33-1}$$

Section 1633.2.9 also has the following limits on F_{px}:

$$0.5 C_a I w_{px} \le F_{px} \le 1.0 C_a I w_{px}$$

For level 7, $x = 7$.

$$F_{p7} = \frac{[17.3 + (33.3 + 37.7 + 22.3)](405)}{(405 + 405 + 214)} = (0.108)(405) = 43.7\text{k}$$

Check limits:

$$0.5 C_a I w_{px} = 0.5\,(0.40)(1.0)\,405 = 81.0\text{k}$$

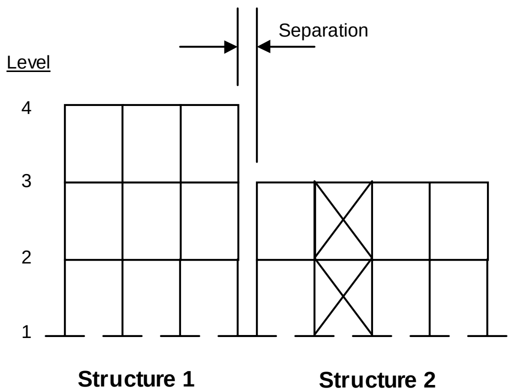
$$1.0 C_a I w_{px} = 1.0\,(0.40)(1.0)\,405 = 162.0\text{k}$$

$$\therefore\ F_{p7} = \underline{\underline{81.0\ \text{kips}}}$$

Note that the redundancy factor, in this example $\rho = 1.2$, is *not* applied to the loads E_h due to F_{px} (such as chord forces and floor-to-frame shear connections).

Example 50
Building Separations §1633.2.11

Building separations are necessary to prevent or reduce the possibility of two adjacent structures impacting during an earthquake. Requirements for building separations are given in §1633.2.11. In this example, the static displacements and information about each structure are given below.



Structure 1		*Structure 2*	
Level	Δ_S	*Level*	Δ_S
4	1.38 in.	—	—
3	1.00	3	0.75 in
2	0.47	2	0.35
1	0	1	0
R = 8.5		R = 7.0	

Find the required separations for the following situations:

1. **Separation within the same building.**

2. **Separation from an adjacent building on the *same* property.**

3. **Separation from an adjacent building on *another* property.**

Calculations and Discussion — Code Reference

1. Separation within the same building. §1633.2.11

Expansion joints are often used to break a large building, or an irregular building, into two or more parts above the foundation level. This effectively creates separate "structures" within the same "building." The code requires that the structures be separated by the amount Δ_{MT}.

where

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \tag{33-2}$$

Δ_{M1} = maximum inelastic displacement of Structure 1

Δ_{M2} = maximum inelastic displacement of Structure 2

The required separation is determined in the following two steps.

a. **Determine inelastic displacements of each structure.** **§1630.9.2**

To determine the minimum separation between parts of the same "building" that are separated by an expansion joint, the maximum inelastic floor displacements under code seismic forces must be determined for each structure. These are

For Structure 1

$$\Delta_{M1} = 0.7R\Delta_s = 0.7 \times 8.5 \times 1.0 = 5.95 \text{ in.} \qquad (30\text{-}17)$$

For Structure 2

$$\Delta_{M2} = 0.7R\Delta_s = 0.7 \times 7.0 \times .75 = 3.68 \text{ in.} \qquad (30\text{-}17)$$

b. **Determine the required separation.** **§1633.2.11**

The required separation is determined from the individual maximum inelastic displacements of each structure as follows:

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} = \sqrt{(5.95)^2 + (3.68)^2} = \underline{\underline{7.0 \text{ in}}}. \qquad (33\text{-}2)$$

2. **Separation from an adjacent building on the *same* property.**

If Structures 1 and 2 above were adjacent, individual buildings on the same property, the solution to this problem is the same as that shown above in Step 1. The code makes no distinction between an "internal" separation in the same building and the separation required between two adjacent buildings on the same property.

$$\Delta_{MT} = \underline{\underline{7.0 \text{ in}}}.$$

3. **Separation from an adjacent building on *another* property.** **§1633.2.11**

If Structure 1 is a building under design and Structure 2 is an existing building on another property, we would generally not have information about the seismic displacements of Structure 2. Often even basic information about the structural system of Structure 2 may not be known. In this case, separation must be based only on information about Structure 1. The maximum static displacement of Structure 1 is 1.38 inches and occurs at the roof (Level 4). The inelastic displacement is calculated as:

$$\Delta_M = 0.7R\Delta_s = 0.7 \times 8.5 \times 1.38 = \underline{\underline{8.2 \text{ in}}}. \qquad (30\text{-}17)$$

Structure 1 must be set back 8.2 inches from the property line, unless a smaller separation is justified by a rational analysis based on maximum ground motions. Such an analysis is difficult to do, and is generally not done except in very special cases.

Example 51
Flexible Nonbuilding Structure §1634.2

A tall cylindrical steel vessel is supported by a heavy, massive concrete foundation. The following information is given:

Weight of tank and maximum
normal operating contents = 150 k

Occupancy Category 2

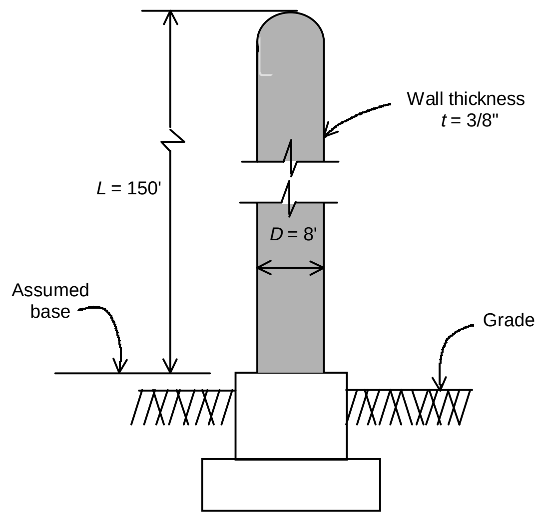
Zone 4

$I = 1.25$ (toxic contents per Table 16-K)

$C_a = 0.44$

$C_v = 0.64$

$N_v = 1.0$



Determine the following:

1. **Period of vibration.**

2. **Design base shear.**

3. **Vertical distribution of seismic forces.**

4. **Overturning moment at base.**

Calculations and Discussion — Code Reference

1. Period of vibration.

In this example, only the case with the vessel full of contents will be considered. In actual practice, other conditions may need to be considered. For calculation purposes, the base is assumed to be located at the top of the pier. The weight of the vessel is assumed to be uniformly distributed over its height. The period of the vessel must be determined by Method B. This is required by §1634.1.4. For this particular vessel, the expression for the period of a thin-walled cantilever cylinder may be used.

$$T = 7.65 \times 10^{-6} \left(\frac{L}{D}\right)^2 \left(\frac{wD}{t}\right)^{\frac{1}{2}}$$

where:

$L = 150\text{ ft}$, $D = 8\text{ ft}$

$t = 3/8$ in.

$W = 150\text{ k}$

$$w = \frac{W}{L} = \frac{150{,}000}{150} = 1000 \text{ plf}$$

$$\frac{wD}{t} = \frac{1000 \times 8}{(0.375/12)} = 256{,}000$$

$$\frac{L}{D} = \frac{150}{8} = 18.75$$

$$T = 7.65 \times 10^{-6} \times 18.75^2 \times \sqrt{256{,}000} = \underline{\underline{1.36 \text{ sec}}}$$

Because the period is greater than .06 seconds, the vessel is considered flexible.

It should be noted that the value of the period T determined using Method B is not subject to the 30-percent limit mentioned in §1630.2.2, Item 2. This is because Method A is intended for buildings and is not applicable to structural systems that differ from typical building configurations and characteristics. Refer to Section C109.1.4 of the SEAOC Blue Book for further discussion.

2. Design base shear.

The design base shear for nonbuilding structures is calculated from the same expressions as for buildings. These are given in §1630.2.1. In addition, nonbuilding structures such as the vessel must also satisfy the requirements of §1634.5.

$$V = \frac{C_v I}{RT} W \qquad (30\text{-}4)$$

$R = 2.9$ and $\Omega_o = 2.0$ Table 16-P

$$V = \frac{0.64\,(1.25)}{2.9\,(1.36)}(150) = 30.4 \text{ kips}$$

Under §1634.5 Item 1, design base shear must not be less than the following:

$$V = 0.56 C_a I W = 0.56\,(.44)(1.25)150 = 46.2 \text{ kips} \qquad (34\text{-}2)$$

nor in Zone 4 less than

$$V = \frac{1.6 Z N_v I}{R} W = \frac{1.6\,(.4)(1.0)(1.25)}{2.9}(150) = 41.4 \text{ kips} \qquad (34\text{-}3)$$

$$\therefore\ V = \underline{\underline{46.2 \text{ kips}}}$$

3. Vertical distribution of seismic forces.

Requirements for the vertical distribution of seismic forces are given in §1634.5 Item 2. This specifies the use of the same vertical distribution of force as for buildings, either Equation (30-13) or a dynamic analysis. The following shows use of the static procedures of Equation (30-13).

$$V = F_t + \sum_{i=1}^{n} F_i \tag{30-13}$$

where

$$T = 1.36 \sec > 0.7$$

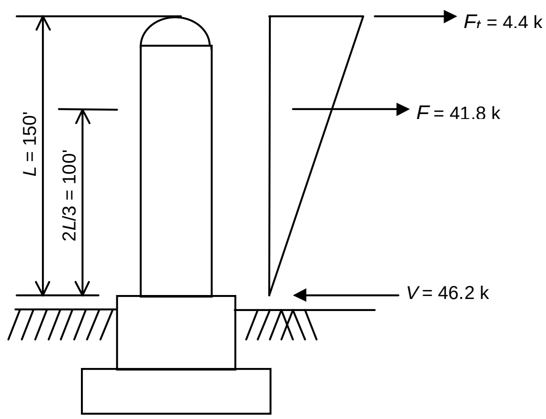
$$F_t = 0.07TV$$

$$\therefore F_t = 0.07\,(1.36)(46.2) = \underline{\underline{4.4\text{ k}}} < 0.25V \quad o.k. \tag{30-14}$$

$$\therefore F = V - F_t = 46.2 - 4.4 = \underline{\underline{41.8\text{ k}}} \text{ acting at } 2L/3$$

(centroid of triangular distribution)

The vertical distribution of seismic forces on the vessel is shown below.



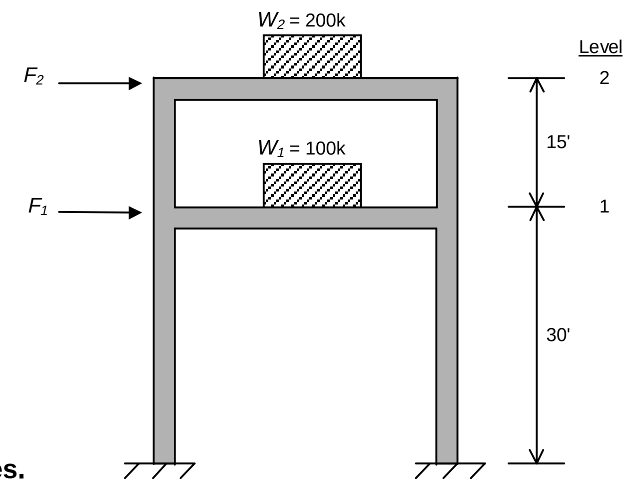
4. Overturning moment at base.

$$M = 4.4\,(150) + 41.8\,(100) = \underline{\underline{4{,}840\text{ k}-\text{ft}}} \text{ (at the top of the foundation)}$$

Example 52
Lateral Force on Nonbuilding Structure §1634.2

A nonbuilding structure with a concrete intermediate moment-resisting frame (IMRF) supports some rigid aggregate storage bins. Weights W_1 and W_2 include the maximum normal operating weights of the storage bins and contents as well as the tributary frame weight. The following information is given:

Zone 4
$I = 1.0$
Soil Profile Type D
$C_a = 0.44$
$C_v = 0.64$
$N_v = 1.0$
$T = 2.0\,\text{sec}$



Determine the following:

1. **Design base shear.**

2. **Vertical distribution of seismic forces.**

Calculations and Discussion — Code Reference

1. **Design base shear.** **§1634.2**

Because this is a flexible structure, the general expressions for design base shear given in §1630.2.1 must be used. Note that the Exception of §1634.2 permits use of an IMRF in Zones 3 and 4, provided the height of the structure is less than 50 feet and R does not exceed 2.8.

The total base shear in a given direction is determined from

$$V = \frac{C_v I}{RT} W = \frac{0.64(1.0)}{2.8(2.0)}(200+100) = 0.114(300) = 34.2\text{ k} \qquad (30\text{-}4)$$

However, the total base shear need not exceed

$$V \le \frac{2.5 C_a I}{R} W = \frac{2.5(0.44)(1.0)}{2.8}(200+100) = 117.9\text{ kips} \qquad (30\text{-}5)$$

The total design base shear cannot be less than

$$V \ge 0.11 C_a I W = 0.11(0.44)(1.0)(200+100) = 14.5\text{ kips} \qquad (30\text{-}6)$$

In Seismic Zone 4, the total base shear also cannot be less than

$$V \geq \frac{0.8ZN_v}{R}W = \frac{0.8(0.4)(1.0)}{2.8}(200+100) = 34.3\,\text{kips} \tag{30-7}$$

In this example, design base shear is controlled by Equation (30-7).

$$V = \underline{\underline{34.3\,\text{kips}}}$$

2. Vertical distribution of seismic forces. §1634.2

The design base shear must be distributed over the height of the structure in the same manner as that for a building structure.

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} = \frac{(V - F_t)(W_x h_x)}{(W_1 h_1 + W_2 h_2)} \tag{30-15}$$

Because $T > 0.7$ seconds, a concentrated force F_t must be applied to the top level.

$$F_t = 0.07TV = 0.07(2.0)(34.3) = \underline{\underline{4.90\,\text{k}}} \tag{30-14}$$

$$F_2 = 4.90 + \frac{(34.3 - 4.90)(200)(45)}{[200(45) + 100(30)]} = \underline{\underline{26.9\,\text{kips}}} \tag{30-15}$$

$$F_1 = \frac{(34.3 - 4.90)(100)(30)}{[200(45) + 100(30)]} = \underline{\underline{7.4\,\text{kips}}} \tag{30-15}$$

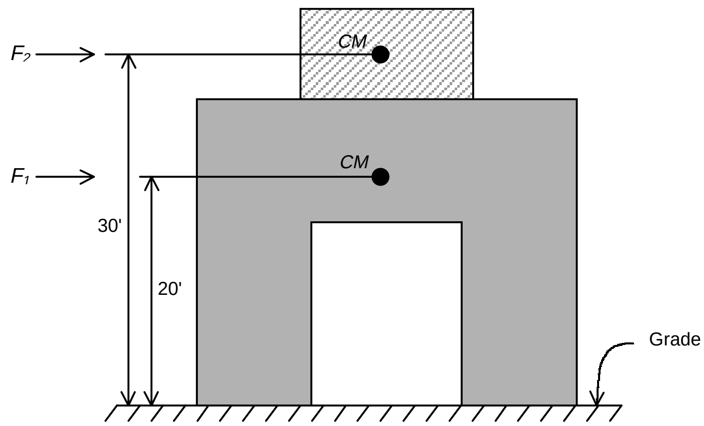
Commentary

Section 1634.1.2 permits use of $\rho = 1.0$ for load combinations for nonbuilding structures using §1634.3, §1634.3 or §1634.5 for determination of seismic forces.

Example 53
Rigid Nonbuilding Structure §1634.3

The code has special requirements for the determination of seismic forces for design of rigid nonbuilding structures. In this example, rigid ore crushing equipment is supported by a massive concrete pedestal and seismic design forces are to be determined. The following information is given:

Zone 4
$C_a = 0.4$
$I = 1.0$
$T = 0.02$ sec
$W_{EQUIPMENT} = 100$ k
$W_{SUPPORT} = 200$ k



Determine the following:

1. **Design base shear.**

2. **Vertical distribution of seismic forces.**

Calculations and Discussion — Code Reference

1. **Design base shear.** **§1634.3**

For rigid nonbuilding structures, Equation (34-1) is used to determine design base shear.

$$V = 0.7 C_a I W = 0.7\,(0.4)(1.0)(200+100) = \underline{\underline{84 \text{ kips}}} \qquad (34\text{-}1)$$

2. **Vertical distribution of seismic forces.** **§1634.3**

Design base shear is distributed according to the distribution of mass

$$F_1 = \frac{200}{300}(84) = \underline{\underline{56.0 \text{ kips}}}$$

$$F_2 = \frac{100}{300}(84) = \underline{\underline{28.0 \text{ kips}}}$$

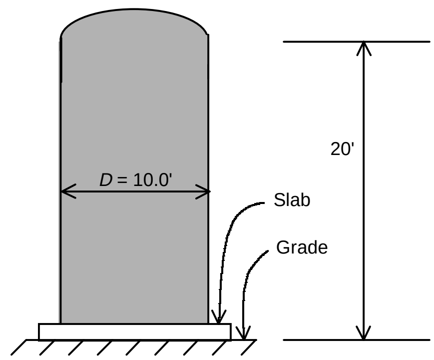
Commentary

Section 1634.1.2 permits use of $\rho = 1.0$ for load combinations for nonbuilding structures using §1634.3, §1634.4 or §1634.5 for determination of seismic forces.

Example 54
Tank With Supported Bottom §1634.4

A small liquid storage tank is supported on a concrete slab. The tank does not contain toxic or explosive substances. The following information is given:

Zone 4
$C_a = 0.4$
$I_p = 1.0$
Weight of tank and maximum normal operating contents = 120 kips



1. Find the design base shear. **§1634.4**

Calculations and Discussion — Code Reference

The tank is a nonbuilding structure, and seismic requirements for tanks with supported bottoms are given in §1634.4. This section requires that seismic forces be determined using the procedures of §1634.3* for rigid structures. Base shear is computed as

$$V = 0.7C_a IW = 0.7\,(0.4)(1.0)(120) = \underline{\underline{33.6 \text{ kips}}} \tag{34-1}$$

The design lateral seismic force is to be applied at the center of mass of the tank and its contents.

*Note: There is a typographical error on page 2-21 in some versions of the 1997 UBC in §1634.4. Section 1632 should be “Section 1634.3.”

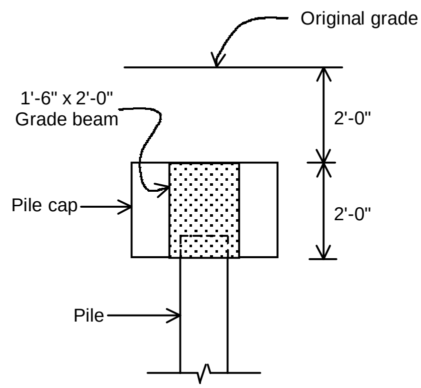
Commentary

The above procedures are intended for tanks that have relatively small diameters and where the forces generated by fluid sloshing modes are small. For large diameter tanks, the effects of sloshing must be considered. Refer to American Water Works Association Standard ANSI/AWWA D100-84 “Welded Steel Tanks for Water Storage,” or American Petroleum Institute Standard 650, “Welded Steel Tanks for Oil Storage” for more detailed guidance. Also see Section C109.5.1 of the SEAOC Blue Book for a discussion of tank anchorage methods.

Example 55
Pile Interconnections §1807.2

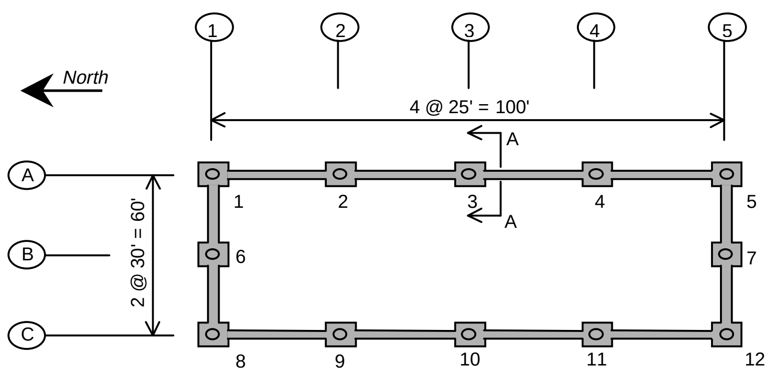
A two-story masonry bearing wall structure has a pile foundation. Piles are located around the perimeter of the building. The foundation plan of the building is shown below. The following information is given:

Zone 4
$I = 1.0$ (standard occupancy)
Pile cap size: 3'-0" square x 2'-0" deep
Grade beam: 1'-6" x 2'-0"
Allowable lateral bearing = 200 psf
per ft. of depth below natural grade.



Section A-A: Typical pile cap

Pile Cap	*Dead Load*	*Reduced Live Load*	*Seismic*	
			N/S	*E/W*
3	46 k	16 k	14 k	0
10	58	16	14	0



Foundation pla

Determine the following:

1. **Interconnection requirements.**

2. **Interconnection force between pile caps 3 and 10.**

3. **Required "tie" restraint between pile caps 3 and 10.**

Calculations and Discussion — Code Reference

1. Interconnection requirements. §1807.2

The code requires that individual pile caps of every structure subject to seismic forces be interconnected with ties. This is specified in §1807.2. The ties must be capable of resisting in tension and compression, a minimum horizontal tie force equal to 10 percent of the larger column vertical load. The column vertical load is to be considered the dead, reduced live, and seismic loads on the pile cap. An exception to §1807.2 allows use of “equivalent restraint.”

2. Interconnection force between pile caps 3 and 10.

Maximum loads on each pile cap under E/W seismic forces are

$$\text{Pile cap 3} = 46 + 16 + 0 = 62\text{ kips}$$

$$\text{Pile cap 10} = 58 + 16 + 0 = 74\text{ kips}$$

Minimum horizontal tie force is 10 percent of largest column vertical load

$$P = 0.10\left(74\right) = \underline{\underline{7.40\text{ kips}}}$$

3. Required “tie” restraint between pile caps 3 and 10.

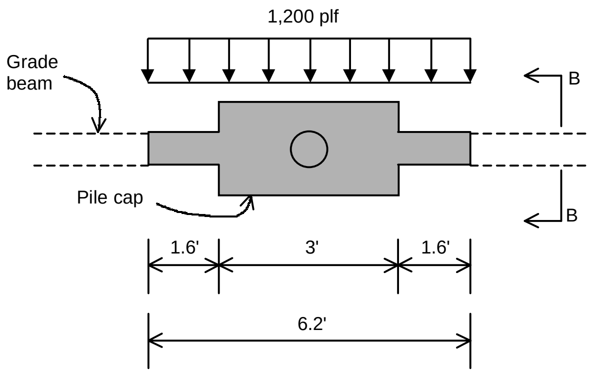
The choices are to add a grade beam (i.e., tie beam) connecting pile caps 3 and 10, or to try to use passive pressure restraint on the pile cap in lieu of a grade beam. The latter is considered an “equivalent restraint” under the exception to §1807.2.

Check passive pressure resistance.

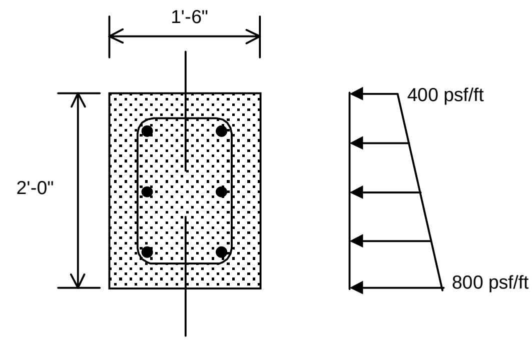
$$\text{Passive pressure} = \frac{\left(400 + 800\right)}{2}\left(2\text{ ft}\right) = 1{,}200\text{ plf}$$

$$\text{Required length} = \frac{7{,}400\text{ lbs}}{1{,}200\text{ plf}} = 6.2\text{ ft}$$

This is greater than 3'-0" pile cap width, but pile cap and a tributary length of N/S grade beam on either side of the pile cap may be designed to resist “tie” forces using passive pressure. This system is shown below, and if this is properly designed, no grade beam between pile caps 3 and 10 (or similar caps) is required.



Equivalent restraint system in pla



Section B-B: Grade beam

Commentary

Normally, buildings on pile foundations are required to have interconnecting ties between pile caps. This is particularly true in the case of highrise buildings and buildings with heavy vertical loads on individual pile caps. Ties are essential in tall buildings. Ties are also necessary when the site soil conditions are poor such that lateral movements, or geotechnical hazards, such as liquefaction, are possible. Also note that while §1807.2 has the wording "tension or compression," the intent is that the ties must resist the required forces in both tension and compression.

In design of relatively lightweight one- and two-story buildings, the exception to the interconnecting tie requirement of §1807.2 may permit a more economical foundation design. However, when interconnecting ties are omitted, a geotechnical engineer should confirm the appropriateness of this decision.