

Seismic Design Manual

Volume II

**Building Design Examples:
Light Frame, Masonry and Tilt-up**

April 2000



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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 H. Shea, Albany, California

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Preface

This document is the second volume of the three-volume SEAOC *Seismic Design Manual*. The first volume, “Code Application Examples,” was published in April 1999. These documents have been developed by the Structural Engineers Association of California (SEAOC) with funding provided by SEAOC. Their 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 Officials (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. While the *Manual* illustrates how the provisions of the code are used, the examples shown do not necessarily illustrate the only appropriate methods of seismic design, and the document is not intended to establish a minimum standard of care. Engineering judgment needs to be exercised when applying these examples to real projects.

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: 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.

Volume II contains six examples. These illustrate the seismic design of the following structures: (1) a two-story wood light frame residence, (2) a three-story wood light frame building, (3) a three-story cold formed light frame building, (4) a one-story masonry building with panelized wood roof, (5) a one-story tilt-up building with panelized wood roof, and (6) the design of a tilt-up wall panel with large openings.

Work on the final volume, *Building Design Examples, Volume III—Steel, Concrete and Cladding*, is nearing completion and is scheduled for release in late Spring 2000.

It is SEAOC’s present intention to update the *Seismic Design Manual* with each edition of the building code used in California. Work is currently underway on a 2000 International Building Code version.

Ronald P. Gallagher
Project Manager

Acknowledgments

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The *Seismic Design Manual* was written by a group of highly qualified structural engineers. These individuals are both California registered civil and 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:

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 Joseph R. Maffei
 Douglas S. Thompson
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Volume II was written principally by Douglas S. Thompson (Examples 1, 2, and 3), Jon P. Kiland (Example 4), Ronald P. Gallagher (Example 5), and John W. Lawson (Example 6). Many useful ideas and helpful suggestions were offered by the other Consultants. Consultant work on Volume 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
 Martin W. Johnson
 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:

Ricardo Arevalo	Gary Ho	Michael Riley
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Close collaboration with the SEAOC Seismology Committee was maintained during the development of the document. The 1999-2000 Committee reviewed the document and provided many helpful comments and suggestions. Their assistance is gratefully acknowledged.

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Suggestions for Improvement

In keeping with two of its Mission Statements: (1) “to advance the structural 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:

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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.seaoc.org> or on the ICBO website at <http://www.icbo.org>. SEAOC, at its sole discretion, may or may not issue written errata.

Seismic Design Manual

Volume II

**Building Design Examples:
Light Frame, Masonry and Tilt-up**



Introduction

Seismic design of new light frame, masonry and tilt-up buildings for the requirements of the 1997 Uniform Building Code (UBC) is illustrated in this document. Six examples are shown: (1) a two-story wood frame residence, (2) a large three-story wood frame building, (3) a three-story cold formed steel light frame building, (4) a one-story masonry (concrete block) building with panelized wood roof, (5) a one-story tilt-up building with panelized wood roof, and (6) the design of a tilt-up wall panel with large openings.

The buildings selected are for the most part representative of construction types found in Zones 3 and 4, particularly California and the Western States. Designs have been largely taken from real world buildings, although some simplifications were necessary for purposes of illustrating significant points and not presenting repetitive or unnecessarily complicated aspects of a design.

The examples are not complete building designs, or even complete seismic designs, but rather they are examples of the significant seismic design aspects of a particular type of building.

In developing these examples, SEAOC has endeavored to illustrate correct use of the *minimum* provisions of the code. The document is intended to help the reader understand and correctly use the design provisions of UBC Chapters 16 (Design Requirements), 19 (Concrete), 21 (Masonry), 22 (Steel) and 23 (Wood). Design practices of an individual structural engineer or office, which may result in a more seismic-resistant design than required by the minimum requirements of UBC, are not given. When appropriate, however, these considerations are discussed as alternatives.

In some examples, the performance characteristics of the structural system are discussed. This typically includes a brief review of the past earthquake behavior and mention of design improvements added to recent codes. SEAOC believes it is essential that structural engineers not only know how to correctly interpret and apply the provisions of the code, but that they also understand their basis. For this reason, many examples have commentary included on past earthquake performance.

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

How to Use This Document

Generally, each design example is presented in the following format. First, there is an “Overview” of the example. This is a description of the building to be designed. This is followed by an “Outline” indicating the tasks or steps to be illustrated in each example. Next, “Given Information” provides the basic design information, including plans and sketches given as the starting point for the design. This is followed by “Calculations and Discussion”, which provides the solution to the example. Some examples have a subsequent section designated “Commentary” The commentary is intended to provide a better understanding of aspects of the example and/or to offer guidance to the reader on use of the information generated in the example. Finally, references and suggested reading are given under “References.” Some examples also have a “Forward” and/or section “Factors Influencing Design” that provide remarks on salient points about the design.

Because the document is based on the UBC, UBC notation is used throughout. However, notation from other codes is also used. In general, 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 (Volume 2) being understood. “Formula (32-2)” is designated Equation (32-2) or just (32-2) in the right-hand margins of the examples. Similarly, the phrase “Table 16-O” is understood to be 1997 UBC Table 16-O. Throughout the document, reference to specific code provisions, tables, and equations (the UBC calls the latter formulas) is given in the right-hand margin under the heading Code Reference.

When the document makes reference to other codes and standards, this is generally done in abbreviated form. Generally, reference documents are identified in the right-hand margin. Some examples of abbreviated references are shown below.

Right-Hand Margin Notation

23.223, Vol. 3

96 AISI E3.3

91 NDS Table 5A

Table 1-A, AISC-ASD

More Complete Description

Section 23.223 of Volume 3, of the 1997 *Uniform Building Code* (UBC).

Section E3.3 of the 1996 Edition of the American Iron and Steel Institute (AISI) *Specification for the Design of Cold-Formed Steel Structural Members*.

Table 5A of the 1991 *National Design Specification for Wood Construction* (NDS).

Table 1-A of Ninth Edition, American Institute of Steel Construction (AISC) *Manual of Steel Construction, Allowable Stress Design*.

Notation

The following notations are used in this document. These are generally consistent with that used in the UBC and other codes such as ACI, AISC, AISI and NDS. Some additional notations have also been added. The reader is cautioned that the same notation may be used more than once and may carry entirely different meaning in different situations. For example, E can mean the tabulated elastic modulus under the NDS definition (wood) or it can mean the earthquake load under §1630.1 of the UBC (loads). When the same notation is used in two or more definitions, each definition is prefaced with a brief description in parentheses (e.g., wood or loads) before the definition is given.

A	=	(wood diaphragm) area of chord cross section, in square inches
A	=	(wood shear wall) area of boundary element cross section, in square inches (vertical member at shear wall boundary)
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_p	=	the effective area (in square inches) of the projection of an assumed concrete failure surface upon the surface from which the anchor protrudes.
A_s	=	area of tension reinforcing steel
A_{se}	=	equivalent area of tension reinforcing steel
A_x	=	the torsional amplification factor at Level x .
A_{conc}	=	net concrete section area
a	=	depth of equivalent rectangular stress block
a_p	=	numerical coefficient specified in §1632 and set forth in Table 16-O of UBC.

Notation

B_m	=	nominal tensile strength of anchor bolt in masonry, in pounds.
b	=	(concrete beam) width of compression face of member
b	=	(wood diaphragm) diaphragm width, in feet
b	=	(wood shear wall) wall width, in feet
b_{tu}	=	factored tensile force supported by anchor bolt in masonry, in pounds
C_a	=	seismic coefficient, as set forth in Table 16-Q of UBC.
C_d	=	penetration depth factor
C_D	=	load duration factor
C_M	=	wet service factor
C_t	=	numerical coefficient given in §1630.2.2 of UBC.
C_v	=	seismic coefficient, as set forth in Table 16-R of UBC.
c	=	distance from neutral axis to extreme fiber
D	=	(loads) dead load on a structural element.
D	=	(wood) diameter
D_e	=	the length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.
d	=	(wood) dimension of wood member (assembly)
d	=	(concrete or masonry) distance from extreme compression fiber to centroid of tension reinforcement
d	=	(loads) distance from lateral resisting element to the center of rigidity
d	=	(wood) pennyweight of nail or spike
d_a	=	deflection due to anchorage details in wood shear wall (rotation and slip at tie-down bolts), in inches

E	=	(wood diaphragm) elastic modulus of chords, in psi
E	=	(wood shear wall) elastic modulus of boundary element (vertical member at shear wall boundary), in psi
E_c	=	modulus of elasticity of concrete, in psi
E_m	=	modulus of elasticity of masonry, in psi
E, E'	=	(wood) tabulated and allowable modulus of elasticity, in psi
e	=	diaphragm eccentricity
e_n	=	nail deformation in inches (see Table 23-2-K of UBC)
$E, E_h, E_m, E_v,$	=	(loads) earthquake loads set forth in §1630.1 of UBC.
F_b, F_b'	=	tabulated and allowable bending design value, in psi
$F_{c\perp}, F_{c\perp}'$	=	tabulated and allowable compression design value perpendicular to grain, in psi
F_v, F_v'	=	tabulated and allowable compression shear design value parallel to grain (horizontal shear), in psi
F_x	=	design seismic force applied to Level 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	=	(loads) that portion of the base shear, V , considered concentrated at the top of the structure in addition to F_n .
F_t	=	torsional shear force
F_v	=	direct shear force
F_y	=	specified yield strength of structural steel.
f_b	=	extreme fiber bending stress
f_c	=	(wood) actual compression stress parallel to grain

Notation

f_c'	=	specified compressive strength of concrete.
$f_{c\perp}$	=	(wood) actual compression stress perpendicular to grain
f_i	=	lateral force at Level i for use in Formula (30-10) of UBC.
f_m'	=	specified compressive strength of masonry, in psi
f_p	=	equivalent uniform load.
f_r	=	(masonry) modulus of rupture, in psi
f_y	=	specified tension yield strength of reinforcing steel.
f_v	=	(wood) actual shear stress parallel to grain
G	=	modulus of rigidity of plywood, in pounds per square inch (see Table 23-2-J of UBC)
g	=	acceleration due to gravity.
h	=	(concrete) height of wall between points of support, in inches
h	=	(wood shear wall) wall height, in feet
h_i, h_n, h_x	=	height in feet above the base to level i, n or x , respectively
I	=	importance factor given in Table 16-K of UBC.
I_{cr}	=	moment of inertia of cracked concrete or masonry section
I_g	=	moment of inertia of gross concrete or masonry section about centroidal axis, neglecting reinforcement
I_p	=	importance factor specified in Table 16-K of UBC.
k	=	(wood) wall stiffness
L	=	(loads) live load on a structural element, except roof live load
L_r	=	(loads) roof live load

L	=	(wood) span length of bending member
L	=	(wood diaphragm) diaphragm length, in feet
l_c	=	(concrete) vertical distance between wall supports, in inches
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.
M	=	maximum bending moment
M_{cr}	=	nominal cracking moment strength in concrete or masonry
M_n	=	nominal moment strength
M_s	=	the maximum moment in the wall resulting from the application of the unfactored load combinations
M_u	=	factored moment at section
M.C.	=	moisture content based on oven-dry weight of wood, in percent
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.
P	=	total concentrated load or total axial load
P_c	=	(concrete) design tensile strength of anchors, in pounds
P_u	=	factored axial load

Notation

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 p . 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.
T	=	(loads) torsional moment
t	=	thickness
t	=	(plywood) effective thickness of plywood for shear, in inches (see Tables 23-2-H and 23-2-I of UBC)
t_m	=	thickness of main member
t_s	=	thickness of side member
V	=	(wood) shear force.
V	=	(loads) 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_m	=	nominal shear strength of masonry
V_n	=	(concrete or masonry) nominal shear strength
V_n	=	(wood) fastener load, in pounds
V_s	=	nominal shear strength of shear reinforcement
V_u	=	(masonry) required shear strength
V_x	=	the design story shear in Story x .
v	=	(wood diaphragm) maximum shear due to design loads in the direction under consideration, plf
v	=	(wood shear wall) maximum shear due to design loads at the top of the wall, in plf

W	=	(wood) total uniform load.
W	=	(loads) the total seismic dead load defined in §1630.1.1 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 or 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.
\bar{x}, \bar{y}	=	distance to centroid
Z	=	seismic zone factor as given in Table 16-I of UBC.
Z, Z'	=	(wood) nominal and allowable lateral design value for a single fastener connection.
Δ	=	(wood) the calculated deflection of wood diaphragm or shear wall, in inches.
Δ_M	=	maximum inelastic response displacement, which is the total 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_{cr}	=	deflection at M_{cr}
Δ_n	=	deflection at M_n
Δ_s	=	(concrete) deflection at M_s
Δ_u	=	deflection due to factored loads, in inches.
γ	=	load/slip modulus for a connection, in pounds per inch.
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.

Notation

ϕ	=	strength-reduction factor
ρ	=	(loads) redundancy/reliability factor given by Formula (30-3) of UBC.
ρ	=	(concrete and masonry) ratio of area of flexural tensile reinforcement, A_s , to area bd .
ρ_b	=	reinforcement ratio producing balanced strain conditions.
Ω_o	=	seismic force amplification factor, which is required to account for structural overstrength and set forth in Table 16-N of UBC.
$\Sigma(\Delta_c X)$	=	sum of individual chord-splice slip values on both sides of wood diaphragm, each multiplied by its distance to the nearest support.

References

The following codes and standards are referenced in this document. Other reference documents are indicated at the end of each Design Example.

ACI-318, 1995, American Concrete Institute, *Building Code Regulations for Reinforced Concrete*, Farmington Hills, Michigan

AISC, American Institute of Steel Construction, *Manual of Steel Construction, Allowable Stress Design-ASD*, Chicago, Illinois

AISI, 1996, American Iron and Steel Institute, *Specification for the Design of Cold-Formed Steel Structural Members*, Washington, D.C

NDS, 1991, American Forest & Paper Association, *National Design Specification for Wood Construction*, Washington, D.C.

UBC 1997, International Conference of Building Officials, *Uniform Building Code*. Whittier, California.

SEAOC Blue Book, 1999, *Recommended Lateral Force Requirements and Commentary*. Structural Engineers Association of California, Sacramento, California.

Design Example 1

Wood Light Frame Residence



Figure 1-1. Wood light frame residence

Foreword

Small wood frame residences, such as the one in this example, have traditionally been designed using simplified design assumptions and procedures based largely on judgment and precedent. This example illustrates the strict, literal application of the 1997 UBC provisions. Two of the requirements shown, while required by the code, are considerably different than current California practice:

1. The use of wood diaphragms as part of the lateral force resisting system.

Traditionally, light frame dwellings have been designed assuming that such diaphragms behave as infinitely flexible elements. This assumption simplifies the analysis and allows lateral forces to be distributed to the vertical elements of the lateral force resisting system by tributary area methods. The code has had a *definition* of a flexible diaphragm since the 1988 UBC (§1630.6 of the 1997

UBC). UBC §1630.6 permits diaphragms to be treated as flexible, only if the maximum deflection of the diaphragm under the lateral loading is equal to or greater than twice the deflection of the vertical elements supporting the diaphragm in the story below. In this example, the diaphragm has been determined not to meet these criteria, and the design is based on the rigid diaphragm assumption. However, recognizing that the diaphragms in this structure likely behave as semi-rigid elements, neither fully flexible nor fully rigid, in this example an envelope approach has been used in which two analyses are performed. The first analysis uses the traditional flexible diaphragm assumptions and the second analysis is based on rigid diaphragm assumptions. The lateral resisting elements have been designed for the most severe forces produced by either assumption. Refer to the overview portion of this design example for further discussion about using the envelope approach.

Although these examples are a literal application of the 1997 UBC, the SEAOC Code and Seismology committees are of the joint opinion that the use of the more traditional design approach can provide acceptable life-safety performance for most one- and two-family dwellings. The commentary below provides more discussion of these issues:

2. The use of a system with limited ductility specifically cantilevered columns.

In this example, the cantilevered columns are used to provide lateral resistance at the garage door openings. In conventional practice, these would be designed for forces calculated using the R value associated with that system ($R=2.2$), with the balance of the structure designed with an R value with light framed shear walls ($R=5.5$). UBC §1630.4.4 requires that the R value used in each direction, may not be greater than the least value for any of the systems used in that same direction. Therefore, in this design example, because the R value for the cantilevered columns at the garage has an R value of 2.2, the entire structure in this direction has been designed using this R value.

Rigid versus flexible diaphragm assumptions.

Small, light frame detached one- and two-family dwellings have traditionally been designed using flexible diaphragm assumptions, or by a “hybrid” approach of treating closely spaced walls as a unit (i.e., as rigidly connected) and treating the remaining diaphragm as flexible. Also, light frame detached one- and two-family dwellings have been built with the conventional construction provisions of the code without an engineering design. These light frame structures have historically performed satisfactorily from a life-safety standpoint when subjected to strong seismic shaking. Two exceptions to light frame structures performing satisfactorily—both of which were addressed in the 1997 UBC by more stringent requirements—have been related to problems with the height-to-length ratio of shear wall panels and the use of plaster and drywall materials to resist seismic forces.

In the Commentary of the 1999 SEAOC Blue Book (§C805.3.1), it is recognized that lateral forces for many structures with wood diaphragms, mostly large buildings, may be better represented as rigid, as opposed to flexible, diaphragms. Relative to the small structure used in this example, the use of the rigid diaphragm assumptions generally will not significantly improve the seismic behavior.

While the building response remains elastic, the rigid diaphragm assumptions will better reflect the initial stiffness of the building system. However, it is not practically possible to accurately calculate the stiffness of all the various elements, including the stiffness contributed by finishes and nonstructural elements and taking into account the fact that stiffness of these elements will degrade as the ground shaking intensifies. As a result, the use of the rigid diaphragm assumptions may not be significantly better than the traditional flexible diaphragm assumption for structures of this type.

At the time of this publication, both the SEAOC Code and Seismology Committees agree that many one- and two-family residential structures can be safely designed using the traditional flexible diaphragm assumptions. Consequently, SEAOC *recommends* modification of the 1997 UBC provisions to allow use of the flexible diaphragm assumption for the design of one- and two-family dwellings. The engineer is cautioned, however, to discuss this with the building official prior to performing substantive design work.

Cantilever column elements in light frame construction.

The UBC requirement that buildings be designed using the least value R for combinations along the same axis was developed with two considerations in mind. The first is that in most structures, the building's ability to resist seismic forces can be limited to the weakest element in the structure. The second is purely a method of discouraging the more nonductile systems. The potential for $P\Delta$ instability of cantilevered column systems limits the column's capacity to carry large gravity loads when subjected to large building drifts. Therefore, the code has assigned a low R value to this system.

However, cantilever columns used in one- and two-family dwellings are typically lightly loaded, and can not develop this $P\Delta$ instability. Further, the literal application of §1630.44 would discourage the use of ordinary moment frames and cantilever column systems in favor for the use of slender shear walls that have been known to perform poorly. Consequently, the 1999 SEAOC Blue Book §105.4.4 (page 12) recommends the following alternative approach:

Exception: For light frame buildings in occupancy groups 4 and 5 and of two stories or less in height, the lateral force resisting elements are permitted to be designed using the least value of R for the different structural systems found on each independent line of resistance. The value of R used for design of

diaphragms for a given direction of loading in such structures shall not be greater than the least value used for any of the systems in that same direction.

Therefore, SEAOC *recommends* this alternative approach. The cantilever columns (together with any shear walls along that line of force, if present) would be designed using an $R = 2.2$, with the shear walls located along other lines of force designed using $R = 5.5$. In other words, the lateral load is factored up for the line with the cantilever column elements, but the conventional R value is used on the remainder of the structure. Consult with your local building official, however, before using this recommendation.

Overview

This design example illustrates the seismic design of a 2,800-square-foot single family residence. The structure, shown in Figures 1-1, 1-2, 1-3, 1-4 and 1-5, is of wood light frame construction with wood structural panel shear walls, roof, and floor diaphragms. Roofing is clay tile. Due to the high h/w (height/width) ratios of the walls next to the garage doors, cantilevered column elements are used to provide lateral support. As shown in Figure 1-3, there is an out-of plane offset from the cantilevered column elements on Line E to the glulam beams (GLBs) supporting the shear walls above Line D. The wood structural panel shear walls over the GLBs in the garage do not meet the required h/w ratios without the addition of straps and blocking above and below the window.

The residence cannot be built using conventional construction methods for reasons shown in Part 8 of this design example. The following steps illustrate a detailed analysis for some of the important seismic requirements of the 1997 UBC that pertain to design of wood light frame buildings. As stated in the introduction of this manual, these design examples, including this one, are not complete building designs. Many aspects of building design are not included, and only selected parts of the seismic design are illustrated. As is common for Type V construction (see UBC §606), a complete wind design is also necessary, but is not given in this design example.

Although the code criteria only recognize two diaphragm categories, flexible and rigid, the diaphragms in this design example are judged to be semi-rigid. Consequently, the analysis in this design example will use the envelope method, which considers the worst loading condition from both the flexible and rigid diaphragm analyses for vertical resisting elements. It should be noted that the envelope method, although not explicitly required by the code, will produce a more predictable performance than will use of only flexible or rigid diaphragm assumptions.

This design example will first determine the shear wall nailing and tiedown requirements obtained using the flexible diaphragm assumption to determine shear wall rigidities for the rigid diaphragm analysis.

The method of determining shear wall rigidities used in this design example is by far more rigorous than normal practice, but is *not* the only method available to determine shear wall rigidities. The Commentary at the end of this design example illustrates two other simplified approaches that would also be appropriate.

Outline

This example will illustrate the following parts of the design process:

- 1.** Design base shear and vertical distributions of seismic forces.
- 2.** Lateral forces on shear walls and shear wall nailing assuming flexible diaphragms.
- 3.** Rigidities of shear walls and cantilever columns at garage.
- 4.** Centers of mass and rigidity of diaphragms.
- 5.** Distribution of lateral forces to the shear walls with rigid diaphragms.
- 6.** Reliability/redundancy factor ρ .
- 7.** Diaphragm deflections and whether diaphragms are flexible or rigid.
- 8.** Does residence meet requirements for conventional construction provisions?
- 9.** Design shear wall frame over garage on line D.
- 10.** Diaphragm shears at the low roof over garage.
- 11.** Detail the wall frame over the GLB on line D.
- 12.** Detail the anchorage of wall frame to the GLB on line D.
- 13.** Detail the continuous load path at the low roof above the garage doors.

Given Information

Roof weights (slope 5:12):		Floor weights:	
Tile roofing	10.0 psf	Flooring	1.0 psf
½-in. sheathing	1.5	5/8" sheathing	1.8
Roof framing	4.0	Floor framing	4.0
Insulation	1.0	Miscellaneous	0.4
Miscellaneous	0.2	Gyp ceiling	<u>2.8</u>
Gyp ceiling	<u>2.8</u>		10.0 psf
D (along slope) =	19.5 psf		

D = dead load

D = (horiz. proj.) = $19.5 (13/12) = 21.1$ psf (the roof and ceilings are assumed to be on a 5:12 slope, vaulted)

Weights of respective diaphragm levels, including exterior and interior walls:

$$\begin{aligned}
 W_{roof} &= 64,000 \text{ lb (roof and tributary walls)} \\
 W_{floor} &= 39,000 \text{ lb (floor and tributary walls above and below)} \\
 W &= \overline{103,000 \text{ lb}}
 \end{aligned}$$

Weights of diaphragms are typically determined by adding the tributary weights of the walls to the diaphragm, e.g., add one-half the height of walls at the second floor to the roof and one-half the height of second floor walls plus one-half the height of first floor walls to second floor diaphragm. It is acceptable practice to ignore the weight of shear walls parallel to the direction of seismic forces to the upper level and add 100 percent of the parallel shear wall weight to the level below, instead of splitting the weight between floor levels. Weights of bearing partitions (not shear walls) should still be split between floors. Unlike commercial construction, the code minimum of 20 psf (vertical load) and 10 psf (lateral load) is often exceeded in residential construction.

Framing lumber is Douglas Fir-Larch grade stamped No. 1S-Dry.

APA-rated wood structural panels for shear walls will be 15/32-inch thick Structural I, 32/16 span rating, 5-ply with Exposure I glue, however, 4-ply is also acceptable. Three-ply 15/32-inch sheathing has lower allowable shears and the inner ply voids can cause nailing problems.

The roof is 15/32-inch thick APA-rated sheathing (equivalent to C-D sheathing in Table 23-II-4), 32/16 span rating with Exposure I glue.

The floor is 19/32-inch thick APA-rated Sturd-I-floor 16 inches o.c. rating (or APA-rated sheathing, 42/20 span rating) with Exposure I glue.

Boundary members for the shear walls are 4x posts.

Common wire nails are to be used for diaphragms, shear walls, and straps.
 Sinker nails are to be used for design of the shear wall sill plate nailing at the second floor. (Note: many nailing guns use the smaller diameter box and sinker nails instead of common nails. Closer nail spacing may be required for smaller diameter nails).

Seismic and site data:

$Z = 0.4$ (Zone 4)

$I = 1.0$ (standard occupancy)

Seismic source type = B

Distance to seismic source = 12 km

Soil profile type = S_C

Table 16-I

Table 16-K

S_C has been determined by geotechnical investigation. Without a geotechnical investigation, S_D can be used as a default value.

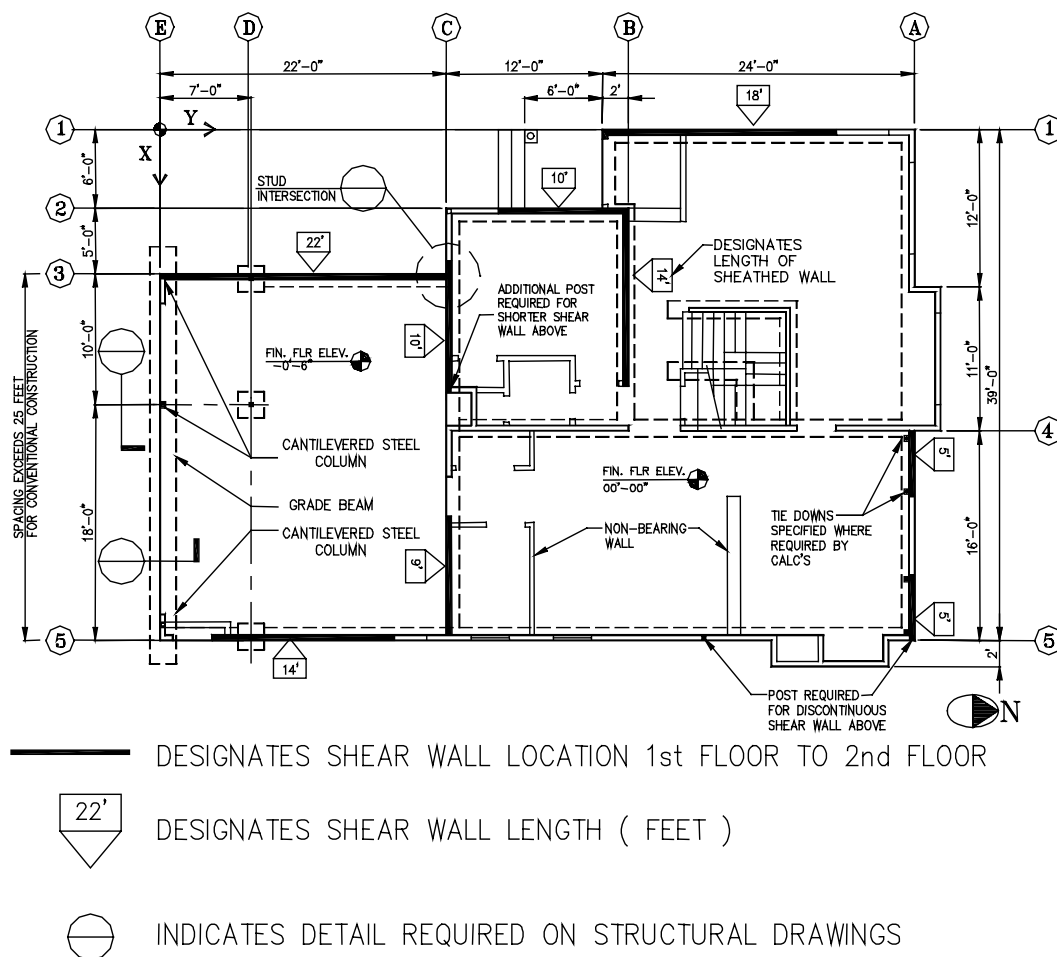


Figure 1-2. Foundation plan (ground floor)

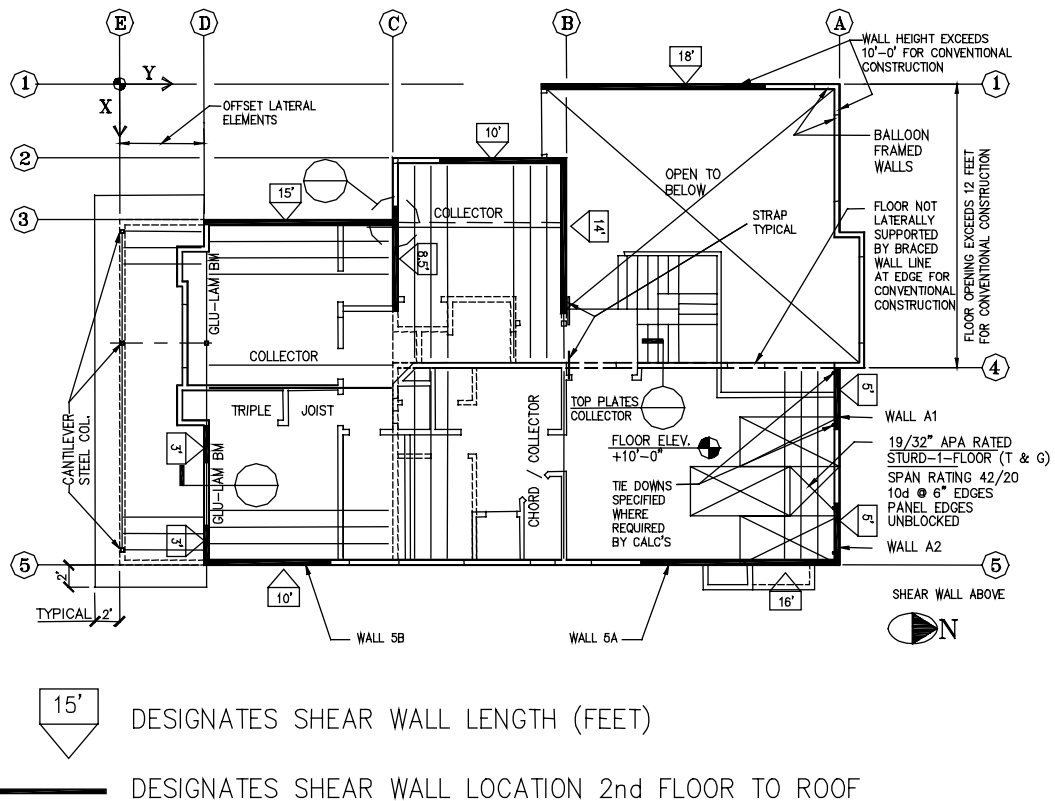


Figure 1-3. Second floor framing plan and low roof framing plan

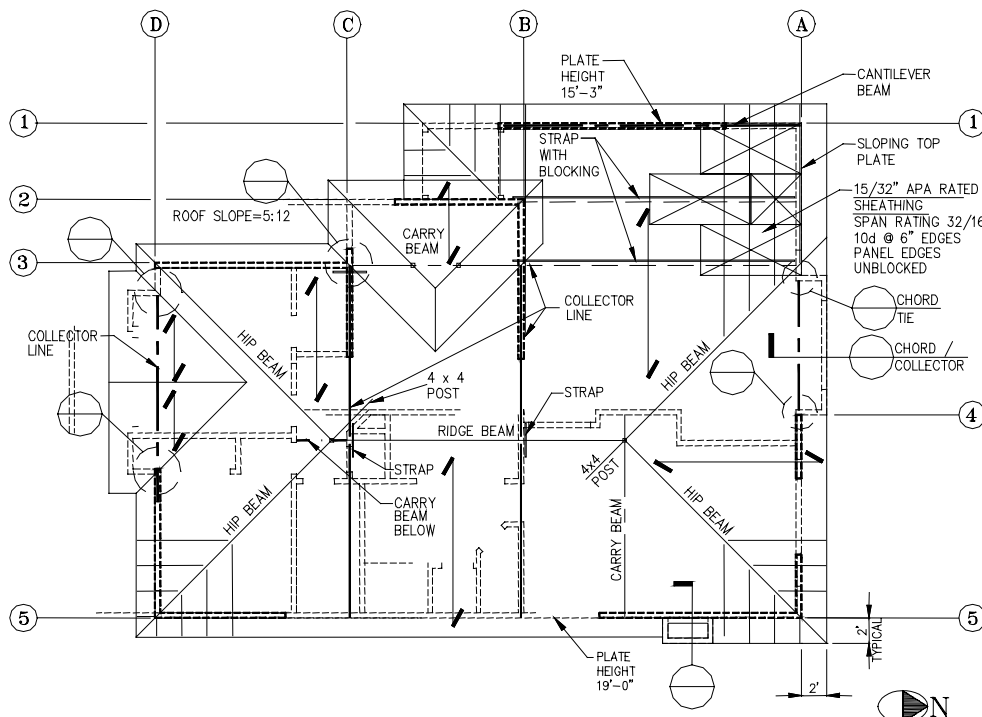


Figure 1-4. Roof framing plan

Figures 1-2 through 1-4 depict the shear walls as dark solid lines. This has been done for clarity in this example. Actual drawings commonly use other graphic depictions. Practice varies on how framing plans are actually shown and on which level the shear walls are indicated.

Actual drawings commonly do not call out shear wall lengths. However, building designers should be aware that some building departments now require shear wall lengths to be called out on plans.

Factors That Influence Design

Prior to starting the seismic design of the residence, three important related aspects of the design bear discussion. These are the effect of moisture content on lumber, the level of engineering design required to meet code requirements in present-day California practice, and effects of box nails on wood structural panel shear walls.

Moisture content in lumber connections.

91 NDS Table 7.3.3

This design example is based on dry lumber. Project specifications typically call for lumber to be grade stamped S-Dry (Surfaced Dry). Dry lumber has a moisture content (MC) less than or equal to 19 percent. Partially Seasoned or Green lumber grade-stamped S-GRN (surfaced green) has a MC between 19 percent and 30 percent. Wet lumber has a MC greater than 30 percent. Construction of structures using lumber with moisture contents greater than 19 percent can produce shrinkage problems in the structures. Also, many engineers and building officials are not aware of the reduction requirements, or wet service factors, related to installation of nails, screws, and bolts (fasteners) into lumber with moisture contents greater than 19 percent at time of installation. For fasteners in lumber with moisture contents greater than 19 percent at the time of installation, the wet service factor, $C_M = 0.75$ for nails and $C_M = 0.67$ for bolts, lags and screws (91 NDS Table 7.3.3). In other words, in lumber whose moisture content exceeds 19 percent, there is a 25 percent to 33 percent reduction in the strength of connections, diaphragms, and shear walls that is permanent. Drying of the lumber after installation of the connectors does not improve the connector capacity. The engineer should exercise good engineering judgment in determining whether it is prudent to base the structural design on dry or green lumber. Other areas of concern are geographical area and time of year the structure will be built. It is possible for green lumber (or dry lumber that has been exposed to rain) to dry out to a moisture content below 19 percent. For 2x framing, this generally takes about two to 3 weeks of exposure to dry air. Thicker lumber takes even longer. Moisture contents can easily be verified by a hand-held “moisture meter.”

Level and type of engineering design required for California residences.

The residence structure in this design example was chosen because it contains many of the structural problem areas that are commonly present in residential construction. These include:

1. The discontinuous shear wall at the north end of the line 5. (Although this is not a code violation per se, selection of a shear wall location that is continuous to the foundation would improve performance).
2. Lack of a lateral resisting element along line 4. (Although this is not a code violation per se, the addition of a shear wall at this location would improve performance).
3. The reduced scope of many structural engineering service contracts, such as “calculation and sketch” projects where the structural engineer provides a set of calculations and sketches of important structural details and the architect produces the actual plans and specifications. This often leads to poorly coordinated drawings and missing structural information. This method also makes structural observation requirements of the building code less effective when the engineer responsible for the design is not performing the site observation. Refer to the Commentary at the end of this design example for further discussion on this subject.

An important factor in the design of California residences, and residences in other high seismic zones, is the level of sophistication and rigor required by the designer. In this design example, a complete, rigorous analysis has been performed. In some jurisdictions, this may not be required by the building official or may not be warranted given the specifics of the design and the overall strength of the lateral force resisting system. The designer must choose between use of the more rigorous approach of considering a rigid diaphragm with torsional resistance characteristics with the more common approach of considering flexible diaphragms with tributary mass. The former may not be necessary in some situations, while at the same time recognizing that the laws of physics must be obeyed. In all cases, the completed structure must have a continuous lateral load path to resist lateral forces. Complete detailing is necessary, even for simple structures.

Effects of box nails on wood structural panel shear walls.

This design example uses common nails for fastening wood structural panels. Based on cyclic testing of shear walls and performance in past earthquakes, the use of common nails is preferred. UBC Table 23-II-I-1 lists allowable shears for wood structural panel shear walls for “common or galvanized box nails.” Footnote number five of Table 23-II-I-1, states that the galvanized nails shall be “hot-dipped or tumbled” (these nails are not gun nails). Most contractors use gun nails for diaphragm and shear wall installations. The UBC does not have a table for allowable shears for wood structural panel shear walls or diaphragms using box nails.

Box nails have a smaller diameter shank and a smaller head size. Using 10d box nails would result in a 19 percent reduction in allowable load for diaphragms and shear walls as compared to 10d common nails. Using 8d box nails would result in a 22 percent reduction in allowable load for diaphragms and shear walls as compared to 8d common nails. This is based on comparing allowable shear values listed in Tables 12.3A and 12.3B in the 1997 NDS for one-half-inch side member thickness (t_s) and Douglas Fir-Larch framing. In addition to the reduction of the shear wall and diaphragm capacities, when box nails are used, the walls will also drift more than when common nails are used.

A contributor to the problem is that when contractors buy large quantities of nails (for nail guns), the word “box” or “common” does not appear on the carton label. Nail length and diameters are the most common listing on the labels. This is why it is *extremely* important to list the required nail lengths and diameters on the structural drawings for all diaphragms and shear walls. Another problem is that contractors prefer box nails because their use reduces splitting, eases driving, and they cost less.

Just to illustrate a point, if an engineer designs for “dry” lumber (as discussed above) and “common” nails, and subsequently “green” lumber and “box” nails are used in the construction, the result is a compounding of the reductions. For example, for 10d nails installed into green lumber, the reduction would be 0.81 times 0.75 or a 40 percent reduction in capacity.

Calculations and Discussion

Code Reference

1.

Design base shear and vertical distribution of seismic forces.

§1630.2.2

This example uses the total building weight W applied to each respective direction. The results shown will be slightly conservative since W includes the wall weights for the direction of load, which can be subtracted out. This approach is simpler than using a separated building weight W for each axis under consideration.

1a.

Design base shear.

Period using Method A (see Figure 1-5 for section through structure):

$$T = C_t(h_n)^{3/4} = .020(23)^{3/4} = .21 \text{ sec.} \quad (30-8)$$

where:

h_n is the center of gravity (average height) of diaphragm above the first floor.

With seismic source type *B* and distance to source = 12 km

$$N_a = 1.0 \quad \text{Table 16-S}$$

$$N_v = 1.0 \quad \text{Table 16-T}$$

For soil profile type *S_C* and *Z* = 0.4

$$C_a = 0.40N_a = 0.40(1.0) = 0.40 \quad \text{Table 16-Q}$$

$$C_v = 0.56N_v = 0.56(1.0) = 0.56 \quad \text{Table 16-R}$$

North-south direction:

For light framed walls with wood structural panels that are both shear walls and bearing walls:

$$R = 5.5 \quad \text{Table 16-N}$$

Design base shear is:

$$V = \frac{C_v I}{RT} W = \frac{0.56(1.0)}{5.5(.21)} W = 0.485W \quad (30-4)$$

(Note that design base shear in the 1997 UBC is now on a strength design basis)

but need not exceed:

$$V = \frac{2.5C_a I}{R} W = \frac{2.5(.40)(1.0)}{5.5} W = 0.182W \quad (30-5)$$

A check of Equations 30-6 and 30-7 indicates these do not control:

$$\therefore V_{N-S} = \underline{\underline{0.182W}}$$

Comparison of the above result with the simplified static method permitted under §1630.2.3 shows that it is more advantageous to use the standard method of determining the design base shear.

$$V = \frac{3.0C_a}{R} W = \frac{3.0(.40)}{5.5} W = 0.218W > 0.182W \quad (30-11)$$

All of the tables in the UBC for wood diaphragms and shear walls are based on allowable loads.

It is desirable to keep the strength level forces throughout the design of the structure for two reasons:

1. Errors in calculations can occur and confusion on which load is being used—strength or allowable stress design. This design example will use the following format:

$$\begin{aligned}
 V_{base\ shear} &= \text{strength} \\
 F_{px} &= \text{strength} \\
 F_x &= \text{force to wall (strength)} \\
 v &= \text{wall shear at element level (ASD)} \\
 v = \frac{F_x}{1.4b} &= \text{ASD}
 \end{aligned}$$

2. This design example will not be applicable in the future, when the code will be all strength design.

$$E = \rho E_h + E_v = 1.0E_h + 0 = 1.0E_h \quad (30-1)$$

where:

E_v is allowed to be assumed as zero for allowable stress design, and ρ is assumed to be 1.0. This is the case for most of Type V residential construction structures. Since the maximum element story shear is not yet known, the value for ρ will have to be verified. This is done later in Part 6.

The basic load combination for allowable stress design is:

$$D + \frac{E}{1.4} = 0 + \frac{E}{1.4} = \frac{E}{1.4} \quad (12-9)$$

$$V_{N-S} = 0.182W$$

$$\therefore V_{N-S} = 0.182(103,000\text{lb}) = 18,750\text{lb} \quad \S 1612.3.1$$

East-west direction:

Since there are different types of lateral resisting elements in this direction, determine the controlling R value.

For light framed walls with wood structural panels that are both shear walls and bearing walls:

$$R = 5.5$$

For cantilevered column elements:

$$R = 2.2$$

Table 16-N

For combinations along the same axis, the UBC requires the use of least value for any of the systems utilized in that same direction, therefore the value for the cantilevered column elements must be used for the entire east-west direction. This provision for combinations along the same axis first appeared in the 1994 UBC.

$$R = 2.2$$

§1630.4.4

Design base shear is:

$$V = \frac{C_v I}{RT} W = \frac{0.56(1.0)}{2.2(.21)} W = 1.21W \quad (30-4)$$

but need not exceed:

$$V = \frac{2.5C_a I}{R} W = \frac{2.5(.40)(1.0)}{2.2} W = 0.454W \quad (30-5)$$

A check of Equations 30-6 and 30-7 indicates that these do not control:

$$\therefore V_{E-W} = 0.454W$$

This is less than that obtained with the simplified static method:

$$V = \frac{3.0C_a}{R} W = \frac{3.0(.40)}{2.2} W = 0.545W > 0.454W \quad (30-11)$$

$$V_{E-W} = 0.454W$$

$$V_{E-W} = 0.454(103,000\text{lb}) = \underline{\underline{46,750\text{lb}}} \quad \text{§1612.3.1}$$

Discussion of R factors.

The UBC places a severe penalty on the use of cantilevered column elements. The design base shear for the east-west direction is two and a half times that for the north-south direction. Some engineers use the greater R factor for light framed walls (e.g., $R = 5.5$), determine the design base shear, and then factor up the force for the respective frame element by using the ratio of the R for the shear walls over the R for the frame element (e.g., $5.5/2.2 = 2.5$). However, under a strict interpretation of the UBC, the factoring up approach does not appear to meet the intent of the UBC requirements. Another approach could be to design the residence

using a rigid diaphragm assumption with the wood shear walls taking 100 percent of the lateral force using $R = 5.5$. Then design the cantilever columns using $R = 2.2$ and a flexible diaphragm. Usually in residential construction, cantilevered column elements are preferred over moment frames by engineers and builders because of the elimination of field welding.

The 1999 Blue Book has added an exception for light frame buildings in Occupancy Groups 4 and 5 and of two stories or fewer in height. The local building department should be consulted on whether or not they will accept this exception. A higher force level could be counter productive in terms of splitting caused by added close nailing.

An ordinary moment-resisting frame could be used with an R value equal to 4.5. This would produce design base shear values only 22 percent higher than in the north-south direction. Additionally, the architecture could be modified to provide shear wall lengths that meet the h/w ratio limit of 2:1. With the plate height at 9'-0", the minimum wall length needed would be 4'-6". Another solution would be to increase the concrete curb height at the base of the wall such that the h/w ratio limit of 2:1 is not exceeded. For illustrative purposes, this design example uses the cantilevered column elements with the higher design base shear for the entire east-west direction. This conforms to the 1997 UBC. Pre-manufactured proprietary trussed wall systems and factory-built wood shear wall systems are also available. Special design considerations should be given when using these systems as outlined below:

1. Building system R values are to be based on officially adopted evaluation reports, such as ICBO reports.
2. Pre-manufactured systems should not be used in the same line as field-built shear walls because of deformation compatibility uncertainties.
3. Pre-manufactured systems should be limited to the first floor level only (of multi-story wood frame buildings) until testing is completed for these systems that sit on wood framing and are not rigidly attached to a concrete foundation.
4. Many of these "systems" exceed not only the new aspect ratio limit of 2:1, but also exceed the old aspect ratio limit of $3\frac{1}{2}:1$. Some are as narrow as 16 inches wide, leaving unanswered the question of whether this is a shear wall or a cantilever column (by comparison, if the "system" were a steel channel with the same width, it would be considered a cantilever column).
5. Many building officials are requesting that the same aspect (2:1) ratio limit for wood structural panel shear walls be adhered to for the pre-manufactured systems.

1b.**Vertical distribution of seismic forces.**

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

$$F_{px} = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (30-15)$$

where:

h_x is the average height at level i of the sheathed diaphragm in feet above the base.

Since $T = 0.21$ seconds < 0.7 seconds, $F_t = 0$

Determination of F_{px} is shown in Table 1-1.

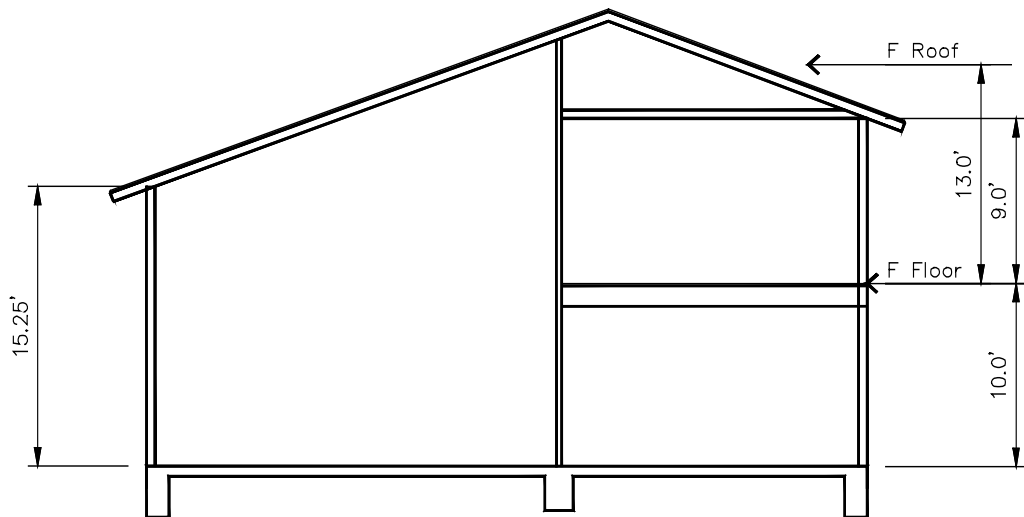


Figure 1-5. Cross-section through residence

Table 1-1. Vertical distribution of seismic forces

Level	w_x (lb)	h_x (ft)	$w_x h_x$ (lb-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_{pxN-S} (lb)	$\frac{F_{pxN-S}}{w_x}$	F_{pxE-W} (lb)	$\frac{F_{pxE-W}}{w_x}$
Roof	64,000	23.0	1,472,000	79	14,800	0.231	36,950	0.577
Floor	39,000	10.0	390,000	21	3,950	0.101	9,800	0.251
Σ	103,000	—	1,862,000	100	18,750	0.182	46,750	0.454

2.**Lateral forces on shear walls and shear wall nailing assuming flexible diaphragms.**

Determine the forces on shear walls. As has been customary practice in the past, this portion of the example assumes flexible diaphragms. The UBC does not require torsional effects to be considered for flexible diaphragms. The effects of torsion and wall rigidities will be considered later in Part 5 of this design example.

The selected method of determining loads to shear walls is based on tributary areas with simple spans between supports. Another method of determining loads to shear walls can assume a continuous beam. A continuous beam approach may not be accurate because of shear deformations in the diaphragm. The tributary area approach works with reasonable accuracy for a continuous beam with 100 percent shear deflection and zero bending deflection. This design example uses the exact tributary area to the shear walls, an approach that is fairly comprehensive. An easier and more common method would be to use a uniform load equal to the widest portion of the diaphragm, which results in conservative loads to the shear walls.

2a.**Forces on east-west shear walls.**

Roof diaphragm:

Roof area = 2,164 sq ft

$$f_{p \text{ roof}} = \frac{36,950 \text{ lb}}{2,164 \text{ sf}} = 17.07 \text{ psf}$$

$$w_1 = (17.07 \text{ psf})(43.0 \text{ ft}) = 734 \text{ plf}$$

$$w_2 = (17.07 \text{ psf})(37.0 \text{ ft}) = 632 \text{ plf}$$

$$w_3 = (17.07 \text{ psf})(32.0 \text{ ft}) = 546 \text{ plf}$$

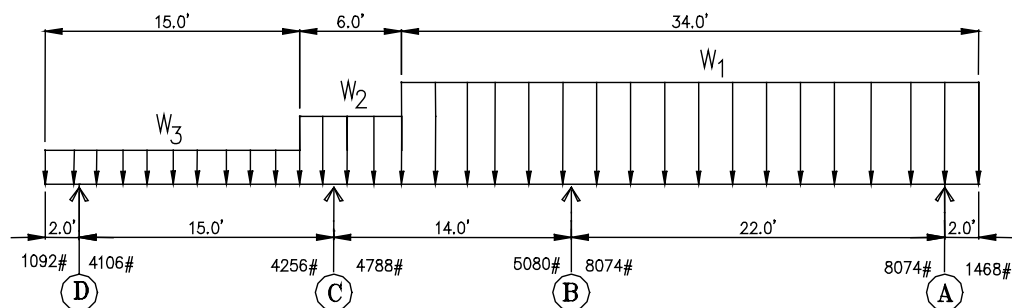


Figure 1-6. Roof diaphragm loading for east-west forces

Check sum of forces:

$$1,092 + 4,106 + 4,256 + 4,788 + 5,080 + 8,074 + 8,074 + 1,468 = 36,938 \text{ lb}$$

$$V_{Roof} = 36,938 \text{ lb} \approx 36,950 \text{ lb} \quad o.k.$$

Note that Figures 1-6, 1-7, 1-8 and 1-9 are depicted as a continuous beam. From a technical standpoint, “nodes” should be shown at the interior supports. In actuality, with the tributary area approach, these are considered as separate simple span beams between the shear wall “supports” (Figure 1-6 has three separate single span beams).

Floor diaphragm:

Second floor area = 1,542 sf

$$f_{p \text{ floor}} = \frac{9,800 \text{ lb}}{1,542 \text{ sf}} = 6.36 \text{ psf}$$

$$w_4 = (6.36 \text{ psf})(16.0 \text{ ft}) = 102 \text{ plf}$$

$$w_5 = (6.36 \text{ psf})(20.0 \text{ ft}) = 127 \text{ plf}$$

$$w_6 = (6.36 \text{ psf})(33.0 \text{ ft}) = 210 \text{ plf}$$

$$w_7 = (6.36 \text{ psf})(28.0 \text{ ft}) = 178 \text{ plf}$$

$$w_8 = (6.36 \text{ psf})(32.0 \text{ ft}) = 204 \text{ plf}$$

$$P_D = (1,092 \text{ lb} + 4,106 \text{ lb}) = 5,198 \text{ lb}$$

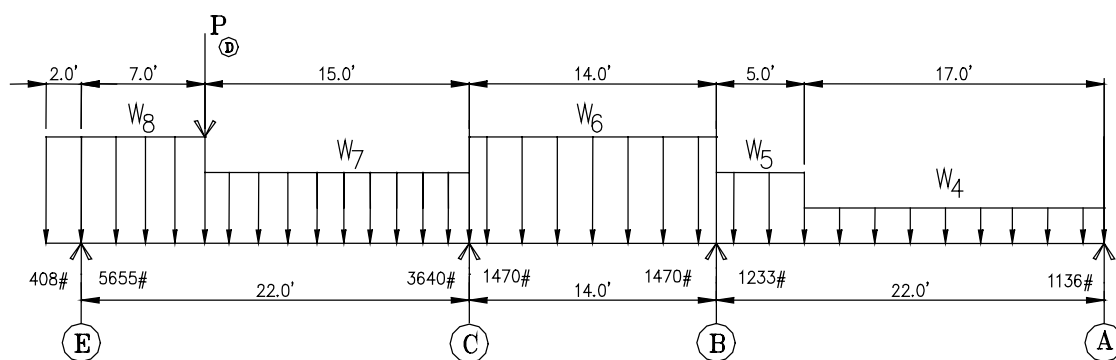


Figure 1-7. Second floor diaphragm loading for east-west forces

Check sum of forces:

$$408 + 5,655 + 3,640 + 1,470 + 1,470 + 1,233 + 1,136 = 15,012 \text{ lb}$$

Subtract P_D from the sum of forces:

$$15,012 - 5,198 = 9,814 \text{ lb}$$

$$V_{\text{floor}} = 9,814 \text{ lb} \approx 9,800 \text{ lb } o.k.$$

2b.

Required edge nailing for east-west shear walls using 10d common nails.

Table 23-II-I-1

Table 1-2. East-west shear walls at roof level (second floor to roof)^{1, 2, 3, 4, 5, 6, 7, 8, 9}

Wall (grid line)	$\sum F_{above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	b (ft)	$v = \frac{F_{tot}}{(b)1.4}$ (plf)	Sheathing ⁽⁵⁾ 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in.)
A	0	9,542	9,542	10.0	681 ⁽⁶⁾	One	870	2 ^{(2) (4)}
B	0	13,154	13,154	14.0	671 ⁽⁶⁾	Two	1330	3 ⁽⁴⁾
C	0	9,044	9,044	8.5	760 ⁽⁶⁾	Two	1330	3 ⁽⁴⁾
D	0	5,198	5,198	6.0	619 ⁽⁶⁾	Two ⁽⁸⁾	1740	2 ^{(2) (4)}
Σ	0	36,938	36,938	38.5				

Notes:

1. Minimum framing thickness. The 1994 and earlier editions of the UBC required 3x nominal thickness stud framing and blocking at abutting panel edges when 10d common nails were spaced 2 inches on center *or* when sheathing is installed on both sides of the studs without staggered panel joints. The 1997 UBC (Table 23-II-I-1 footnotes) requires 3x nominal thickness stud framing at abutting panel edges and at foundation sill plates when the allowable stress design shear values exceed 350 pounds per foot *or* if the sheathing is installed on both sides of the studs without staggered panel joints.
2. Sill bolt washers. Section 1806.6.1 requires a minimum of 2-inch-square by 3/16-inch-thick plate washers to be used for each foundation sill bolt (regardless of allowable shear values in the wall). These changes were a result of the splitting of framing studs and sill plates observed in the Northridge earthquake and in cyclic testing of shear walls. The plate washers are intended to help resist uplift forces on shear walls. Because of vertical displacements of holdowns, these plate washers are required even if the wall has holdowns designed to take uplift forces at the wall boundaries. The washer edges shall be parallel/perpendicular to the sill plate.
3. Errata to the First Printing of the 1997 UBC (Table 23-II-I-1 footnotes) added an exception to the 3x foundation sill plates by allowing 2x foundation sill plates when the allowable shear values are less than 600 pounds per foot, provided that sill bolts are designed for 50 percent of allowable values.
4. Refer to Design Example 2 for discussions about fasteners for pressure—preservative treated wood and the gap at bottom of sheathing.
5. APA Structural I rated wood structural panels may be either plywood or oriented strand board (OSB).
6. Note forces are strength level and shear in wall is divided by 1.4 to convert to allowable stress design.
7. It should be noted that having to use a nail spacing of 2 inches is an indication that more shear wall length should be considered. However, in this example, the close nail spacing is a direct result of $R = 2.2$ for the cantilever column elements. Some jurisdictions, and many engineers, as a matter of judgment, put a limit of 1,500 plf on wood shear walls.
8. A minimum of 3-inch nail spacing with sheathing on only one side is required to satisfy shear requirements. In this design example, sheathing has been provided on both sides with closer nail spacing in order to increase the stiffness of this short wall.
9. The 1999 Blue Book recommends special inspection when the nail spacing is closer than 4-inch on center.

Table 1-3. East-west shear walls at floor level (first floor to second floor)

Wall (grid line)	$\sum F_{above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	b (ft)	$v = \frac{F_{tot}}{(b)1.4}$ (plf)	Sheathing 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in)
A	9,542	1,136	10,678	10.0	763 ⁽²⁾	One	870	2
B	13,154	2,703	15,857	14.0	809 ⁽²⁾	Two	1330	3
C	9,044	5,110	14,154	19.0	532 ⁽²⁾	Two ⁽³⁾	1330	3
D	5,198	0	0	0	0			
E	0	6,063	6,063	Frame	Frame			
Σ	36,938	15,012	46,752	43.0				

Notes: See notes for Table 1-2.

2c. Forces on north-south shear walls.

Roof diaphragm:

$$f_{p \text{ roof}} = \frac{14,800 \text{ lb}}{2,164 \text{ sq ft}} = 6.84 \text{ psf}$$

$$w_1 = (6.84 \text{ psf})(55.0 \text{ ft}) = 376 \text{ plf}$$

$$w_2 = (6.84 \text{ psf})(40.0 \text{ ft}) = 274 \text{ plf}$$

$$w_3 = (6.84 \text{ psf})(34.0 \text{ ft}) = 233 \text{ plf}$$

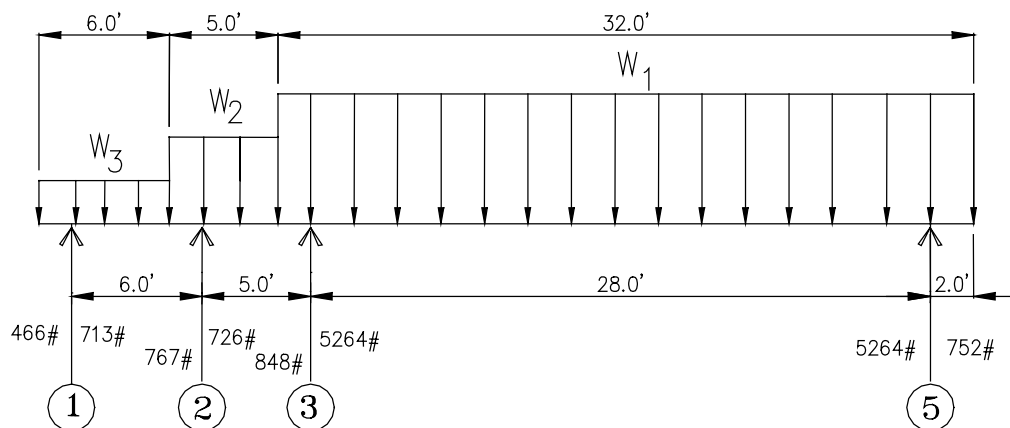


Figure 1-8. Roof diaphragm loading for north-south forces

Check sum of forces:

$$466 + 713 + 767 + 726 + 848 + 5,264 + 5,264 + 752 = 14,800 \text{ lb}$$

$$V_{\text{roof}} = 14,800 \text{ lb} \approx 14,800 \text{ lb} \quad o.k.$$

Floor diaphragm:

$$f_{p \text{ floor}} = \frac{3,950 \text{ lb}}{1,542 \text{ sq ft}} = 2.56 \text{ psf}$$

$$w_4 = (2.56 \text{ psf})(9.0 \text{ ft}) = 23.0 \text{ plf}$$

$$w_5 = (2.56 \text{ psf})(60.0 \text{ ft}) = 154 \text{ plf}$$

$$w_6 = (2.56 \text{ psf})(43.0 \text{ ft}) = 110 \text{ plf}$$

$$w_7 = (2.56 \text{ psf})(38.0 \text{ ft}) = 97.2 \text{ plf}$$

$$w_8 = (2.56 \text{ psf})(23.0 \text{ ft}) = 58.9 \text{ plf}$$

$$w_9 = (2.56 \text{ psf})(14.0 \text{ ft}) = 35.8 \text{ plf}$$

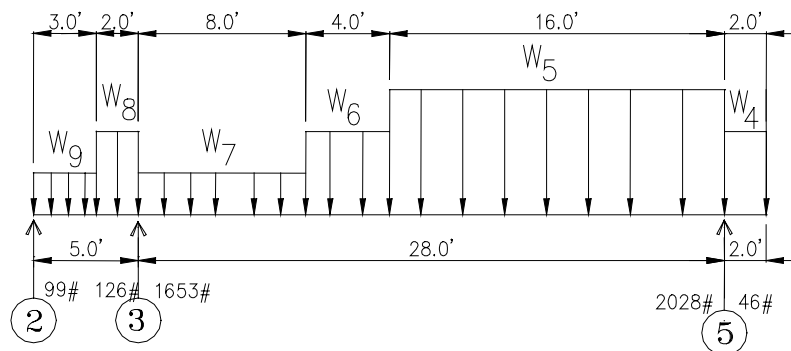


Figure 1-9. Second floor diaphragm loading for north-south forces

Check sum of forces:

$$99 + 126 + 1,653 + 2,028 + 46 = 3,952 \text{ lb}$$

$$V_{\text{floor}} = 3,952 \text{ lb} \approx 3,950 \text{ lb} \quad o.k.$$

2d.

Required edge nailing for north-south shear walls using 10d common nails.

Table 23-II-I-1

Table 1-4. North-south shear walls at roof level (second floor to roof)

Wall	$\sum F_{above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	b (ft)	$v = \frac{F_{tot}}{(b)1.4}$ (plf)	Sheathing 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in.)
1	0	1,179	1,179	18.0	47	One	510	4
2	0	1,493	1,493	10.0	107	One	510	4
3	0	6,112	6,112	15.0	291	One	510	4
5	0	6,016	6,016	26.0	165	One	510	4
Σ	0	14,800	14,800	69.0				

Table 1-5. North-south shear walls at floor level (first floor to second floor)

Wall	$\sum F_{above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	b (ft)	$v = \frac{F_{tot}}{(b)1.4}$ (plf)	Sheathing 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in.)
2	1,493	99	1,592	10.0	114	One	510	4
3	6,112	1,779	7,891	22.0	256	One	510	4
5	6,016	2,074	8,090	14.0	413	One	510	4
Σ	13,621	3,952	17,573	46.0				

3.

Rigidities of shear walls and cantilever columns at garage.

3a.

Estimation of wood shear wall rigidities.

Determination of the rigidities of wood shear walls is often difficult and inexact, even for design loads. In addition, when walls are loaded substantially beyond their design limits, as occur under strong earthquake motions, rigidity determination becomes even more difficult. It is complicated by a number of factors that make any exact determination, in a general sense, virtually impossible short of full-scale testing.

There is the well-known expression for shear wall deflection found in UBC Standard 23-2. This expression, shown below, is used to estimate deflections of shear walls with fixed bases and free tops for design level forces.

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

§23.223, Vol. 3

The expression above was developed from static tests of solid wood shear walls, many typically 8-foot x 8-foot in size. Until recently, there was very little cyclic testing of wood shear walls (to simulate actual earthquake behavior) or testing of walls with narrow aspect ratios.

In modern wood frame building construction, shear walls take many forms and sizes, and these are often penetrated by ducts, windows, and door openings. Also, many walls in residences are not designed as shear walls, yet have stiffness from their finish materials (gypsum board, stucco, etc.). In multi-story structures, walls are stacked on the walls of lower floors, producing indeterminate structural systems. In general, it is difficult to calculate wall rigidities with the UBC equation alone. As will be shown in subsequent paragraphs, things like shrinkage can significantly effect deflection and subsequent stiffness calculations. Further, in strong earthquake motions, shear walls may see forces and displacements several times larger than those used in design, and cyclic degradation effects can occur that significantly change the relative stiffness of shear walls at the same level.

It can be argued that wall rotation of the supporting wall below needs to be considered when considering shear wall rigidities. However, considering rotation of the supporting wall below would be similar to measuring the shear wall as the cumulative height, as opposed to the accepted floor-to-floor clear height. Not considering rotation of the supporting wall below is appropriate for determining relative wall rigidities.

At the present time, there are number of ways to estimate shear wall rigidities, particularly when only relative rigidities are desired (see Blue Book §C805.3). These include:

1. Rigidity based on estimated nail slip.
2. Rigidity calculated from UBC Standard 23-2 (the four term equation given above).
3. Rigidity incorporating both UBC Standard 23-2 and shrinkage.
4. Several other procedures.

Only one of these approaches is given in this design example. By using this one approach, SEAOC does not intend to establish a standard procedure or indicate a standard of care for calculation of wood shear wall rigidities. It is merely one of the present-day methods.

At present, CUREe (California Universities for Research in Earthquake Engineering) is conducting a large testing program to study earthquake effects on wood structures, including research on shear walls and diaphragms. It is expected that in the years ahead, new approaches will be developed and/or existing approaches reaffirmed or refined. Until then, the practicing structural engineer must use judgment in the method selected to determine wood shear wall rigidities.

It is recommended that the local building official be contacted for determination of what is acceptable in a particular jurisdiction.

3b.**Discussion of rigidity calculation using the UBC deflection equation.**

Since the rigidity, k , of a shear wall or cantilever column is based on its displacement, Δ , the displacements will first be computed using the F_{tot} forces already determined above in Tables 1-2 and 1-3.

Compute values for k :

$$F = k\Delta$$

$$\text{or } k = F/\Delta$$

The basic equation to determine the deflection of a shear wall is the four-term equation shown below.

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a \quad \text{\$23.223, Vol. 3}$$

The above equation is based on a uniformly nailed, cantilever shear wall with a horizontal point load at the top, panel edges blocked, and reflects tests conducted by the American Plywood Association. The deflection is estimated from the contributions of four distinct parts. The first part of the equation accounts for cantilever beam action using the moment of inertia of the boundary elements. The second term accounts for shear deformation of the sheathing. The third term accounts for nail slippage/bending, and the fourth term accounts for tiedown assembly displacement (this also should include bolt/nail slip and shrinkage). End stud elongation due to compression or tension is not considered, nor the end rotations of the base support. The UBC references this in §2315.1.

Testing on wood shear walls has indicated that the above formula is reasonably accurate for aspect ratios (h/w) lower than or equal to 2:1. For higher aspect ratios, the wall drift increases significantly, and testing showed that displacements were not adequately predicted. Use of the new aspect ratio requirement of 2:1 (1997 UBC) makes this formula more accurate for determining shear wall deflection/stiffness than it was in previous editions of the UBC, subject to the limitations mentioned above.

Recent testing on wood shear walls has shown that sill plate crushing under the boundary element can increase the deflection of the shear wall by as much as 20 percent to 30 percent. For a calculation of this crushing effect, see the deflection of wall frame at line D later in Part 3c.

Fastener slip/nail deformation values (e_n).

Volume 3 of the UBC has Table 23-2-K for obtaining values for e_n . However, its use is somewhat time-consuming, since interpolation and adjustments are necessary. Footnote 1 to Table 23-2-K requires the values for e_n to be decreased 50 percent for seasoned lumber. This means that the table is based on nails being driven into *green* lumber and the engineer must use one-half of these values for nails driven in *dry* lumber. The values in Table 23-2-K are based on tests conducted by the APA. The 50 percent reduction for dry lumber is a conservative factor. The actual tested slip values with dry lumber were less than 50 percent of the green lumber values.

It is recommended that values for e_n be computed based on fastener slip equations from Table B-4 of APA Research Report 138. Note that this Research Report is the basis for the formulas and tables in the UBC. Both the Research Report and the UBC will produce the same values. Using the fastener slip equations from Table B-4 of Research Report 138 will save time, and also enable computations to be made by a computer. For 10d common nails there are two basic equations:

When the nails are driven into *green* lumber:

$$e_n = (V_n/977)^{1.894} \quad \text{APA Table B-4}$$

When the nails are driven into *dry* lumber:

$$e_n = (V_n/769)^{3.276} \quad \text{APA Table B-4}$$

where:

V_n is the fastener load in pounds per fastener.

These values are based on Structural I sheathing and must be increased by 20 percent when the sheathing is not Structural I. The language in footnote a in UBC Table B-4 states “Fabricated green/tested dry (seasoned)...” is very misleading. The values in the table are actually *green values*, since the lumber is fabricated when green. Don’t be misled by the word “seasoned.”

It is uncertain whether or not the d_a factor is intended to include wood shrinkage and crushing due to shear wall rotation, because the code is not specific. This design example includes both shrinkage and crushing these in the d_a factor.

Many engineers have a concern that if the contractor installs the nails at a different spacing (too many or too few), then the rigidities will be different than those calculated. However, nominal changing of the nail spacing in a given wall does not significantly change the stiffness.

Determination of the design level displacement Δ_s .**§1630.9.1**

For both strength and allowable stress design, the 1997 UBC requires building drifts to be determined by the load combinations of §1612.2, which covers load combinations using strength design or load and resistance factor design. Errata for the second and third printing of the UBC unexplainably referenced §1612.3 for allowable stress design. The reference to §1612.3 is *incorrect* and will be changed back to reference §1612.2 in the fourth and later printings.

Wood design using the 1997 UBC now means that the engineer must use both strength-level forces and allowable stress forces. This can create some confusion, since the code requires drift checks to be strength-level forces. However, all of the design equations and tables in Chapter 23 are based on allowable stress design. Drift and shear wall forces will be based on strength-level forces. Remember that the structural system factor R is based on using strength-level forces.

3c.**Estimation of roof level rigidities.****Roof design level displacements.**

To determine roof level wall rigidities, roof level displacements must first be determined. Given below are a series of calculations, done in table form, to estimate the roof level displacements Δ_s in each shear wall connecting to the roof (Table 1-7). Because there is a wall with openings supported by a GLB on line D, the Δ_s for this wall must also be determined. Finally, roof level wall rigidities are summarized in Table 1-8 and a drift check is given in Table 1-9.

Table 1-6. Determine tiedown assembly displacements for roof level shear walls¹

Wall	ASD		Strength Design					
	Uplift ^{1.4(2)} (lb)	Tiedown ⁽³⁾ Device	Uplift (lb)	Tiedown ⁽⁴⁾ Elongation (in.)	Tiedown Assembly Displacement			$d_a^{(8)}$ (in.)
					Shrink ⁽⁵⁾	Crush ⁽⁶⁾	Slip ⁽⁷⁾	
A1	5,915	Bolted	8,280	0.13	0.19	0.02	0.04	0.38
A2	5,915	Bolted	8,280	0.13	0.19	0.02	0.04	0.38
B	5,975	Bolted	8,365	0.13	0.19	0.02	0.04	0.38
C	7,430	Bolted	10,400	0.17	0.19	0.04	0.05	0.45
1	0	Not required	0	0	0.02	0.02	0	0.04
2	0	Not required	0	0	0.19	0.02	0	0.21
3	830	Strap	1,160	0.004	0.19	0.02	0.002	0.21
5a	0	Not required	0	0	0.19	0.02	0	0.21
5b	0	Not required	0	0	0.19	0.02	0	0.21

Notes:

1. Tiedown assembly displacement is calculated at the second floor level.
2. Uplift force is determined by using the *net* overturning force ($M_{OT} - M_{OR}$) divided by the distance between the centroid of the tiedown to the end of the shear wall. With 4x members at the ends of the wall, this equates to the length of the wall minus 1¾ inches for straps, or the length of wall minus 5½ inches when using a bolted holdown with 2-inch offset from post to anchor bolt. Using allowable stress design, tiedown devices

need only be sized by using the ASD uplift force. The strength design uplift force is used to determine tiedown assembly displacement in order to determine strength-level displacements.

3. Continuous tie rod holdown systems can also be used. See Design Example 2 for method of calculating tiedown assembly displacement.
4. Tiedown elongation is based on actual uplift force divided by tiedown capacity times tiedown elongation at capacity (from manufacturer's catalog). Example for tiedown elongation at A1: tiedown selected has a 15,000 lb allowable load for a 5½-inch-thick (net) member. From the manufacturer's ICBO Evaluation Report, the tiedown deflection at the highest allowable design load (15,000 lb) is 0.12 inches. Since there are two tiedown devices (one above and one below the floor), the total elongation is twice the tiedown deflection of one device. Therefore the total tiedown elongation is $(8,280/15,000)0.12 \times 2 = 0.13$ inches.
5. Wood shrinkage based on a change from 19 percent moisture content (MC) to 13 percent MC with 19 percent MC being assumed for S-Dry lumber per project specifications. The MC of 13 percent is the assumed final MC at equilibrium with ambient humidity for the project location. The final equilibrium value can be higher in coastal areas and lower in inland or desert areas. This equates to $(0.002)(d)(19-13)$, where d is the dimension of the lumber (see Figure 1-10).

Shrinkage:

$$\begin{aligned} 2 \times \text{DBL Top Plate} + 2 \times \text{sill plate} &= (0.002)(3 \times 1.5 \text{ in.})(19 - 13) &= 0.05 \\ 2 \times 12 \text{ Floor Joist} &= (0.002)(11.25)(19 - 13) &= \underline{0.14} \\ & &= 0.19 \text{ in.} \end{aligned}$$

The use of pre-manufactured, dimensionally stable, wood I joists are considered not to shrink, and would thereby reduce the shrinkage to 0.05 inches.

6. Per 91 NDS 4.2.6, when compression perpendicular to grain ($f_{c\perp}$) is less than $0.73F'_{c\perp}$ crushing will be approximately 0.02 inches. When $f_{c\perp} = F'_{c\perp}$ crushing is approximately 0.04 inches. The effect of sill plate crushing is the downward effect with uplift force at the opposite end of the wall and has the same rotational effect as the tiedown displacement. Short walls that have no uplift forces will still have a wood crushing effect and contribute to rotation of the wall.
7. Per 91 NDS 7.3.6 $\gamma = (270,000)(1)^{1.5} = 270,000 \text{ lb/in.}$ plus 1/16" oversized hole for bolts. For nails, values for e_n can be used. Example for slip at tiedown at A1 (tiedown has five 1-inch diameter bolts to post):

$$\begin{aligned} \text{Load/bolt} &= 8,280/5 = 1,656 \text{ lb/bolt} \\ &= (270,000)(1)^{1.5} = 270,000 \text{ lb/in.} \\ \text{slip} &= (1,656/270,000) = 0.006 \text{ in.} \end{aligned}$$

Since there are two tiedown devices (one above and one below the floor), the total slip is twice the bolt slip. Good detailing practice should specify the tiedown bolts to be re-tightened just prior to closing in. This can accomplish two things: it takes the slack out of the oversized bolt hole and compensates for some wood shrinkage. This design example will assume that about one-half of the bolt hole slack is taken out.

$$\text{Therefore, total slip equals } (0.006 \times 2) + \left(\frac{1}{16}\right)\frac{1}{2} = 0.04 \text{ inches.}$$

8. d_a is the total tiedown assembly displacement. This also could include mis-cuts (short-studs) and lack of square cut ends.

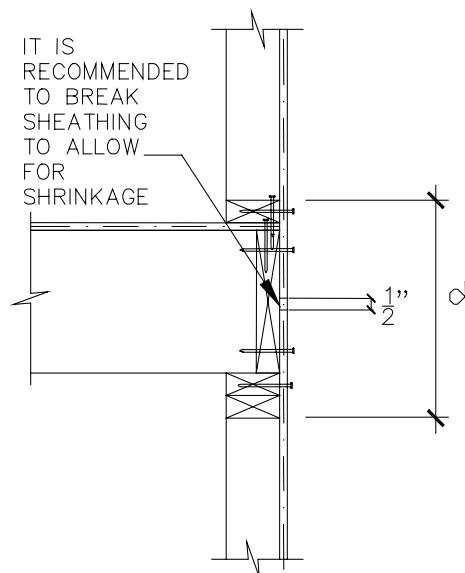


Figure 1-10. Second floor diaphragm connection to shear wall

Table 1-7. Deflections of the shear walls at the roof level^{1,2,6,10}

Wall	ASD v (plf)	Strength v (plf)	h (ft)	$A^{(3)}$ (sq in.)	E (psi)	b (ft)	$G^{(4)}$ (psi)	t (in.)	V_n (lb)	$e_n^{(5)}$ (in.)	d_a (in.)	$\Delta_S^{(7)}$ (in.)
A1	681	953	9.0	19.25	1.7E6	5.0	90,000	0.535	159	0.0057	0.38	0.93
A2	681	953	9.0	19.25	1.7E6	5.0	90,000	0.535	159	0.0057	0.38	0.93
B ⁽⁸⁾	336	470	10.0	19.25	1.7E6	14.0	90,000	0.535	118	0.0022	0.38	0.39
C ⁽⁸⁾	380	532	10.0	19.25	1.7E6	8.5	90,000	0.535	133	0.0032	0.45	0.68
1	47	66	15.25	19.25	1.7E6	18.0	90,000	0.535	22	8.8E-6	0.04	0.06
2	107	150	9.0	12.25	1.7E6	10.0	90,000	0.535	50	0.0001	0.21	0.22
3	291	407	9.0	12.25	1.7E6	15.0	90,000	0.535	136	0.0034	0.21	0.23
5a ⁽⁹⁾	194	271	9.0	12.25	1.7E6	16.0	90,000	0.535	90	0.0009	0.21	0.18
5b ⁽⁹⁾	120	168	9.0	12.25	1.7E6	10.0	90,000	0.535	56	0.0002	0.21	0.23

Notes:

- $\Delta_S = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a$ §23.223, Vol. 3
- h values are from the bottom of the sill plate to the bottom of the framing at diaphragm level (top plates).
- A values are for 4×6 posts for walls A1, A2, B, C, and wall 1. A values are for 4×4 posts for walls 2, 3, 5a, and 5b.
- G values are for Structural I sheathing. Testing of shear walls has indicated that the G values are slightly higher for oriented strand board (OSB) than plywood, but not enough to warrant the use of different values.
- e_n values for Structural I sheathing with dry lumber $= (V_n/769)^{3.276}$
- The use of a computer spreadsheet is recommended. This will not only save time, but also eliminate possible arithmetic errors with these repetitive calculations.
- Deflection of walls (Δ_S) is based on strength level forces. The shear wall deflections must be determined using the strength design forces. The calculated deflection of a shear wall is linear up to about two times

- the allowable stress design values. Since there are tiedown assembly displacements, and dead loads that resist overturning, the factoring up approach of ASD forces is not appropriate.
8. When sheathing is applied to both sides of the wall, the deflection of the shear wall is determined by using one-half the values from Table 1-2.
 9. In-plane shears to walls 5a and 5b are proportioned based on relative lengths (not per §23.223, Volume 3). Example for wall at line 5a: $R = 16^2 / (16^2 + 10^2) = 72$ percent, which is appropriate for two walls in a line, but not necessarily for three or more walls in line. Attempting to equate deflections is desirable. However, the calculations are iterative and indeterminate, and the results are very similar.
 10. For deflection of shear wall at line D, see the following Part 3c.

Determine deflection of wall frame at line D (with force transfer around openings).

The deflection for the shear wall can be approximated by using an analysis similar to computing the stiffness for a concrete wall with an opening in it. The deflection for the solid wall is computed, then a deflection for a horizontal window strip is subtracted, and the deflection for the wall piers added back in.

Engineering judgment may be used to simplify this approximation. However, the method shown below is one way to approximate the deflection.

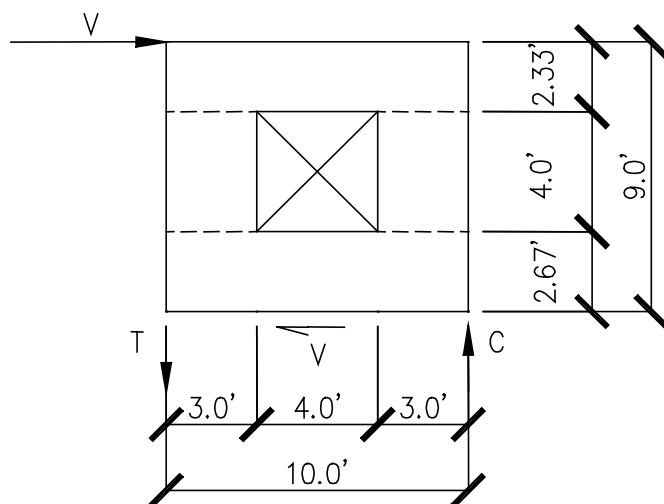


Figure 1-11. Elevation of wall frame on line D

First, determine deflection of the entire wall, without an opening:

Deflection of solid wall:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a \quad \text{\S 23.223 Vol. 3}$$

Sheathing is on both sides of wall with 10d common nails @ 2 inches o. c. Wall has 2×6 studs with 4×6 at ends.

$$V = 5,198 \text{ lb}$$

$$v = \frac{5,198 \text{ lb}}{(2)10.0 \text{ ft}} = 260 \text{ plf}$$

With edge nailing at 2 inches on center:

$$V_n = \text{load per nail} = 260(2/12) = 43 \text{ lb/nail}$$

$$e_n = (43/769)^{3.276} = 0.0001 \text{ inch}$$

With a tiedown elongation of 0.05 in., wood shrinkage of 0.13 in., and wood crushing of .02, it gives a tiedown assembly displacement of 0.20 in.

For crushing: from Part 9e, the strength level overturning moment $M_{OT} = 52,452 \text{ ft-lb}$. Dividing by the distance $L = 9.7 \text{ ft}$ computes the seismic downward component of the 4×6 post:

$$P = 52,452/9.7 = 5,407 \text{ lb}$$

$$f_c = P/A$$

$$f_c = 5,407/(3.5 \times 5.5) = 281 \text{ psi} < 0.73(625) = 456 \text{ psi}$$

$$\therefore \text{crush} = 0.02 \text{ in}$$

For shrinkage of GLB fabricated to AITC specifications at 17 percent MC: $0.002(17-13) 16.5 = 0.13 \text{ in}$.

For strap: $\frac{PL}{AE} + \text{strap nail slip} = 0.05 \text{ in.}$

$$d_a = 0.05 + 0.13 + 0.02 = 0.20 \text{ in.}$$

$$\Delta = \frac{8(260)9.0^3}{1.7E6(19.25)10.0} + \frac{260(9.0)}{(90,000)0.535} + 0.75(9.0)0.0001 + \frac{9.0(0.20)}{10.0} = 0.23 \text{ in.}$$

Second, determine deflection of window strip:

$$V = 5,198 \text{ lb (strength)}$$

With sheathing on both sides:

$$v = \frac{5,198 \text{ lb}}{(2)10.0 \text{ ft}} = 260 \text{ plf}$$

$$V_n = \text{load per nail} = 260(2/12) = 43 \text{ lb/nail}$$

$$e_n = (43/769)^{3.276} = 0.0001 \text{ in}$$

Since the boundary elements are connected to continuous posts that extend above and below the opening, the value of d_a equals the sheathing nail deformation value calculated above (boundary element “chord” elongation is neglected):

$$d_a = 0.0001 \text{ in.}$$

$$-\Delta = \frac{8(260)4.0^3}{1.7E6(19.25)10.0} + \frac{260(4.0)}{(90,000)0.535} + 0.75(4.0)0.0001 + \frac{4.0(0.0001)}{10.0} = 0.02 \text{ in.}$$

Note that this deflection is negative because it is subtracted from the sum of the deflections, as shown later.

Third, determine deflection of wall piers:

$$V = \frac{5,198 \text{ lb}}{2} = 2,599 \text{ lb}$$

$$v = \frac{2,599 \text{ lb}}{(2)3.0 \text{ ft}} = 433 \text{ plf}$$

$$V_n = \text{load per nail} = 433(2/12) = 72 \text{ lb/nail}$$

$$e_n = (72/769)^{3.276} = 0.0004 \text{ in.}$$

Since the boundary elements are connected to continuous posts that extend above and below the opening, the value of d_a equals the sheathing nail deformation value calculated for the wall piers.

$$d_a = 0.0004 \text{ in.}$$

$$\Delta = \frac{8(433)4.0^3}{1.7E(19.25)3.0} + \frac{433(4.0)}{(90,000)0.535} + 0.75(4.0)0.0004 + \frac{4.0(0.0004)}{3.0} = 0.04 \text{ in.}$$

Last, determine the sum of the deflections:

$$\Delta = 0.23 - 0.02 + 0.04 = 0.25 \text{ in}$$

Thus the stiffness of the wall is $(0.23/0.25)$, or 92 percent of that of the solid wall.

Determine deflection of wall due to deflection of GLB (see Figure 1-12).

Δh = Shear wall deflection due to deflection of the support beam

$$\tan \theta = \frac{\Delta V}{b} = \frac{\Delta h}{h}$$

$$\therefore \Delta h = \frac{h(\Delta V)}{b}$$

$$R_{OT} = \frac{Vh}{b}$$

$$R_{OT} = \frac{5,198 \text{ lb}(9.0 \text{ ft})}{10.0 \text{ ft}} = 4,678 \text{ lb (strength)}$$

For 5.125×16.5 GLB 24FV4:

$$E = 1,800,000 \text{ psi}$$

$$I = 1,918 \text{ in.}^4$$

$$\Delta V = \frac{R_{OT} a^2 b^2}{3EI L}$$

$$\Delta V = \frac{4,678(8.0 \times 12)^2 (10.0 \times 12)^2}{3(1.8 \text{E}6)1,918(18.0 \times 12)} = 0.278 \text{ in.}$$

$$\Delta h = \frac{h(\Delta V)}{b}$$

$$\Delta h = \frac{(9.0 \times 12)(0.278)}{(10.0 \times 12)} = 0.25 \text{ in.}$$

Total deflection of shear wall including GLB rotation and tiedown assembly displacement:

$$\Delta h = 0.25 + 0.25 = 0.50 \text{ in.}$$

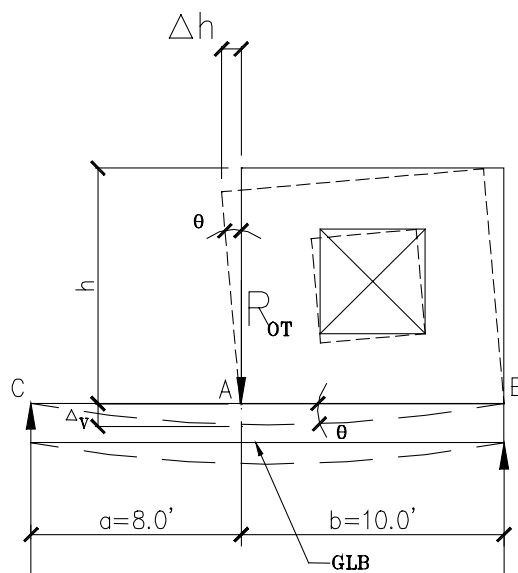


Figure 1-12. Wall elevation at line D

Table 1-8. Wall rigidities at roof level¹ (walls from second floor to roof)

Wall	$\Delta_s^{(2)}$ (in.)	F_{tot} (lb)	$k = \frac{F_{tot}}{\Delta_s}$ (k/in.)	$k = \frac{F_{tot}}{\Delta_s}$ (k/in.)
A1	0.93	4,771	5.130	10.26
A2	0.93	4,771	5.130	
B	0.39	13,154		33.73
C	0.68	9,044		13.30
D	0.50	5,198		10.40
1	0.06	1,179		19.65
2	0.22	1,493		6.79
3	0.23	6,112		26.57
5a	0.18	4,332	24.07	31.39
5b	0.23	1,684	7.32	

Notes:

1. Deflections and forces are based on strength force levels.
2. Δ_s is the design level displacement from Table 1-7 and calculations of wall frame.

Determination of Δ_M .**§1630.9.2**

Before checking drift, the maximum inelastic response displacement Δ_M must be computed. This is done as follows:

$$\Delta_M = 0.7R\Delta_S$$

$$R = 5.5 \text{ for the north-south direction}$$

$$R = 2.2 \text{ for the east-west direction}$$

$$\Delta_M = 0.7(5.5)\Delta_S = 3.85\Delta_S \text{ for the north-south direction}$$

$$\Delta_M = 0.7(2.2)\Delta_S = 1.54\Delta_S \text{ for the east-west direction}$$

Determination of maximum drift.**§1630.10.2**

The calculated story drift using Δ_M shall not exceed the maximum Δ_M which is 0.025 times the story height for structures that have a fundamental period less than 0.7 seconds. The building period for this design example was calculated to be 0.21 seconds, which is less than 0.7 seconds, therefore the 0.025 drift limitation applies.

Table 1-9. Drift check at roof level

	Wall	Δ_S (in.)	h (ft)	Δ_M (in.)	Max. $\Delta_M^{(1)}$ (in.)	Status
East-West	A1	0.93	9.0	1.43	2.70	ok
	A2	0.93	9.0	1.43	2.70	ok
	B	0.39	10.0	0.60	3.00	ok
	C	0.68	10.0	1.05	3.00	ok
	D	0.50	9.0	0.77	2.70	ok
North-South	1	0.06	15.25	0.23	4.57	ok
	2	0.22	9.0	0.85	2.70	ok
	3	0.23	9.0	0.88	2.70	ok
	5a	0.18	9.0	0.69	2.70	ok
	5b	0.23	9.0	0.88	2.70	ok

3d.**Estimation of second floor level rigidities.****First floor level design displacements.**

First floor level rigidities are determined by first calculating tiedown displacements (Table 1-10) and then deflections of shear walls at the second floor level (Table 1-11). The drift check, discussed in Part 3c, is given in Table 1-12, and wall rigidities calculated in Table 1-13.

Table 1-10. Tiedown assembly displacements for first floor level walls¹

Wall	ASD		LRFD					
	Uplift/1.4 ⁽²⁾ (lb)	Tiedown Device	Uplift (lb)	Tiedown ⁽³⁾ Elongation (in.)	Tiedown Assembly Displacement			d_a (in.)
					Shrink ⁽⁴⁾	Crush ⁽⁵⁾	Slip ⁽⁶⁾	
A1	13,450	Bolted	18,830	0.15	0.01	0.04	0.05	0.25
A2	13,450	Bolted	18,830	0.15	0.01	0.04	0.05	0.25
B	12,675	Bolted	17,745	0.14	0.01	0.04	0.05	0.24
C1	11,335	Bolted	15,870	0.13	0.01	0.04	0.04	0.22
C2	3,890	Bolted	5,445	0.04	0.01	0.02	0.04	0.11
2	0	Not req'd	0	0	0.01	0.02	0	0.03
3	825	Strap	1,155	0.05	0.01	0.02	0.002	0.08
5	400	Strap	560	0.03	0.01	0.02	0.002	0.06

Notes:

1. Tiedown assembly displacement is calculated at the foundation.
2. Uplift force is determined by using the net overturning force ($M_{OT} - M_R$), divided by the distance to the centroids of the boundary elements assuming 4x members at the ends of the shear wall. This equates to the length of the wall minus 3½ inches for straps, or the length of wall minus 7¼ inches when using a bolted holdown, which includes a 2-inch offset from post to tiedown bolt.
3. Tiedown elongation is based on actual uplift force divided by tiedown capacity multiplied by the tiedown elongation at capacity from manufacturer's catalog. Example of tiedown elongation at A1: Tiedown selected has a 15,000 lb allowable load for a 5½-inch member. From the manufacturer's ICBO approval, the tiedown deflection at the highest allowable design load (15,000 lb) is 0.12 inches, giving a tiedown elongation of $(18,830/15,000)0.12 = 0.15$ inches. Since the tiedown device has an average ultimate strength of 55,000 lb, the displacement can be assumed to be linear and therefore extrapolated.
4. Wood shrinkage is based on a change from 15 percent MC to 13 percent MC. This equates to $0.002 \times d \times (15-13)$. Where d is 2.5 inches for a 3× sill plate. Pressure-treated lumber has a moisture content of less than 15 percent at completion of treatment.
5. Per 91 NDS 4.2.6, when compression perpendicular to grain ($f_{c\perp}$) is less than $0.73F'_c \perp$ crushing will be approximately 0.02 inches, when $f_{c\perp} = F'_c \perp$ crushing is approximately 0.04 inches.
6. Per 91 NDS 7.3.6 $\gamma = \text{load/slip modulus} = (270,000)(D^{1.5})$ plus 1/16" oversized hole for bolts. For nails, values for e_n can be used. Example for slip at tiedown at A1 (Tiedown has five 1-inch diameter bolts to post).

$$\text{Load/bolt} = 18,830/5 = 3,766 \text{ lb/bolt}$$

Design Example 1 ■ Wood Light Frame Residence

$$\gamma = (270,000)(1)^{1.5} = 270,000 \text{ lb/in.}$$

$$\text{Slip} = (3,766/270,000) = 0.014 \text{ in.}$$

Good detailing should specify the tiedown bolts to be re-tightened just prior to closing in. This can accomplish two things: it takes the slack out of the oversized bolt hole, and compensates for some wood shrinkage. This design example assumes that about one-half of the bolt hole slack is taken out.

$$\text{Therefore the total slip} = (0.014) + \left(\frac{1}{16}\right)\frac{1}{2} = 0.05 \text{ in.}$$

Table 1-11. Deflections of the shear walls at the second floor level ^{1,2,3,4} (§23.222 Vol. 3)

Wall	ASD v (plf)	Strength v (plf)	h (ft)	A (sq in.)	E (psi)	b (ft)	G (psi)	t (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
A1	763	1,067	9.0	19.25	1.7E6	5.0	90,000	0.535	178	0.0083	0.25	0.74
A2	763	1,067	9.0	19.25	1.7E6	5.0	90,000	0.535	178	0.0083	0.25	0.74
B	404	566	9.0	19.25	1.7E6	14.0	90,000	0.535	141	0.0039	0.24	0.29
C1 ⁽⁵⁾	279	391	9.0	19.25	1.7E6	10.0	90,000	0.535	98	0.0012	0.22	0.29
C2 ⁽⁵⁾	251	351	9.0	19.25	1.7E6	9.0	90,000	0.535	88	0.0008	0.11	0.19
2	114	159	9.0	12.25	1.7E6	10.0	90,000	0.535	53	0.0002	0.03	0.06
3	256	359	9.0	12.25	1.7E6	22.0	90,000	0.535	120	0.0023	0.08	0.12
5	413	578	9.0	12.25	1.7E6	14.0	90,000	0.535	192	0.0106	0.06	0.23

Notes:

1. h values are from bottom of sill plate to bottom of framing at diaphragm level (top plates).

$$2. \Delta_s = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a \quad \text{\$23.223, Vol. 3}$$

3. G values are for Structural I sheathing. Testing of shear walls has indicated that the G values are slightly higher for OSB than plywood, but not enough to warrant different values.

4. e_n values for Structural I sheathing with dry lumber = $(V_n/769)^{3.276}$

5. Shear distributed to walls C1 and C2 are proportioned based on relative lengths. Attempting to equate deflections is desirable, however the calculations are iterative and indeterminate, and the results are very similar. The average Δ for walls A, B, and C at the second floor level is 0.42 inches. For deformation compatibility, it has been decided to size the cantilever column elements at line E for the deflections nearest shear wall at C, where the average is $\Delta = 0.24$ inches. Another approach would be to use a weighted average that includes the force in the wall. For example, if 99 percent of the load is carried by a stiff wall with $\Delta = 0.10$ inches and 1 percent is carried by wall with $\Delta = 1.00$ inches, then the weighted average approach is appropriate.

$\Delta = 0.10 \times 0.99 + 1.0 \times 0.01 = 0.11$ inches, this assumes no rotation and a rigid diaphragm. If the diaphragm is flexible, then deflection compatibility is not an issue. The engineer should exercise good engineering judgment in determining deformation compatibility.

Table 1-12. Drift check at second floor level

	Wall	Δ_s (in.)	h (ft)	Δ_M (in.)	Max. $\Delta_M^{(1)}$ (in.)	Status
East-West	A1	0.74	9.0	1.14	2.70	ok
	A2	0.74	9.0	1.14	2.70	ok
	B	0.29	9.0	0.44	2.70	ok
	C1	0.29	9.0	0.44	2.70	ok
	C2	0.19	9.0	0.29	2.70	ok
North-South	2	0.06	9.0	0.23	2.70	ok
	3	0.12	9.0	0.46	2.70	ok
	5	0.23	9.0	0.88	2.70	ok

Drift for cantilever columns at line E.

The cantilever column is assumed to be fixed at the base. This can be accomplished by setting the column on a footing and then casting the grade beam around the column. With this type of connection, the stresses in the flange of the column caused by concrete bearing at the top of the grade beam should be checked. Another approach is to provide a specially detailed base plate with anchor bolts that are bolted to the top of the grade beam. The bolts and base plate will allow for some rotation, which should be considered in computing the column deflections. The grade beam should have a stiffness of at least 10 times greater than that of the column for the column to be considered fixed at the base. It is common for columns of this type to have drift control the size of the column rather than bending.

$$\Delta = \frac{PL^3}{3EI}$$

It should be noted that if the steel columns were not needed to resist lateral forces (gravity columns only), and all lateral forces were resisted by the wood shear walls, then only relative rigidities of the wood shear walls would need to be calculated.

From Figure 1-7 at line E, the force to each of the three cantilever columns:

$$P = (5,655\text{ lb} + 408\text{ lb})/3 = 2,021\text{ lb/column}$$

$$I_{req'd} = \frac{2,022(9 \times 12)^3}{3(29 \times 10^6)0.24} = 122\text{ in.}^4$$

Use $TS10 \times 5 \times 3/8$

$$I_x = 128 \text{ in.}^4$$

$$\Delta_{TS} = \left(\frac{122}{128} \right) 0.24 = 0.23 \text{ in.}$$

$$M = (2,021 / 1.4) = 12,992 \text{ ft} \cdot \text{lb (allowable stress design)}$$

$$f_b = \frac{M}{S} = \frac{12,992 \times 12}{25.5} = 6,115 \text{ psi} < 0.66(46,000) \quad o.k.$$

Table 1-13. Wall rigidities at second floor level
(walls from first to second floor)⁽¹⁾

Wall	$\Delta_s^{(2)} \text{ (in.)}$	$F_{tot} \text{ (lb)}$	$k = \frac{F_{tot}}{\Delta_s} \text{ (k/in.)}$	$k = \frac{F_{tot}}{\Delta_s} \text{ (k/in.)}$
A1	0.74	5,339	7.215	14.43
A2	0.74	5,339	7.215	
B	0.29	15,857		54.68
C1	0.29	7,820	26.96	60.30
C2	0.19	6,334	33.34	
E	0.23	6,063		26.36
2	0.06	1,592		26.53
3	0.12	7,891		65.76
5	0.23	8,090		35.17

Notes:

1. Deflections and forces are based on strength force levels.
2. Δ_s is the design level displacement from Table 1-11.

4.

Determine centers of mass and rigidity of diaphragms.

It has been a common practice for practicing engineers to assume flexible diaphragms and distribute loads to shear walls based on tributary areas. This has been done for many years and is a well-established conventional design assumption. In this design example, the rigid diaphragm assumption will be used. This is not intended to imply that seismic design of residential construction in the past should have been necessarily performed in this manner. However, recent earthquakes and testing of wood panel shear walls have indicated that expected drifts are considerably higher than what was known or assumed in the past. This knowledge of the increased drifts of short wood panel shear walls has increased the need for the engineer to consider the relative rigidities of shear walls. This, and the fact that diaphragms tend to be much more rigid than the shear walls, has necessitated consideration of diaphragm rigidities. In this Part, the diaphragms are assumed to be rigid. See Part 7 for later confirmation of this assumption.

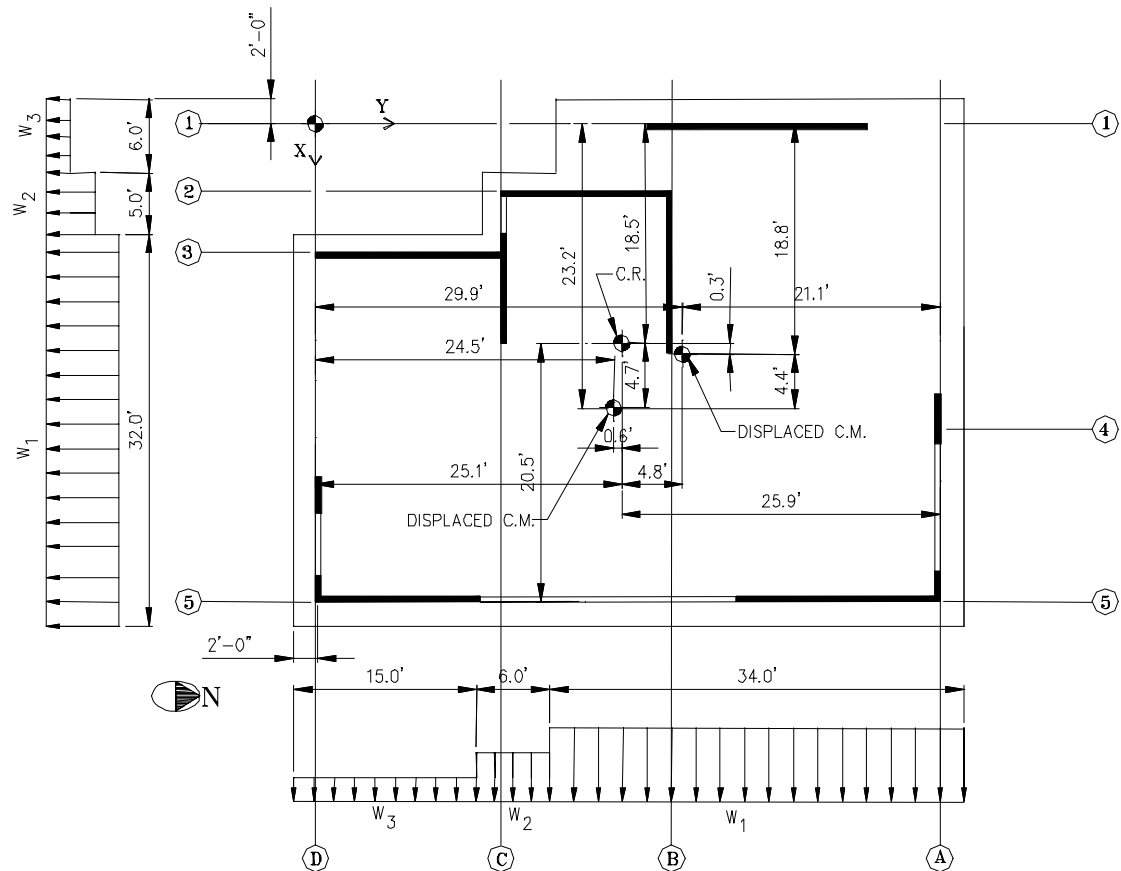
4a. For roof diaphragm.

Figure 1-13. Roof diaphragm centers of rigidity and mass

Determine center of mass of roof diaphragm from wall loads.

Using diaphragm loading from flexible diaphragm analysis for east-west direction (Figure 1-6) and summing forces about line D:

$$\begin{aligned}
 734 \text{ plf}(34.0 \text{ ft}) &= 24,956 \text{ lb} \times \left(\frac{34}{2} + 6 + (15 - 2) \right) \text{ ft} = 898,416 \text{ ft} \cdot \text{lb} \\
 632 \text{ plf}(6.0 \text{ ft}) &= 3,792 \text{ lb} \times \left(\frac{6}{2} + (15 - 2) \right) \text{ ft} = 60,672 \text{ ft} \cdot \text{lb} \\
 546 \text{ plf}(15.0 \text{ ft}) &= 8,190 \text{ lb} \times \left(\frac{15}{2} - 2 \right) \text{ ft} = 45,045 \text{ ft} \cdot \text{lb} \\
 \hline
 &36,938 \text{ lb} \qquad \qquad \qquad 1,004,133 \text{ ft} \cdot \text{lb}
 \end{aligned}$$

$$\therefore \bar{y}_m = \frac{\sum wx}{\sum w} = \frac{1,004,133 \text{ ft} - \text{lb}}{36,938 \text{ lb}} = \underline{\underline{27.2 \text{ ft @ roof}}}$$

Using diaphragm loading from flexible diaphragm analysis for north-south direction (Figure 1-8) and summing forces about line 1:

$$376 \text{ plf}(32.0 \text{ ft}) = 12,032 \text{ lb} \times 25.0 \text{ ft} = 300,800 \text{ ft} - \text{lb}$$

$$274 \text{ plf}(5.0 \text{ ft}) = 1,370 \text{ lb} \times 6.5 \text{ ft} = 8,905 \text{ ft} - \text{lb}$$

$$233 \text{ plf}(6.0 \text{ ft}) = 1,398 \text{ lb} \times 1.0 \text{ ft} = 1,398 \text{ ft} - \text{lb}$$

$$\underline{\underline{10,800 \text{ lb}}} \qquad \underline{\underline{311,103 \text{ ft} - \text{lb}}}$$

$$\therefore \bar{x}_m = \frac{\sum wy}{\sum w} = \frac{311,103 \text{ ft} - \text{lb}}{14,800 \text{ lb}} = \underline{\underline{21.0 \text{ ft @ roof}}}$$

Determine center of rigidity for roof diaphragm.

Using the rigidity values R from Table 1-8 and the distance y from line D to the shear wall:

$$\bar{y} = \frac{\sum (k_{xx}y)}{\sum k_{xx}} \quad \text{or} \quad \bar{y} \sum k_{xx} = \sum k_{xx}y$$

$$\bar{y}(10.40 + 13.30 + 33.73 + 10.26) = 10.40(0) + 13.30(15.0) + 33.73(29.0) + 10.26(51.0)$$

$$\therefore \bar{y}_r = \frac{1700.9}{67.69} = \underline{\underline{25.1 \text{ ft @ roof}}}$$

$$\bar{x} = \frac{\sum (k_{yy}x)}{\sum k_{yy}} \quad \text{or} \quad \bar{x} \sum k_{yy} = \sum k_{yy}x$$

$$\bar{x}(19.65 + 6.79 + 26.57 + 31.39) = 19.65(0) + 6.79(6.0) + 26.57(11.0) + 31.39(39.0)$$

$$\therefore \bar{x}_r = \frac{1557.2}{84.40} = \underline{\underline{18.5 \text{ ft @ roof}}}$$

Using diaphragm loading from flexible diaphragm analysis for north-south direction (Figure 1-9) and summing forces about line 2:

$$\begin{array}{rclcl}
 23 \text{ plf}(2.0 \text{ ft}) & = & 46 \text{ lb} \times 34.0 \text{ ft} & = & 5,564 \text{ ft} \cdot \text{lb} \\
 154 \text{ plf}(16.0 \text{ ft}) & = & 2,464 \text{ lb} \times 25.0 \text{ ft} & = & 61,600 \text{ ft} \cdot \text{lb} \\
 110 \text{ plf}(4.0 \text{ ft}) & = & 440 \text{ lb} \times 15.0 \text{ ft} & = & 6,600 \text{ ft} \cdot \text{lb} \\
 97.2 \text{ plf}(8.0 \text{ ft}) & = & 778 \text{ lb} \times 9.0 \text{ ft} & = & 7,002 \text{ ft} \cdot \text{lb} \\
 58.9 \text{ plf}(2.0 \text{ ft}) & = & 118 \text{ lb} \times 4.0 \text{ ft} & = & 471 \text{ ft} \cdot \text{lb} \\
 35.8 \text{ plf}(3.0 \text{ ft}) & = & 107 \text{ lb} \times 1.5 \text{ ft} & = & 160 \text{ ft} \cdot \text{lb} \\
 & & \hline
 & & 3,953 \text{ lb} & & 77,397 \text{ ft} \cdot \text{lb}
 \end{array}$$

$$\therefore \bar{x}_m = \frac{77,397 \text{ ft} \cdot \text{lb}}{3,953 \text{ lb}} = \underline{\underline{19.6 \text{ ft @ second floor}}}$$

Determine center of rigidity for floor diaphragm.

Using the rigidity values k from Table 1-13 and the distance y from line E to the shear wall:

$$\bar{y}_r(26.36 + 60.30 + 54.68 + 14.43) = 26.36(0) + 60.30(22.0) + 54.68(36.0) + 14.43(58.0)$$

$$\therefore \bar{y}_r = \frac{4132.0}{155.77} = \underline{\underline{26.5 \text{ ft @ second floor}}}$$

Using the rigidity values k from Table 1-13 and the distance x from line 2 to the shear wall:

$$\bar{x}_r(26.53 + 65.76 + 35.17) = 26.53(0.0) + 65.76(5.0) + 35.17(33.0)$$

$$\therefore \bar{x}_r = \frac{1489.4}{127.5} = \underline{\underline{11.7 \text{ ft @ second floor}}}$$

5.

Distribution of lateral forces to the shear walls with rigid diaphragms.**§1630.6**

Using the rigid diaphragm assumption, the base shear was distributed to the two levels in Part 1. In this Part, the story forces are distributed to the shear walls that support each level.

The code requires that the story force at the center of mass to be displaced from the calculated center of mass a distance of 5 percent of the building dimension at that level perpendicular to the direction of force. This is to account for accidental torsion. The code requires the most severe load combination to be considered and also permits the negative torsional shear to be subtracted from the direct load shear. However, lateral forces must be considered to act in each direction of the two principal axis. This design example does not consider eccentricities between the center of masses between levels. In this example, these eccentricities are small and are therefore considered insignificant. The engineer must exercise good engineering judgment in determining when these effects need to be considered.

5a.

For the roof diaphragm (Figure 1-13).

Forces in the east-west (x) direction:

$$\text{Distance to the calculated } CM : \bar{y}_m = 27.2 \text{ ft}$$

$$\text{Displaced } e_y = (0.05 \times 55 \text{ ft}) = 2.7 \text{ ft}$$

$$\text{New } \bar{y} \text{ to displace } CM = 27.2 \text{ ft} \pm 2.7 \text{ ft} = 29.9 \text{ ft or } 24.5 \text{ ft}$$

$$\text{Distance to the calculated } CR : \bar{y}_r = 25.1 \text{ ft}$$

$$e_y = 29.9 - 25.1 = 4.8 \text{ ft or } e_y = 25.1 - 24.5 = 0.6$$

Note that displacing the center of mass by 5 percent can result in the CM being on either side of the CR and can produce added torsional shears to all walls.

$$T_x = F_x e_y = 36,950 \text{ lb} (4.8 \text{ ft}) = 177,360 \text{ ft} - \text{lb}$$

or

$$T_x = F_x e_y = 36,950 \text{ lb} (0.6 \text{ ft}) = 22,170 \text{ ft} - \text{lb}$$

Forces in the north-south (y) direction:

$$\text{Distance to the calculated } CM : \bar{x}_m = 21.0 \text{ ft}$$

$$\text{Displaced } e_x = (0.05 \times 43 \text{ ft}) = 2.2 \text{ ft}$$

$$\text{New } \bar{x} \text{ to displace } CM = 21.0 \text{ ft} \pm 2.2 \text{ ft} = 23.2 \text{ ft or } 18.8 \text{ ft}$$

$$\text{Distance to the calculated } CR : \bar{x}_r = 18.5 \text{ ft}$$

$$e_x = 23.2 - 18.5 = 4.7 \text{ ft or } e_x = 18.8 - 18.5 = 0.3$$

$$T_y = F_y e_x = 14,800 \text{ lb} (4.7 \text{ ft}) = 69,560 \text{ ft} - \text{lb}$$

or

$$T_y = F_y e_x = 14,800 \text{ lb} (0.3 \text{ ft}) = 4,440 \text{ ft} - \text{lb}$$

$$F_{e-w} = 36,950 \text{ lb (Table 1-1)}$$

$$F_{n-s} = 14,800 \text{ lb (Table 1-1)}$$

$$T_x = 177,360 \text{ ft} - \text{lb for walls A and B}$$

$$T_x = 22,170 \text{ ft} - \text{lb for walls C and D}$$

$$T_y = 69,560 \text{ ft} - \text{lb for wall 5}$$

$$T_y = 4,440 \text{ ft} - \text{lb for walls 1, 2, and 3}$$

5b.

For roof wall forces.

The direct shear force F_v is determined from:

$$F_v = F \frac{R}{\sum R}$$

and the torsional shear force F_t is determined from:

$$F_t = T \frac{Rd}{J}$$

where:

$$J = \sum R d_x^2 + \sum R d_y^2$$

R = rigidity of lateral resisting element

d = distance from lateral resisting element to the center of rigidity

$$T = Fe$$

Table 1-14. Distribution of forces to shear walls below the roof level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	10.26			25.9	265.7	6,883	5601	1247	6,848
	B	33.73			3.9	131.5	513	18,412	617	19,029
	C	13.30			10.1	134.3	1,357	7,260	79	7,339
	D	10.40			25.1	261.0	6,552	5,677	151	5,828
	Σ	67.69					15,305	36,950		
North-South	1		19.65	-18.5		-363.5	6,725	3,446	-42	3,404
	2		6.79	-12.5		-84.8	1,061	1,191	-10	1,181
	3		26.57	-7.5		-199.3	1,495	4,659	-24	4,635
	5		31.39	20.5		643.5	13,192	5,504	1,185	6,689
	Σ		84.40				22,473	14,800		
	Σ						37,778			

For simplicity, many engineers will add 5 percent or 10 percent of the direct force shears to account for torsional effects. The average torsional force added to the shears walls in this design example is 11 percent of the direct force. Adding only 5 percent of the wall shears can be unconservative.

Torsional forces are subtracted from direct forces for this design example as now allowed by code. This only occurs when both of the displaced center of mass is on

the same side of the center of rigidity for a given direction. When the center of rigidity occurs between the two displaced centers of mass, then torsional forces can not be subtracted (which occurs at the roof in the east-west direction). Many engineers still neglect these negative forces.

5c.

For the floor diaphragm (Figure 1-14).

Forces in the east-west (x) direction:

$$\begin{aligned}
 \text{Distance to the calculated CM : } \bar{y}_m &= 24.3 \text{ ft} \\
 \text{Displaced } e_y &= (0.05 \times 60 \text{ ft}) = 3.0 \text{ ft} \\
 \text{New } \bar{y} \text{ to displace CM} &= 24.3 \text{ ft} \pm 3.0 \text{ ft} = 27.3 \text{ ft or } 21.3 \text{ ft} \\
 \text{Distance to the calculated CR : } \bar{y}_r &= 26.5 \text{ ft} \\
 e_y &= 27.3 - 26.5 = 0.8 \text{ ft} \\
 \text{or} \\
 e_y &= 26.5 - 21.3 = 5.2 \text{ ft} \\
 T_x = F_x e_y &= 46,750 \text{ lb}(0.8 \text{ ft}) = 37,400 \text{ ft} - \text{lb} \\
 \text{or} \\
 T_x = F_x e_y &= 46,750 \text{ lb}(5.2 \text{ ft}) = 243,100 \text{ ft} - \text{lb}
 \end{aligned}$$

Forces in the north-south (y) direction:

$$\begin{aligned}
 \text{Distance to the calculated CM : } \bar{x}_m &= 19.6 \text{ ft} \\
 \text{Displaced } e_x &= (0.05 \times 35 \text{ ft}) = 1.7 \text{ ft} \\
 \text{New } \bar{x} \text{ to displace CM} &= 19.6 \text{ ft} \pm 1.7 \text{ ft} = 21.3 \text{ ft or } 17.9 \text{ ft} \\
 \text{Distance to the calculated CR : } \bar{x}_r &= 11.7 \text{ ft} \\
 e_x &= 21.3 - 11.7 = 9.6 \text{ ft or } e_x = 17.9 - 11.7 = 6.2 \text{ ft} \\
 T_y = F_y e_x &= 18,750 \text{ lb}(9.6 \text{ ft}) = 180,000 \text{ ft} - \text{lb} \\
 \text{or} \\
 T_y = F_y e_x &= 18,750 \text{ lb}(6.2 \text{ ft}) = 116,250 \text{ ft} - \text{lb} \\
 F_{e-w} &= (36,950 + 9,800) = 46,750 \text{ lb (adding forces from roof and floor from Table 1-1)} \\
 T_x &= 37,400 \text{ ft} - \text{lb for walls A and B} \\
 T_x &= 243,100 \text{ ft} - \text{lb for walls C and E} \\
 F_{n-s} &= (14,800 + 3,950) = 18,750 \text{ lb (adding forces from roof and floor from Table 1-1)} \\
 T_y &= 116,250 \text{ ft} - \text{lb for walls 2 and 3} \\
 T_y &= 180,000 \text{ ft} - \text{lb for wall 5}
 \end{aligned}$$

Table 1-15. Distribution of forces to shear walls below the second floor level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	14.43			31.5	454.5	14,318	4,331	276	4,607
	B	54.68			9.5	519.5	4,935	16,410	316	16,726
	C	60.30			4.5	271.3	1,221	18,097	1,072	19,169
	E	26.36			26.5	698.5	18,511	7,910	2,760	10,670
	Σ	155.77					38,985	46,750		
North-South	2		26.53	-11.7		-310	3,632	3,903	-585	3,318
	3		65.76	-6.7		-440	2,952	9,674	-831	8,843
	5		35.17	21.3		749	15,956	5,173	2,191	7,364
	Σ		127.46				22,540	18,750		
	Σ						61,525			

Table 1-16. Comparison of loads on shear walls using flexible versus rigid diaphragm analysis and recheck of nailing in walls

Wall	$F_{flexible}$ (lb)	F_{rigid} (lb)	Rigid/ Flexible Ratio	b (ft)	$v = \frac{F_{max}}{(b)1.4}$ (plf)	Sheathing 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in.)
Roof Level								
A	9,542	6,848	0.72	10.0	682	One	870	2
B	13,154	19,029	1.44	14.0	970	Two	1330	3
C	9,044	7,339	0.81	8.5	760	Two	1330	3
D	5,198	5,828	1.12	6.0	693	Two	1740	2
1	1,179	3,404	2.89	18.0	135	One	510	4
2	1,493	1,181	0.79	10.0	107	One	510	4
3	6,112	4,635	0.76	15.0	292	One	510	4
5	6,016	6,689	1.11	26.0	184	One	510	4
Floor Level								
A	10,678	4,607	0.43	10.0	762	One	870	2
B	15,857	16,726	1.05	14.0	853	Two	1330	3
C	14,154	19,169	1.35	19.0	721	Two	1330	3
E	6,063	10,670	1.76	—	—	—	—	—
2	1,592	3,318	2.08	10.0	237	One	510	4
3	7,891	8,843	1.12	22.0	287	One	510	4
5	8,090	7,364	0.91	14.0	413	One	510	4

Shear walls with shears that exceed 350 pounds per lineal foot will require 3x framing at abutting panel edges with staggered nails. See also notes at bottom of Table 1-2.

Where rigid diaphragm analysis shows seismic forces to the shear walls are higher than from flexible diaphragm analysis, the wall stability and anchorage must be re-evaluated. Engineering judgment should be used to determine if a rigid diaphragm analysis should be repeated due to changes in wall rigidity.

If rigid diaphragm loads are used, the diaphragm shears should be rechecked for total load divided by diaphragm length along the individual wall lines.

6.

Reliability/redundancy factor ρ .**§1630.1.1**

The reliability/redundancy factor penalizes lateral force resisting systems without adequate redundancy. In this example (in Part 1), the reliability/redundancy factor was assumed to be $\rho = 1.0$. This will now be checked.

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

where:

r_{max} = the maximum element-story shear ratio.

For shear walls, the ratio for the wall with the largest shear per foot at or below two-thirds the height of the building is calculated. Or in the case of a three-story building, the lower two levels. The value of r_{max} is computed from the total lateral load in the wall multiplied by $10/l_w$ and divided by the story shear.

l_w = length of wall in feet

A_B = the ground floor area of the structure in square feet

$$r_i = \frac{V_{max}(10/l_w)}{F}$$

$A_B = 1,542 \text{ sq ft}$

For east-west direction:

Using strength-level forces for wall C:

$$r_{max} = \frac{16,726(10/14.0)}{46,750} = 0.26$$

$$\rho = 2 - \frac{20}{0.26\sqrt{1,542}} = 0.04 < 1.0 \text{ minimum} \quad o.k.$$

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

Therefore, there is no increase in base shear required due to lack of reliability/redundancy. The SEAOC Seismology Committee added the sentence “The value of the ratio $10/l_w$ need not be taken as greater than 1.0” in the 1999 SEAOC Blue Book—which will not penalize longer walls, but in this design example has no effect.

Note that the cantilevered column elements are not considered to be a moment frame and are not subject to the r_i and ρ requirements of §1630.1.

For north-south direction:

Using strength-level forces for wall 5:

$$r_{max} = \frac{8,090(10/14.0)}{18,750} = 0.31$$

$$\rho = 2 - \frac{20}{0.31\sqrt{1,542}} = 0.36 < 1.0 \text{ minimum} \quad o.k.$$

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

Therefore, for both directions there is no increase in base shear required due to lack of reliability/redundancy.

7.

Diaphragm deflections and whether diaphragms are flexible or rigid.

This step is shown only as a reference for how to calculate horizontal diaphragm deflections. Since the shear wall forces were determined using both flexible and rigid diaphragm assumptions, there is no requirement to verify that the diaphragm is actually rigid or flexible.

The design seismic force in the roof and floor diaphragms using Equation 33-1 must first be found. The design seismic force is then divided by the diaphragm area to determine the horizontal loading in pounds per square foot (refer to Figures 1-13 and 1-14). The design seismic force shall not be less than $0.5C_a Iw_{px}$ nor greater than $1.0C_a Iw_{px}$.

The basic equation to determine seismic forces on a diaphragm is shown below. The following will compute the seismic forces in the north-south direction.

$$F_{px} = \frac{F_t + \sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (33-1)$$

$F_t = 0$ in this example because $T < 0.7$ seconds.

Note that the forces in the east-west direction are higher.

$$F_{p \text{ roof}} = \frac{(36,950 \times 64,000)}{64,000} = 36,950 \text{ lb}$$

$$F_{p \text{ roof}} = \frac{36,950 \text{ lb}}{2,164 \text{ sq ft}} = 17.07 \text{ psf}$$

For the uppermost level, the above calculation will always produce the same force as computed in Eq. (30-15).

$$F_{p \text{ floor}} = \frac{(36,950 \times 9,800) \times 39,000}{(39,000 + 64,000)} = 17,701 \text{ lb (governs)}$$

$$F_{p \text{ min}} = 0.5C_a Iw_{px} = 0.5(0.40)(1.0)w_{px} = 0.20(39,000) = 7,800 \text{ lb} \quad \S 1633.2.9$$

$$F_{p \text{ floor}} = \frac{17,701 \text{ lb}}{1,542 \text{ sq ft}} = 11.48 \text{ psf}$$

In this example, the roof and floor diaphragms spanning between line A and line B will be used to illustrate the method. The basic equation to determine the deflection of a diaphragm is shown below.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b} \quad \S 23.222 \text{ Vol. 3}$$

The above equation is based on a uniformly loaded, uniformly nailed, simple span diaphragm with blocked panel edges and is based on monotonic tests conducted by the American Plywood Association (APA). The equation has four separate parts. The first part of the equation accounts for beam bending, the second accounts for shear deformation, the third accounts for nail slippage/bending, and the last part accounts for chord slippage. The UBC references this in §2315.1.

For the purpose of this calculation, assume the diaphragm is a simple span supported at A and B (refer to Figures 1-13 and 1-14). In reality, with continuity at B, the actual deflection will be less.

7a.

Roof diaphragm.

Check diaphragm shear.

Based on the $F_{p \text{ roof}} = 17.07 \text{ psf}$ as computed above, find roof shear to line A for the east-west direction.

1. Area of roof including over hangs is 22' x 43'.
2. Wall length is 39 ft.
3. Diaphragm shears are converted to allowable stress design by dividing by 1.4.

$$v = \frac{(17.07)43.0(22.0)}{1.4(39.0)^2} = 148 \text{ plf} < 190 \text{ plf allowable}$$

From Table 23-II-H, the allowable shear of 190 plf is based on 15/32-inch APA-rated wood structural panels with unblocked edges and 10d nails spaced at 6 inches on center at boundaries and panel edges. APA-rated wood structural panels may be either plywood or oriented strand board (OSB).

Check diaphragm deflection.

The UBC specifies that the deflection be calculated on a unit load basis. In other words, the diaphragm deflection should be based on the *same load* as the load used for the lateral resisting elements, not F_{px} total force at the level considered. Since the code now requires building drifts to be determined by the load combinations of §1612.2 (see Part 3b for additional comments), determine strength loads on building diaphragm.

$$f_{p \text{ roof}} = \frac{36,950 \text{ lb}}{2,164 \text{ sq ft}} = 17.07 \text{ psf}$$

$$v = \frac{(17.07 \text{ psf}) 43.0 \text{ ft} (22.0 \text{ ft})}{2(39.0 \text{ ft})} = 207 \text{ plf}$$

With nails at 6 inches on center, the load per nail is $207(6/12) = 104 \text{ lb/nail} = V_n$

$$L = 22.0 \text{ ft}$$

$$b = 39.0 \text{ ft}$$

$$G = 90,000 \text{ psi}$$

Table 23-2-J Vol. 3

$$E = 1,700,000 \text{ psi}$$

$$A_{2 \times 4 \text{ chords}} = 5.25 \text{ sq in.} \times 2 = 10.50 \text{ sq in.}$$

Sum of individual chord-splice slip.

Note that the area for 2 – 2 × 4 top plates (chord) has been used. All top plates are connected with metal straps. If a metal strap is not used, then use of the area for one top plate is recommended. Also note that the top plates at line 1 are 2 – 2 × 6. The deflection calculation will conservatively use the chord area of the 2 2x4s at line 5.

Fastener slip/nail deformation values (e_n).

$$e_n = 1.20(104/769)^{3.276} = 0.0017$$

$$t = 0.298 \text{ in. (for CDX or Standard Grade)}$$

Table 23-2-H

The chord-splice of the diaphragm will be spliced with a 12 gauge metal strap using 10d nails. Assume a chord splice of the diaphragm at mid-span. The slippage for both the diaphragm chords is to be included. The nail slip value from APA Research Report 138 can be used:

$$e_n = (V_n/769)^{3.276} = (120/769)^{3.276} = 0.002 \text{ in.}$$

where:

The allowable load is 120 pound per nail (from NDS Table 12.3F for a 10d nail in a 12-gauge strap).

$V_n = 120 \text{ lb/nail}$ in the strap. The elongation of the metal strap is assumed to be 0.03 inches.

Therefore, the chord slip is:

$$\Delta_c = 0.002 + 0.03$$

$$\Delta_c = 0.032 \text{ in.}$$

$$\sum (\Delta_c X) = (0.032)11.0 \text{ ft}(2) = 0.70 \text{ in.} - \text{ft}$$

Where the distance to the nearest support is 11'-0" and to get the sum for both chords you multiply by 2.

$$\Delta = \frac{5(207)22.0^3}{8(1.7E6)10.50(39.0)} + \frac{207(22.0)}{4(90,000)0.298} + 0.188(22.0)0.0017 + \frac{0.70}{2(39.0)} = 0.06 \text{ in.}$$

This deflection is based on a *blocked* diaphragm. The UBC does not have a formula for an *unblocked* diaphragm. The APA is currently working on a simplified formula for unblocked diaphragms. Based on diaphragm deflection test results performed by the APA, an unblocked diaphragm will deflect between 2 to 2½ times more than that of a blocked diaphragm or can be proportioned to allowable shears. The roof diaphragm is also sloped at 5:12, which is believed to increase the deflection (but has not been confirmed with tests). This design example has unblocked panel edges for the floor and roof diaphragms, so a conversion factor is necessary. It is assumed that the unblocked diaphragm will deflect:

$$\Delta = 0.06(2.5) = \underline{0.15 \text{ in.}}$$

Note that at gable ended roofs, when the chord is in the plane of the roof (pitched), the chord connection at the ridge should be carefully detailed to accommodate the uplift component of the chord.

7b.

Floor diaphragm.

Check diaphragm shear.

Based on the $F_p \text{ floor} = 11.48 \text{ psf}$ as computed in Part 7 above, find floor shear to line A for the east-west direction (area of floor is 22×16).

Diaphragm shears are converted to allowable stress design by dividing by 1.4 where:

$$v = \frac{(11.48 \text{ psf})16.0'(22.0')}{(1.4)2(16.0)} = 90 \text{ plf} < 190 \text{ plf}$$

Table 23-II-H

Allowable shear of 190 plf is based on 15/32-inch APA-rated sheathing with unblocked edges and 10d nails spaced at 6 inches on center at boundaries and panel edges supported on framing. APA-rated wood structural panels may be either plywood or oriented strand board (OSB).

Check diaphragm deflection:

$$f_{p \text{ floor}} = \frac{9,800}{1,542} = 6.36 \text{ psf}$$

$$v = \frac{(6.36 \text{ psf})16.0 \text{ ft} (22.0 \text{ ft})}{2(16.0 \text{ ft})} = 70 \text{ psf}$$

With nails at 6 inches on center the load per nail is $70(6/12) = 35 \text{ lb/nail} = V_n$

$$L = 22.0 \text{ ft}$$

$$b = 16.0 \text{ ft}$$

$$G = 90,000 \text{ psi}$$

Table 23-2-J, Vol. 3

$$E = 1,700,000 \text{ psi}$$

$$A_{2 \times 4 \text{ chords}} = 5.25 \text{ sq in.} \times 2 = 10.50 \text{ sq in.}$$

$$e_n = 1.2(35/769)^{3.276} = 4.8E-05$$

$$t = 0.319 \text{ in}$$

Table 23-2-H

Using an assumed single chord-splice slip of 0.032-inch at the mid-span of the diaphragm:

$$\Sigma \Delta_c X = (0.032)11.0 \text{ ft} (2) = 0.70 \text{ in.}$$

$$\Delta = \frac{5(70)22.0^3}{8(1.7E6)10.50(16.0)} + \frac{70(22.0)}{4(90,000)0.319} + 0.188(22.0)4.8E-05 + \frac{0.70}{2(16.0)} = 0.04 \text{ in.}$$

Converting to an unblocked diaphragm:

$$\Delta = 0.04(2.5) = \underline{\underline{0.10 \text{ in.}}}$$

7C.

Flexible versus rigid diaphragms.

§1630.6

The maximum diaphragm deflection is 0.15 inches, assuming a simple span for the diaphragm. The average story drift is on the order of 0.62 inches (see Part 4, Tables 1-9 and 1-12 for the computed deflections of the shear walls). For the diaphragms to be considered flexible, the maximum diaphragm deflection will have to be more than two times the average story drift, or 1.25 inches. This would be eight times the computed “simple span” deflections of the diaphragms. As defined by the UBC,

the diaphragms are considered rigid. Since some amount of diaphragm deformation will occur, the analysis is highly complex and beyond the scope of what is normally done for this type of construction.

Diaphragm deflection analysis and testing to date has been performed on level/flat diaphragms. There has not been any testing of sloped (e.g. roof) and complicated diaphragms as found in the typical wood-framed single-family residence. Consequently, some engineers perform their design based on the roof diaphragm being flexible and the floor diaphragm being rigid.

In this procedure, the engineer should exercise good engineering judgment in determining if the higher load of the two methodologies is actually required. In other words, if the load to two walls by rigidity analysis is found to be 5 percent to line A, 95 percent to line B, but by flexible analysis it is found to be 50 percent to line A and 50 percent to line B, the engineer should probably design for the larger of the two loads for the individual walls. Note that the same definition of a flexible diaphragm has been in the UBC since the 1988 edition. However, it generally has not been enforced by building officials for Type V construction. The draft of the IBC 2000 has repeated this same definition in Chapter 23 (wood) definitions. For further discussion, see the Commentary at end of this example.

8.

Does residence meet requirements of conventional construction provisions.

§2320

The UBC has had prescriptive provisions for Type V (light frame) construction for many years. It used to be quite common for building officials to allow developers, architects, building designers, and homeowners to build structures under these provisions *without* any engineering design. The size and style of current single-family residences now being constructed—with vaulted ceilings and large floor openings, tile roofs, and larger window sizes—require an engineering design be done. Due to misuse of the conventional construction requirements, more stringent limitations on the usage of these provisions were placed in the 1994 UBC. Following is an analysis of the construction of the residence proposed in this design example compared with conventional construction requirements and an explanation of why an engineering design is required for both vertical and lateral loads. As engineered design code changes continue to get more restrictive, the “gap” between the double standard (i.e. conventional construction vs. engineered design) continues to widen.

The structure must be checked against the individual requirements of §2320.1. Additionally, because this structure is in Seismic Zone 4, it must also be checked against §2320.5. Results of these checks are shown below.

Roof total loads.

Dead load of roof exceeds the 15 psf limit

§2320.1

Unusually shaped buildings.

Exterior braced wall panels at line D over the garage are horizontally offset from the bracing systems at the floor below and therefore not in one vertical plane. §2320.5.4.1

Floor opening exceeds 12 feet and 50 percent of the least floor dimension at line A. §2320.5.4.4

Floor is not laterally supported by braced wall lines on all edges. §2320.5.4.2

Cantilever column bracing at the garage door does not conform to prescribed methods. §2320.11.3

Stud height exceeds 10'-0" without lateral support at line 1. §2320.11.1

Braced wall lines.

Spacing between braced wall lines 3 and 5 exceeds 25 feet maximum. §2320.5.1

Minimum individual panel length is less than 4'-0" at second floor at line D. §2320.11.3

∴ The residence cannot be designed using the conventional construction provisions of the code.

9.

Design shear wall over garage on line D.

$$V = 5,828 \text{ lb (from Table 1-16)}$$

Converting to allowable stress design for the wall frame:

$$V = 5,828 / 1.4 = 4,159 \text{ lb (refer to Figures 1-11 and 1-15)}$$

Determine h/w aspect ratios for the shear walls:

$$h/w = 9.0 / 3.0 = 3.0$$

Maximum $h/w = 2.0$ for Seismic Zones 3 and 4 Table 23-II-G

Therefore, the wall piers need to be designed to transfer forces around opening. Figure 23-II-1

$$\text{New } h/w \text{ ratio} = 4.0 / 3.0 = 1.33 < 2.0 \quad o.k.$$

9a.**Design of wall frame (perforated shear wall with force transfer around opening).**

It is possible to get the misleading impression from Table 23-II-1 that all a designer needs to do is add some blocking and straps in order to reduce the h/w ratio. This design example has a structure with 9'-0" plate heights, which makes using a wall frame feasible. However, when the plate height is 8'-0", which is a more common plate height, there are chord development and panel nailing capacity problems. Most often, the wall shears above and below the opening will be higher than in the wall piers. This design example analyzes the wall frame and neglects gravity loads, although from a technically correct standpoint, some engineers will argue that vertical loads need to be considered when determining wall shears. The standard practice of neglecting gravity loads when considering wall shears is considered appropriate. Gravity loads are considered for anchorage of the wall in Part 9b.

Using statistics, determine the shears and forces in each free body panel. This is a two-step procedure as follows:

First: Find forces acting on upper left corner of wall frame (Figure 1-15).

Second: Break up wall frame into free-body panel sections and balance forces for each panel starting with upper left corner forces already determined (Figure 1-16).

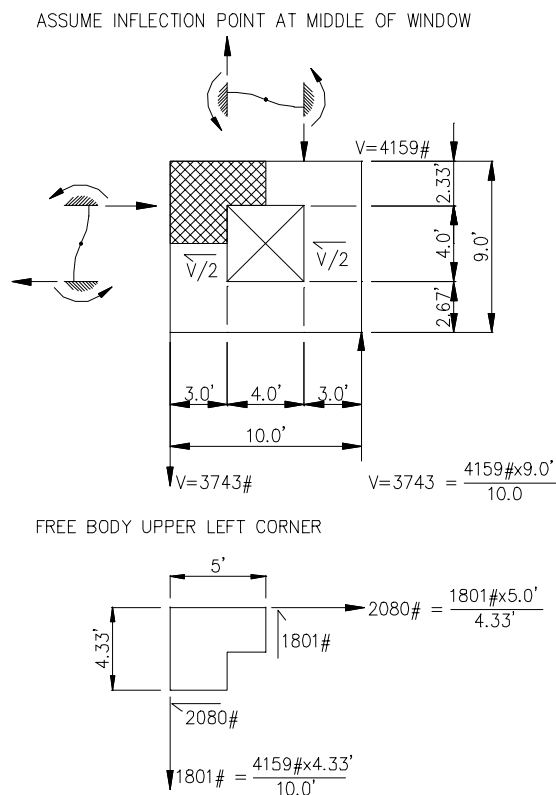


Figure 1-15. Wall frame elevation at line D

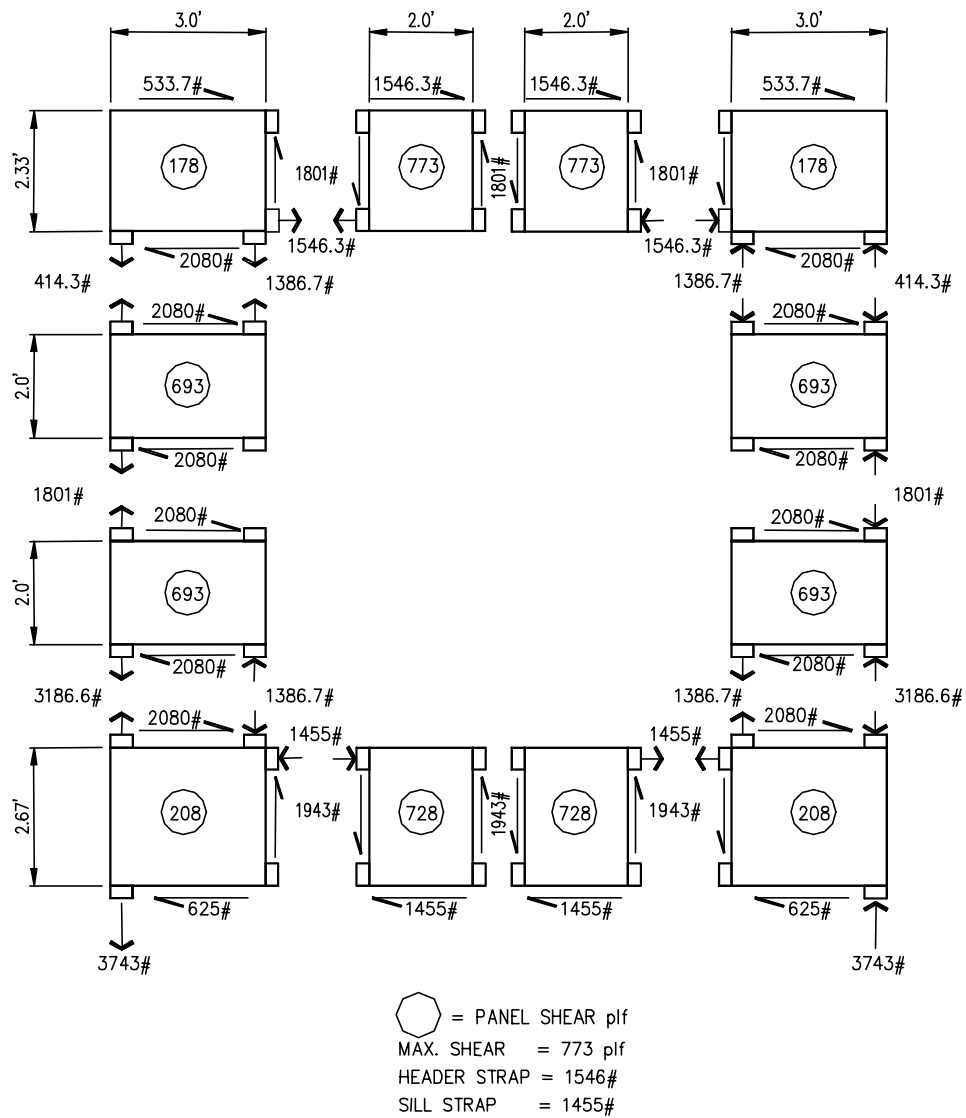


Figure 1-16. Free-body individual panels of wall on line D

Many engineers will arbitrarily add tiedowns at the window jamb members (Figure 1-18). However, with this type of design, the tiedowns at these locations are not necessary, but shear stresses above and below the window may become higher. Adding tiedowns at the window jambs would increase the wall frame performance and help prevent sill plate uplift at the window jambs, which occurs (to some degree) when they are not provided.

9b.**Design horizontal tie straps above and below windows (Figure 1-18).**

Determine the tie force for the horizontal strap (from Figure 1-16). Tie force is maximum at header beam.

$$F_{tie} = 1,546 \text{ lb}$$

Consult ICBO Evaluation Reports for the allowable load capacity of premanufactured straps.

Check penetration depth factor:

C_d : for 10d nail thru-strap and ½" sheathing

$$\text{penetration} = 3.0 - 0.060 - 0.5 = 2.4"$$

$$\text{Required penetration for full value} = 12D = 12 \times 0.148 = 1.8 < 2.4" \quad o.k.$$

Allowable load per 10d common nail with 16 ga metal
side plate = 113 lb

91 NDS Table 12.3F

$$\text{Number of 10d nails required each end} = \frac{1,546 \text{ lb}}{113 \text{ lb/nail} \times 1.33} = 10.3 \text{ nails}$$

(nailing does not control)

Use a continuous 16 gauge x 1¼-inch strap across the opening head and sill to blocking.

$$\text{Allowable strap load is } (1.25)(0.06)(0.6 \times 33)(1.33) = 1,975 \text{ lb} > 1,546 \text{ lb} \quad o.k.$$

9c.**Load combinations using allowable stress design.****§1612.3**

The basic load combinations of §1612.3.1 do not permit stress increases. However, the alternate basic load combinations of §1612.3.2 do permit stress increases.

The Errata to the first printing of the UBC added $0.9D \pm \frac{E}{1.4}$ to the alternate basic load combinations as Eq. (12-16-1).

Since this exact same load combination is listed in the basic load combinations, the UBC is in contradiction and is confusing (to say the least). This design example uses the alternate basic load combinations with the one-third stress increase.

9d.**Check shear panel nailing in wall frame.**

From Figure 1-16:

Maximum panel shear = 773 plf

2-inch edge nailing with sheathing both sides *o.k.*

Table 23-II-I-1

v allowable = $2 \times 870 = 1,740$ plf

Note that sheathing on both sides of this wall does not appear to be required by the code. To eliminate sheathing on one side, a complete design would recheck the force distribution with the reduced wall rigidity. An inspection of Figure 1-13 would indicate that the center of rigidity would shift to the north and hence add more torsional force to the wall.

9e.**Determine anchorage of wall to the supporting GLB.**

The former UBC provision of using 85 percent of the dead loads for consideration of uplift effects has now been replaced with the basic load combinations in UBC §1612.3.1 or §1612.3.2

From Figure 1-17:

$$w_{DL} = 100 \text{ plf (triangle loading from hip roof)}$$

$$P_{DL} = 700 \text{ lb}$$

$$Wall_{DL} = 1,100 \text{ lb}$$

$$E_h = V = 5,828 \text{ lb}$$

$$M_{OT} = 5,828 \text{ lb (9.0 ft)} = 52,452 \text{ ft} \cdot \text{lb (strength level)}$$

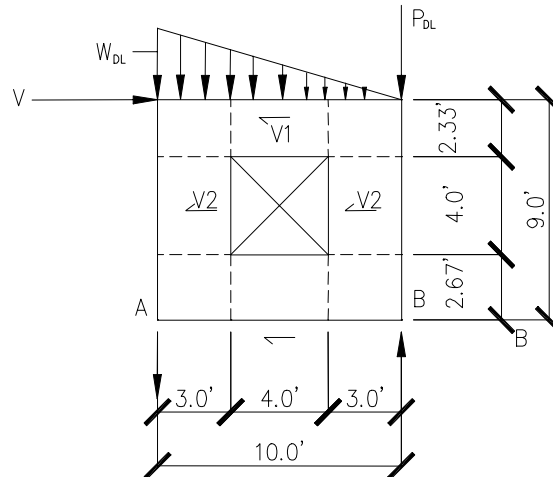


Figure 1-17. Wall frame elevation at D at second floor

Determine anchorage at A:

$$M_R = 100 \text{ plf } (10.0 \text{ ft}/2)(10.0 \times 2/3) + 1,100(10.0 \text{ ft}/2) + 700 \text{ lb } (10.0 \text{ ft}) = 8,833 \text{ ft} \cdot \text{lb}$$

With a 4×6 post at each end wall $L = 10.0 - \frac{3.5 \text{ in.}}{12} = 9.7 \text{ ft}$

$$\text{The critical loading condition is: } 0.9D \pm \frac{E}{1.4} \quad (12-10)$$

$$\text{Uplift at A} = \frac{(52,452/1.4) - (8,833 \times 0.9)}{9.7 \text{ ft}} = 3,043 \text{ lb}$$

Determine anchorage at B:

$$M_R = 100 \text{ plf} (10.0 \text{ ft}/2)(10.0/3) + 1,100 (10.0 \text{ ft}/2) + 700 \text{ lb} (10.0 \text{ ft}) = 14,167 \text{ ft} - \text{lb}$$

$$\text{Uplift at B} = \frac{(52,452/1.4) - (14,167 \times 0.9)}{9.7\text{ft}} = 2,548\text{lb}$$

Elements supporting discontinuous systems

§1630.8.2

Since location A does not continue to the foundation, check special seismic load combination for elements supporting discontinuous systems.

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

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

where:

$$f_1 = 0.0 \text{ for roof live loads (non-snow)} \quad \S 1612.4$$

$$f_1 = 0.5 \text{ for live loads} \quad \S 1612.4$$

$$E_m = \Omega_o E_h \quad (30-2)$$

Determine the seismic force overstrength factor Ω_o §1630.3.1

$$\Omega_o = 2.8 \text{ for wood structural panel wall} \quad \text{Table 16-N}$$

$$\Omega_o = 2.0 \text{ for cantilevered column building systems} \quad \text{Table 16-N}$$

For east-west axis of structure $R = 2.2$ for cantilevered building systems
Therefore, $\Omega_o = 2.0$

Determine anchorage force at A for special seismic load combination:

$$E_m = \Omega_o E_h = 2.0(5,828 \text{ lb}) = 11,656 \text{ lb}$$

$$M_{OT} = 11,656 \text{ lb}(9.0 \text{ ft}) = 104,904 \text{ ft} \cdot \text{lb}$$

$$\text{Therefore, uplift} = \frac{(104,904/1.4) - (8,833 \times 0.9)}{(10.0 \text{ ft} - 0.3 \text{ ft})} = 6,905 \text{ lb}$$

Consult ICBO Evaluation Reports for the allowable load capacity of premanufactured straps.

Allowable load per 10d nail common with 14 ga metal side plate = 115 lb

91 NDS Table 12.3F

From Part 9b, with 3-inch nails penetration factor $C_d = 1.0$.

For allowable stress design, the allowable stress increase factor is 1.7 for steel. §1630.8.2.1

$$\text{Number of 10d common nails required} = \frac{6,905 \text{ lb}}{115 \text{ lb/nail}(1.7)(1.33)} = 26.5 \text{ nails}$$

Use a continuous 14 gauge x 3-inch strap bent around GLB.

Note that §1630.8.2.1 allows the combination of allowable stress increase of 1.7 with the duration of load increase in Chapter 23.

Note that the adequacy of the GLB to resist the overturning of the wall must be checked using the special seismic load combinations. As permitted in §1612.4 and §1630.8.2.1, an allowable stress increase of 1.7 can be used in addition to the duration of load increase of 1.33 for C_D .

Also, the boundary post at the wall corner must be checked for orthogonal effects with shear wall 5 (and on other locations in the structure with common corners). §1633.1

10.

Diaphragm shears at the low roof over garage (Figure 1-20).

From Table 16-M, this has plan irregularity type 4.

The diaphragm between lateral resisting elements C and E is required to transfer the design seismic force from shear wall D due to the offset between D and E. UBC §1633.2.9 requires the diaphragm force used in UBC Equation (33-1) to be used. UBC §1630.8.2 references special seismic load combinations of §1612.4 and does not allow the one-third increase permitted under §1612.3.2

From Part 7 in this design example:

$$f_p \text{ floor} = 11.48 \text{ psf}$$

From Table 16-P: Ω_o for cantilever column type structures is 2.0.

$$f_p \Omega_o = 11.48 \times 2.0 = 22.96 \text{ psf}$$

For simplification of analysis, assume the diaphragm over the garage is a simple span between lateral resisting elements at lines C and E.

$$\text{Load from wall D above} = 5,828 \text{ lb}$$

$$V_E = 22.96(28.0 \text{ ft})(22.0/2) + 5,828 \text{ lb}(15.0 \text{ ft}/22.0 \text{ ft}) = 11,045 \text{ lb}$$

$$v_E = 11,045 \text{ lb}/1.4(28.0) = 281 \text{ plf} > 215 \text{ plf (for unblocked)} \quad n.g. \quad \text{Table 23-II-H}$$

Therefore, panel edges need to be blocked. Since the allowable shear values in Table 23-II-H already include a increase for short-term loading, (C_D), the duration of load increase (§1612.3.1 and §1612.3.2) cannot be used concurrently with the 1.7 increase, as prohibited in §2316.2, Item 5.

From Table 23-II-H, the allowable diaphragm shear for 19/32-inch APA sheathing, with 10d common wire nails spaced at 6-inch centers, with blocked edges, is 320 plf.

$$320 \text{ plf} > 281 \text{ plf} \quad o.k.$$

∴ Use 10d @6 inches o.c. with blocked edges on 19/32-inch sheathing.

11.

Detail the wall frame over the GLB.

Wall frame details must be shown on the drawings. Depending on the variations, when multiple wall frames are on a project, it is necessary at times to have individual details for each condition. While the detail shown in Figure 1-18 is somewhat generic, it should be noted that a separate anchorage detail (keynote 10) may be necessary where the end of the GLB is connected to the supporting post.

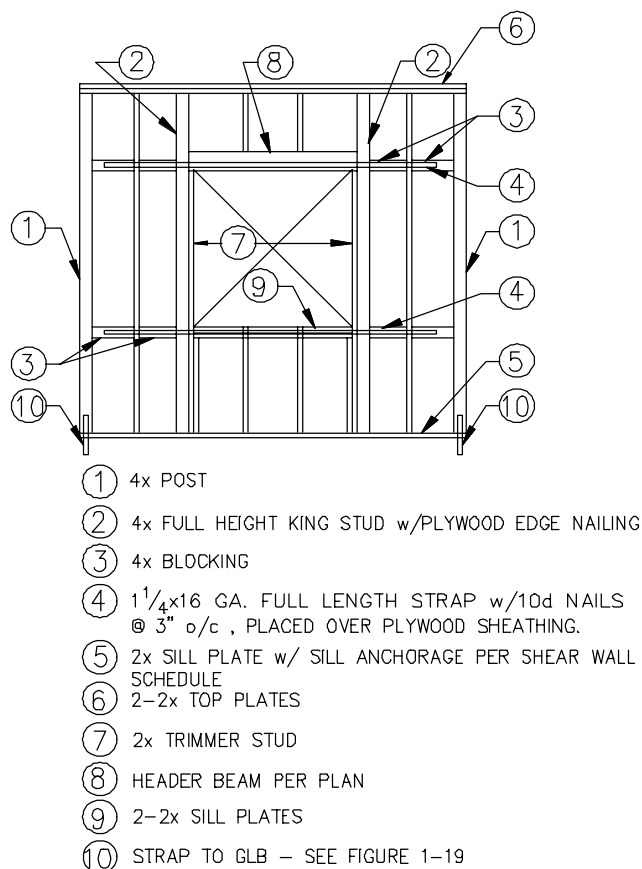


Figure 1-18. Details of wall frame on line D at second floor

12.**Detail the anchorage of wall frame to the GLB.**

Cross-grain shrinkage of the GLB may be a problem when using a connection of the type shown in Figure 1-19. Also, nails above the neutral axis of the GLB should be left out from the design to avoid cross-grain tension. In other words, only the nails below the neutral axis are considered effective for uplift forces. To avoid confusion in the field, all nail holes are to be filled. It should be noted that a separate anchorage detail may be necessary where the end of the GLB is connected to the supporting post (intersection of grids D and 5).

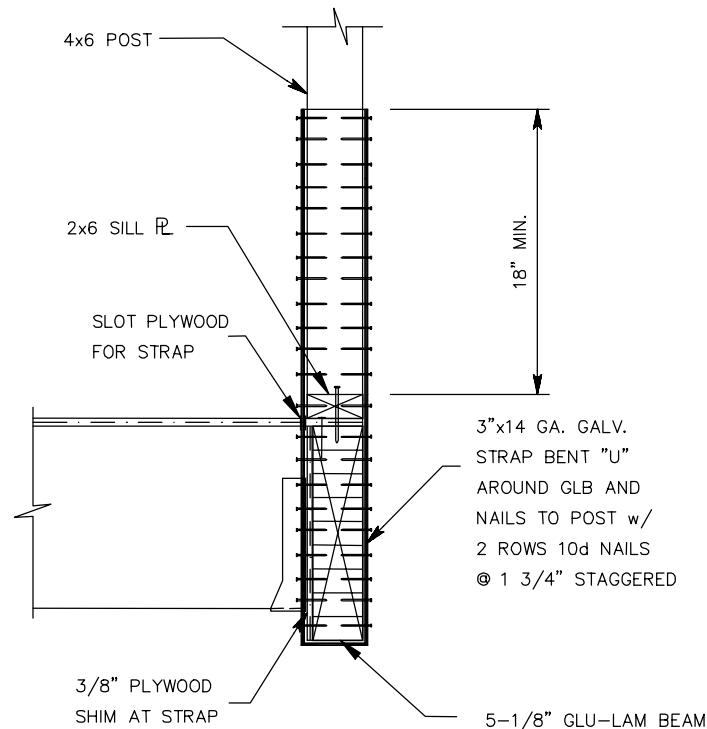


Figure 1-19. Detail of anchorage at point A (see also Figure 1-18)

13.**Detail the continuous load path at the low roof above the garage doors.**

The low roof above the garage is an important part of the continuous load path. Historically, this type of detail has been mis-detailed and mis-constructed. This detail has two load paths: the loads from the roof can either go through the pitched roof, or down the wall to the GLB and across the horizontal diaphragm to the exterior wall.

Figure 1-20 shows one way that the shear transfer can be made. Also note that the chord/drag tie of the top plates will be interrupted by the GLB-to-post connection and will require detailing at grids D3 and D5.

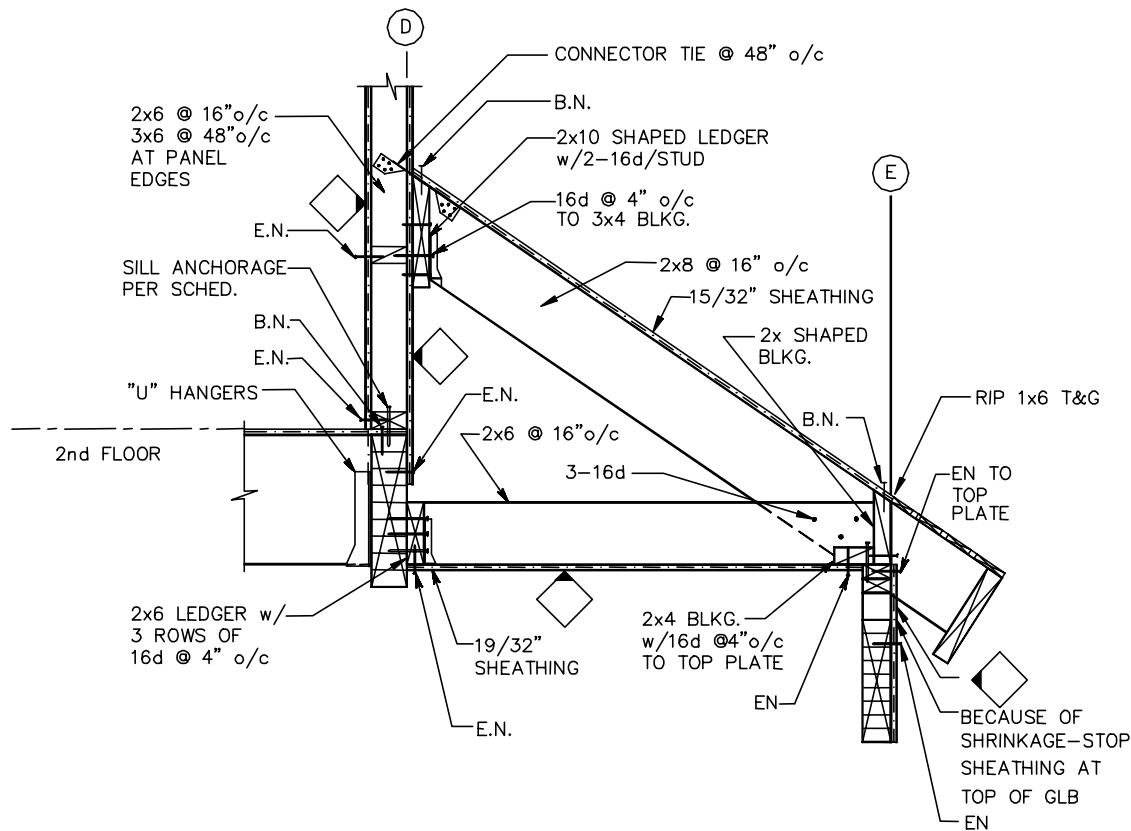


Figure 1-20. Detail of load path for low roof over garage

Commentary

Following are some issues and topics related to the seismic design of wood frame residences that can be used to improve design practices and/or understanding of important aspects of design.

“Calc and sketch” philosophy.

In wood frame construction, particularly for single-family residences, it has been a common design practice to have an engineer provide only calculations and sketches for the architect to include on the architectural drawings.” This is done to provide a cost savings to the owner. This approach has some significant problems based on reviews of how residential framing is actually being constructed, the “calc and sketch only” service is a practice which should be discontinued, with a few exceptions.

Architects and building officials need to be encouraged to adopt the following standards:

1. Any new building (or remodel requiring the existing building to be brought into conformance with the current building code) that cannot be clearly shown to conform with building code conventional construction framing requirements should require submittal of structural drawings and calculations signed for by a licensed civil or structural engineer.
2. Structural framing plans and details should be separate from the architectural drawings.

Most new wood residential building designs are complex and beyond the scope and intent of the prescriptive conventional construction requirements of the UBC. Misuse of these conventional requirements has led to structures with incomplete lateral force systems, resulting in poor performance in earthquakes. Since the engineer generally is not asked to review the architect’s final drawings, the use of calculations and sketches lends itself to poorly coordinated drawings and missing structural information. The common practice of referring to details on architectural drawings as “similar” leads to further confusion as to the design intent. The structural observation requirements of the code, when enforced (many jurisdiction do not require structural observation for single-family residences), are even less effective, since the architect did not design the structural system and often can not identify what is missing or incorrect.

Rigid versus flexible diaphragm.

This design example illustrates seismic design using both flexible and rigid diaphragms. It also illustrates that most one- and two-family dwellings have rigid

diaphragms as defined by code. This being the case, a design based on flexible diaphragm assumption would not be required if the design is based on the rigid diaphragm assumption. Using the common approach of basing wall rigidities on deflections of shear walls and other vertical elements, the engineer first needs to know or assume how the shear walls will be constructed (e.g., nail size and spacing). Without performing a preliminary analysis, the procedure of just doing a design based on rigid diaphragms may be subject to a trial and error process. One method (as used in this design example) to avoid this process is to first perform an analysis based on flexible diaphragms, then use the construction required from the flexible diaphragms for determining the wall rigidities.

Part 2 of this design example uses flexible diaphragms to determine shear wall construction. Parts 3, 4, and 5 of this design example use rigid diaphragms per UBC requirements. The shear wall deflections used in this design example use UBC equations. This needs to be viewed as one possible approach that is substantiated by the code. However, other approaches can also be used. Two of these are given below:

1. The rigidities of the shear walls can be based on the length of the wall times the allowable shear capacity. This method can be appropriate provided the tiedown assembly displacements are kept to a minimum. This may involve using specific types of tiedown devices that limit displacements to less than 1/8".
2. Shear wall rigidities can be based on graphs of the four-term shear wall code deflection equation (see Part 3b). As shown in Figure 1-21, a chart of these is included in this section and is also considered appropriate in determining wall rigidities.

Tiedown location.

When designing shear walls, the engineer needs to consider where the tiedown posts will actually be located. The tiedown posts occur where shear walls stack from floor to floor. The lower level wall requires tiedown devices on each side of the tiedown post. However, the upper shear wall only requires a tiedown device on one side of the tiedown post. Since the posts must align between story levels, the upper level tiedown post will need to be offset inward in order to line up with the post below.

Based on actual tiedown post locations, the upper level shear wall design may have to be rechecked once the lower level shear wall design is complete. The use of tiedown devices on each side of the post will improve the shear wall performance, since eccentricity in the connection, as occurs when there is only a single-sided tiedown, is avoided. Double-sided tiedowns are generally preferred over single-sided.

Design comments.

This design example illustrates a detailed analysis for some of the important seismic requirements of the 1997 UBC. To complete this design, the engineer will have to check all the major structural elements along the various lateral load paths of the residence, including the foundations. The seismic calculations and details for this example residence are approximately 50 percent complete. Normal engineering design of this type of structure may omit many of the calculations shown in this example and rely on good engineering judgment. This design example illustrates a very comprehensive approach to the engineering calculations. This design example fills a void in the available engineering literature on the subject—many engineers have stated that there simply are not sufficient reference documents available on this subject.

In the so called “big one,” it is expected that actual peak earthquake forces may be 2 to 3 times greater than the equivalent static forces required by the UBC and used in this example. The use of good detailing practices with ductile elements to absorb energy, clear construction documents with adequate detailing, structural site observation, and special inspection are considered every bit as important as a comprehensive set of structural calculations.

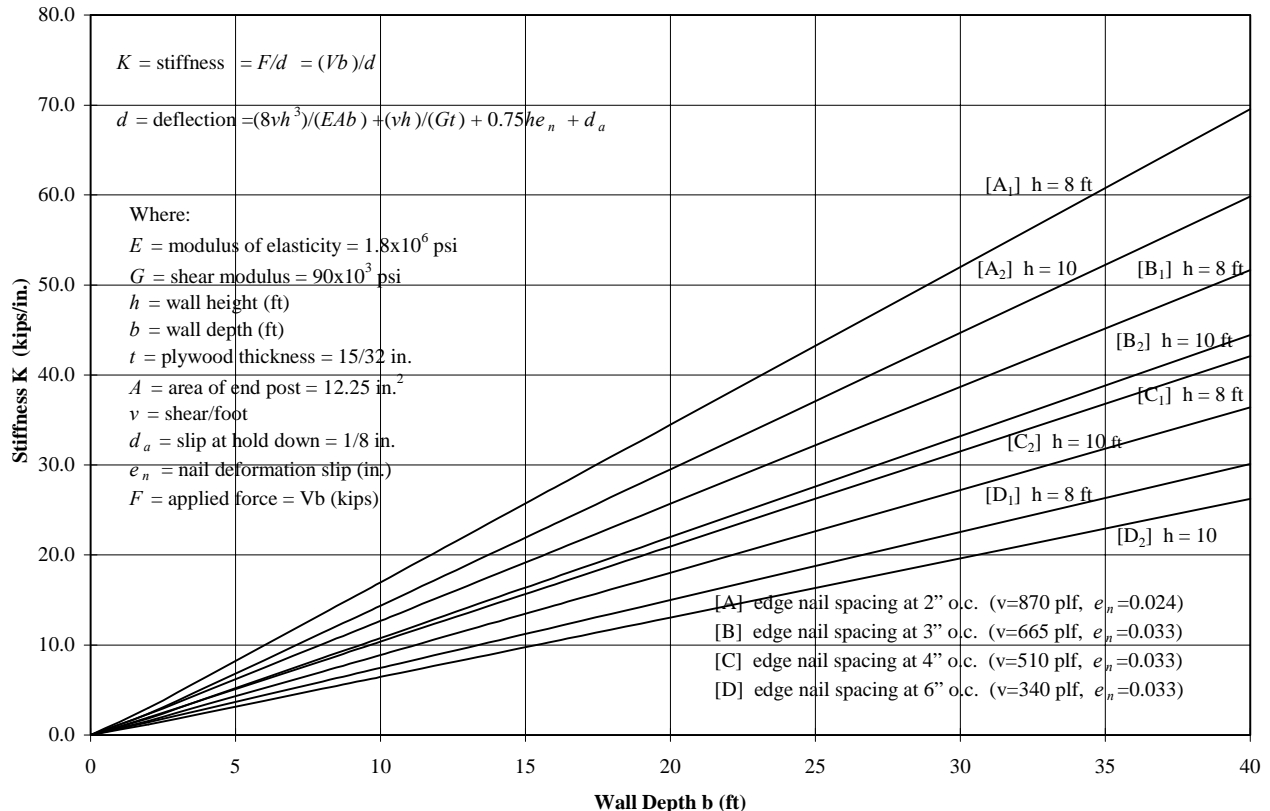


Figure 1-21. Stiffness of one-story 1/2-inch Structural-I plywood shear walls

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Design Example 2

Wood Light Frame Three-Story Structure

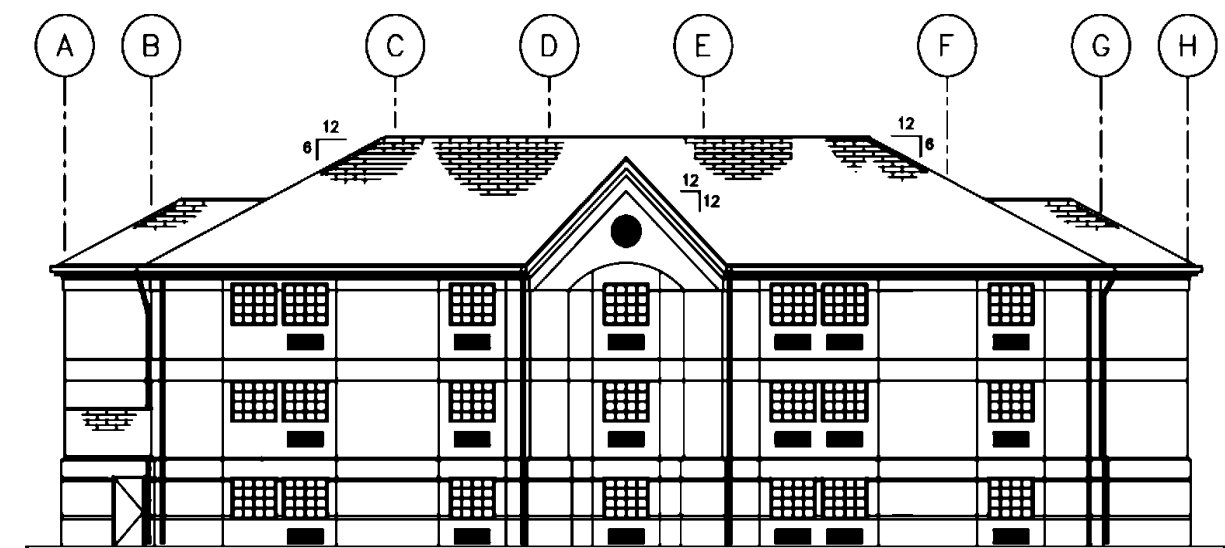


Figure 2-1. Wood light frame three-story structure elevation

Foreword

After careful consideration and extensive discussion, SEAOC is recommending that large wood frame structures, such as the three-story building in this design example, be designed for seismic forces considering *both* rigid and flexible diaphragm assumptions. This method represents a significant change from current practice. At present, California practice has almost exclusively used the flexible diaphragm assumption for determining distribution of story shears to shear walls. There are two principal reasons for considering both rigid and flexible diaphragms.

First, since adoption of the 1988 UBC, there has been a *definition* of diaphragm flexibility in the code (§1630.6 of the 1997 UBC). Arguably, when introduced in 1988, this definition may not have been intended to apply to wood framed diaphragms. After considerable discussion and re-evaluation, it is now the joint opinion of the SEAOC Code and Seismology Committees that this definition should be considered in wood framed diaphragms. The application of this definition in wood construction often requires the use of the rigid diaphragm assumption, and subsequent calculation of shear wall rigidities, for distribution of story shears to shear walls. In fact, this definition results in many, if not most, diaphragms in wood frame construction being considered rigid.

Many engineers feel that exclusive use of the flexible diaphragm assumption results in underestimation of forces on some shear walls. For example, a rigid

diaphragm analysis is judged more appropriate when the shear walls are more flexible compared to the diaphragm, particularly where one or more lines of shear walls (or other vertical resisting elements) are more flexible than the others are.

Second, in some instances, the use of flexible diaphragm assumptions can actually force the engineer to provide a more favorable lateral force resisting system than would occur by only using rigid diaphragm assumptions. Flexible diaphragm assumptions encourage the placement of shear walls around the perimeter of the floor and roof area, therefore minimizing the need to have wood diaphragms to resist torsional forces.

In this design example, the floor diaphragms are constructed using screw shank nails, sheathing is glued to the framing members (to reduce floor squeaks), and lightweight concrete fill is placed over the floor sheathing (for sound insulation). Additionally, gyp board is applied to the framing underside for ceiling finish. These materials in combination provide significantly stiffer diaphragms than those represented by the diaphragm deflection equation of UBC standard 23-2.

For the part of the analysis that assumes a rigid diaphragm, the engineer must also select a method to estimate shear wall rigidities (and rigidities of other vertical resisting elements). This also requires use of judgment because at the present time there is no consensus method for estimating rigidities. In the commentary of Design Example 1, several alternatives are discussed.

Prior to starting design of a wood light frame structure, users of this document should check with the local jurisdiction regarding both the level of analysis required and acceptable methodologies.

Overview

This design example illustrates the seismic design of a three-story 30-unit hotel structure. The light frame structure, shown in Figures 2-1, 2-2, 2-3, and 2-4, has wood structural panel shear walls, and roof and floor diaphragms. The roofs have composite shingles and are framed with plated trusses. The floors have a 1½-inch lightweight concrete topping framed with engineered I joists. The primary tiedowns for the shear walls use a continuous tiedown system.

This structure cannot be built using conventional construction methods for reasons shown in Part 6 of this design example. The following sections illustrate a detailed analysis for some of the important seismic requirements of the 1997 UBC. This design example is not a complete building design, and many aspects of a complete design, including wind design (see UBC §606), are not included. Only selected items of the seismic design are illustrated.

In general, the UBC recognizes only two diaphragm categories: flexible and rigid. However, the diaphragms in this design example are considered to be semi-rigid.

Hence, the analysis will use the envelope method, which considers the worst loading condition from the flexible and rigid diaphragm analyses for each vertical shear resisting element. It should be noted that the envelope method, although not explicitly required by code, is deemed necessary and good engineering practice for this design example.

Initially, the shear wall nailing and tiedown requirements are determined using the flexible diaphragm assumption. Secondly, use these shear wall forces to determine shear wall rigidities for the rigid diaphragm analysis. Finally, further iterations may be required with significant stiffness redistributions.

The method of determining shear wall rigidities used in this design example is by far more rigorous than normal practice but is *not* the only method available to determine shear wall rigidities. The commentary following Design Example 1 illustrates two other simplified approaches that would also be appropriate for this design example.

Outline

This example will illustrate the following parts of the design process:

- 1. Design base shear and vertical distributions of seismic forces.**
- 2. Lateral forces on the shear walls and required nailing assuming flexible diaphragms.**
- 3. Rigidities of shear walls.**
- 4. Distribution of lateral forces to the shear walls.**
- 5. Reliability/redundancy factor ρ .**
- 6. Does structure meet requirements of conventional construction provisions?**
- 7. Diaphragm deflections to determine if the diaphragm is flexible or rigid.**
- 8. Tiedown forces for shear wall on line C.**
- 9. Tiedown connection at the third floor for the shear wall on line C.**
- 10. Tiedown connection at the second floor for the shear wall on line C.**
- 11. Anchor bolt spacing and tiedown anchor embedment for shear wall on line C.**

12. Detail of tiedown connection at the third floor for shear wall on line C (Figure 2-9).
13. Detail of tiedown connection at the second floor for shear wall at line C. (Figure 2-10).
14. Detail of wall intersection at exterior shear walls (Figure 2-11).
15. Detail of tiedown connection at foundation (Figure 2-12).
16. Detail of shear transfer at interior shear wall at roof (Figure 2-13).
17. Detail of shear transfer at interior shear walls at floors (Figure 2-14).
18. Detail of shear transfer at interior shear walls at foundation (Figure 2-15).
19. Detail of sill plate at foundation edge (Figure 2-16).
20. Detail of shear transfer at exterior wall at roof (Figure 2-17).
21. Detail of shear transfer at exterior wall at floor (Figure 2-18).

Given Information

Roof weights (slope 6:12):

Roofing	3.5 psf
½" sheathing	1.5
Trusses	3.5
Insulation	1.5
Miscellaneous	0.7
Gyp ceiling	<u>2.8</u>
DL (along slope)	13.5 psf

Floor weights:

Flooring	1.0 psf
Lt. wt. concrete	14.0
5/8" sheathing	1.8
Floor framing	5.0
Miscellaneous	0.4
Gyp ceiling	<u>2.8</u>
	25.0 psf

DL (horiz. proj.) = 13.5 (13.41/12) = 15.1 psf

Stair landings do not have lightweight concrete fill

Area of floor plan is 5,288 sq ft

Weights of respective diaphragm levels, including tributary exterior and interior walls:

W_{roof}	=	135,000 lb
$W_{3rd\ floor}$	=	2300,000 lb
$W_{2nd\ floor}$	=	<u>230,000 lb</u>
W	=	595,000 lb

Weights of diaphragms are typically determined by taking one-half height of walls at the third floor to the roof and (with equal story heights) full height of walls for the third and second floor diaphragms.

Framing lumber is Douglas Fir-Larch (DF-L) grade stamped No. 1 S-Dry.

(Note: The designer must recognize the increased potential for shrinkage problems when green lumber is used. The shrinkage of lumber can effect the architectural and mechanical systems as well as the structural system. The potential for wood shrinkage problems proportionally increases with the number of stories in the structure.)

Foundation sill plates are pressure-treated Hem-Fir.

APA-rated wood structural panels for shear walls will be 15/32-inch-thick Structural I, 32/16 panel index span rating, 5-ply with Exposure I glue is specified. However, 4-ply is also acceptable. Three-ply 15/32-inch sheathing has lower allowable shears and the inner ply voids can cause nailing problems.

The roof is 15/32-inch-thick APA-rated sheathing (equivalent to C-D in Table 23-II-4), 32/16 span rating with Exposure I glue.

The floor is 19/32-inch-thick APA-rated Sturd-I-Floor 24" o/c rating (or APA-rated sheathing, 48/24 span rating) with Exposure I glue.

Common wire nails are used for diaphragms, shear walls, and straps.

Sinker nails will be used for design of the shear wall sill plate nailing at the second and third floor. (Note: Many nailing guns use the smaller diameter box and sinker nails instead of common nails. Closer nail spacing may be required if the smaller diameter nails are used).

Seismic and site data:

(Zone 4)

$I = 1.0$ (standard occupancy)

Seismic source Type = B

Distance to seismic source = 12 km

Soil profile type = S_C

Table 16-I

Table 16-K

S_C has been determined by geotechnical investigation. Without a geotechnical investigation, S_D can be used as a default value.

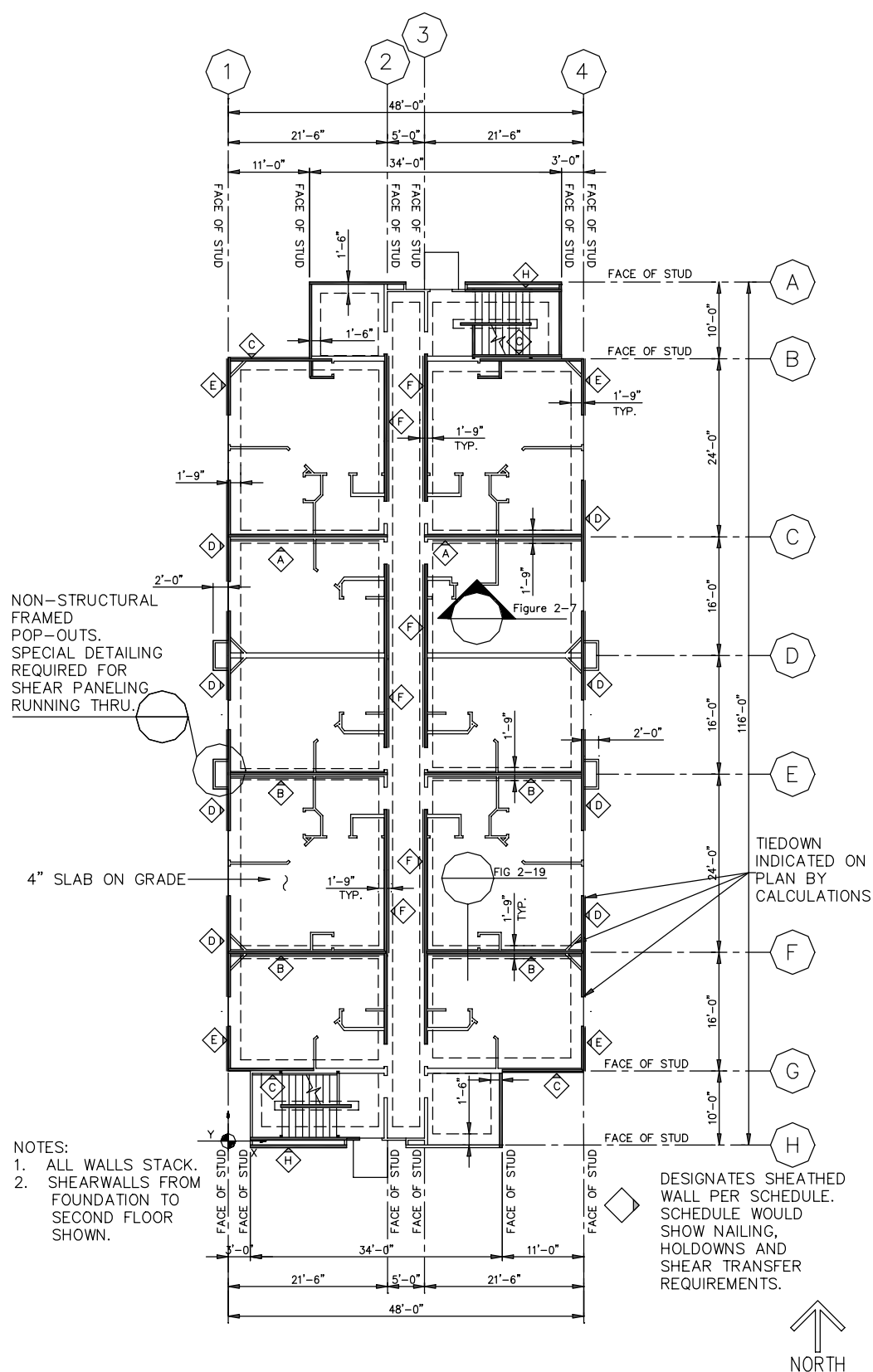


Figure 2-2. Foundation plan (ground floor)

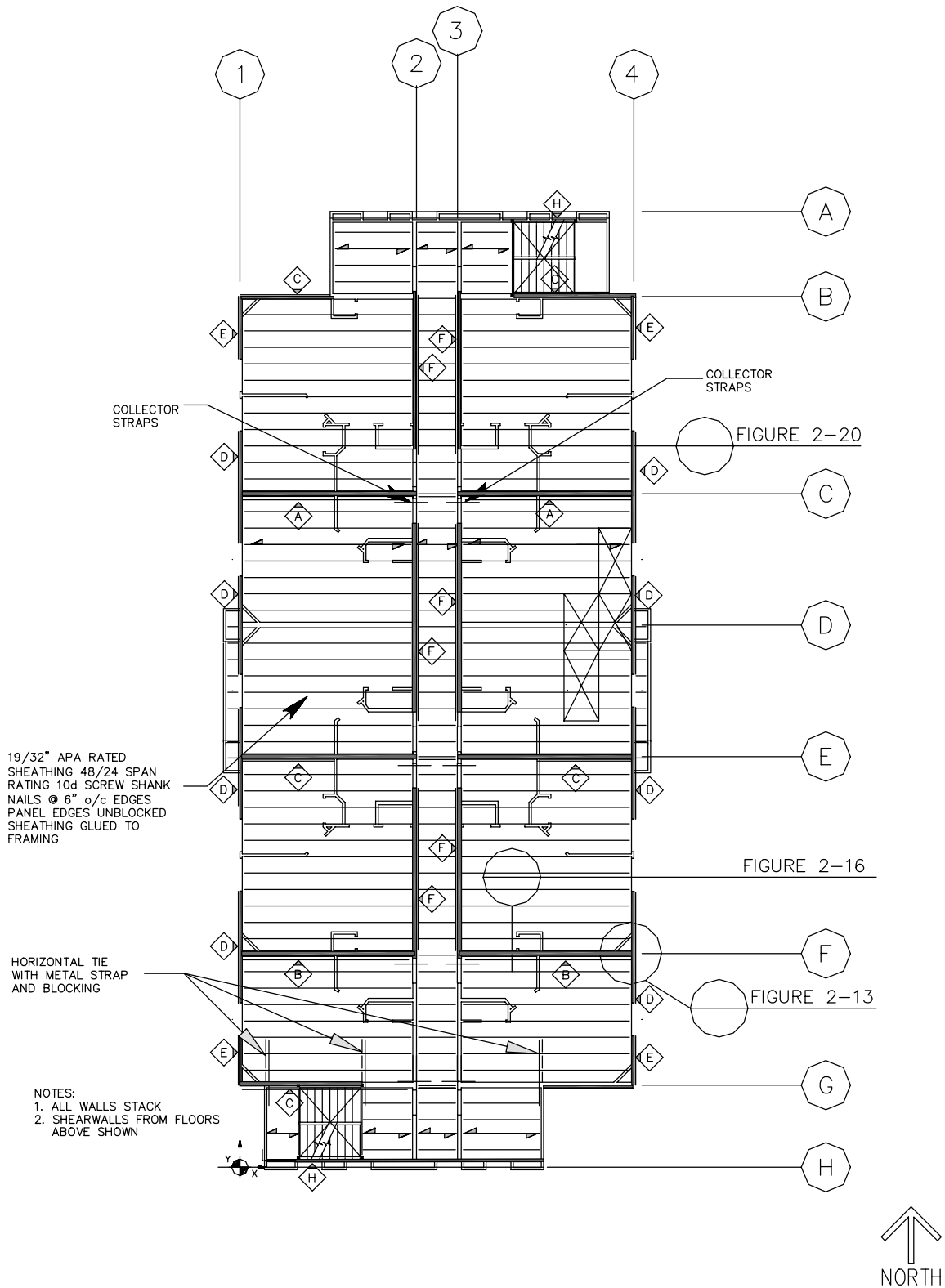


Figure 2-3. Floor framing plan (second and third floors)

Note: Shear walls on lines 2 and 3 do not extend from the third floor to the roof.

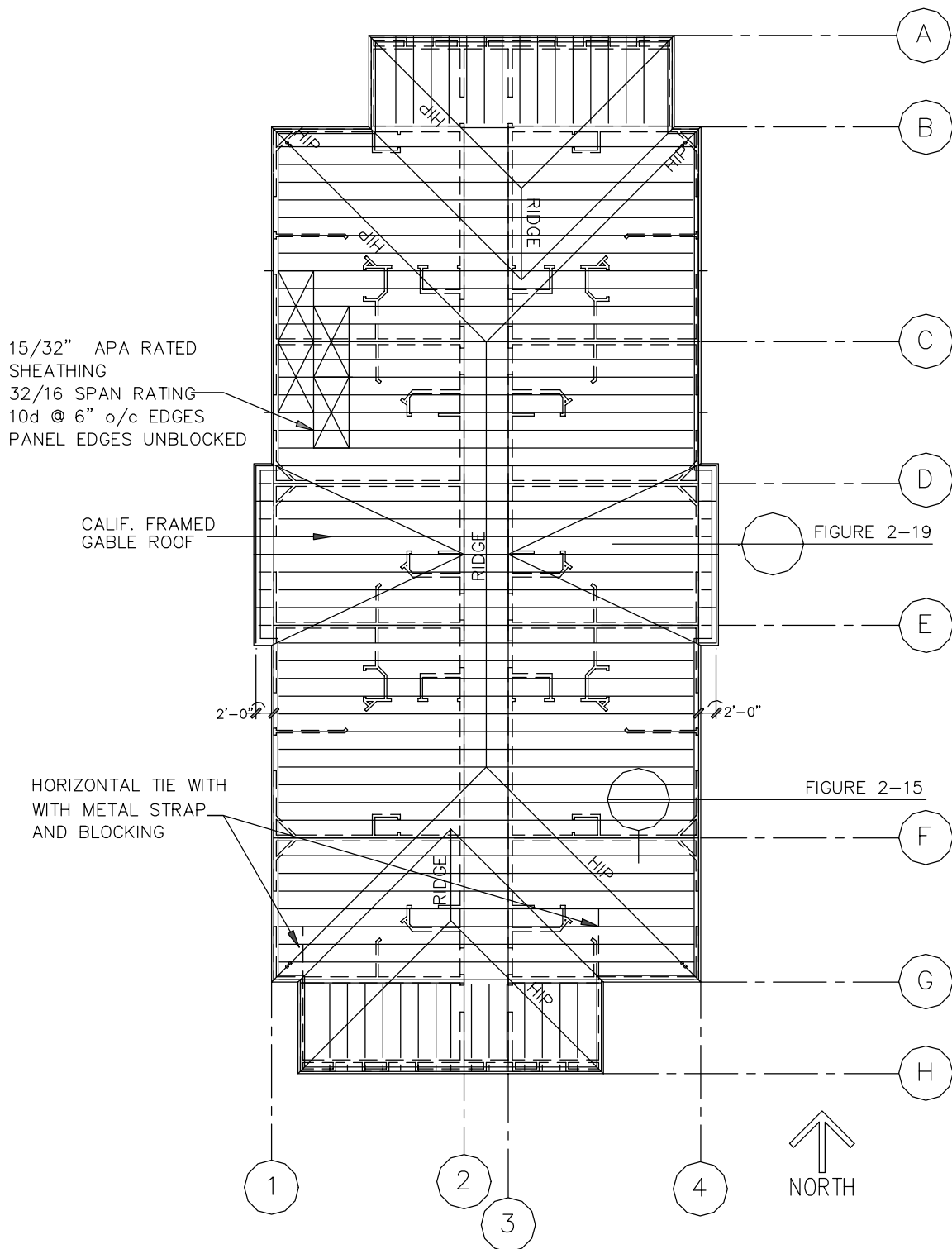


Figure 2-4. Roof framing plan

Factors That Influence Design

Before starting the example, four important related aspects of the design will be discussed. These are the effect of moisture content on lumber, the use of pre-manufactured roof trusses, proper detailing of shear walls at building pop-outs, and effects of box nails on wood structural panel shear walls.

Moisture content in lumber connections.

This design example is based on dry lumber. Project specifications typically call for lumber to be grade-stamped S-Dry (Surfaced Dry). Dry lumber has a moisture content (MC) less than or equal to 19 percent. Partially seasoned or green lumber grade stamped S-GRN (surfaced green) has a MC between 19 percent and 30 percent. Wet lumber has a MC greater than 30 percent. Construction of structures using lumber with moisture contents greater than 19 percent can produce shrinkage problems. Note that UBC §2304.7 requires consideration of lumber shrinkage. Also, many engineers and building officials are not aware of the reduction requirements or wet service factors related to installation of nails, screws, and bolts (fasteners) into lumber with moisture content greater than 19 percent. For fasteners installed in lumber with moisture content greater than 19 percent, the wet service factor $C_M = 0.75$ for nails and $C_M = 0.67$ for bolts, lags and screws (91 NDS Table 7.3.3) are used.

For construction using lumber of MC greater than 19 percent, there is a 25 percent to 33 percent reduction in the strength of connections, diaphragms, and shear walls that is permanent. The engineer needs to exercise good engineering judgment in determining whether it is prudent to base the structural design on dry or green lumber. Other areas of concern are the geographical area and the time of year the structure is built. It is possible for green lumber (or dry lumber that has been exposed to rain) to dry out to a moisture content below 19 percent on the construction site. For 2× framing, this generally takes about 2 to 3 weeks of exposure to dry air, 4× lumber takes even longer. Drying occurs when the surfaces are exposed to air on all sides, not while stacked on pallets (unless shimmed with stickers). Moisture content can easily be verified by a hand held “moisture meter.”

Use of pre-manufactured roof trusses to transfer lateral forces.

The structural design in this design example uses the pre-manufactured wood roof trusses. Under seismic forces, these must transfer the lateral forces from the roof diaphragm to the tops of the interior shear walls. To accomplish this, special considerations must be made in the design and detailed on the plans. In particular, any trusses that are to be used as collectors or lateral drag struts should be clearly indicated on the structural framing plan. The magnitude of the forces, the means by which the forces are applied to the trusses and transferred from the trusses to the shear walls must be shown on the plans. In addition, if the roof sheathing at the hip

ends breaks above the joint between the end jack trusses and the supporting girder truss, the lateral forces to be resisted by the end jacks should be specified so that an appropriate connection can be provided to resist these forces. The drawings also must specify the load combinations and whether or not a stress increase is permitted. If ridge vents are being used, special detailing for shear transfers must be included because normal diaphragm continuity is disrupted.

Proper detailing of shear walls at building pop-outs.

The structure for this design example has doubled-framed walls for party walls and exterior “planted-on” box columns (pop-outs). The designer should not consider these walls as shear walls unless special detailing and analysis is provided to substantiate that there is a viable lateral force path to that wall and the wall is adequately braced.

Effects of box nails on wood structural panel shear walls.

This design example uses common nails for fastening wood structural panels. Based on cyclic testing of shear walls and performance in past earthquakes, the use of common nails is preferred. UBC Table 23-II-I-1 lists allowable shears for wood structural panel shear walls for “common or galvanized box nails.” Footnote number five of Table 23-II-I-1, states that the galvanized nails shall be “hot-dipped or tumbled” (these nails are not gun nails). Most contractors use gun nails for diaphragm and shear wall installations. The UBC does not have a table for allowable shears for wood structural panel shear walls or diaphragms using box nails.

Box nails have a smaller diameter shank and a smaller head size. Using 10d box nails would result in a 19 percent reduction in allowable load for diaphragms and shear walls as compared to 10d common nails. Using 8d box nails would result in a 22 percent reduction in allowable load for diaphragms and shear walls as compared to 8d common nails. This is based on comparing allowable shear values listed in Tables 12.3A and 12.3B in the 1997 NDS for one-half-inch side member thickness (t_s) and Douglas Fir-Larch framing. In addition to the reduction of the shear wall and diaphragm capacities, when box nails are used, the walls will also drift more than when common nails are used.

A contributor to the problem is that when contractors buy large quantities of nails (for nail guns), the word “box” or “common” does not appear on the carton label. Nail length and diameters are the most common listing on the labels. This is why it is *extremely* important to list the required nail lengths and diameters on the structural drawings for all diaphragms and shear walls. Another problem is that contractors prefer box nails because their use reduces splitting, eases driving, and they cost less.

Just to illustrate a point, if an engineer designs for “dry” lumber (as discussed above) and “common” nails, and subsequently “green” lumber and “box” nails are used in the construction, the result is a compounding of the reductions. For

example, for 10d nails installed into green lumber, the reduction would be 0.81 times 0.75 or a 40 percent reduction in capacity.

Calculations and Discussion

Code Reference

1.

Design base shear and vertical distributions of seismic forces.

§1630.2.2

1a.

Design base shear.

Determine period using Method A (see Figure 2-5 for section through structure):

$$T = C_t (h_n)^{3/4} = .020(33.63)^{3/4} = 0.28 \text{ sec} \quad (30-8)$$

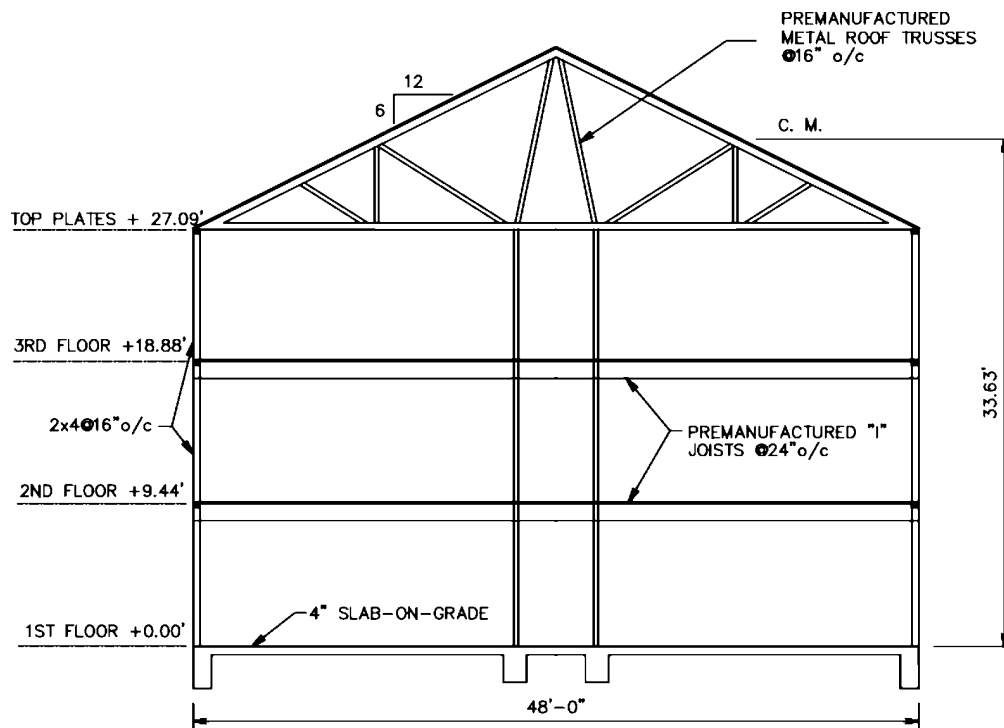


Figure 2-5. Typical cross-section through building

With seismic source type B and distance to source = 12 km

$$N_a = 1.0 \quad \text{Table 16-S}$$

$$N_v = 1.0 \quad \text{Table 16-T}$$

For soil profile type S_C and $Z = 0.4$

$$C_a = 0.40 N_a = 0.40(1.0) = 0.40 \quad \text{Table 16-Q}$$

$$C_v = 0.56 N_v = 0.56(1.0) = 0.56 \quad \text{Table 16-R}$$

Because the stud walls are both wood structural panel shear walls and bearing walls

Table 16-N

$$R = 5.5$$

Design base shear is:

$$V = \frac{C_v I}{RT} W = \frac{0.56(1.0)}{5.5(0.28)} W = 0.364W \quad (30-4)$$

Note: design base shear is now on a strength design basis.

but need not exceed:

$$V = \frac{2.5 C_a I}{R} W = \frac{2.5(0.40)(1.0)}{5.5} W = \underline{\underline{0.182W}} \quad (30-5)$$

$$V = 0.11 C_a I W = 0.11(0.40)(1.0)W = 0.044W < 0.182W$$

Check Equation 30-7:

$$V = \frac{0.8 Z N_v I}{R} W = \frac{0.8 \times 0.4 \times 1.0 \times 1.0}{5.5} W$$

$$V = 0.058W < 0.182W$$

All of the tables in the UBC for wood diaphragms and shear walls are based on allowable loads.

It is desirable to use the strength level forces throughout the design of the structure for two reasons:

1. Errors in calculations can occur and which load is being used—strength design or allowable stress design—may be confused. This design example will use the following format:

$$\begin{aligned} V_{base\ shear} &= \text{strength} \\ F_{px} &= \text{strength} \\ F_x &= \text{force-to-wall (strength)} \\ v &= \text{wall shear at element level (ASD)} \\ v = \frac{F_x}{1.4b} &= \text{ASD} \end{aligned}$$

2. Future editions of the code will use only strength design.

$$E = \rho E_h + E_v = 1.0E_h + 0 = 1.0E_h \quad (30-1)$$

where:

E_v is permitted to be taken as zero for allowable stress design, and ρ will be assumed to be 1.0 (under most cases is 1.0 for Type V construction with interior shear walls). Since the maximum element story shear is not yet known, the assumed value for ρ will have to be verified. (This will be shown in Part 5.)

The basic load combination for allowable stress design for horizontal forces is:

$$D + \frac{E}{1.4} = 0 + \frac{E}{1.4} = \frac{E}{1.4} \quad (12-9)$$

For vertical downward loads:

$$D + \frac{E}{1.4} \text{ or } D + 0.75 \left[L + (L_r \text{ or } S) + \frac{E}{1.4} \right] \quad (12-10, 12-11)$$

For vertical uplift:

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

$$V = 0.182W \quad \S 1612.3.1$$

$$\therefore V = 0.182(595,000 \text{ lb}) = \underline{\underline{108,290 \text{ lb}}}$$

1b.**Vertical distributions of forces.**

The base shear must be distributed to each level. This is done as follows:

$$F_{px} = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (30-15)$$

Where h_x is the average height at level i of the sheathed diaphragm in feet above the base.

Since $T = 0.28$ second < 0.7 second, $F_t = 0$

Determination of F_{px} is shown in Table 2-1.

§1630.5

Note: Although not shown here, designers must also check wind loading. In this example, wind loading may control the design in the east-west direction.

Table 2-1. Vertical distribution of seismic forces

Level	w_x (k)	h_x (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_{px} (k)	$\frac{F_{px}}{w_x}$	F_{tot} (k)
Roof	135.0	33.6	4,536	41.1	44.5	0.330	44.5
3 rd Floor	230.0	18.9	4,347	39.4	42.7	0.186	87.2
2 nd Floor	230.0	9.4	2,162	19.5	21.1	0.092	108.3
Σ	595.0		11,045		108.3		

2.**Lateral forces on the shear walls and required nailing assuming flexible diaphragms.**

In this step, forces on shear walls due seismic forces will be determined. As has been customary practice in the past, this portion of the example assumes flexible diaphragms. The UBC does not require torsional effects to be considered for flexible diaphragms. The effects of torsion and wall rigidities will be considered in Part 4 of this design example.

Under the flexible diaphragm assumptions, loads to shear walls are determined based on tributary areas with simple spans between supports. Another method of determining loads to shear walls can assume a continuous beam. This design example uses the total building weight W applied to each respective direction. The results shown will be slightly conservative, since the building weight W includes the wall weights for the direction of load, which can be subtracted out. This example converts the story forces into seismic forces per square foot of floor or

roof area. This may result in losing a certain amount of precision, but in turn results in much simpler calculations. This approach is generally considered acceptable unless there is seen to be a concentration of dead load in a particular area (e.g., a mechanical penthouse).

A detailed analysis will include the derivation of these tributary weights, which includes the tributary exterior and interior wall weights.

Using forces from Table 2-1 and the area of the floor plan = 5,288 sf, calculate tributary weights.

For roof diaphragm:

Roof area = 5,288 sq ft

$$f_{p\text{ roof}} = \frac{44.5 \times 1,000}{5,288} = 8.415 \text{ psf}$$

For third floor diaphragm:

Floor area = 5,288 sq ft

$$f_{p\text{ 3rd}} = \frac{42.7 \times 1,000}{5,288} = 8.075 \text{ psf}$$

For second floor diaphragm:

Floor area = 5,288 sq ft

$$f_{p\text{ 2nd}} = \frac{21.1 \times 1,000}{5,288} = 3.990 \text{ psf}$$

Table 2-2. Forces to walls and required panel nailing for east-west direction^{1, 2, 3}

Wall	Trib Area (sq ft)	$\sum F_{Above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	$b^{(4)}$ (ft)	$v = \frac{F_{tot}^{(5)}}{(1.4)b}$ (plf)	Sheathed 1 or 2 sides	Allowable Shear ⁽⁶⁾ (plf)	Edge Nail Spacing (in.)
<i>Shear Walls at Roof Level⁽⁷⁾</i>									
A	170	0	1,430	1,430	12.5	85	1	340	6
B	746	0	6,280	6,280	22.0	205	1	340	6
C	1,344	0	11,310	11,310	43.0	190	1	340	6
E	1,344	0	11,310	11,310	43.0	190	1	340	6
F	960	0	8,080	8,080	43.0	135	1	340	6
G	554	0	4,660	4,660	22.0	155	1	340	6
H	170	0	1,430	1,430	12.5	85	1	340	6
Σ	5,288	0	44,500	44,500	198				
<i>Shear Walls at Third Floor Level</i>									
A	170	1,430	1,375	2,805	12.5	160	1	340	6
B	746	6,280	6,025	12,305	22.0	400	1	510	4
C	1,344	11,310	10,850	22,160	43.0	370	1	510	4
E	1,344	11,310	10,850	22,160	43.0	370	1	510	4
F	960	8,080	7,750	15,830	43.0	265	1	510	4
G	554	4,660	4,475	9,135	22.0	300	1	510	4
H	170	1,430	1,375	2,805	12.5	160	1	340	6
Σ	5,288	44,500	42,700	87,200	198				
<i>Shear Walls at Second Floor Level</i>									
A	170	2,805	680	3,485	12.5	200	1	340	6
B	746	12,305	2,975	15,280	22.0	500	1	665	3
C	1,344	22,160	5,365	27,525	43.0	460	1	665	3
E	1,344	22,160	5,365	27,525	43.0	460	1	665	3
F	960	15,830	3,830	19,660	43.0	330	1	665	3
G	554	9,135	2,210	11,345	22.0	370	1	665	3
H	170	2,805	680	3,485	12.5	200	1	340	6
Σ	5,288	87,200	21,100	108,300	198				

Notes:

1. Minimum framing thickness: The 1994 and earlier editions of the UBC required 3× nominal thickness stud framing at abutting panel edges when 10d common nails were spaced 3 inches on center or closer (2" on center for 8d) or if sheathing is installed on both sides of the studs without staggered panel joints. The 1997 UBC (Table 23-II-I-1 Footnote 2 and 3) requires 3× nominal thickness stud framing at abutting panels and at foundation sill plates when the allowable shear values exceed 350 pounds per foot or if the sheathing is installed on both sides of the studs without staggered panel joints.
2. Sill bolt washers: Section 1806.6.1 of the 1997 UBC requires that a minimum of 2-inch-square by 3/16-inch-thick plate washers be used for each foundation sill bolt (regardless of allowable shear values in the wall). These changes were a result of splitting of framing studs and sill plates observed in the Northridge earthquake and in cyclic testing of shear walls. The plate washers are intended to help resist uplift forces on shear walls. Because of observed vertical displacements of tiedowns, these plate washers are required even if the wall has tiedowns designed to take uplift forces at the wall boundaries. The washer edges shall be parallel/perpendicular to the sill plate. Errata to the First Printing of the 1997 UBC (Table 23-II-I-1 Footnote 3) added an exception to the 3× foundation sill plates by allowing 2× foundation sill plates when the allowable shear values are less than 600 pounds per foot, provided that sill bolts are designed for 50 percent of allowable values.
3. The 1999 SEAOC Blue Book recommends special inspection when the nail spacing is closer than 4" on center.

4. The shear wall length used for wall shears is the “out-to-out” wall length.
5. Note that forces are strength level and that shear in wall is divided by 1.4 to convert to allowable stress design.
6. APA Structural I rated wood structural panels may be either plywood or oriented strand board (OSB). Allowable shear from UBC Table 23-II-I-1.
7. Shear walls at lines C, E, and F extend to the bottom of the prefabricated wood trusses at the roof level. Shear transfer is obtained by framing clips from the bottom chord of the trusses to the top plates of the shear walls. Project plans call for trusses at these lines to be designed for these horizontal forces (see also comments in Part 8). Roof shear forces are also transferred to lines A, B, G, and H.

Table 2-3. Forces to walls and required panel nailing for north-south direction^{1, 2, 3}

Wall	Trib. Area (sq ft)	$\sum F_{Above}$ (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	$b^{(4)}$ (ft)	$v = \frac{F_{tot}^{(4)}}{(1.4)b}$ (plf)	Sheathed 1 or 2 sides	Allowable Shear (plf)	Edge Nail Spacing (in.)
<i>Shear Walls at Roof Level (5)</i>									
1	2,644	0	22,250	22,250	64.5	250	1	340	6
2	0	0	0	0	0	0			
3	0	0	0	0	0	0			
4	2,644	0	22,250	22,250	64.5	250	1	340	6
Σ	5,288	0	44,500	44,500	129.0				
<i>Shear Walls at Third Floor Level</i>									
1	1,202	22,250	9,705	31,955	64.5	355	1	510	4
2	1,442	0	11,645	11,645	60.0	140	1	340	6
3	1,442	0	11,645	11,645	60.0	140	1	340	6
4	1,202	22,250	9,705	31,955	64.5	355	1	510	4
Σ	5,288	44,500	42,700	87,200	249.0				
<i>Shear Walls at Second Floor Level</i>									
1	1,202	31,955	4,795	36,750	64.5	410	1	510	4
2	1,442	11,645	5,755	17,400	60.0	210	1	340	6
3	1,442	11,645	5,755	17,400	60.0	210	1	340	6
4	1,202	31,955	4,795	36,750	64.5	410	1	510	4
Σ	5,288	87,200	21,100	108,300	249.0				

Notes:

1. *Minimum framing thickness:* The 1994 and earlier editions of the UBC required 3× nominal thickness stud framing at abutting panel edges when 10d common nails were spaced 3 inches on center or closer (2" on center for 8d) or if sheathing is installed on both sides of the studs without staggered panel joints. The 1997 UBC (Table 23-II-I-1 Footnote 2 and 3) requires 3× nominal thickness stud framing at abutting panels and at foundation sill plates when the allowable shear values exceed 350 pounds per foot or if the sheathing is installed on both sides of the studs without staggered panel joints.
2. *Sill bolt washers:* Section 1806.6.1 of the 1997 UBC requires that a minimum of 2-inch-square by 3/16-inch-thick plate washers be used for each foundation sill bolt (regardless of allowable shear values in the wall). These changes were a result of splitting of framing studs and sill plates observed in the Northridge earthquake and in cyclic testing of shear walls. The plate washers are intended to help resist uplift forces on shear walls. Because of observed vertical displacements of tiedowns, these plate washers are required even if the wall has tiedowns designed to take uplift forces at the wall boundaries. The washer edges shall be parallel/perpendicular to the sill plate. Errata to the First Printing of the 1997 UBC (Table 23-II-I-1 Footnote 3) added an exception to the 3× foundation sill plates by allowing 2× foundation sill plates when the allowable shear values are less than 600 pounds per foot, provided that sill bolts are designed for 50 percent of allowable values.
3. The 1999 SEAOC Blue Book recommends special inspection when the nail spacing is closer than 4" on center.
4. Note that forces are strength level and that shear in wall is divided by 1.4 to convert to allowable stress design.

5. The interior shear walls at lines 2 and 3 were not used to brace the roof diaphragm. This is because installing wall sheathing (blocking panels) perpendicular to plated trusses is labor intensive. Often it is not installed correctly, and occasionally it is not even installed due to contractor error. This approach will increase the third floor diaphragm transfer (redistribution) forces. With rigid diaphragms, you must carefully follow the load paths.

3.

Rigidities of shear walls.

3a.

Rigidity calculation using the UBC deflection equation.

Determination of wood shear wall rigidities is not a simple task. In practice, approximate methods are often used. The method illustrated in this example is by far the most rigorous method used in practice. There are other methods that are more simplified, and use of these other more simplified methods is often appropriate. The alternate methods are briefly discussed in the Commentary to Design Example 1.

It must be emphasized, that at the present time *every* method is approximate, particularly for multistory structures such as in this example. Until more definite general procedures are established through further testing and research, the designer must exercise judgment in selecting an appropriate method to be used for a given structure. When in doubt, consult with the local building official regarding methods acceptable to the jurisdiction. At the time of this publication, the type of seismic design required for a project of this type varies greatly from jurisdiction to jurisdiction.

Wall rigidities are approximate. The initial rigidity R of the structure can be significantly higher due to stucco, drywall, stiffening effects of walls not considered, and areas over doors and windows. During an earthquake, some low-stressed walls may maintain their stiffness and others degrade in stiffness. Some walls and their collectors may attract significantly more lateral load than anticipated in flexible or rigid diaphragm analysis. It must be understood that the method of analyzing a structure using rigid diaphragms takes significantly more engineering effort. However, use of the rigid diaphragm method indicates that some lateral resisting elements can attract significantly higher seismic demands than from tributary area (i.e., flexible diaphragm) analysis methods.

In this example, shear wall rigidities (k) are computed using the basic stiffness equation:

$$F = k\Delta$$

or:

$$k = \frac{F}{\Delta}$$

The basic equation to determine the shear wall deflections is shown below. This should be viewed as one possible approach that can be substantiated with code equations. There are other approaches that can also be used.

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a \quad \text{\$23.223 Vol. 3}$$

where:

v = shear in the wall in pounds per lineal foot

h = height from the bottom of the sill plate to the underside of the framing at diaphragm level above (top plates)

A = area of the boundary element in square inches

At the third floor, the boundary elements consist of 2-2x4s
(see Figure 2-9)

At the second floor, the boundary elements consist of 3-2x4s
(see Figure 2-10)

At the ground floor, the boundary elements consist of 3-3x4s:

b = is the shear wall length in feet

G = shear modulus values from Table 23-2-J, in pounds per square inch

t = equivalent thickness values from Table 23-2-I, in inches

V_n = load per fastener (nail) in pounds

e_n = nail slip values are for Structural I sheathing with dry lumber = $(V_n/769)^{3.276}$

d_a = displacement of the tiedown due to anchorage details in inches

The above equation is based on tests conducted by the American Plywood Association and on a uniformly nailed, cantilever shear wall with fixed base and free top, a horizontal point load at top, and panel edges blocked, and deflection is estimated from the contributions of four distinct parts. The first part of the equation accounts for cantilever beam action using the moment of inertia of the boundary elements. The second term accounts for shear deformation of the sheathing. The third term accounts for nail slippage/bending, and the fourth term accounts for tiedown assembly displacement (this also should include bolt/nail slip and shrinkage). The UBC references this equation in §2315.1.

The engineer should be cautioned to use the units as listed in §23.223 (and as listed above). Do not attempt to change the units.

Testing on wood shear walls has indicated that the above deflection formula is reasonably accurate for wall aspect ratios (h/w) lower than or equal to 2:1. For higher aspect ratios, the wall drift increases significantly, and displacements were not be adequately predicted by the formula. Using the new aspect ratio requirement of 2:1 (UBC 1997) makes this formula more accurate for determining shear wall deflection/stiffness than it was in previous editions of the UBC, subject to the limitations mentioned above.

Recent testing on wood shear walls has shown that sill plate crushing under the boundary element can increase the shear wall deflection by as much as 20 to 30 percent. For a calculation of this crushing effect, see the deflection of wall frame at line D later in this same Part 11c.

Faster slip/nail deformation values (e_n).

Volume 3 of the UBC has Table 23-2-K for obtaining values for e_n . However, its use is somewhat time-consuming since interpolation and adjustments are necessary. Footnote 1 to Table 23-2-K requires the values for e_n to be decreased 50 percent for seasoned lumber. This means that the table is based on nails being driven into *green* lumber and the engineer must use one-half of these values for nails driven in *dry* lumber. The values in Table 23-2-K are based on tests conducted by the APA. The 50 percent reduction for dry lumber is a conservative factor. The actual tested slip values with dry lumber were less than 50 percent of the green lumber values.

It is recommended that values for e_n be computed based on fastener slip equations from Table B-4 of APA Research Report 138. This research report is the basis for the formulas and tables in the UBC. Both the research report and the UBC will produce the same values. However, using the fastener slip equations from Table B-4 of Research Report 138 will save time and also enable computations to be made by a computer.

For 10d common nails used in this example, there are two basic equations:

$$\text{When nails are driven into green lumber: } e_n = (V_n/977)^{1.894} \quad \text{APA Table B-4}$$

$$\text{When nails are drive into dry lumber: } e_n = (V_n/769)^{3.276} \quad \text{APA Table B-4}$$

where:

$$V_n = \text{fastener load in pounds per fastener}$$

These values from the above formulas are based on Structural I sheathing and must be increased by 20 percent when the sheathing is not Structural I. The language in Footnote A in Research Report 138, Table B-4, which states “Fabricated green/tested dry (seasoned)...” is potentially misleading. The values in the table are actually *green values*, since the assembly is fabricated when green. Don’t be misled by the word “seasoned.”

It is uncertain whether or not the d_a factor is intended to include wood shrinkage and crushing due to shear wall rotation, because the code is not specific. This design example includes shrinkage and crushing in the d_a factor.

Many engineers are concerned that if the contractor installs the nails at a different spacing (too many or too few), then the rigidities will be different than those calculated. However, nominal changing of the nail spacing in a given wall does not significantly change the stiffness.

3b.

Calculation of shear wall rigidities.

In this example, shear wall rigidities are calculated using the four-term code deflection equation in §23.223 of Volume 3. These calculations are facilitated by the use of a spreadsheet program, which eliminates possible arithmetic errors from the many repetitive computations that must be made.

The first step is to calculate the displacement (i.e., vertical elongation) of the tiedown assemblies and the crushing effect of the boundary element. This is the term d_a . The force considered to act on the tiedown assembly is the net uplift force determined from the flexible diaphragm analyses of Part 2. These forces are summarized in Tables 2-4, 2-9, and 2-13 for the roof at the third floor and second floor, respectively.

After the tiedown assembly displacements are determined, the four-term deflection equation is used to determine the deflection Δ_s of each shear wall. These are summarized in Tables 2-5 and 2-6 for the roof level, and in Tables 2-10 and 2-11 for the third floor level, and in Table 2-14 and 2-15 for the second floor level.

Finally, the rigidities of the shear walls are summarized in Tables 2-7, 2-12, and 2-16 for the roof, third floor, and second floor, respectively.

For both strength and allowable stress design, the 1997 UBC now requires building drifts to be determined by the load combinations of §1612.2, which covers load combinations using strength design or load and resistance factor design. Errata for the second and third printing of the UBC unexplainably referenced §1612.3 for allowable stress design. The reference to §1612.3 is *incorrect* and will be changed back to reference §1612.2 in the fourth and later printings.

Using strength level forces for wood design using the 1997 UBC now means that the engineer will use both strength-level forces and allowable stress forces. This can create some confusion, since the code requires drift checks to be strength-level forces. However, all of the design equations and tables in Chapter 23 are based on allowable stress design. Drift and shear wall rigidities should be calculated from the strength-level forces. Remember that the structural system factor R is based on using strength-level forces.

3c.**Estimation of roof level rigidities.**

To determine roof level wall rigidities, roof level displacements must first be determined. Given below are a series of calculations, done in table form, to estimate the roof level displacements Δ_s in each shear wall. First, the shear wall tiedown assembly displacements are determined (Table 2-4). These, and the parameters given in Table 2-5, are used to arrive at the displacements Δ_s for each shear wall at the roof level (Table 2-5 and 2-6). Rigidities are estimated in Table 2-7 for walls in both directions. Once the Δ_s displacements are known, a drift check is performed. This is summarized in Table 2-8.

Table 2-4. Determine tiedown assembly displacements at roof level ¹

Wall	ASD		Strength Design					
	Uplift/1.4 ⁽²⁾ (lb)	Tiedown Device	Uplift (lb)	Tiedown ⁽³⁾ Elongation (in.)	Tiedown Assembly Displacement			d_a ⁽⁷⁾ (in.)
					Shrink ⁽⁴⁾	Crush ⁽⁵⁾	Slip ⁽⁶⁾	
A	0	Not required	0	0	0.05	0.02	0	0.07
B1	840	Strap	1,175	0.04	0.05	0.02	0.002	0.11
B2	840	Strap	1,175	0.04	0.05	0.02	0.002	0.11
C1	100	Strap	140	0.02	0.05	0.02	0.002	0.09
C2	100	Strap	140	0.02	0.05	0.02	0.002	0.09
E1	100	Strap	140	0.02	0.05	0.02	0.002	0.09
E2	100	Strap	140	0.02	0.05	0.02	0.002	0.09
F1	0	Not required	0	0	0.05	0.02	0	0.07
F2	0	Not required	0	0	0.05	0.02	0	0.07
G1	500	Strap	700	0.02	0.05	0.02	0.002	0.09
G2	500	Strap	700	0.02	0.05	0.02	0.002	0.09
H	0	Not required	0	0	0.05	0.02	0	0.07
1a, 4a	120	Strap	170	0.02	0.05	0.02	0.002	0.09
1b, 4b	0	Not required	0	0	0.05	0.02	0	0.07
1c, 4c	0	Not required	0	0	0.05	0.02	0	0.07
1d, 4d	0	Not required	0	0	0.05	0.02	0	0.07
1e, 4e	0	Not required	0	0	0.05	0.02	0	0.07
1f, 4f	120	Strap	170	0.02	0.05	0.02	0.002	0.09

Notes:

1. Tiedown assembly displacements for the roof level are calculated for the tiedowns at the third floor level.
2. Uplift force is determined by using the *net* overturning moment ($M_{OT} - M_{OR}$) divided by the distance between the *centroids* of the boundary elements with 4x members at the ends of the shear wall. This equates to the length of the wall minus 3½ inches for straps or the

length of wall minus 7¼ inches when using a bolted tiedown with 2-inch offset from post to anchor bolt. Using allowable stress design, tiedown devices need only be sized by using the ASD uplift force. The strength design uplift force is used to determine tiedown assembly displacement in order to determine strength-level displacements.

3. The continuous tiedown (rod) system selected for this structure will have a “shrinkage compensating” system. Most of these systems have shrinkage compensation by either pre-tensioning of cables or a “self-ratcheting” hardware connector and are proprietary. The device selected in this design example has adjusting grooves at 1/10-inch increments, meaning the most the “system” will have not compensated for in shrinkage and crushing will be 1/10-inch. If the selected device does not have a shrinkage compensating device then, shrinkage of floor framing, sill plates, compression bridges, crushing of bridge support studs, and collector studs will need to be considered. See Design Example 1, Part 3c for an example calculation for a bolted connection. The tiedown rod at line B will elongate as follows:

$$\text{for } 5/8" \text{ rod: } \Delta = \frac{PL}{AE} = 6,090\text{lb}(4.5)(12)/0.31(29\text{E}6) = 0.04\text{in}$$

Note that the rod length is 4.5 feet (Figure 2-12). The elongation for the portion of the rod at the level below will be considered at the level below.

For level below (Table 2-13) rod length is 9.44 feet (Figure 2-12):

$$\text{for } 5/8" \text{ rod: } \Delta = \frac{PL}{AE} = 12,040\text{lb}(9.44)(12)/0.31(29\text{E}6) = 0.15\text{in}.$$

4. Wood shrinkage is based on a change in moisture content (MC) from 19 percent to 15 percent, with 19 percent MC being assumed for S-Dry lumber per project specifications. The MC of 15 percent is the assumed final MC at equilibrium with ambient humidity for the project location. The final equilibrium value can be higher in coastal areas and lower in inland or desert areas. This equates to $(0.002)(d)(19 - 15)$, where d is the dimension of the lumber (see Figure 2-11). Pressure-treated lumber has moisture content of less than 16 percent at treatment completion. Shrinkage of $2 \times \text{DBL Top Plate} + 2 \times \text{DBL sill plate} = (0.002)(4 \times 1.5\text{in})(19 - 15) = 0.05\text{in}$.
5. Per 91 NDS 4.2.6, when compression perpendicular to grain ($f_{c\perp}$) is less than $0.73F'_{c\perp}$ crushing will be approximately 0.02 inches. When $f_{c\perp} = F'_{c\perp}$ crushing is approximately 0.04 inches. The effect of sill plate crushing is the downward effect at the opposite end of the wall with uplift force and has the same rotational effect as the tiedown displacement. Short walls that have no uplift forces will still have a crushing effect and contributes to rotation of the wall.
6. Per 91 NDS 7.3.6 load/slip modulus $\gamma = (270,000)(D^{1.5})$, plus an additional 1/16" for the oversized hole for bolts. For nails, values for e_n can be used.
7. d_a is the total tiedown assembly displacement. This also could include mis-cuts (short studs) and lack of square cut ends.

Table 2-5. Deflections of shear walls at the roof level in east-west direction

Wall	ASD v (plf)	Strength v (plf)	h (ft)	A (in. ²)	E (psi)	b (ft)	G (psi)	t (in.)	Nail Spacing (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
A	85	119	8.21	10.5	1.7E6	12.5	90,000	0.535	6	60	0.0002	0.07	0.07
B1	205	287	8.21	10.5	1.7E6	11.0	90,000	0.535	6	144	0.0041	0.11	0.16
B2	205	287	8.21	10.5	1.7E6	11.0	90,000	0.535	6	144	0.0041	0.11	0.16
B						22.0							
C1	190	266	8.21	10.5	1.7E6	21.5	90,000	0.535	6	133	0.0032	0.09	0.10
C2	190	266	8.21	10.5	1.7E6	21.5	90,000	0.535	6	133	0.0032	0.09	0.10
C						43.0							
E1	190	266	8.21	10.5	1.7E6	21.5	90,000	0.535	6	133	0.0032	0.09	0.10
E2	190	266	8.21	10.5	1.7E6	21.5	90,000	0.535	6	133	0.0032	0.09	0.10
E						43.0							
F1	135	189	8.21	10.5	1.7E6	21.5	90,000	0.535	6	95	0.0011	0.07	0.07
F2	135	189	8.21	10.5	1.7E6	21.5	90,000	0.535	6	95	0.0011	0.07	0.07
F						43.0							
G1	155	217	8.21	10.5	1.7E6	11.0	90,000	0.535	6	109	0.0017	0.09	0.12
G2	155	217	8.21	10.5	1.7E6	11.0	90,000	0.535	6	109	0.0017	0.09	0.12
G						22.0							
H	85	119	8.21	10.5	1.7E6	12.5	90,000	0.535	6	60	0.0002	0.07	0.07

Table 2-6. Deflections of shear walls at the roof level in north-south direction

Wall	ASD v (plf)	Strength v (plf)	h (ft)	$A^{(2)}$ (in.)	E (psi)	b (ft)	G (psi)	t (in.)	Nail Spacing (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
1a, 4a	250	350	8.21	10.5	1.7E6	8.0	90,000	0.535	6	175	0.0078	0.09	0.21
1b, 4b	250	350	8.21	10.5	1.7E6	14.0	90,000	0.535	6	175	0.0078	0.07	0.16
1c, 4c	250	350	8.21	10.5	1.7E6	11.5	90,000	0.535	6	175	0.0078	0.07	0.17
1d, 4d	250	350	8.21	10.5	1.7E6	11.5	90,000	0.535	6	175	0.0078	0.07	0.17
1e, 4e	250	350	8.21	10.5	1.7E6	11.5	90,000	0.535	6	175	0.0078	0.07	0.17
1f, 4f	250	350	8.21	10.5	1.7E6	8.0	90,000	0.535	6	175	0.0078	0.09	0.21
1, 4						64.5							

Table 2-7. Shear wall rigidities at roof level ¹

Wall	$\Delta_s^{(2)}$ (in.)	F (lb)	$k_i = \frac{F}{\Delta_s}$ (k/in.)	k_{total} (k/in.)
A	0.07	1,430	20.43	20.43
B1	0.16	3,140	19.62	
B2	0.16	3,140	19.62	
B		6,280	39.24	39.24
C1	0.10	5,655	56.55	
C2	0.10	5,655	56.55	
C		11,310	113.1	113.1
E1	0.10	5,655	56.55	
E2	0.10	5,655	56.55	
E		11,310	113.1	113.1
F1	0.07	4,040	57.71	
F2	0.07	4,040	57.71	
F		8,080	115.4	115.4
G1	0.12	2,330	19.42	
G2	0.12	2,330	19.42	
G		4,660	38.84	38.84
H	0.07	1,430	20.42	20.42
1a, 4a	0.21	2,760	13.14	
1b, 4b	0.16	4,830	30.19	
1c, 4c	0.17	3,965	23.32	
1d, 4d	0.17	3,970	23.35	
1e, 4e	0.17	3,965	23.32	
1f, 4f	0.21	2,760	13.14	
1, 4		22,250	126.5	126.5

Notes:

1. Deflections and forces are based on strength force levels.
2. Δ_s are the design level displacements from Tables 2-5 and 2-6.

3d.**Drift check at roof level.****§1630.10.2**

To determine drift, the maximum inelastic response displacement Δ_M must be determined. This is defined in §1630.9.2 and computed as follows:

$$\Delta_M = 0.7R\Delta_S \quad (30-17)$$

$$R = 5.5$$

Table 16-N

$$\Delta_M = 0.7(5.5)\Delta_S$$

Under §1630.10.2, the calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period less than 0.7 seconds. The building period for this design example was calculated to be 0.28 seconds, which is less than 0.7 seconds, therefore the 0.025 drift limitation applies. The drift check is summarized in Table 2-8.

Table 2-8. Drift check at roof level

	Wall	Δ_S (in.)	Height (ft.)	Δ_M (in.)	Max. Δ_M (in.)	Status
East-West	A	0.07	8.21	0.27	2.46	ok
	B	0.16	8.21	0.62	2.46	ok
	C	0.10	8.21	0.38	2.46	ok
	E	0.10	8.21	0.38	2.46	ok
	F	0.07	8.21	0.27	2.46	ok
	G	0.12	8.21	0.46	2.46	ok
	H	0.07	8.21	0.27	2.46	ok
North-South	1a, 4a	0.21	8.21	0.81	2.46	ok
	1b, 4b	0.16	8.21	0.62	2.46	ok
	1c, 4c	0.17	8.21	0.65	2.46	ok
	1d, 4d	0.17	8.21	0.65	2.46	ok
	1e, 4e	0.17	8.21	0.65	2.46	ok
	1f, 4f	0.21	8.21	0.81	2.46	ok

3e.**Estimation of third floor level rigidities.**

Shear wall rigidities at the third floor are estimated in the same manner as those at the roof. The calculations are summarized in Tables 2-9, 2-10, 2-11, and 2-12. A drift check is not shown.

Table 2-9. Tiedown assembly displacements at third floor level¹

Wall	ASD		Strength Design					
	Uplift/1.4 ⁽²⁾ (lb)	Tiedown Device	Uplift (lb)	Tiedown Elongation ⁽³⁾ (in.)	Tiedown Assembly Displacement			d_a ⁽⁷⁾ (in.)
					Shrink ⁽⁴⁾	Crush ⁽⁵⁾	Slip ⁽⁶⁾	
A	135	Strap	190	0.02	0.05	0.02	0.002	0.09
B1	4,350	Rod	6,090	0.04	0	0	0.10	0.14
B2	4,350	Rod	6,090	0.04	0	0	0.10	0.14
C1	2,000	Strap	2,800	0.02	0.05	0.02	0.002	0.09
C2	2,000	Strap	2,800	0.02	0.05	0.02	0.002	0.09
E1	2,000	Strap	2,800	0.02	0.05	0.02	0.002	0.09
E2	2,000	Strap	2,800	0.02	0.05	0.02	0.002	0.09
F1	550	Strap	770	0.02	0.05	0.02	0.002	0.09
F2	550	Strap	770	0.02	0.05	0.02	0.002	0.09
G1	2,800	Rod	3,920	0.02	0	0	0.10	0.12
G2	2,800	Rod	3,920	0.02	0	0	0.10	0.12
H	135	Strap	190	0.02	0.05	0.02	0.002	0.09
1a, 4a	2,275	Strap	3,185	0.02	0.05	0.02	0.002	0.09
1b, 4b	0	Not req'd	0	0	0.05	0.02	0	0.07
1c, 4c	0	Not req'd	0	0	0.05	0.02	0	0.07
1d, 4d	0	Not req'd	0	0	0.05	0.02	0	0.07
1e, 4e	0	Not req'd	0	0	0.05	0.02	0	0.07
1f, 4f	2,275	Strap	3,185	0.02	0.05	0.02	0.002	0.09
2a, 3a	0	Not req'd	0	0	0.05	0.02	0	0.07
2b, 3b	0	Not req'd	0	0	0.05	0.02	0	0.07
2c, 3c	0	Not req'd	0	0	0.05	0.02	0	0.07

Notes:

1. Tiedown assembly displacements for the third floor level are calculated for the tiedowns at the second floor level.
2. Footnotes 2-6, see Table 2-4.

Table 2-10. Deflections of shear walls at third floor level in east-west direction

Wall	ASD v (plf)	Strength v (plf)	h (ft)	A (in. ²)	E (psi)	b (ft)	G (psi)	t (in.)	Space (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
A	160	224	9.43	15.7	1.7E6	12.5	90,000	0.535	6	112	0.0018	0.09	0.13
B1	400	560	9.43	15.7	1.7E6	11.0	90,000	0.535	4	187	0.0097	0.14	0.31
B2	400	560	9.43	15.7	1.7E6	11.0	90,000	0.535	4	187	0.0097	0.14	0.31
B						22.0							
C1	370	518	9.43	15.7	1.7E6	21.5	90,000	0.535	4	173	0.0075	0.09	0.20
C2	370	518	9.43	15.7	1.7E6	21.5	90,000	0.535	4	173	0.0075	0.09	0.20
C						43.0							
E1	370	518	9.43	15.7	1.7E6	21.5	90,000	0.535	4	173	0.0075	0.09	0.20
E2	370	518	9.43	15.7	1.7E6	21.5	90,000	0.535	4	173	0.0075	0.09	0.20
E						43.0							
F1	265	371	9.43	15.7	1.7E6	21.5	90,000	0.535	4	124	0.0025	0.09	0.13
F2	265	371	9.43	15.7	1.7E6	21.5	90,000	0.535	4	124	0.0025	0.09	0.13
F						43.0							
G1	300	420	9.43	15.7	1.7E6	11.0	90,000	0.535	4	140	0.0038	0.12	0.22
G2	300	420	9.43	15.7	1.7E6	11.0	90,000	0.535	4	140	0.0038	0.12	0.22
G						22.0							
H	160	224	9.43	15.7	1.7E6	12.5	90,000	0.535	6	112	0.0018	0.09	0.13

Table 2-11. Deflections of shear walls at the third floor level in north-south direction

Wall	ASD v (plf)	Strength (v) (plf)	h (ft)	A (in. ²)	E (psi)	b (ft)	G (psi)	t (in.)	Space (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
1a, 4a	355	497	9.43	15.7	1.7E6	8.0	90,000	0.535	4	166	0.0066	0.09	0.27
1b, 4b	355	497	9.43	15.7	1.7E6	14.0	90,000	0.535	4	166	0.0066	0.07	0.20
1c, 4c	355	497	9.43	15.7	1.7E6	11.5	90,000	0.535	4	166	0.0066	0.07	0.21
1d, 4d	355	497	9.43	15.7	1.7E6	11.5	90,000	0.535	4	166	0.0066	0.07	0.21
1e, 4e	355	497	9.43	15.7	1.7E6	11.5	90,000	0.535	4	166	0.0066	0.07	0.21
1f, 4f	355	497	9.43	15.7	1.7E6	8.0	90,000	0.535	4	166	0.0066	0.09	0.27
1, 4						64.5							
2a, 3a	140	196	9.43	15.7	1.7E6	18.0	90,000	0.535	6	98	0.0012	0.07	0.09
2b, 3b	140	196	9.43	15.7	1.7E6	24.0	90,000	0.535	6	98	0.0012	0.07	0.08
2c, 3c	140	196	9.43	15.7	1.7E6	18.0	90,000	0.535	6	98	0.0012	0.07	0.09
2, 3						60.0							

Table 2-12. Shear wall rigidities at third floor ¹

Wall	Δ_s (in.) ⁽²⁾	F (lb)	$k_i = \frac{F}{\Delta_s}$ (k/in.)	k_{total} (k/in.)
A	0.13	2,805	21.58	21.58
B1	0.31	6,152	19.84	
B2	0.31	6,153	19.84	
B		12,305	39.68	39.68
C1	0.20	11,080	55.40	
C2	0.20	11,080	55.40	
C		22,160	110.80	110.80
E1	0.20	11,080	55.40	
E2	0.20	11,080	55.40	
E		22,160	110.80	110.80
F1	0.13	7,915	60.88	
F2	0.13	7,915	60.88	
F		15,830	121.70	121.70
G1	0.22	4,568	20.76	
G2	0.22	4,567	20.76	
G		9,135	41.52	41.52
H	0.13	2,805	21.58	21.58
1a, 4a	0.27	3,965	14.68	
1b, 4b	0.20	6,936	34.68	
1c, 4c	0.21	5,696	27.12	
1d, 4d	0.21	5,696	27.12	
1e, 4e	0.21	5,696	27.12	
1f, 4f	0.27	3,966	14.68	
1, 4		31,955	145.40	145.40
2a, 3a	0.09	3,494	38.82	
2b, 3b	0.08	4,657	58.21	
2c, 3c	0.09	3,494	38.82	
2, 3		11,645	135.80	135.80

Notes:

1. Deflections and forces are based on strength levels.
2. Δ_s are the design level displacements from Tables 2-10 and 2-11.

3f.**Estimation of second floor level rigidities.**

Shear wall rigidities at the second floor level are estimated in the same manner as those for the roof and third floor. The calculations are summarized in Tables 2-13, 2-14, 2-15, and 2-16. A drift check is not shown.

Table 2-13. Tiedown assembly displacements at second floor level¹

Wall	ASD		Strength Design					
	Uplift/1.4 ⁽²⁾ (lb)	Tiedown Device	Uplift (lb)	Tiedown Elongation ⁽³⁾ (in.)	Tiedown Assembly Displacement			$d_a^{(7)}$ (in.)
					Shrink ⁽⁴⁾	Crush ⁽⁵⁾	Slip ⁽⁶⁾	
A	1,090	Strap	1,525	0.02	0.01	0.02	0.002	0.05
B1	8,600	Rod	12,040	0.15	0	0	0.10	0.25
B2	8,600	Rod	12,040	0.15	0	0	0.10	0.25
C1	4,380	Rod	6,130	0.08	0	0	0.10	0.18
C2	4,380	Rod	6,130	0.08	0	0	0.10	0.18
E1	4,380	Rod	6,130	0.08	0	0	0.10	0.18
E2	4,380	Rod	6,130	0.08	0	0	0.10	0.18
F1	1,565	Rod	2,200	0.03	0	0	0.10	0.13
F2	1,565	Rod	2,200	0.03	0	0	0.10	0.13
G1	5,700	Rod	7,980	0.10	0	0	0.10	0.20
G2	5,700	Rod	7,980	0.10	0	0	0.10	0.20
H	1,090	Strap	1,525	0.02	0.01	0.02	0.002	0.05
1a, 4a	5,240	Rod	7,340	0.10	0	0	0.10	0.20
1b, 4b	0	Not req'd	0	0	0.01	0.02	0	0.03
1c, 4c	1,000	Strap	1,400	0.02	0.01	0.02	0.002	0.05
1d, 4d	1,000	Strap	1,400	0.02	0.01	0.02	0.002	0.05
1e, 4e	1,000	Strap	1,400	0.02	0.01	0.02	0.002	0.05
1f, 4f	5,240	Rod	7,340	0.10	0	0	0.10	0.20
2a, 3a	0	Not req'd	0	0	0.01	0.02	0	0.03
2b, 3b	0	Not req'd	0	0	0.01	0.02	0	0.03
2c, 3c	0	Not req'd	0	0	0.01	0.02	0	0.03

Notes:

1. Tiedown assembly displacements for the second floor level are calculated for the tiedowns at the first floor level.
2. See Table 2-4 for footnotes 2-6.

Table 2-14. Deflections of shear walls at the second floor level in east-west direction

Wall	ASD v (plf)	Strength v (plf)	h (ft)	A (in. ²)	E (psi)	b (ft)	G (psi)	t (in.)	Space (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
A	200	280	9.43	26.2	1.7E6	12.5	90,000	0.535	6	140	0.0038	0.05	0.12
B1	500	700	9.43	26.2	1.7E6	11.0	90,000	0.535	3	175	0.0078	0.25	0.42
B2	500	700	9.43	26.2	1.7E6	11.0	90,000	0.535	3	175	0.0078	0.25	0.42
B						22.0							
C1	460	644	9.43	26.2	1.7E6	21.5	90,000	0.535	3	161	0.0060	0.18	0.25
C2	460	644	9.43	26.2	1.7E6	21.5	90,000	0.535	3	161	0.0060	0.18	0.25
C						43.0							
E1	460	644	9.43	26.2	1.7E6	21.5	90,000	0.535	3	161	0.0060	0.18	0.25
E2	460	644	9.43	26.2	1.7E6	21.5	90,000	0.535	3	161	0.0060	0.18	0.25
E						43.0							
F1	330	462	9.43	26.2	1.7E6	21.5	90,000	0.535	3	115	0.0020	0.13	0.16
F2	330	462	9.43	26.2	1.7E6	21.5	90,000	0.535	3	115	0.0020	0.13	0.16
F						43.0							
G1	370	518	9.43	26.2	1.7E6	11.0	90,000	0.535	3	130	0.0030	0.20	0.30
G2	370	518	9.43	26.2	1.7E6	11.0	90,000	0.535	3	130	0.0030	0.20	0.30
G						22.0							
H	200	280	9.43	26.2	1.7E6	12.5	90,000	0.535	6	140	0.0038	0.05	0.12

Table 2-15. Deflections of shear walls at the second floor level in north-south direction

Wall	ASD v (plf)	Strength v (plf)	h (ft)	A (in. ²)	E (psi)	b (ft)	G (psi)	t (in.)	Space (in.)	V_n (lb)	e_n (in.)	d_a (in.)	Δ_s (in.)
1a, 4a	410	574	9.43	26.2	1.7E6	8.0	90,000	0.535	4	191	0.0104	0.20	0.43
1b, 4b	410	574	9.43	26.2	1.7E6	14.0	90,000	0.535	4	191	0.0104	0.03	0.21
1c, 4c	410	574	9.43	26.2	1.7E6	11.5	90,000	0.535	4	191	0.0104	0.05	0.23
1d, 4d	410	574	9.43	26.2	1.7E6	11.5	90,000	0.535	4	191	0.0104	0.05	0.23
1e, 4e	410	574	9.43	26.2	1.7E6	11.5	90,000	0.535	4	191	0.0104	0.05	0.23
1f, 4f	410	574	9.43	26.2	1.7E6	8.0	90,000	0.535	4	191	0.0104	0.20	0.43
1, 4						64.5							
2a, 3a	210	294	9.43	26.2	1.7E6	18.0	90,000	0.535	6	147	0.0044	0.03	0.10
2b, 3b	210	294	9.43	26.2	1.7E6	24.0	90,000	0.535	6	147	0.0044	0.03	0.10
2c, 3c	210	294	9.43	26.2	1.7E6	18.0	90,000	0.535	6	147	0.0044	0.03	0.10
2, 3						60.0							

Table 2-16. Wall rigidities at second floor ¹

Wall	$\Delta_s^{(2)}$ (in.)	F (lb)	$k_i = \frac{F}{\Delta_s}$ (k/in.)	k_{total} (k/in.)
A	0.12	3,485	29.04	29.04
B1	0.42	7,640	18.19	
B2	0.42	7,640	18.19	
B		15,280	36.38	36.38
C1	0.25	13,762	55.05	
C2	0.25	13,763	55.05	
C		27,525	110.1	110.1
E1	0.25	13,762	55.05	
E2	0.25	13,763	55.05	
E		27,525	110.1	110.1
F1	0.16	9,830	61.44	
F2	0.16	9,830	61.44	
F		19,660	122.8	122.8
G1	0.30	5,672	18.91	
G2	0.30	5,673	18.91	
G		11,345	37.82	37.82
H	0.12	3,485	29.04	29.04
1a, 4a	0.43	4,558	10.60	
1b, 4b	0.21	7,978	37.99	
1c, 4c	0.23	6,552	28.48	
1d, 4d	0.23	6,552	28.48	
1e, 4e	0.23	6,552	28.48	
1f, 4f	0.43	4,558	10.60	
1, 4		36,750	144.6	144.6
2a, 3a	0.10	5,221	52.21	
2b, 3b	0.10	6,958	69.58	
2c, 3c	0.10	5,221	52.21	
2, 3		17,400	174.0	174.0

Notes:

1. Deflections and forces are based on strength force levels.
2. Δ_s are the design level displacements from Tables 2-14 and 2-15.

4.**Distribution of lateral forces to the shear walls.****§1630.6**

The base shear was distributed to the three levels in Part 2. In this step, the story forces are distributed to the shear walls supporting each level using the rigid diaphragm assumption. See Part 7 for a later confirmation of this assumption.

It has been a common engineering practice to assume flexible diaphragms and distribute loads to shear walls based on tributary areas. This has been done for many years and is a well-established conventional design assumption. In this design example, the rigid diaphragm assumption will be used. This is not intended to imply that seismic design of wood light frame construction in the past should

have been necessarily performed in this manner. However, recent earthquakes and testing of wood panel shear walls have indicated that drifts can be considerably higher than what was known or assumed in the past. This knowledge of the increased drifts of short wood panel shear walls and the fact that the diaphragms tend to be much more rigid than the shear walls has increased the need for the engineer to consider the relative rigidities of shear walls.

The code requires that the story force at the center of mass to be displaced from the calculated center of mass (CM) a distance of 5 percent of the building dimension at that level perpendicular to the direction of force. This is to account for accidental torsion. The code requires the most severe load combination to be considered and also permits the negative torsional shear to be subtracted from the direct load shear. The net effect of this is to add 5 percent accidental eccentricity to the calculated eccentricity.

However, lateral forces must be considered to act in each direction of the two principal axis. This design example does not consider eccentricities between the centers of mass between levels. In this design example, these eccentricities are small and are therefore considered insignificant. The engineer must exercise good engineering judgment in determining when those effects need to be considered.

The direct shear force F_v is determined from:

$$F_v = F \frac{R}{\sum R}$$

and the torsional shear force F_t is determined from:

$$F_t = T \frac{Rd}{J}$$

where:

$$J = \sum R d_x^2 + \sum R d_y^2$$

R = shear wall rigidity

d = distance from the lateral resisting element (e.g., shear wall) to the center of rigidity (CR)

$$T = Fe$$

$$F = 44,500 \text{ lb (for roof diaphragm)}$$

e = eccentricity

4a.**Determine center of rigidity, center of mass, eccentricities for roof diaphragm.***Forces in the east-west (x) direction:*

$$\bar{y}_r = \frac{\sum k_{xx} y}{\sum k_{xx}} \quad \text{or} \quad \bar{y}_r \sum k_{xx} = \sum k_{xx} y$$

Using the rigidity values k from Table 2-7 and the distance y from line H to the shear wall:

$$\begin{aligned} \bar{y}_r (20.43 + 39.24 + 113.1 + 113.1 + 115.4 + 38.84 + 20.42) &= 20.43(116) + 39.24(106) \\ &+ 113.1(82.0) + 113.1(50.0) + 115.4(26.0) + 38.84(10.0) + 20.42(0) \end{aligned}$$

Distance to calculated CR $\bar{y}_r = \frac{24,847.3}{460.53} = \underline{\underline{53.9 \text{ ft}}}$

The building is symmetrical about the x -axis (Figure 2-6) and the center of mass is determined as:

$$\bar{y}_m = \frac{116.0}{2} = \underline{\underline{58.0 \text{ ft}}}$$

The minimum 5 percent accidental eccentricity for east-west forces, e_y , is computed from the length of the structure perpendicular to the applied story force.

$$e_y = (0.05 \times 116 \text{ ft}) = \pm 5.8 \text{ ft}$$

The new \bar{y}_m to the displaced CM = $58.0 \text{ ft} \pm 5.8 \text{ ft} = 63.8 \text{ ft}$ or 52.2 ft

The total eccentricity is the distance between the displaced center of mass and the center of rigidity

$$y_r = \underline{\underline{53.9 \text{ ft}}}$$

$$\therefore e_y = 63.8 - 53.9 = \underline{\underline{9.9 \text{ ft}}} \quad \text{or} \quad 52.2 - 53.9 = \underline{\underline{-1.7 \text{ ft}}}$$

Note that displacing the center of mass 5 percent can result in the CM being on either side of the CR and can produce added torsional shears to all walls.

Note that the 5 percent may not be conservative. The contents-to-structure weight ratio can be higher in wood framing than in heavier types of construction. Also, the location of the calculated center of rigidity is less reliable than in other structural systems. Use engineering judgment when selecting the eccentricity e .

Forces in the north-south (y) direction:

The building is symmetrical about the y-axis (Figure 2-6). Therefore, the distance to the CM and CR is:

$$\bar{x}_m = \frac{48.0}{2} = \underline{\underline{24.0 \text{ ft}}}$$

$$e'_x = (0.05)(48 \text{ ft}) = \underline{\underline{\pm 2.4 \text{ ft}}}$$

Because, the CM and CR locations coincide,

$$e_x = e'_x$$

$$\therefore e_x = \underline{\underline{2.4 \text{ ft}}} \text{ or } \underline{\underline{-2.4 \text{ ft}}}$$

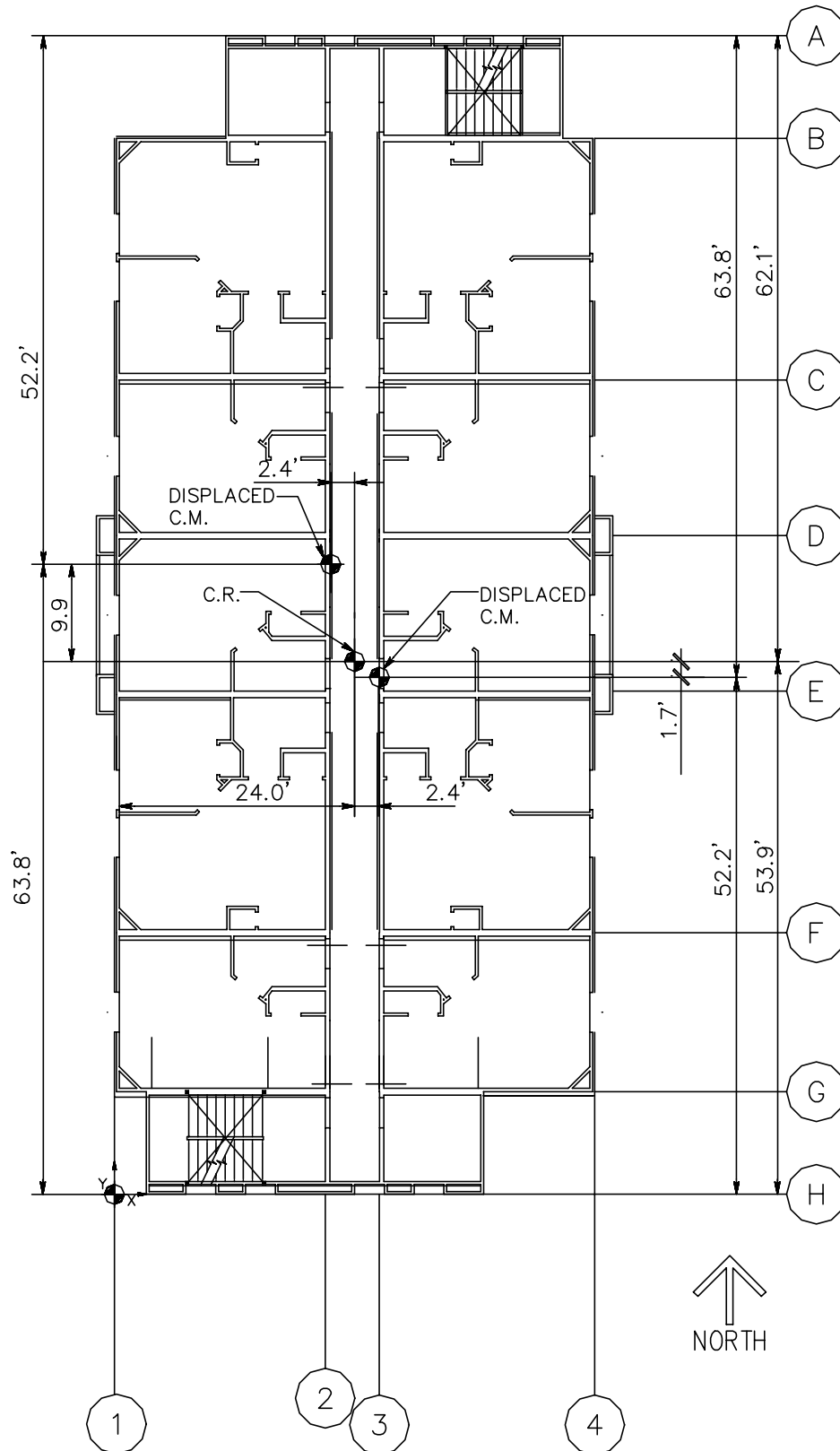


Figure 2-6. Center of rigidity and location of displaced centers of mass for second and third floor diaphragms

4b. Determine total shears on walls at roof level.

The total shears on the walls at the roof level are the direct shears F_v and the shears due to torsion (combined actual torsion and accidental torsion), F_t .

Torsion on the roof diaphragm is computed as follows:

$$T_x = Fe_y = 44,500 \text{ lb}(9.9 \text{ ft}) = 440,550 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

$$\text{or } T_x = 44,500 \text{ lb}(1.7 \text{ ft}) = 75,650 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$$

$$T_y = Fe_x = 44,500 \text{ lb}(2.4 \text{ ft}) = 106,800 \text{ ft} \cdot \text{lb}$$

Since the building is symmetrical for forces in the north-south direction, the torsional forces can be subtracted for those walls located on the opposite side from the displaced center of mass. The critical force will then be used for the design of these walls. Table 2-17 summarizes the spreadsheet for determining combined forces on the roof level walls.

4c. Determine the center of rigidity, center of mass, and eccentricities for the third and second floor diaphragms.

Since the walls stack with uniform nailing, it can be assumed that the center of rigidity for the third floor and the second floor diaphragms will coincide with the center of rigidity of the roof diaphragm.

Torsion on the third floor diaphragms

$$F = (44,500 + 42,700) = 87,200 \text{ lb}$$

$$T_x = Fe_y = 87,200 \text{ lb}(9.9 \text{ ft}) = 863,280 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

$$\text{or } 87,200 \text{ lb}(1.7 \text{ ft}) = 148,240 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$$

$$T_y = Fe_x = 87,200 \text{ lb}(2.4 \text{ ft}) = 209,280 \text{ ft} \cdot \text{lb}$$

Results for the third floor are summarized in Table 2-18.

Torsion on the second floor diaphragms:

$$F = (44,500 + 42,700 + 21,100) = 108,300 \text{ lb}$$

$$T_x = Fe_y = 108,300 \text{ lb}(9.9 \text{ ft}) = 1,072,170 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

or $108,300 \text{ lb}(1.7 \text{ ft}) = 184,110 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$

$$T_y = Fe_x = 108,300 \text{ lb}(2.4 \text{ ft}) = 259,920 \text{ ft} \cdot \text{lb}$$

Results for the second floor are summarized in Table 2-19.

4d.

Comparison of flexible vs. rigid diaphragm results.

Table 2-20 summarizes wall forces determined under the separate flexible and rigid diaphragm analysis. Since nailing requirements were established in the flexible diaphragm analysis of Part 2, they must be checked for results of the rigid diaphragm analysis and adjusted if necessary (also given in Table 2-20).

Table 2-17. Distribution of forces to shear walls below the roof level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	20.43			62.1	1,269	78,786	1,970	+865	2,835
	B	39.24			52.1	2,044	106,513	3,791	+1394	5,185
	C	113.10			28.1	3,178	89,305	10,932	+2167	13,099
	E	113.10			3.9	441	1,720	10,932	+52	10,984
	F	115.40			27.9	3,220	89,829	11,153	+377	11,530
	G	38.84			43.9	1,705	74,853	3,752	+200	3,952
	H	20.42			53.9	1,101	59,324	1,970	+129	2,099
	Σ	460.53					500,330	44,500		
North-South	1		126.5	24.0		3,036	72,864	22,250	+502	22,752
	4		126.5	-24.0		-3,036	72,864	22,250	-502	21,748
	Σ		253.0				145,728	44,500		
	Σ						646,058			

Table 2-18. Distribution of forces to shear walls below the third floor level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	21.58			62.1	1,340	83,221	4,024	1,685	5,709
	B	39.68			52.1	2,067	107,708	7,399	2,559	9,998
	C	110.8			28.1	3,113	87,489	20,660	3,914	24,574
	E	110.8			3.9	432	1,685	20,660	93	20,693
	F	121.7			27.9	3,395	94,732	22,692	733	23,425
	G	41.52			43.9	1,823	80,018	7,741	393	8,134
	H	21.58			53.9	1,163	62,694	4,024	251	4,275
	Σ	467.66					517,547	87,200		
North-South	1		145.4	24.0		3,490	83,750	22,544	1,064	23,608
	2		135.8	2.5		340	849	21,056	259	21,315
	3		135.8	-2.5		-340	849	21,056	-259	20,797
	4		145.4	-24.0		-3,490	83,750	22,544	-1,064	21,480
	Σ		562.4				169,198	87,200		
	Σ						686,745			

Table 2-19. Distribution of forces to shear walls below second floor level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	29.04			62.1	1,803	111,990	6,617	2,682	9,299
	B	36.38			52.1	1,911	98,750	8,290	2,843	11,133
	C	110.1			28.1	3,094	86,936	25,088	4,602	29,690
	E	110.1			3.9	429	1,675	25,088	109	25,197
	F	122.8			27.9	3,426	95,589	27,982	875	28,857
	G	37.82			43.9	1,660	72,887	8,618	424	9,042
	H	29.04			53.9	1,565	84,367	6,617	400	7,017
	Σ	475.28					552,194	108,300		
North-South	1		144.6	24.0		3470	83,290	24,576	1,251	25,827
	2		174.0	2.5		435	1,088	29,574	157	29,731
	3		174.0	-2.5		-435	1,088	29,574	-157	29,417
	4		144.6	-24.0		-3470	83,290	24,576	-1,251	23,325
	Σ		637.2				168,756	108,300		
	Σ						720,950			

Table 2-20. Comparison of loads on shear walls using flexible versus rigid diaphragm analysis and recheck of nailing in walls

Wall	$F_{flexible}$ (lb)	F_{rigid} (lb)	Rigid/ Flexible ratio	b (ft)	$v = \frac{F_{max}}{(b)1.4}$ (plf)	Plywood 1 or 2 sides	Allowable Shear (plf) ⁽¹⁾⁽²⁾	Edge Nail Spacing (in.)
<i>Roof Level</i>								
A	1,430	2,835	+98%	12.5	165	1	340	6
B	6,280	5,185	-17%	22.0	205	1	340	6
C	11,310	13,099	+15%	43.0	220	1	340	6
E	11,310	10,984	-3%	43.0	190	1	340	6
F	8,080	11,530	+43%	43.0	195	1	340	6
G	4,660	3,952	-15%	22.0	155	1	340	6
H	1,430	2,099	+46%	12.5	120	1	340	6
1	22,250	22,752	+2%	64.5	255	1	340	6
4	22,250	22,752 ⁽³⁾	+2%	64.5	255	1	340	6
<i>Third Floor</i>								
A	2,805	5,709	+103%	12.5	330	1	340	6
B	12,305	9,998	-18%	22.0	400	1	510	4 ⁽²⁾
C	22,160	24,574	+11%	43.0	415	1	510	4
E	22,160	20,693	-7%	43.0	370	1	510	4
F	15,830	23,425	+48%	43.0	390	1	510	4
G	9,135	8,134	-11%	22.0	300	1	510	4
H	2,805	4,275	+52%	12.5	245	1	340	6
1	31,955	23,608	-26%	64.5	355	1	510	4
2	11,645	21,315	+83%	60.0	255	1	340	6
3	11,645	21,315 ⁽³⁾	+83%	60.0	255	1	340	6
4	31,955	23,608 ⁽³⁾	-26%	64.5	355	1	510	4
<i>Second Floor</i>								
A	3,485	9,299	+167%	12.5	535	1	510	4
B	15,280	11,133	-27%	22.0	500	1	665	3
C	27,525	29,690	+7%	43.0	495	1	665	3
E	27,525	25,197	-9%	43.0	460	1	665	3
F	19,660	28,857	+47%	43.0	480	1	665	3
G	11,345	9,042	-20%	22.0	370	1	665	3
H	3,485	7,017	+100%	12.5	400	1	510	4
1	36,750	25,827	-30%	64.5	410	1	510	4
2	17,400	29,731	+70%	60.0	355	1	340	6 ⁽⁵⁾
3	17,400	29,731 ⁽³⁾	+70%	60.0	355	1	340	6 ⁽⁵⁾
4	36,750	25,827 ⁽³⁾	-30%	64.5	410	1	510	4

Notes:

1. Allowable shears from UBC Table 23-II-I-1
2. Shear walls with shears that exceeds 350 pounds per lineal foot will require 3× framing at abutting panel edges with staggered nails. See also notes at bottom of Table 1-3.
3. Designates the force used was the higher force for the same wall at the opposite side of the structure.
4. The shear of 535 plf exceeds allowable of 510 plf therefore the nail spacing will need to be decreased to 3 inch spacing. A redesign will not be necessary.
5. The shear of 355 plf exceeds allowable of 340 plf, therefore the nail spacing will need to be decreased to 4-inch spacing. A redesign will not be necessary.

Where forces from rigid diaphragm analysis are higher than those from the flexible diaphragm analysis, wall stability and anchorage must be re-evaluated. However, engineering judgment may be used to determine if a complete rigid diaphragm analysis should be repeated due to changes in wall rigidity.

If rigid diaphragm loads are used, the diaphragm shears should be rechecked for total load divided by diaphragm length along the individual wall lines.

5.

Determine reliability/redundancy factor ρ .

§1630.1.1

The reliability/redundancy factor penalizes lateral force resisting systems that do not have adequate redundancy. In Part 1 of this example, the reliability/redundancy factor was previously assumed to be $\rho = 1.0$. This will now be checked.

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

where:

r_{max} = the maximum element-story shear ratio.

For shear walls, the ratio for the wall with the largest shear per foot at or below two-thirds the height of the building is calculated. Or in the case of a three-story building, the ground level and the second level are calculated (see the SEAOC Blue Book Commentary §C105.1.1.1). The total lateral load in the wall is multiplied by $10/l_w$ and divided by the story shear.

l_w = length of wall in feet

A_B = the ground floor area of the structure in square feet

$$r_i = \frac{V_{max}(10/l_w)}{F}$$

$A_B = 5,288 \text{ sq ft}$

For ground level.

For east-west direction:

Using strength-level forces for wall A:

$$r_{max} = \frac{(9,299)(10/12.5)}{108,300} = 0.068$$

$$\rho = 2 - \frac{20}{0.068\sqrt{5,288}} = -2.0 < 1.0 \text{ minimum} \quad o.k.$$

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

Therefore, there is no increase in base shear due to lack of reliability/redundancy.

For north-south direction:

Using strength-level forces for walls 1 and 4:

Load to wall:

$$36,750 \times 11.5 / 64.5 = 6,552 \text{ lb}$$

$$r_i = \frac{(6,552)(10/11.5)}{108,300} = 0.053$$

Note that this is the same as using the whole wall.

$$r_{max} = \frac{(36,750)(10/64.5)}{108,300} = 0.053$$

$$\rho = 2 - \frac{20}{0.053\sqrt{5,288}} = -3.2 < 1.0 \text{ minimum} \quad o.k.$$

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

Therefore, for both directions there is no increase in base shear required due to lack of reliability/redundancy.

For second level.

For east-west direction:

Using strength-level forces for wall B:

$$r_{max} = \frac{(24,574 \times 5)(10/21.5)}{87,200} = 0.065$$

$$\rho = 2 - \frac{20}{0.065\sqrt{5,288}} = -2.2 < 1.0 \text{ minimum } o.k.$$

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

Therefore, there is no increase in base shear due to lack of reliability/redundancy.

For north-south direction:

Using strength-level forces for walls 1 and 4:

$$r_{max} = \frac{(31,955)(10/64.5)}{87,200} = 0.057$$

$$\rho = 2 - \frac{20}{0.057\sqrt{5,288}} = -2.8 < 1.0 \text{ minimum } o.k.$$

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

Therefore, there is no increase in base shear due to lack of reliability/redundancy.

The SEAOC Seismology Committee added the sentence “The value of the ratio of $10/l_w$ need not be taken as greater than 1.0” in the 1999 SEAOC Blue Book—which will not penalize longer walls, but in this design example has no effect.

6.

Determine if structure meets requirements of conventional construction provisions.

While SEAOC is not encouraging the use of conventional construction methods, this step is included because conventional construction is allowed by the UBC (however, it is often misused) and can lead to poor performing structures.

The structure must be checked against the individual requirements of §2320, and because it is in Seismic Zone 4, it must also be checked against §2320.5.2. Results of these checks are shown below.

6a.

Floor total loads.

§1230.5.2

The dead load weight of the floor exceeds the limit of 20 psf limit, and therefore the structure requires an engineering design for vertical and lateral forces.

6b.

Braced wall lines.

§2320.5.2

The spacing of braced wall lines exceeds 25 feet on center, and therefore the entire lateral system requires an engineering design.

Therefore, the hotel structure requires an engineering design for both vertical and lateral loads. If all walls were drywall and the floor weight was less than 20 psf, then use of conventional construction provisions would be permitted by the UBC. However, conventional construction is *not* recommended for this type of structure.

7.

Diaphragm deflections to determine if the diaphragm is flexible or rigid.

This step is shown only as a reference for how to calculate horizontal diaphragm deflections. Since the shear wall forces were determined using both flexible and rigid diaphragm assumptions, there is no requirement to verify that the diaphragm is actually rigid or flexible.

The roof diaphragm has been selected to illustrate the methodology. The design seismic force in the roof diaphragm using Eq (33-1) must first be determined. The design seismic force is then divided by the diaphragm area to determine the horizontal loading in pounds per square foot. These values are used for determining diaphragm shears (and also collector forces). The design seismic force shall not be less than $0.5C_aIW_{px}$ nor greater than $1.0C_aIW_{px}$.

7a.**Roof diaphragm check.**

The roof diaphragm will be checked in two steps. First, the shear in the diaphragm will be determined and compared to allowables. Next, the diaphragm deflection will be calculated. In Part 7b, the diaphragm deflection is used to determine whether the diaphragm is flexible or rigid.

Check diaphragm shear:

The roof diaphragm consists of 15/32"-thick sheathing with 10d @ 6" o/c and panel edges are unblocked. Loading on the segment between C and E, where:

$$v = \frac{(8.41)48.0'(32.0')}{1.4(48.0')^2} = 96 \text{ plf}$$

Diaphragm span = 32.0 ft

Diaphragm depth = 48.0 ft

Diaphragm shears are converted to allowable stress design by dividing by 1.4

From Table 23-II-H, the allowable shear of 190 plf is based on 15/32-inch APA-rated wood structural panels with unblocked edges and 10d nails spaced at 6 inches on center at boundaries and supported panel edges. APA-rated wood structural panels may be either plywood or oriented strand board (OSB).

Check diaphragm deflection:

The code specifies that the deflection is calculated on a unit load basis. In other words, the diaphragm deflection should be based on the same load as the load used for the lateral resisting elements, not F_{px} total force at the level considered. Since the UBC now requires building drifts to be determined by the load combinations of §1612.2 (see Step 4 for additional comments), strength loads on building diaphragm must be determined.

The basic equation to determine seismic forces on a diaphragm is shown below.

$$F_{px} = \frac{F_t + \sum_{i=x}^n F_t}{\sum_{i=x}^n w_i} w_{px} \quad (33-1)$$

where $F_t = 0$ in this example because $T < 0.7$ seconds

$$f_{p\text{ roof}} = \frac{(44.5 \times 135.0)}{135.0} = 44.5 \text{ k}$$

For the uppermost level, the above calculation will always produce the same force as computed in Eq (30-15). Then divide by the area of the diaphragm to find the equivalent uniform force.

$$f_{p\text{ roof}} = \frac{44.5 \times 1,000}{5,288} = 8.41 \text{ psf}$$

In this example, the roof and floor diaphragms spanning between C and E will be used to illustrate the method. The basic code equation to determine the deflection of a diaphragm is shown below.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_C X)}{2b} \quad \text{\S 23.222, Vol. 3}$$

The above equation is based on a uniformly nailed, simple span diaphragm with panel edges blocked and is based on monotonic tests conducted by the American Plywood Association (APA). The equation has four parts. The first part accounts for beam bending, the second accounts for shear deformation, the third accounts for nail slippage/bending, and the last part accounts for chord slippage. The UBC references this in §2315.1.

For the purpose of this design example, the diaphragm is assumed to be a simple span supported at C and E (refer to Figure 2-4). In reality, with continuity, the actual deflection will be less.

With nails at 6 inches on center the strength load per nail is $96 \times 1.4(6/12) = 67 \text{ lb/nail} = V_n$. Other terms in the deflection equation are:

$$L = 32.0 \text{ ft}$$

$$b = 48.0 \text{ ft}$$

$$G = 50,000 \text{ psi}$$

Table 23-2-J Vol. 3

$$E = 1,700,000 \text{ psi}$$

$$A_{2 \times 4 \text{ chords}} = 5.25 \text{ sq in} \times 2 = 10.50 \text{ sq in.}$$

Fastener slip/nail deformation values (e_n) are obtained as follows:

Volume 3 of the UBC uses Table 23-2-K for obtaining nail slip values e_n , however, its use is somewhat time-consuming, since interpolation and adjustments are necessary. Footnote 1 in Table 23-2-K requires the nail slip values e_n be decreased 50 percent for seasoned lumber. This means that the table is based on nails being driven into *green* lumber and the engineer must use half of these values for nails driven in *dry* (seasoned) lumber. The values in Table 23-2-K are based on tests conducted by the APA. The 50 percent nail slip reduction for dry lumber is a conservative factor. The actual tested slips with dry lumber were less than 50 percent of the green lumber slips.

Values for e_n can be computed based on fastener slip equations from Table B-4 of APA Research Report 138. This will save time, be more accurate, and also enable computations to be made by a computer. Using the values of e_n from Volume 3 of UBC requires interpolation and is very time-consuming. For 10d common nails, there are 2 basic equations:

When the nails are driven into *green* lumber: $e_n = (V_n/977)^{1.894}$ APA Table B-4

When the nails are driven into *dry* lumber: $e_n = (V_n/769)^{3.276}$ APA Table B-4

where:

V_n is the fastener load in pounds per fastener

These values are based on Structural I sheathing and must be increased by 20 percent when the sheathing is not Structural I. Footnote a in UBC Table B-4 states “Fabricated green/tested dry (seasoned)...” is very misleading. The values in the table are actually *green values*, since the lumber is fabricated when green. Again, don’t be misled by the word “seasoned.”

$$e_n = 1.20(67/769)^{3.276} = 0.0004$$

$t = 0.298$ in. (for CDX or Standard Grade) Table 23-2-H

Assume chord-splice at the mid-span of the diaphragm that will be nailed. The allowable loads for fasteners are based on limit state design. In other words, the deformation is set at a limit rather than the strength of the fastener. The deformation limit is 0.05 diameters of the fastener. For a 16d nail, a conservative slippage of 0.01 inch will be used.

Using strength level diaphragm shear:

$$\sum (\Delta_c X) = (0.01)16.0 \text{ ft } (2) = 0.32 \text{ in. - ft}$$

$$\Delta = \frac{5(96 \times 1.4)32.0^3}{8(1.7E6)10.50(48.0)} + \frac{96 \times 1.4(32.0)}{4(50,000)0.298} + 0.188(32.0)0.0004 + \frac{0.32}{2(48.0)} = 0.08 \text{ in.}$$

This deflection is based on a *blocked* diaphragm. The UBC does not have a formula for an *unblocked* diaphragm. The APA is currently working on a simplified formula for unblocked diaphragms. Based on diaphragm deflection test results (performed by the APA), an unblocked diaphragm will deflect between 2 to 2½ times that of a blocked diaphragm or can be proportioned to the allowable shears of a blocked diaphragm divided by the unblocked diaphragm. The roof diaphragm is also sloped at 6:12, which is believed to increase the deflection (but this has not been confirmed with tests). This design example has unblocked panel edges for the floor and roof diaphragms, so a conversion factor is necessary. This conversion is for the roof diaphragm. The floors will similarly neglect the stiffening effects of lightweight concrete fill and gluing of sheathing. It is assumed that the unblocked diaphragm will deflect:

$$\therefore \Delta = 0.08(2.5) = \underline{\underline{0.20 \text{ in.}}}$$

7b.

Flexible versus rigid diaphragms.

§1630.6

In this example, the maximum diaphragm deflection was estimated as 0.20 inches. This assumes a simple span for the diaphragm, and the actual deflection would probably be less. The average story drift is on the order of 0.10 inches at the roof (see Step 3c for the computed deflections of the shear walls). For the diaphragms to be considered flexible, the maximum diaphragm deflection will have to be more than two times the average story drift. This is right at the limit of a definition of a flexible diaphragm. The other diaphragm spans would easily qualify as “rigid” diaphragms. As defined by the code, the diaphragms in this design example are considered rigid.

In reality, some amount of diaphragm deformation will occur, and the true analysis is highly complex and beyond the scope of what is normally done for this type of construction. Diaphragm deflection analysis and testing has been performed on level/flat diaphragms. There has not been any testing of sloped and complicated diaphragms, as found in the typical wood framed structure. Therefore, some engineers perform their design based on the roof diaphragm as flexible and the floor diaphragms as rigid.

In using this procedure, the engineer should exercise good engineering judgment in determining if the higher load of the two methodologies is actually required. For example, if the load to two walls by rigidity analysis is found to be 5 percent to line

A and 95 percent to line B, but by flexible analysis it is found to be 50 percent to line A and 50 percent to line B, the engineer should probably design for the larger of the two loads for the individual walls. Note that though the same definition of a flexible diaphragm has been in the UBC since the 1988 edition, it has not been enforced by building officials for Type V construction. The draft of the IBC 2000 has repeated this same definition into Chapter 23 (wood) definitions.

8.

Tiedown forces for the shear wall on line C.

Tiedowns are required to resist the uplift tendency on shear walls caused by overturning moments. In this step, tiedown forces for the three-story shear wall on line C are determined. The design chosen uses continuous tiedowns below the third floor. At the third floor, conventional premanufactured straps are used.

Not included in this design example, but it should be noted: the code has two new provisions for one-hour wall assemblies—Footnotes 17 and 18 of Table 7-B in Volume 1. Footnote 17 requires longer fasteners for gypsum sheathing when the sheathing is applied over wood structural panels. Footnote 18 requires values for F'_c to be reduced to 78 percent of allowable in one-hour walls.

8a.

Discussion on continuous tiedown systems.

The continuous tiedown system is a relatively new method for resisting shear wall overturning. Similar to the many metal connectors used for wood framing connections, most are proprietary and have ICBO approvals. All of the systems have some type of rod and hardware connector system that goes from the foundation to the top of the structure. A common misconception that engineers have with these types of systems is that the elongation of the rod will produce large displacements in the shear walls. Contrary to that perception, these systems are in many instances superior to the one-sided bolted tiedowns.

Investigations after the Northridge earthquake as well as independent testing of the conventional one-sided bolted tiedowns, have concluded that there can be large displacements associated with this type of connection. The large displacements are a result of eccentricity with the boundary element, deflection of the tiedown, wood shrinkage, wood crushing, and oversized holes for the through-bolts.

Some of the proprietary systems compensate for shrinkage either by pre-tensioning of the rod or by a self-ratcheting connector device. Shrinkage-compensating devices are desirable in multi-level wood frame construction. These devices will also compensate for other slack in the tiedown system caused by crushing of plates, seating of posts, studs, etc.

8b.**Determine strength shear wall forces.**

The shear wall on line C is shown on Figure 2-7. Forces at each story are determined as follows (from Table 2-20):

$$F_{roof} = 13,099/2 = 6,550\text{lb}$$

$$F_{third} = (24,574 - 13,099)/2 = 5,738\text{lb}$$

$$F_{second} = (29,690 - 24,574)/2 = 2,558\text{lb}$$

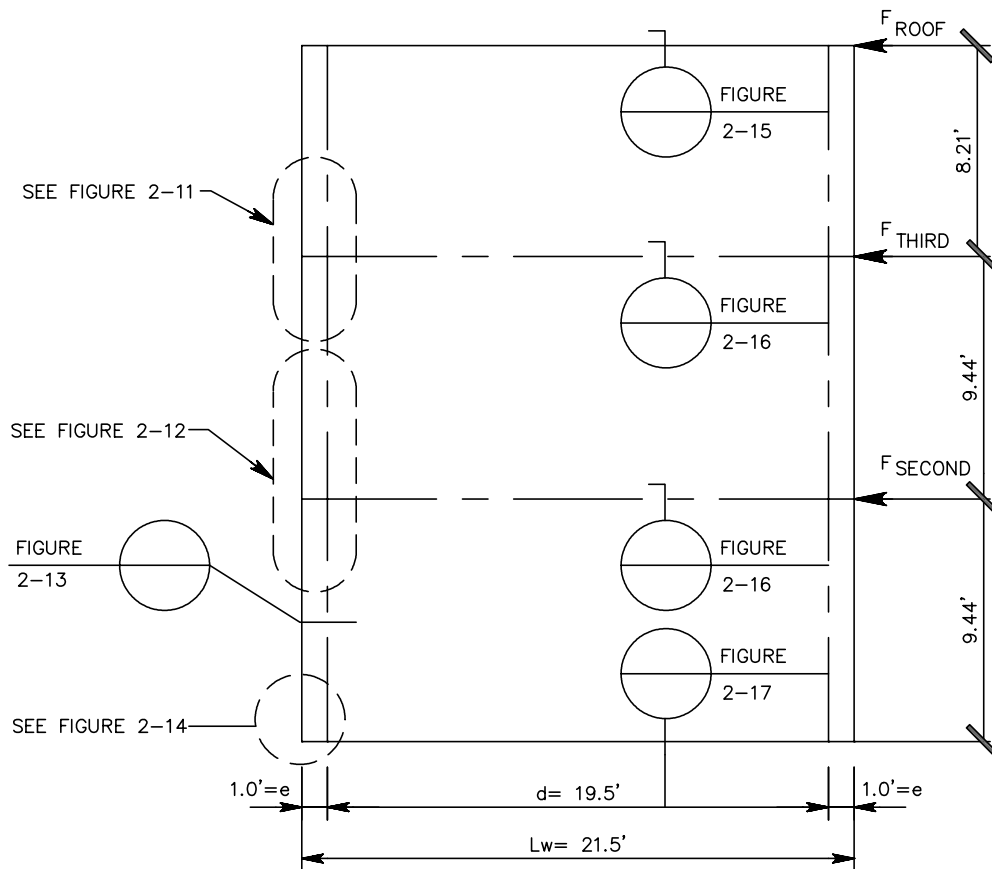


Figure 2-7. Shear wall C elevation

The distance between the centroid of the boundary forces that represent the overturning moment at each level must be estimated. This is shown below.

e = the distance to the center of tiedown rod and boundary studs or collectors studs (Figure 2-12)

$$e = 2 \times 2.5 \text{ in.} + (13/2) = 11.5 \text{ in.} = 0.958 \text{ in.}$$

$$\text{Use } e = 1.0 \text{ ft}$$

d = the distance between centroids of the tiedown and the boundary studs, in feet. (Note that it is also considered acceptable to use the distance from the end of the shear wall to the centroid of the tiedown.)

$$d = 21.5 \text{ ft} - 2(1.0 \text{ ft}) = 19.5 \text{ ft at second floor for third level (Figure 2-12)}$$

$$d = 21.5 \text{ ft} - (2 \times 0.125) = 21.25 \text{ ft at third floor for roof level (Figure 2-11)}$$

The resisting moment M_R is determined from the following loads:

$$W_{\text{roof}} = 13.5 \text{ psf}(2.0 \text{ ft}) = 27.0 \text{ plf}$$

$$W_{\text{floor}} = 25.0 \text{ psf}(2.0 \text{ ft}) = 50.0 \text{ plf}$$

$$W_{\text{wall}} = 10.0 \text{ plf}$$

Table 2-21. Tiedown forces for shear wall C

Level	M_{OT} (ft-lb)	M_R (ft-lb)	$M_R \times 0.9^{(1)}$ (ft-lb)	$\frac{\text{Uplift}}{d}$ $\frac{(M_{OT}/1.4) - 0.9M_R}{d}$ (lb)	Differential Load ⁽²⁾ (lb)
Roof	53,775	25,216	22,694	740	740
Third	169,774	58,590	52,731	3,515	2,775
Second	309,920	91,965	82,769	7,110	3,595

Notes:

1. The UBC no longer has the 0.85 DL provision for stability, this has been replaced with the basic load combinations of §1612.3.1.
2. The differential is the load difference between the uplift force at level x and the level above.

9.**Design tiedown connection at the third floor for the shear wall on line C.**

Figure 2-11 illustrates the typical tiedown connection for the shear wall on line C at the third floor. This is the conventional pre-manufactured strap and is fastened to the framing with nails.

The total uplift force at this level is 740 lb.

$$P_1 = 740 \text{ lb}$$

The tiedowns will be designed using allowable stress design.

§1612.3

The basic load combinations of §1612.2.1 do not permit stress increases.

The alternate basic load combinations of §1612.2.2, however, do permit stress increases.

The Errata to the first printing of the code added $0.9D \pm \frac{E}{1.4}$, Eq. (12-16-1), to the alternate basic load combinations. This exact same load combination is listed in the basic load combinations. This is confusing to many engineers on this topic, because the basic load combinations are based on duration factors (see 1999 SEAOC Blue Book Commentary, §C101.7.3 for further explanation). This design example will use the one-third stress increase of the alternate basic load combination method.

With a 16-gauge $\times 1.25$ -in strap and 10d common nails.

Allowable load per nail is $ZC_D = 113(1.33) = 150 \text{ lb/nail}$

NDS Table 12.3F

Number of nails required $= 740/150 = 4.9 \therefore$ use 5

With nails at 1.5 inches on center the length of strap required is $2(0.75 \text{ in.} + 5 \times 1.5 \text{ in.}) + 6 \text{ in.} = 22.5 \text{ in.}$

\therefore use 24-inch-strap

10.**Design tiedown connection at the second floor for the shear wall on line C.**

As previously mentioned, the second floor tiedown will be part of the continuous tiedown system used below the third level. Refer to Figure 2-12 for illustration of this system and the location of forces P_1 , P_2 , and P_3 .

The total uplift force at the second floor is 3,515 lb (Table 2-21).

$$P_1 = P_2 = \text{total uplift force from above} = 740 \text{ lb}$$

$$P_3 = \text{uplift force for the collector studs} = \text{differential load}/2 = 2,775 \text{ lb}/2 = 1,388 \text{ lb}$$

Since the strap from above is only connected to one pair of collector studs, the total uplift force for the outside set of collectors is equal to the uplift force plus the uplift force on the second floor shear wall from the third floor.

Taking a free-body diagram of the system, the tension in the tiedown rod is increased due to cantilever action between the centroids of the forces. A downward component is actually applied to the interior-most support stud (Figure 2-8):

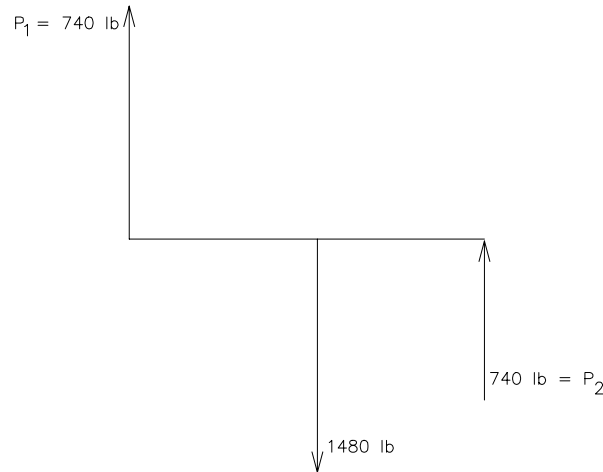


Figure 2-8. Free-body force diagram of compression bridge

Next, the tension in the tiedown rod between the second floor and the compression bridge is the differential load plus the tension load, as computed above. This will produce the total force P_2 on support stud (Figure 2-9):

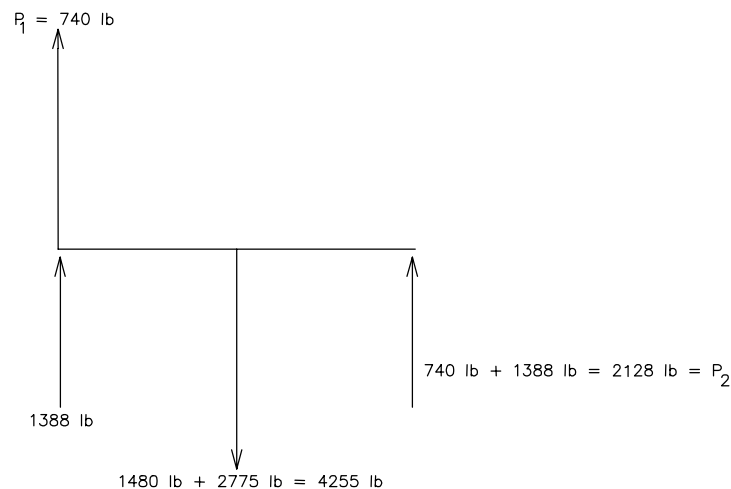


Figure 2-9. Free-body force diagram of compression bridge

Determine spacing for the flat nailing:

$$P_{max} = 2,028 \text{ lb}$$

The allowable lateral load for a 16d common nail in a 1½-inch side member is:

$$ZC_D = 141(1.33) = 187 \text{ lb} \quad \text{NDS Table 12.3B}$$

With 2 rows of 16d nails, the number of nails per row is
 $2,028 \text{ lb} / 2 \times 187 = 5.4 \text{ nails}$

∴ use 6 nails

Maximum spacing = $48 \text{ in} / (6 + 1) = 6.8 \text{ in.}$

∴ Use 6-inch o.c. for the flat nailing

Check compression perpendicular to grain for the bridge support studs to compression bridge:

Critical at P_2

$$f_{c \max} = 2,028 \text{ lb} / (1.5 \times 3.5) = 386 \text{ psi} < F_{c \perp} = 625 \text{ psi} \quad o.k. \quad \text{NDS Supp. Table 4A}$$

Check the bearing perpendicular to grain on bearing plate:

$$F = T_1 = 4,255 \text{ lb}$$

$$f_{c \perp} = 4,255 \text{ lb} / 3.25 \times 5.0 = 262 \text{ psi} < F_{c \perp} = 625 \text{ psi} \quad o.k.$$

Check bearing perpendicular to grain on the top plate from the collector studs from below:

First floor is framed with 3×4 studs

$$\text{Force at } P_3 = 1,388 \text{ lb}$$

$$f_{c \perp} = P/A = 1,388 \text{ lb} / (2.5 \times 3.5) = 160 \text{ psi} < F_{c \perp} = 625 \text{ psi} \quad o.k.$$

Check shear on 4×8 compression bridge (assume tiedown is at center of wall and not at party wall, see Figure 2-12):

$$T_1 = 4,255 \text{ lb}$$

Assuming compression bridge to take all shear:

$$V = \frac{T_1}{2} = \frac{4,255}{2} = 2,130 \text{ lb}$$

$$f_v = \frac{2,130 \times 1.5}{3.5 \times 7.25} = 126 \text{ psi}$$

For Douglas Fir-Larch No. 1:

$$F_v' = F_v C_D = 95 \times 1.33 = 126 \text{ psi } o.k.$$

Check bending on 4×8 compression bridge:

$$T_1 = 4,255 \text{ lb}$$

$$M = \frac{T_1 \times L}{4} = \frac{4,255 \times (10 + 1.5)}{4} = 12,235 \text{ in. - lb}$$

$$S_x \text{ for } 4 \times 8 \text{ with hole for } 5/8" \text{ rod} = (3.5 - 0.69)7.25^2/6 = 24.6 \text{ in}^3$$

$$f_b = \frac{M}{S} = \frac{12,235}{24.6} = 497 \text{ psi}$$

For Douglas Fir Larch No. 1:

$$F_b' = F_b C_D C_F = 1,000(1.33)(1.3) = 1,729 \text{ psi } o.k.$$

Check shear on plates at floor:

Tiedown connector reaction is the *differential load*, which is 3,595 lb.

$$T = 3,595 \text{ lb}$$

Assuming 2 sill plates and 2 top plates to take all shear:

$$V = \frac{T}{2} = \frac{3,595}{2} = 1,800 \text{ lb}$$

$$f_v = \frac{1,800 \times 1.5}{4(1.5 \times 3.5)} = 130 \text{ psi}$$

Since plate have no spits $C_H = 2.0$ (plates rarely check on the edges)

$$F_V' = F_V C_H C_D = 95(2.0)(1.33) = 252 \text{ psi } o.k.$$

Therefore, the tiedown connection shown on Figure 2-12 meets the requirements of code.

11. Design tiedown connection and anchor bolt spacing for shear wall on line C.

11a. Design anchor bolt spacing of sill plate on Line C.

See discussion about fasteners for pressure-preservative treated wood and in Step 19.

From Table 2-20:

$$V = 29,690 \text{ lb}$$

$$v = \frac{V}{L} = \frac{29,690 \text{ lb}}{43 \text{ ft}} = 690 \text{ lb/ft}$$

The 1997 UBC references the 1991 NDS, which specifies in §8.2.3 that the allowable bolt design value, Z , is equal to $t_m = Z_{ts}$ = twice the thickness of wood member. The problem is, there aren't any tables for 6x to 6x members, leaving only the Z formulas. In lieu of using the complex Z formulas, an easier method would be to use the new tables in the 1997 NDS, which are specifically for ledgers and sill plates.

For a side member, thickness = 2.5 inch in Hem-Fir wood (note that designing for Hem-Fir will require a tighter nail and bolt spacing):

$$Z_{11} = 1,350 \text{ lb/bolt}$$

Table 8.2E 97 NDS

$$\text{Required spacing} = \frac{Z_{11} C_D}{v} = \frac{(1,350)(1.33)(1.4)}{690} = 3.6 \text{ ft} = 43 \text{ in.}$$

where

1.4 is the strength conversion factor

∴ Use ¾" diameter bolts at 32 inches on center.

11b. Determine tiedown anchor embedment.

In this calculation, the tiedown anchor will be assumed to occur at the center of the exterior wall. This will produce a lower capacity than if the rod were located at the double-framed wall shown in Figure 2-13.

From Table 2-21:

$$T = 7,110 \text{ lb}$$

$$T_y = 7,110 \times 1.4 \times 1.3 = 13,000 \text{ lb}$$

where 1.4 is the strength conversion factor and 1.3 is for special inspection per §1923.2. Neglecting the area of bolt head bearing surface, the effective area (A_p) of the projected (Figure 2-10), assumed concrete failure surface is:

$$(A_p) = \frac{\pi l_e^2}{2} + 1.75(l_e)^2$$

For $l_e = 15 \text{ in.}$

$$A_p = 406 \text{ in.}^2$$

$$\Phi P_C = \Phi \lambda 4 A_p \sqrt{f'_c} = 0.65 \times 1.0 \times 4 \times 406 \sqrt{3,000} = 57.8 \text{ k}$$

$$P_{SS} = 0.9 \times 0.307 \times 60,000 = 16,580 \text{ lb} > 13,000 \text{ lb (critical)}$$

Provide an oversized hole for the tiedown rod in the foundation sill plate. The rod has no nut or washer to the sill plate, therefore, assume $V = 0 \text{ lb}$ in the rod.

Tiedown bolts resist vertical loads only, anchor bolts are designed to resist the lateral loads.

11C. Check the bearing perpendicular to grain on sill plates.

Assuming all compressive force for overturning will be resisted by end boundary elements, the critical load combination is:

$$D + L + \left(\frac{E}{1.4} \right) \quad (12-13)$$

From Table 2-21, the strength level overturning moment is:

$$M_{OT} = 309,920 \text{ ft} \cdot \text{lb}$$

The seismic compressive force is obtained by dividing by the distance d .

Conversion to allowable stress design is obtained by dividing by 1.4.

$$P_{seismic} = \frac{M_{OT}}{d(1.4)} = \frac{309,920}{19.5(1.4)} = 11,350 \text{ lb}$$

$$P_{DL} = [W_{roof} + (W_{floor}) + W_{wall}(27 \text{ ft})]$$

$$P_{DL} = [27.0 + 2(50.0) + 10.0(27)] \left(\frac{16'' + 8''}{12''} \right) = 795 \text{ lb}$$

$$P_{LL} = (40 \text{ psf} \times 2' \times 2') \left(\frac{16'' + 8''}{12''} \right) = 320 \text{ lb}$$

$$\sum P = 11,350 + 795 + 320 = 12,465 \text{ lb}$$

with full width bearing studs bearing on both sill plates (Figure 2-13), the bearing area is equal to six 3x4 studs.

$$f_c'_{max} = \frac{12,465}{6(8.75)} = 240 \text{ psi} < F_c'_{\perp} = 626 \text{ psi} \quad o.k.$$

where the area of a 3x4 is 8.75 square inches. Note that if a Hem-Fir sill plate is used the allowable compression perpendicular to grain $F_c'_{\perp} = 405 \text{ psi}$.

$$f_c < 0.73F_c'_{\perp} = 0.73(405) = 295 \text{ psi} \quad \text{NDS Supp. Table 4A}$$

Therefore, the assumed crushing effect of 0.02 inches (Table 2-13) is correct.

This crushing will be compensated by the ratcheting effect of the continuous tiedown system as discussed in the notes for Table 2-4.

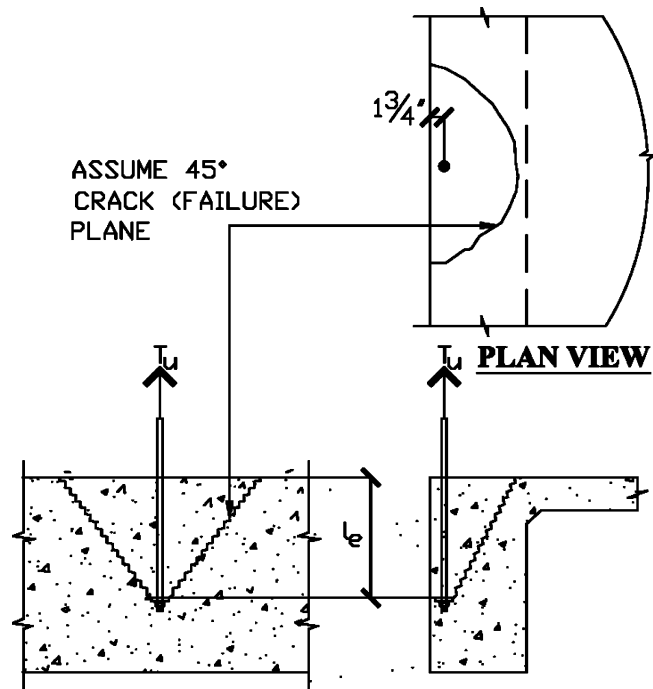


Figure 2-10. Tiedown bolt

12.**Detail of tiedown connection at the third floor for shear wall on line C.**

Note that since the boundary element is a double stud and the wall panel edge nailing is nailed to the end stud, the 16d at 12 inches o.c. internailing of the two tiedown studs should have the capacity to transfer one-half the force to the interior stud (Figure 2-11). These nails may be installed from either side (normally nailed from the outside). See Figure 2-16 for the location of the top plates and commentary about plate locations.

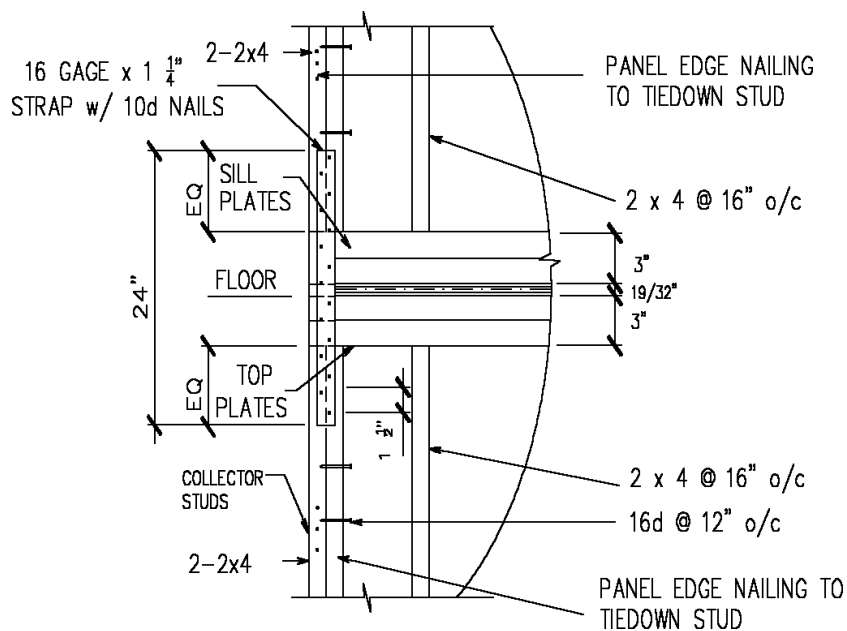


Figure 2-11. Tiedown connection at the third floor for shear wall C.

13.

Detail of tiedown connection at the second floor for shear wall on line C.

This tiedown rod system (Figure 2-12) may also be extended to the third floor instead of using the conventional metal strap shown in Figure 2-11. See Figure 2-16 for the location of the top plates and commentary about plate locations.

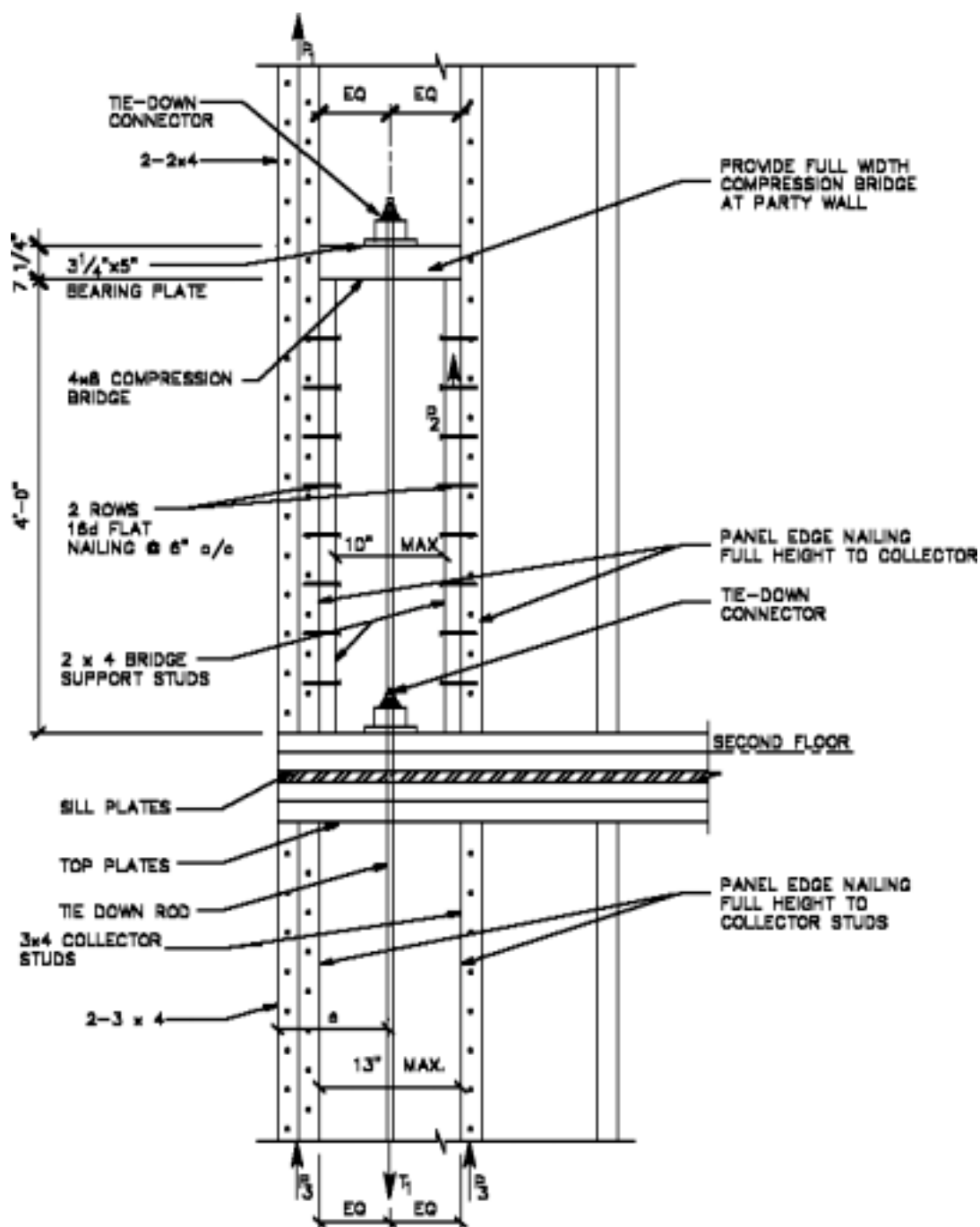


Figure 2-12. Tiedown connection at second floor for shear wall C

14.**Detail of wall intersection at exterior walls.**

The detail shows full-width studs at tiedown (Figure 2-13). This is desirable when sheathing is applied to both stud walls. It is also desirable for bearing perpendicular to grain because the bearing area is doubled. When full-width studs are used for bearing, both sill plates will need to be 3x thickness (*not* as shown in Figure 2-17). Tiedowns may be located at the center of the stud wall that is also sheathed. It is good practice to tie the wall together. In this case, there is no design requirement or minimum shear wall to shear wall connection requirement other than that required by the UBC standard nailing schedule.

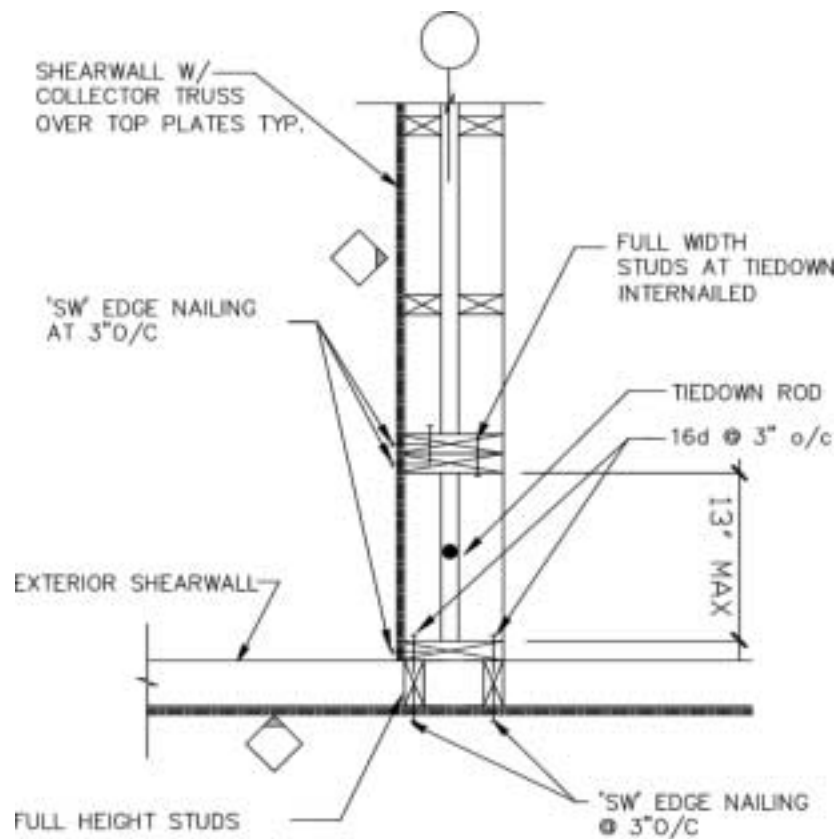


Figure 2-13. Wall intersection at shear wall (plan view)

15.**Detail of tiedown connection at foundation.**

The manufacturer of the tiedown system usually requires the engineer of record to specify the tiedown forces at each level of the structure. This can easily be done in a schedule (Figure 2-14).

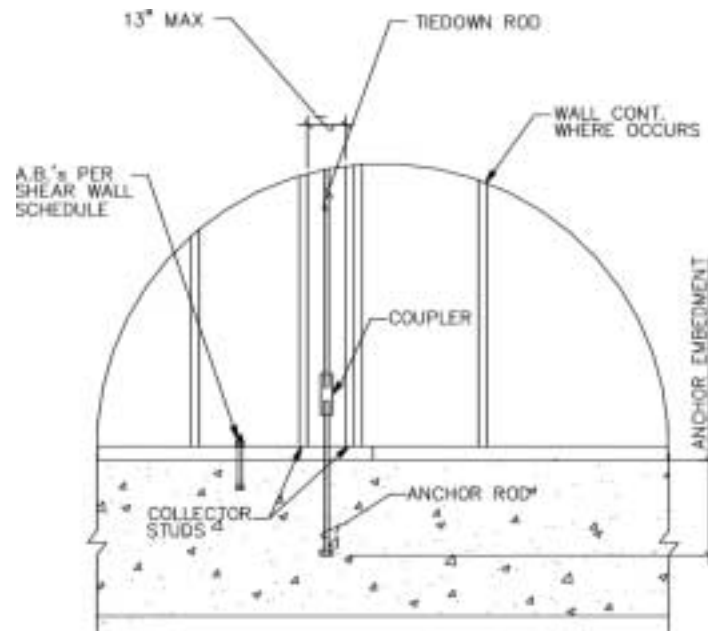


Figure 2-14. Tiedown connection at foundation

16.**Detail of shear transfer at interior shear wall at roof.**

Note: Edge nailing from roof sheathing to collector truss may need to be closer than the roof sheathing edge nailing due to shears being collected from each side of the truss. It is also common to use a double collector truss at these locations. The 2×4 braces at the top of the shear wall need to be designed for compression or provide tension bracing on each side of the wall (Figure 2-15).

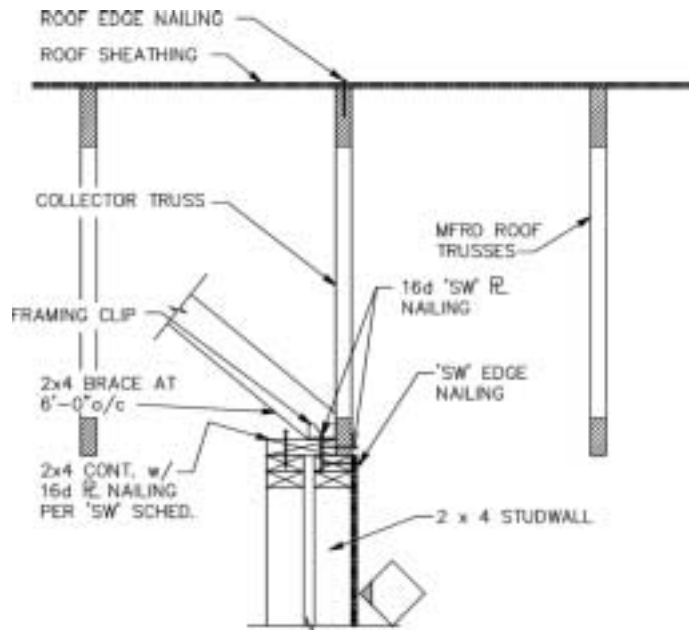


Figure 2-15. Shear transfer at interior shear wall at roof

17.**Detail of shear transfer at interior shear wall at floors.**

This detail uses the double top plates at the underside of the floor sheathing (Figure 2-16). This is advantageous for shear transfer. Another detail that is often used is to bear the floor joists directly on the top plates. However, when the floor joist is on top of the top plates, shear transfer is required through the glue joint in the webs and heavy nailing from the joist chord to the top plate.

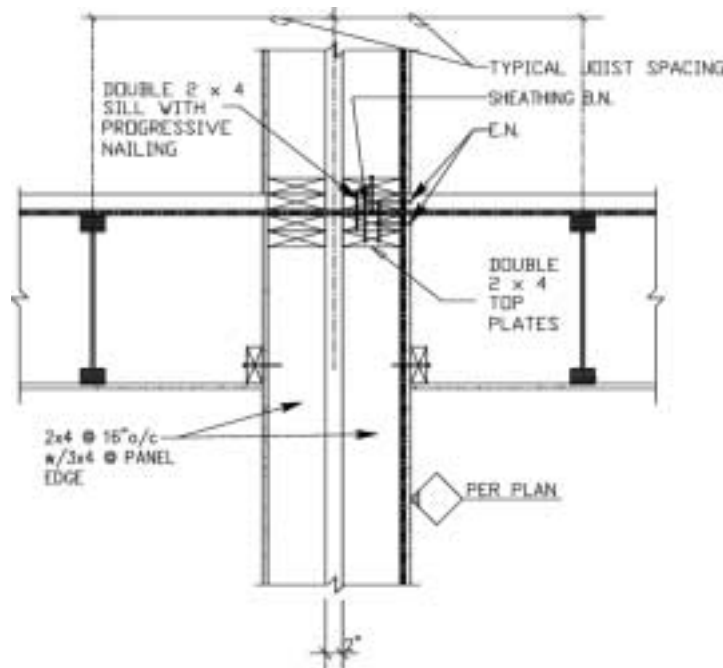


Figure 2-16. Shear transfer at interior shear wall at floor

Note: The nailers for the drywall ceiling need to be installed after the wall sheathing and wall drywall have been installed.

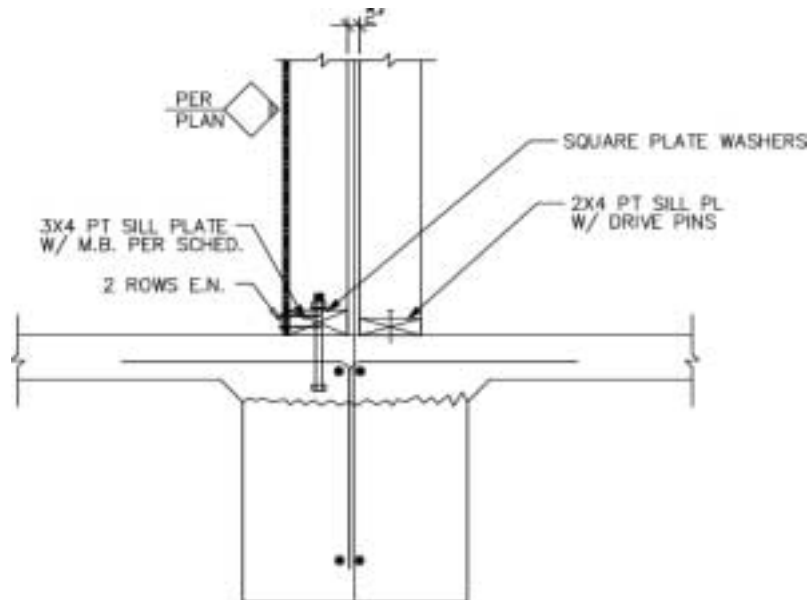
18.**Detail of shear transfer at interior shear wall at foundation.**

Figure 2-17. Shear transfer at foundation

19.**Detail of sill plate at foundation edge.****Fasteners for pressure- or preservative-treated wood.**

Sections 2304.3 and 1811.3 of the 1997 UBC added a new requirement for corrosion-resistant fasteners. Although it does not appear to be the intent of the provision, a literal interpretation of the section would require hot-dipped zinc-coated galvanized nails and anchor bolts. The code change was proposed by the wood industry, and §2304.3 is from a report in the wood handbook by the Forest Products Lab, where fasteners were found to react with the preservative treatment when "... in the presence of moisture..." However, it is uncertain whether a sill plate in a finished "dried-in" building is "in the presence of moisture." This can create a construction problem because hot-dipped zinc coated nails have to be hand-driven, requiring the framer to put down his nail gun and change nailing procedures.

An additional caution for sill plates is the type of wood used. The most common species used on the west coast for pressure treatment is Hem-Fir, which has lower fastener values for nails and bolts than for Douglas-Fir-Larch. A tighter nail spacing to the sill plate is necessary, or a double stagger row can be used. Figure 2-18 shows two rows of edge nailing to the sill plate as a method of compensating for a Hem-Fir sill plate.

Gap at bottom of sheathing.

Investigations into wood-framed construction have found that plywood or oriented strand board sheathing that bear on concrete at perimeter exterior edges can “wick” moisture up from the concrete and cause corrosion of the fasteners and rotting in the sheathing. To help prevent this problem, the sheathing can be placed with a gap above the concrete surface. A $\frac{1}{4}$ -inch gap is recommended for a 3x sill plate and an $\frac{1}{8}$ -inch gap is recommended for a 2x sill plate (Figure 2-18).

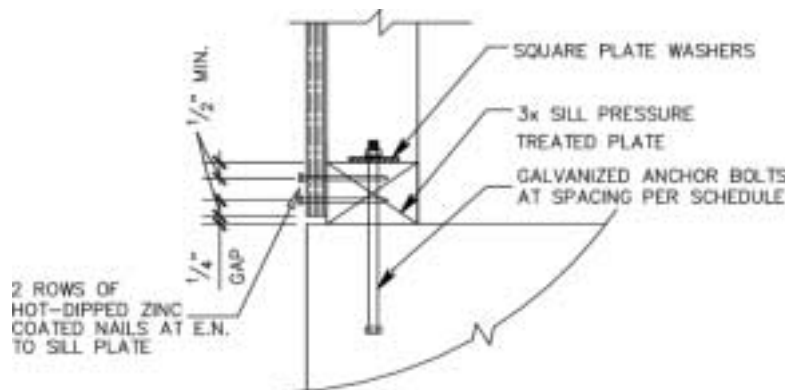


Figure 2-18. Sill plate at foundation edge

Note: The UBC only requires a minimum edge distance of $\frac{3}{8}$ -inch for nails in sheathing. Tests have shown that sheathing with greater edge distances have performed better.

20.

Detail of shear transfer at exterior wall at roof.

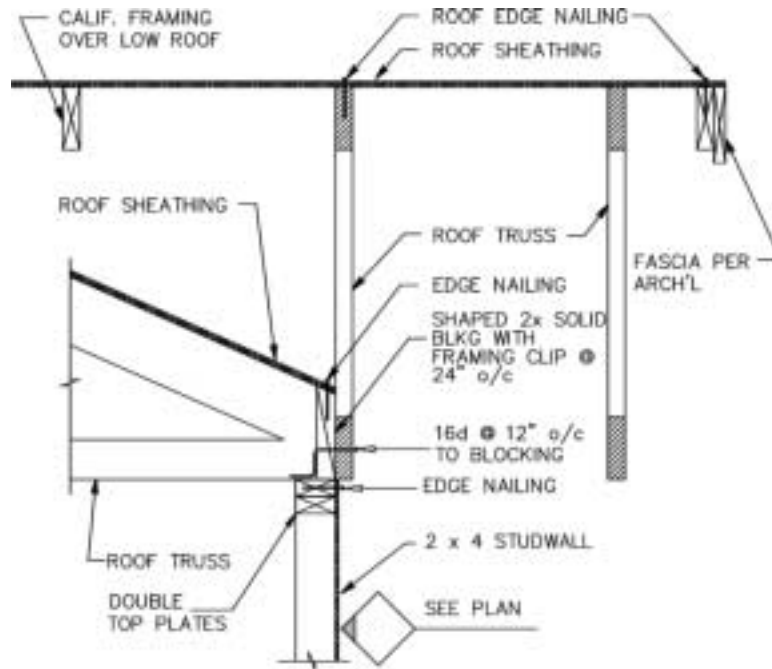


Figure 2-19. Shear transfer at exterior wall at roof

Note: The roof truss directly above the exterior wall is also a “collector” truss. Roof edge nailing to this truss and the 16d nails to the blocking need to be checked for the “collector” load. Double top plates are also a chord and collector.

21.**Detail of shear transfer at exterior wall at floor.**

NOTE: SHEATHING PANELS FOR WALLS SHALL NOT BE SPLICED
AT BOTTOM PLATE OF DOUBLE TOP PLATES.

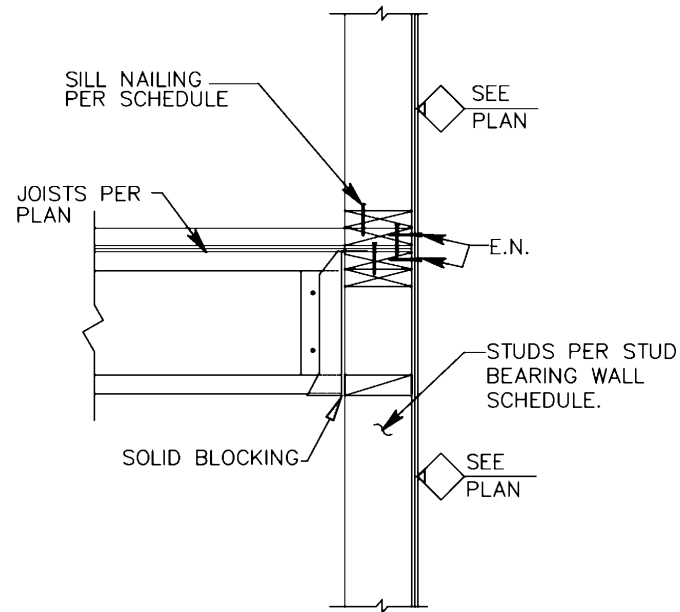


Figure 2-20. Shear transfer at exterior wall at floor

Note: This detail uses double top plates at the underside of the floor sheathing. Another detail that is often used is bearing the floor joists on the double top plates. See Figure 2-16 for additional commentary.

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Design Example 3

Cold-Formed Steel Light Frame Three-Story Structure

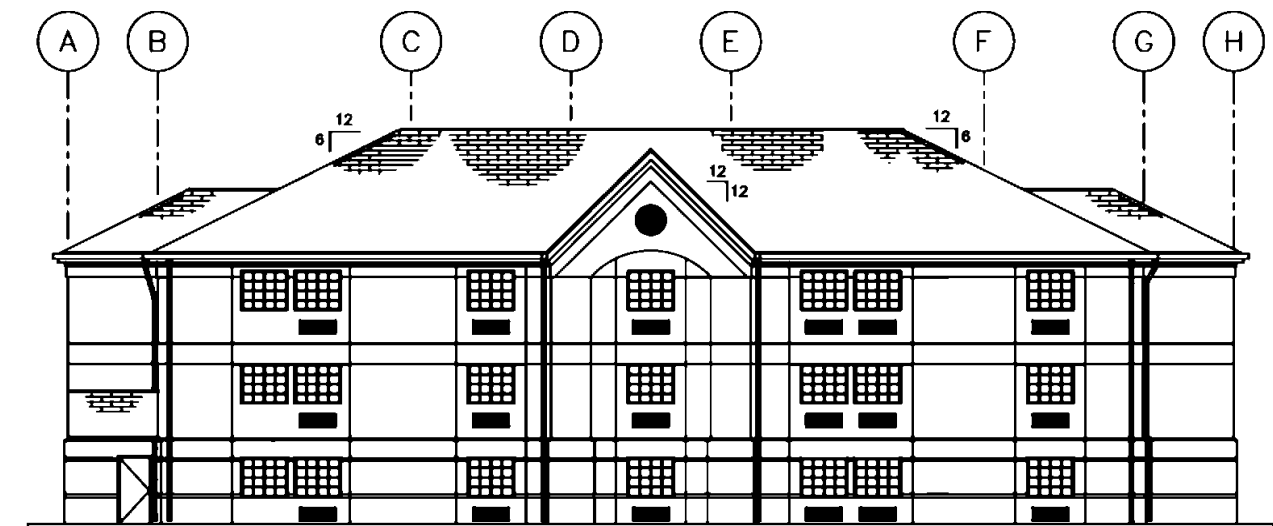


Figure 3-1. Cold-formed light frame three-story structure elevation

Foreword

The building in this example has cold-formed light-gauge steel framing, and shear walls and diaphragms that are sheathed with wood structural panels. This example presents a new approach to the seismic design of this type of building. This is because the past and present California design practice in seismic design of light framed structures has almost exclusively considered flexible diaphragm assumptions when determining shear distribution to shear walls. However, since the 1988 UBC, there has been a *definition* in the code (§1630.6 of the 1997 UBC) that defines diaphragm flexibility. The application of this definition often requires the use of the rigid diaphragm assumption, and calculation of shear wall rigidities for distribution to shear walls. While the latter is rigorous and complies with the letter of the code, it does not reflect present-day practice. In actual practice, for reasons of simplicity and precedence, many structural engineers routinely use the flexible diaphragm assumption.

A rigid diaphragm analysis is recommended where the shear walls can be judged by observation to be flexible compared to the diaphragm, and particularly where one or more lines of either shear walls, moment frames, or cantilever columns are more flexible than the rest of the shear walls.

This design example has floor diaphragms with lightweight concrete fill over the floor sheathing (for sound insulation), making the diaphragms significantly stiffer than that determined using the standard UBC diaphragm deflection equations.

Before beginning design, users of this Manual should check with the local jurisdiction regarding the level of analysis required for cold-formed light framed structures.

Overview

This design example illustrates the seismic design of a three-story cold-formed (i.e., light-gauge) steel structure. The structure is shown in Figures 3-2, 3-3 and 3-4. The building in this example is the same as in Design Example 2, with the exception that light-gauge metal framing is used in lieu of wood. The structure has wood structural panel shear walls, and roof and floor diaphragms. The roofs have composite shingles over the wood panel sheathing that is supported by light-gauge metal trusses. The floors have 1½ inches of lightweight concrete fill and are framed with metal joists.

The following steps illustrate a detailed analysis of some of the important seismic requirements of the 1997 UBC. As stated in the introduction of the manual, this example is not a complete building design. Many aspects have not been included, and only selected steps of the seismic design have been illustrated. As is common for Type V construction (see UBC §606), a complete wind design is also necessary, but is not given here.

Although code requirements recognize only two diaphragm categories, flexible and rigid, the diaphragms in this example are judged to be semi-rigid due to the fact that the diaphragms do deflect. The code also requires only one type of analysis, flexible or rigid. The analysis in this design example will use the *envelope* method. The envelope method considers the worst loading condition from both flexible and rigid diaphragm analyses to determine the design load on each shear-resisting element. It should be noted that the envelope method is not a code requirement, but is deemed appropriate for this design example, because neither flexible nor rigid diaphragm analysis may accurately model the structure.

Outline

This example will illustrate the following parts of the design process.

1. Design base shear and vertical distributions of seismic forces.
2. Rigidities of shear walls.
3. Distribution of lateral forces to the shear walls.
4. Reliability/redundancy factor ρ .
5. Tiedown forces for shear wall on line C.
6. Allowable shear and nominal strength of No. 10 screws.
7. Tiedown connection at third floor for wall on line C.
8. Tiedown connection at the second floor for shear wall on line C.
9. Boundary studs for first floor wall on line C.
10. Shear transfer at second floor on line C.
11. Shear transfer at foundation for walls on line C.
12. Shear transfer at roof at line C.

Given Information

Roof weights (slope 6:12):

Roofing	3.5 psf
½" sheathing	1.5
Trusses	3.5
Insulation	1.5
Miscellaneous	0.7
Gyp ceiling	<u>2.8</u>
DL (along slope)	13.5 psf

Floor weights:

Flooring	1.0 psf
Lt. wt. concrete	14.0
5/8" sheathing	1.8
Floor Framing	5.0
Miscellaneous	0.4
Gyp ceiling	<u>2.8</u>
	25.0 psf

DL (horiz. proj.) = $13.5 (3.41/12) = 15.1$ psf

Stair landings do not have lightweight concrete fill

Area of floor plan is 5,288 sq ft

Weights of respective diaphragm levels, including tributary exterior and interior walls:

$$W_{roof} = 135,000 \text{ lb}$$

$$W_{3rd \text{ floor}} = 230,000 \text{ lb}$$

$$\begin{aligned} W_{2nd \text{ floor}} &= 230,000 \text{ lb} \\ &= \overline{595,000 \text{ lb}} \end{aligned}$$

The same roof, floor, and wall weights used in Design Example 2 are also used in this example. This has been done to better illustrate a side-by-side comparison of cold-formed light-gauge steel construction with the more traditional wood frame construction used in Design Example 2. This side-by-side comparison has been done so that the engineer can have a better “feel” for the similarities and differences between structures with wood studs and structures with cold-formed metal studs. It should be noted that roof, floor, and wall weights for light-gauge steel framed structures are typically lighter than similar structures constructed of wood framing. Because of light-gauge steel framed structures being lighter, a more accurate estimate of building weight for this structure would be about 560 kips instead of the 595 kips used in this example. Consequently, wall shears and overturning forces would be reduced accordingly.

Weights of diaphragms are typically determined by taking one-half height of walls at the third floor to the roof and full height of walls for the third and second floors diaphragms.

Wall framing is ASTM A653, grade 33'-4"×18-gauge metal studs at 16 inches on center. These have a 1-5/8-inch flange with a 3/8-inch return lip. The ratio of tensile strength to yield point is at least 1.08. Studs are painted with primer. ASTM A653 steel is one of three ASTM steel specifications used in light frame steel construction. The others are A792 and A875. The difference between the specifications are primarily the coatings which are galvanized, 55 percent aluminum-zinc (A792), and zinc-5 percent aluminum (A875) respectively. The recommended minimum coating classifications are G60, AZ50 and GF60 respectively. It should be noted that the studs do not require painting with primer.

It should be noted that the changing stud sizes or thickness of studs at various story heights is common (as is done in wood construction). The thickness of studs and tracks should be identified by visible means such as coloring or metal stamping of gauges/sizes on studs and tracks.

APA-rated wood structural panels for shear walls will be 15/32-inch-thick Structural I, 32/16-span rating, 5-ply with Exposure I glue is specified, however 4-ply is also acceptable.

Framing screws are No. 8 by 5/8-inch wafer head self-drilling with a minimum head diameter of 0.292-inch, as required by footnote 2 of Table 22-VIII-C of the UBC.

The roof is 15/32-inch thick APA-rated sheathing, 32/16-span rating with Exposure I glue.

The floor is 19/32-inch thick APA-rated Sturd-I-Floor 24" o/c rating (or APA-rated sheathing, 48/24-span rating) with Exposure I glue.

Seismic and site data:

$Z = 0.4$ (Zone 4)

Table 16-I

$I = 1.0$ (standard occupancy)

Table 16-K

Seismic source type = B

Distance to seismic source = 12 km

Soil profile type = S_C

S_C has been determined by geotechnical investigation. Without a geotechnical investigation, S_D can be used as a default value.

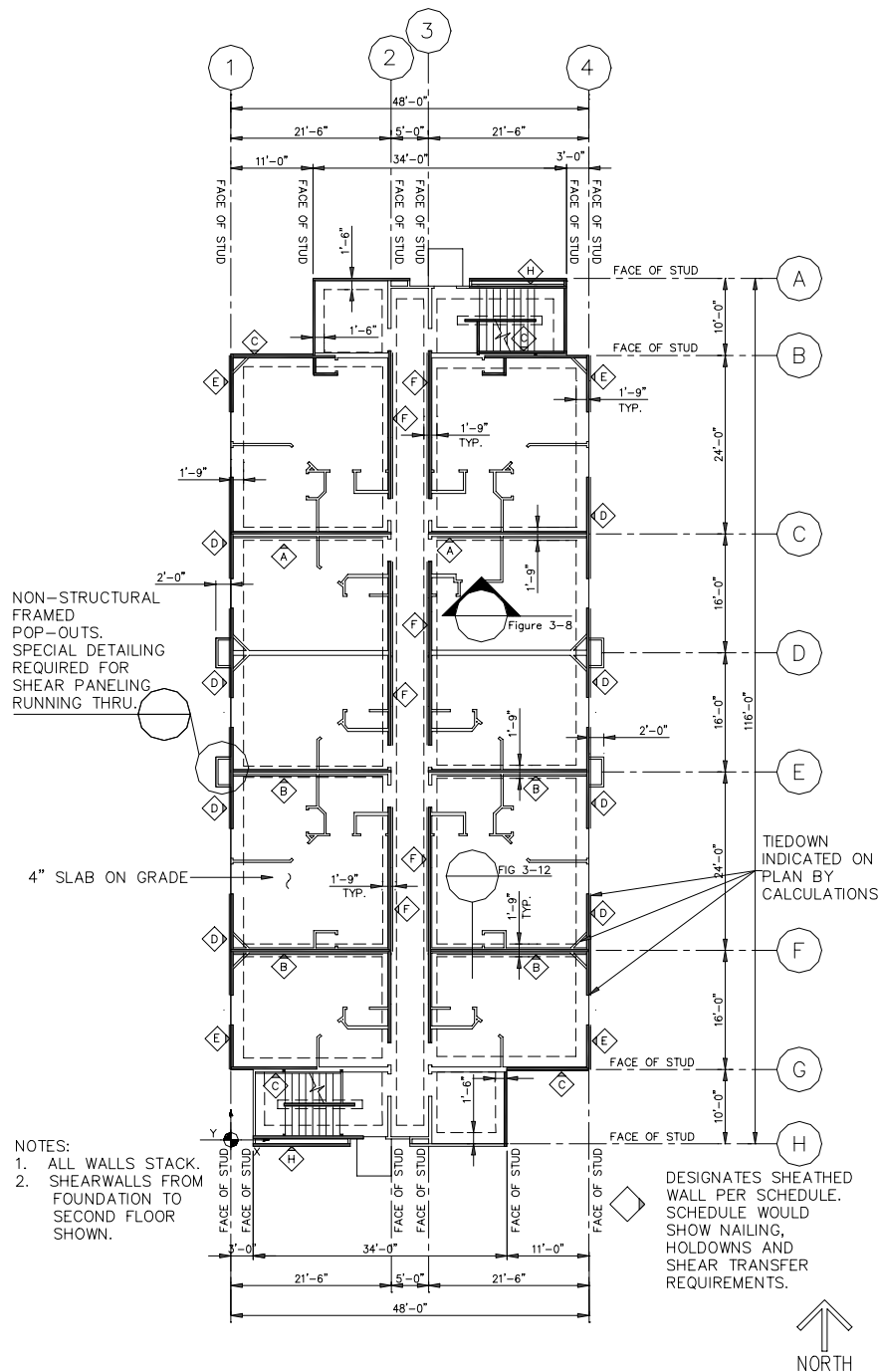
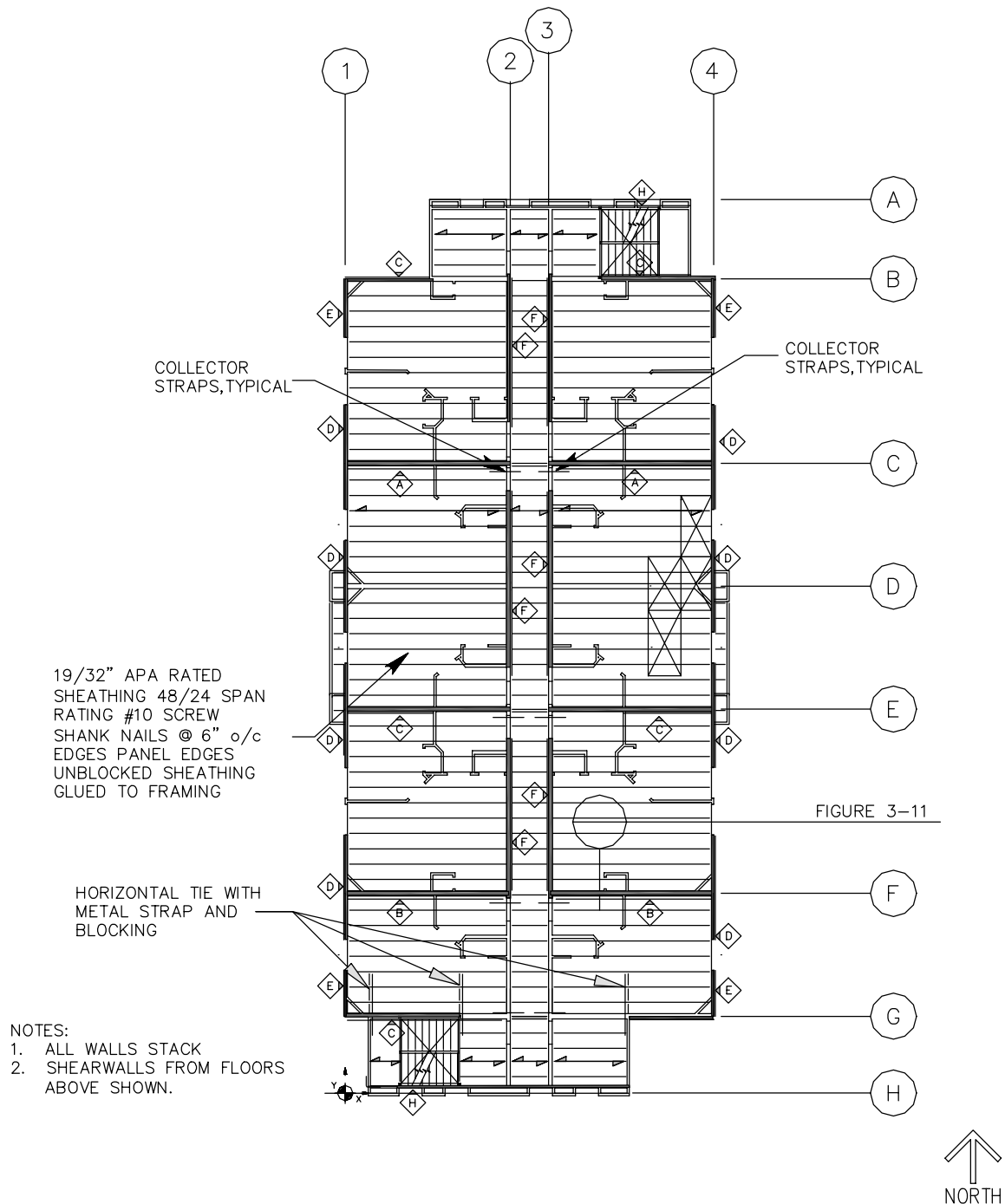


Figure 3-2. Foundation plan (ground floor)



Note: Shear walls on lines 2 and 3 do not extend from the third floor to the roof.

Figure 3-3. Floor framing plan (second and third floors)

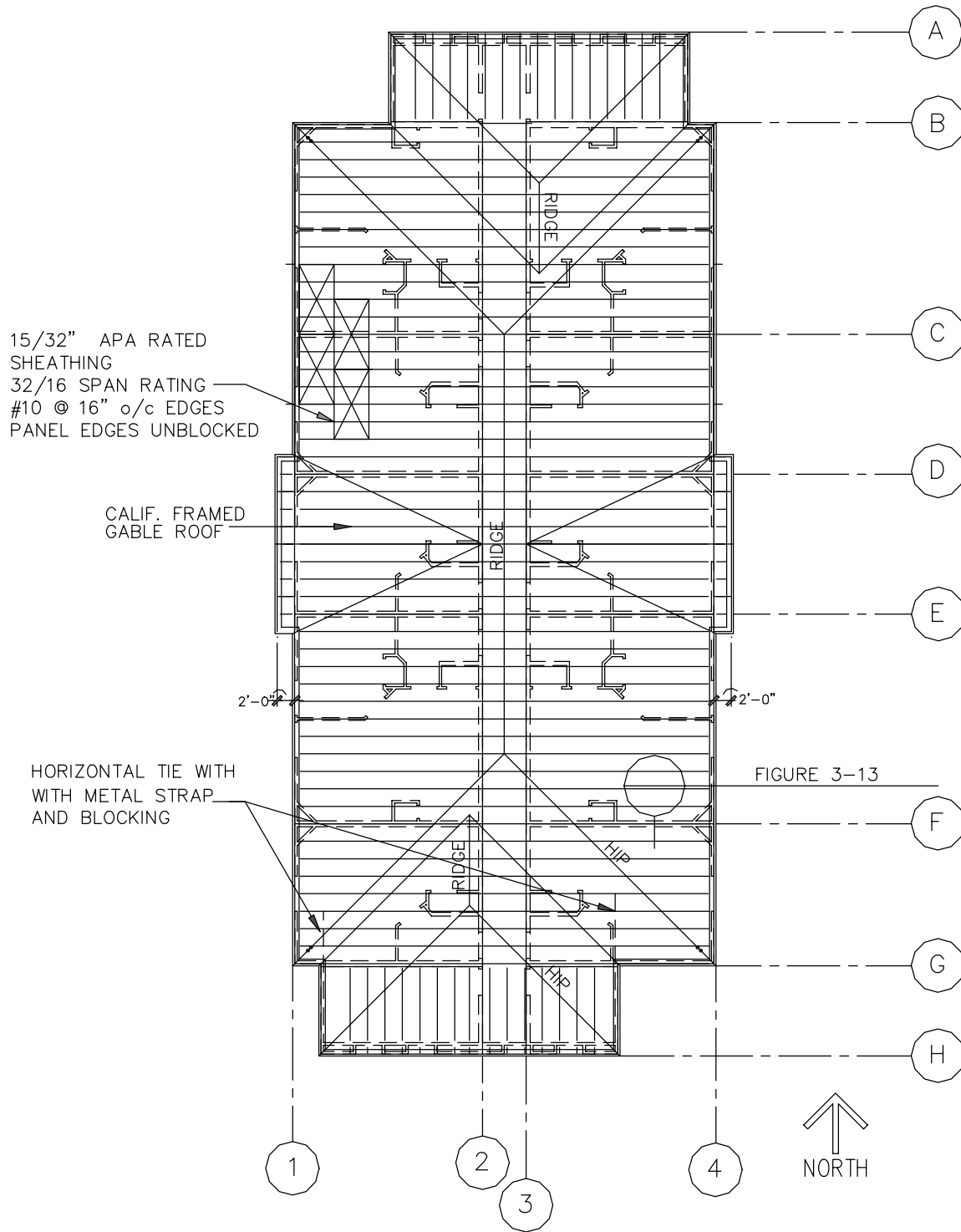


Figure 3-4. Roof framing plan

Factors That Influence Design

Requirements for seismic design of cold-formed steel stud wall systems are specified in Division VIII of the UBC. Division VIII is a new addition to the UBC and it contains information previously found in §2211.11 of the 1994 UBC relating to seismic design. Division VIII has provisions for both wind and seismic forces for shear walls with wood structural panels framed with cold-formed steel studs. The tables for shear walls (Tables 22-VIII-A, 22-VIII-B and 22-VIII-C) are primarily based on static and cyclic tests conducted by the Light-gauge Steel Research Group at the Santa Clara University Engineering Center for the American Iron and Steel Institute (AISI).

Before starting the example, several important aspects of cold-formed construction will be discussed. These are:

- Stud thickness
- Screw type
- Material strength
- Use of pre-manufactured roof trusses to transfer lateral forces
- Proper detailing of shear walls at building “pop-outs”
- AISI Specification for design of cold-formed steel

Stud thickness.

Section 2220.3 of Division VIII states that the uncoated base metal thickness for the studs used with wood structural panels shall not be greater than 0.043-inch. Since an 18-gauge stud has 0.0451-inch thickness, this implies that the heaviest gauge studs that can be used are 20-gauge studs, which can not support a significant bearing or out-of-plane loading. At the time the code change proposal by AISI was submitted to ICBO for inclusion in the 1997 UBC, testing had been performed on only 33 mil (0.033-inch) studs. The SEAOC Seismology Committee felt, and AISI agreed, that there should be a cap on the maximum thickness permitted until testing could be performed on thicker studs. It was felt at the time that limiting the system to 20- and 18-gauge studs would be acceptable for attaching sheathing with #8 screws. Since the UBC is no longer referencing gauge, the 0.043-inch thickness was intended to be a nominal thickness. Subsequent to the code change proposal, AISI has modified this limitation by taking the average thickness between the old 18- and 16-gauges and placed a limitation of 0.043-inch in the AISI code. The 0.043-inch thickness represents 95 percent of the design thickness and is the minimum acceptable thickness delivered to the job site for 18-gauge material based on Section A3.4 of the 1996 AISI Code. Thus, 18-gauge studs can be used, and are used in this example (Table 3-1).

The industry has gone away from the use of the gauge designation and is, for the purposes of framing applications, switching to a mil (thousandths of an inch)

designation. In the future, studs, joists, and track will have their thickness expressed in mils.

Table 3-1. Stud thicknesses

Mils	Min. Delivered Thickness	Min. Design Thickness	Gauge Reference
33	0.033 inch	0.0346 inch	20
43	0.043 inch	0.0451 inch	18
54	0.054 inch	0.0566 inch	16

The reason for the limitation on maximum stud thickness of 43 mils (18-gauge) is for ductility. At the time of this publication (March 2000), the cyclic tests to date of wood structural panels fastened to 16- and 14-gauge studs with screws have shown nonductile (brittle) failures with the screws shearing off at the face of the stud flange. Cyclic tests for the 20- and 18-gauge studs resulted in ductile behavior with the screw fasteners rocking (tilting) about the plane of the stud flange. Tests are still being conducted by AISI and other organizations on wall systems using the thicker 16- and 14-gauge studs in an attempt to come up with a fastening system that will be ductile.

The failure mode of the tests with 33-mil studs for screw spacings of 3 inches and 2 inches on center was end stud *compression* failure. Subsequent to the code change proposal included in the 1997 UBC, the assemblies have been retested using 43 mil end studs, and higher capacities have been proposed for such assemblies.

The values in Table 22-VIII-C are for seismic forces and are *nominal* shear values. Values are to be modified for *both* allowable stress design (ASD) and load and resistance factor design (LRFD or strength design). For ASD, the allowable shear values are determined by dividing the nominal shear values by a factor of safety (Ω) of 2.5. For LRFD the design shear values are determined by multiplying the nominal shear values by a resistance factor (ϕ) of 0.55. Comparing the difference to the two designs: $2.5(0.55)=1.375$. In other words, design shears for LRFD (or strength design) are 1.375 times higher than shears for ASD or working stress design. This is consistent with the ASD conversion factor of 1.4 in §1612.3.

The values in Table 22-VIII-C for 15/32-inch Structural I sheathing using No. 8 screws are almost identical to the values for the same sheathing applied to Douglas Fir with 8d common nails at the same spacing.

Screw type.

Footnote 2 of UBC Table 22-VIII-C requires the framing screws to be *self-drilling*. The reason for the self-drilling screws (or drill point screws) is to be able to penetrate 43-mil steel and thicker steel. Self-piercing screws can also be used in

33-mil steel, but with some difficulty. Both self-drilling and self-piercing screws have performed equally well in the shear tests.

There is a significant concern in screw installation when there is a gap between the stud flange and the sheathing after installation (e.g., jacking). When jacking occurs, the stiffness of the shear wall is *significantly* reduced. The drill point alone will not prevent jacking. Jacking occurs when the drill point spins for a rotation or two before the drill point pierces the metal. Only a blank shaft (i.e. smooth with no threads) for the depth of the sheathing will remove the jacking created by the drill point spin prior to piercing. A detailed drawing or explicit specifications should be included in the design drawings and should specify that the distance from the screw head to the beginning of the thread portion be equal to or less than the thickness of the plywood or OSB (oriented strand board). The “unused portion” of the screw protruding from the connection of sheathing and metal stud can be used as a simple inspection gauge to see if jacking has occurred.

Material strength.

Common practice is for material 16-gauge and heavier to have a yield strength of 50,000 psi; for 18-gauge and lighter, 33,000 psi. This practice holds true for studs and track, but not for manufactured hardware (straps, clips and tiedown devices).

Use of pre-manufactured roof trusses to transfer lateral forces.

The structural design in this design example utilizes pre-manufactured roof trusses to transfer the lateral forces from the roof diaphragm to the tops of the interior shear walls. Special considerations need to be included in the design and detailed on the plans for this including:

1. Provision that any trusses used as collectors (i.e., drag struts) should be clearly indicated on the structural framing plan.
2. The magnitude of the forces, the means by which the forces are applied to the trusses, and how the forces are transferred from the trusses to the shear walls should be shown.
3. If the roof sheathing at the hip ends breaks above the joint between the end jack trusses and the supporting girder truss, the lateral forces to be resisted by the end jacks should be specified so that an appropriate connection can be provided to resist these forces.
4. The drawings should also specify the load combinations and whether or not a stress increase is permitted.
5. If ridge vents are being used, special detailing for shear transfers need to be indicated in the details.

Proper detailing of shear walls at building “pop-outs. ”

The structure for this design example has double framed walls for party walls, exterior “planted-on” box columns (pop-outs). The designer should not consider these walls as shear walls unless special detailing and analysis is provided to substantiate that there is a viable lateral force path to that wall and the wall is adequately braced.

AISI Specification for design of cold-formed steel.

The code uses the 1986 version of AISC *Specification for Design of Cold-Formed Steel Structural Members* as an adopted “Standard” by reference (UBC §2217). Section 2218 amends the 1986 manual. These for the most part are from the 1996 version of the manual. Some sections of the 1996 sections have been used for the solution of this design example.

Calculations and Discussion	Code Reference
<div style="border: 1px solid black; padding: 2px; display: inline-block; width: 30px; text-align: center;">1.</div> Design base shear and vertical distributions of seismic forces.	§1630.2.2
<div style="border: 1px solid black; padding: 2px; display: inline-block; width: 30px; text-align: center;">1a.</div> Design base shear.	
Period using Method A (See Figure 3-5 for section through structure):	
$T = C_t (h_n)^{3/4} = .020(33.63)^{3/4} = 0.28 \text{ sec}$	
With seismic source type <i>B</i> and distance to source = 12 km	
$N_a = 1.0$	Table 16-S
$N_v = 1.0$	Table 16-T

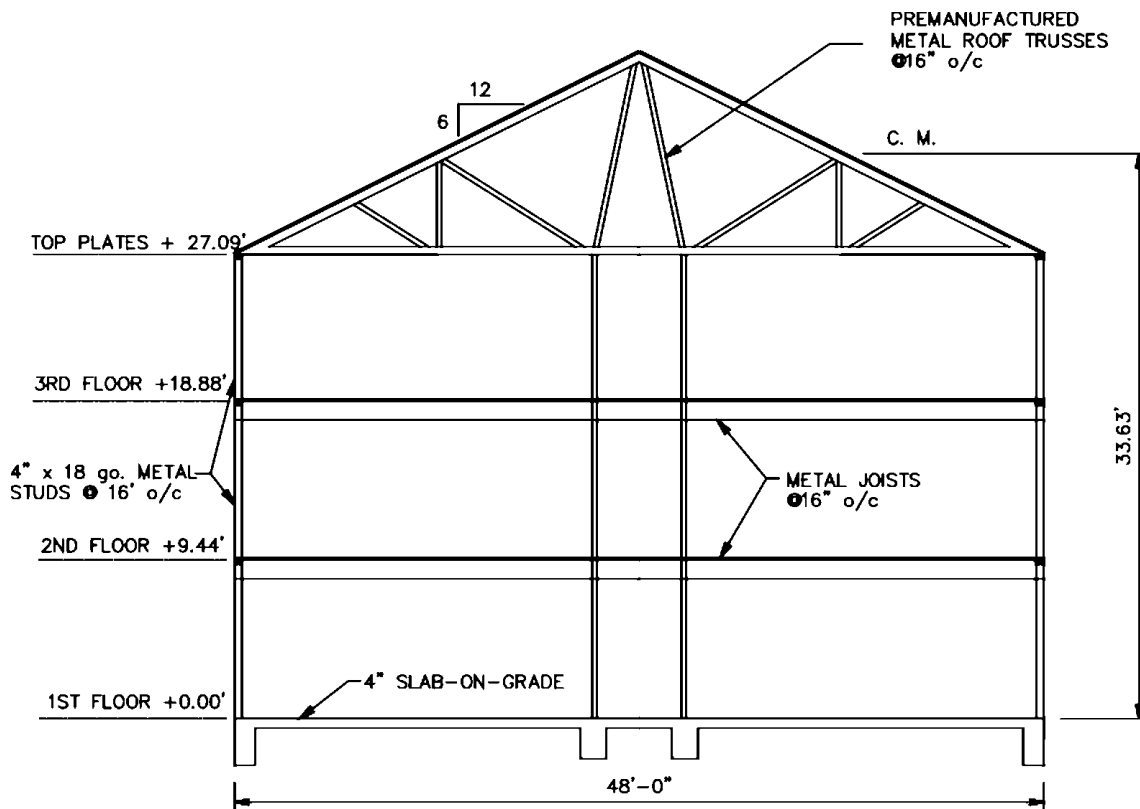


Figure 3-5. Typical cross section through building

For soil profile type S_C and $Z = 0.4$

$$C_a = 0.40 N_a = 0.40(1.0) = 0.40$$

Table 16-Q

$$C_v = 0.56 N_v = 0.56(1.0) = 0.56$$

Table 16-R

Since the stud walls are both wood structural panel shear walls and bearing walls:

$$R = 5.5$$

Table 16-N

Design base shear is:

$$V = \frac{C_v I}{RT} W = \frac{0.56(1.0)}{5.5(0.28)} W = 0.364W \quad (30-4)$$

Note that design base shear is now on a strength design basis, but need not exceed:

$$V = \frac{2.5C_a I}{R} W = \frac{2.5(0.40)(1.0)}{5.5} W = \underline{\underline{0.182W}} \quad (30-5)$$

$$V = 0.11C_a I W = 0.11(0.40)(1.0)W = 0.044W < 0.182W$$

Check Equation 30-7:

$$V = \frac{0.8ZN_v I}{R} W = \frac{0.8 \times 0.4 \times 1.0 \times 1.0}{5.5} W = 0.058W < 0.182W$$

$$V = 0.182W \quad \S 1612.3.1$$

$$\therefore V = 0.182(595,000 \text{ lbs}) = \underline{\underline{108,290 \text{ lb}}}$$

In this Design Example, the designer may choose either allowable stress design or strength design. In Design Example 2, however, allowable strength design must be used.

It is desirable to use the strength level forces throughout the design of the structure for two reasons:

1. Errors in calculations can occur and confusion on which load is being used, strength or allowable stress design. This Design Example uses the following format:

$$V_{base\ shear} = \text{strength}$$

$$F_{px} = \text{strength}$$

$$F_x = \text{force-to-wall strength}$$

$$v = \text{wall shear at element level - ASD}$$

$$v = \frac{F_x}{1.4b} = \text{ASD}$$

2. This design example is not paving the way for the future, when the code will be all strength design.

$$E = \rho E_h + E_v = 1.0E_h + 0 = 1.0E_h \quad (30-1)$$

where:

E_v is permitted to be taken as zero for allowable stress design and initially ρ will be assumed to be 1.0, and in most cases $\rho = 1.0$ for Type V construction with interior shear walls. Since the maximum element story shear is not yet known, the assumed value for ρ will have to be verified. This is done later in Part 4.

The basic load combination for allowable stress design for horizontal forces is:

$$D + \frac{E}{1.4} = 0 + \frac{E}{1.4} = \frac{E}{1.4} \quad (12-9)$$

For vertical downward:

$$D + \frac{E}{1.4} \text{ or } D + 0.75 \left[L + (L_r \text{ or } S) + \frac{E}{1.4} \right] \quad (12-10, 12-11)$$

For vertical uplift:

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

1b.

Vertical distribution of forces.

The design base shear must be distributed to each level, as follows:

$$F_{px} = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^n w_i h_i} \quad (30-15)$$

Where h_x is the average height at level i of the sheathed diaphragm in feet above the base.

Since $T = 0.28$ seconds < 0.7 seconds, $F_t = 0$

Determination of F_{px} is shown in Table 3-2.

Table 3-2. Vertical distribution of seismic forces

Level	w_x (k)	h_x (ft)	$w_x h_x$ (k-ft)	$\frac{w_x h_x}{\sum w_i h_i}$ (%)	F_{px} (k)	$\frac{F_{px}}{w_x}$	F_{tot} (k)
Roof	135	33.6	4,536	41.1	44.5	0.330	44.5
3 rd Floor	230	18.9	4,347	39.4	42.7	0.186	87.2
2 nd Floor	230	9.4	2,162	19.5	21.1	0.092	108.3
Σ	595		11,045		108.3		

Note: Although not shown here, designers must also check wind loading. In this example, wind load may control the design in the east-west direction.

2.

Rigidities of shear walls.

2a.

Deflection of panel assemblies with metal studs.

At the time of this publication, there is not a UBC formula, nor any accepted guideline, for determining the deflection for a diaphragm or shear wall framed with metal studs and structural wood panels. This does not mean that the deflections, drifts, and shear wall rigidities need not be considered (though some engineers may argue otherwise).

The formula in UBC Standard §23.223, Vol. 3, can be used with somewhat reasonable results. Given below is a comparison of results from shear panel tests conducted by the Light-gauge Steel Research Group and those determined using the UBC formula.

For an 8 ft × 8 ft test panel with 15/32-inch APA-rated sheathing and #8 screw fasteners at 6-inch spacing to 3½-inch × 20-gauge studs and 485 pounds per foot shear, the measured deflection was 0.5 inch.

In this Design Example 3, 4-inch × 18-gauge studs are used. Tests have indicated that measured deflections are partially dependent on the stiffness of the studs used. The shear panel test results should not be compared to the nominal shear values from UBC Table 22-VIII-C. Using this table would give an allowable shear of $780/2.5 = 312$ plf. This panel test is used *only* to show the relationship of the measured deflection with results using the UBC formula.

Deflection using the formula of UBC standard §23.223, Vol. 3 is shown below:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a = 0.40 \text{ in.} \approx 0.50 \text{ in. as tested} \quad \text{§23.223, Vol. 3}$$

where:

$$v = 485 \text{ plf}$$

$$h = 8 \text{ ft}$$

$$E = 29 \times 10^6 \text{ psi}$$

$$A = 0.250 \text{ in.}^2 \text{ for } 3 \frac{1}{2} \text{ - inch} \times 20 \text{ gauge stud}$$

$$G = 90,000 \text{ psi}$$

Table 23-2-J, Vol. 3

$$t = 0.298 \text{ in.}$$

Table 23-2-H, Vol. 3

$$V_n = \text{load per screw} = (485)6/12 = 242 \text{ lb/screw}$$

$$e_n = 1.2(242/769)^{3.276} = 0.0272 \text{ in.}$$

$$b = 8 \text{ ft}$$

$$d_a = 0.0625 \text{ in (assumed at } 1/16 \text{ in.)}$$

2b.

Calculation of shear wall rigidities.

In this Design Example 3, shear wall rigidities (k) are computed using the basic stiffness equation.

$$F = k\Delta$$

or

$$k = \frac{F}{\Delta}$$

To simplify the calculations compared to the more rigorous approach used in Design Example 2, this example uses wall rigidities based on the chart in Figure 3-6. This chart is based on the shear wall deflection equation given in UBC Standard §23.223. It should be noted that Design Example 2 considered wood shrinkage and tiedown displacements. With metal framing, shrinkage is zero. This Design Example also assumes a fixed base and pinned top for all shear walls. The chart in Figure 3-6 uses a tiedown displacement (e.g., elongation) of 1/8 inch, which is based on judgment and considered appropriate for this structure.

Actual determinations of shear wall rigidities at the roof, third floor, and second floor are shown in Figures 3-3, 3-4, and 3-5, respectively.

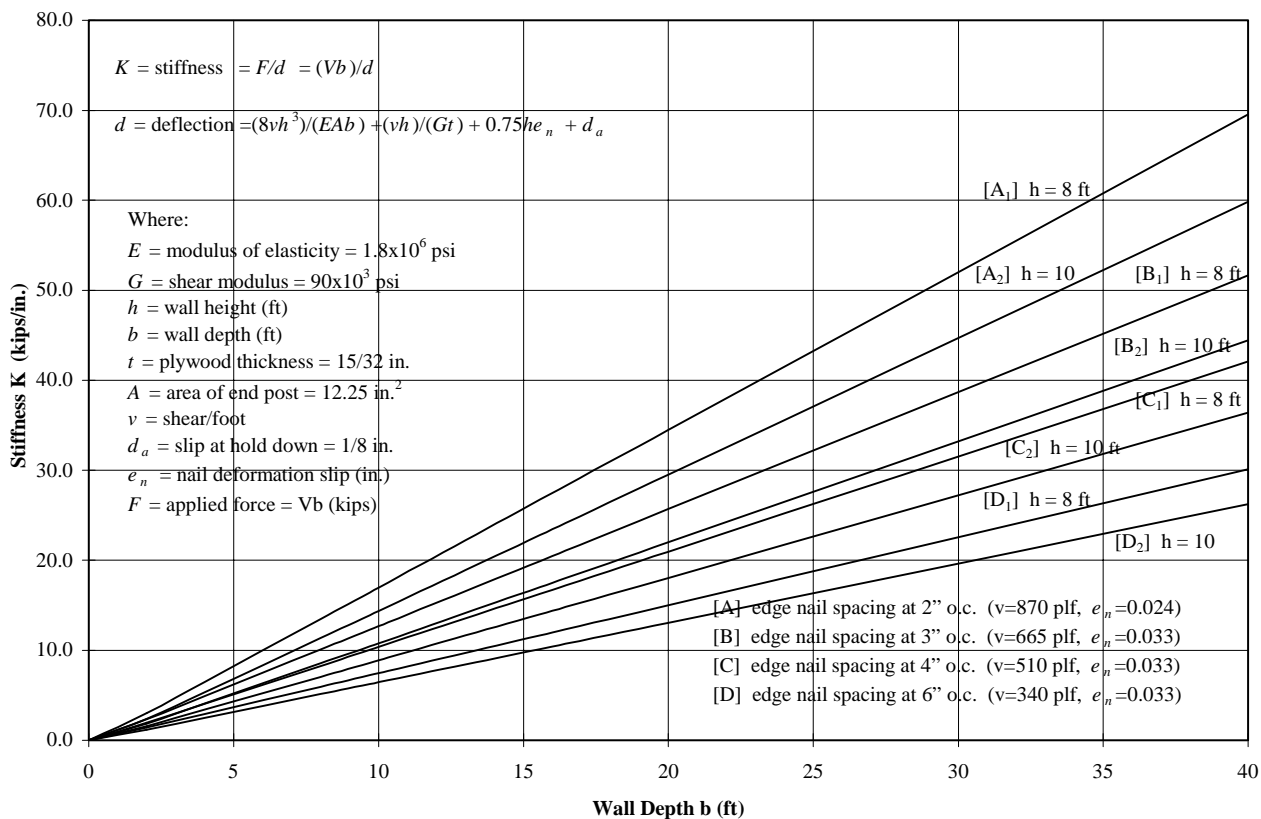


Figure 3-6. Stiffness of one-story Structural-I 15/32-inch plywood shear walls

Table 3-3. Shear wall rigidities at roof level

Wall	Wall Depth <i>b</i> (ft)	Edge Fastener Spacing (in.)	<i>k</i> (From Fig. 3— 6) (k/in.)	<i>k</i> _{total} (k/in.)
A	12.5	6	8.0	8.0
B1	11.0	6	7.5	—
B2	11.0	6	7.5	—
B	—	—	15.0	15.0
C1	21.5	6	15.0	—
C2	21.5	6	15.0	—
C	—	—	30.0	30.0
E1	21.5	6	15.0	—
E2	21.5	6	15.0	—
E	—	—	30.0	30.0
F1	21.5	6	15.0	—
F2	21.5	6	15.0	—
F	—	—	30.0	30.0
G1	11.0	6	7.5	—
G2	11.0	6	7.5	—
G	—	—	15.0	15.0
H	12.5	6	8.0	8.0
1a, 4a	8.0	6	6.0	—
1b, 4b	14.0	6	10.0	—
1c, 4c	11.5	6	8.0	—
1d, 4d	11.5	6	8.0	—
1e, 4e	11.5	6	8.0	—
1f, 4f	8.0	6	6.0	—
1, 4	—	—	46.0	46.0

Table 3-4. Shear wall rigidities at third floor

Wall	Wall Depth b (ft)	Edge Fastener Spacing (in.)	k (From Fig. 3-6) (k/in.)	k_{total} (k/in.)
A	12.5	6	8.0	8.0
B1	11.0	4	10.0	—
B2	11.0	4	10.0	—
B	—	—	20.0	20.0
C1	21.5	4	19.0	—
C2	21.5	4	19.0	—
C	—	—	38.0	38.0
E1	21.5	4	19.0	—
E2	21.5	4	19.0	—
E	—	—	38.0	38.0
F1	21.5	4	19.0	—
F2	21.5	4	19.0	—
F	—	—	38.0	38.0
G1	11.0	4	10.0	—
G2	11.0	4	10.0	—
G	—	—	20.0	20.0
H	12.5	6	8.0	8.0
1a, 4a	8.0	4	7.0	—
1b, 4b	14.0	4	12.0	—
1c, 4c	11.5	4	10.0	—
1d, 4d	11.5	4	10.0	—
1e, 4e	11.5	4	10.0	—
1f, 4f	8.0	4	7.0	—
1, 4	—	—	56.0	56.0
2a, 3a	18.0	6	12.0	—
2b, 3b	24.0	6	15.0	—
2c, 3c	18.0	6	12.0	—
2, 3	—	—	39.0	39.0

Table 3-5. Shear wall rigidities at second floor

Wall	Wall Depth b (ft)	Edge Fastener Spacing (in.)	k (From Fig. 3-6) (k/in.)	k_{total} (k/in.)
A	12.5	6	8.0	8.0
B1	11.0	3	11.0	—
B2	11.0	3	11.0	—
B	—	—	22.0	22.0
C1	21.5	3	22.5	—
C2	21.5	3	22.5	—
C	—	—	45.0	45.0
E1	21.5	3	22.5	—
E2	21.5	3	22.5	—
E	—	—	45.0	45.0
F1	21.5	3	22.5	—
F2	21.5	3	22.5	—
F	—	—	45.0	45.0
G1	11.0	3	11.0	—
G2	11.0	3	11.0	—
G	—	—	22.0	22.0
H	12.5	6	8.0	8.0
1a, 4a	8.0	4	7.0	—
1b, 4b	14.0	4	12.0	—
1c, 4c	11.5	4	10.0	—
1d, 4d	11.5	4	10.0	—
1e, 4e	11.5	4	10.0	—
1f, 4f	8.0	4	7.0	—
1, 4	—	—	56.0	56.0
2a, 3a	18.0	6	12.0	—
2b, 3b	24.0	6	15.0	—
2c, 3c	18.0	6	12.0	—
2, 3	—	—	39.0	39.0

2c.**Determination of the design level displacement Δ_s .****§1630.9.1**

For both strength and allowable stress design, the UBC now requires building drifts to be determined by the load combinations of §1612.2, these being the load combinations that use strength design, or LRFD. An errata for the second and third printing of the UBC unexplainably referenced §1612.3 for allowable stress design. The reference to §1612.3 (Allowable Stress Design) is *incorrect* and will be changed back to reference §1612.2 (Strength Design) in the fourth and later printings.

Shear wall displacements for a structures of this type (generally) are well below the maximum allowed by code and the computation of these displacements is

considered not necessary. Refer to Design Example 2 for an illustration of this procedure.

3.

Distribution of lateral forces to the shear walls.

§1630.6

In this part, story shears are distributed to shear walls with the diaphragms assumed to be rigid. (Refer to Design Example 2 for a code confirmation of the applicability of this assumption).

It has been common practice for engineers to *assume* flexible diaphragms and distribute loads to shear walls based upon tributary areas. The procedures used in this Design Example 3 are not intended to imply that seismic design of light frame construction in the past should have been performed in this manner. Recent earthquakes and testing of wood panel shear walls have indicated that drifts can be considerably higher than what was known or assumed in the past. Knowledge of the increased drifts of short wood panel shear walls has increased the need for the engineer to consider relative rigidities of shear walls.

Section 1630.6 requires the center of mass (CM) to be displaced from the calculated center of mass a distance of 5 percent of the building dimension at that level perpendicular to the direction of force. Section 1630.7 requires the most severe load combination to be considered and also permits the negative torsional shear to be subtracted from the direct load shear. The net effect of this is to add 5 percent accidental eccentricity to the actual eccentricity.

The direct shear force F_{vi} in wall i is determined from:

$$F_{vi} = F \frac{R}{\sum R}$$

and the torsional shear force F_{ti} in wall i is determined from:

$$F_{ti} = T \frac{R_i d_i}{J}$$

where:

i = wall number

$$J = \sum R d_x^2 + \sum R d_y^2$$

R = shear wall rigidity

d = distance from the lateral resisting element (e.g., shear wall) to the center of rigidity (CR).

$$T = Fe$$

F = story shear

e = eccentricity

3a.

Determine center of rigidity, center of mass, eccentricities for roof diaphragm.

Forces in the east-west (x) direction:

$$\bar{y}_r = \frac{\sum k_{xx} y}{\sum k_{xx}} \quad \text{or} \quad \bar{y}_r = \frac{\sum k_{xx} y}{\sum k_{xx}}$$

Using the rigidity values k from Table 3-3 and the distance y from line H to the shear wall:

$$\bar{y} (8.0 + 15.0 + 30.0 + 30.0 + 30.0 + 15.0 + 8.0) = 8.0(116) + 15.0(106) + 30.0(82.0) + 30.0(50.0) + 30.0(26.0) + 15.0(10.0) + 8.0(0)$$

$$\therefore \bar{y}_r = \frac{7,408}{136.0} = \underline{\underline{54.5 \text{ ft}}}$$

The building is symmetrical about the x -axis and the center of mass is determined as:

$$\bar{y}_m = \frac{116.0}{2} = \underline{\underline{58.0 \text{ ft}}}$$

The minimum 5 percent accidental eccentricity for east-west forces, e'_y , is computed from the length of the structure perpendicular to the applied story force.

$$e'_y = (0.05)(116 \text{ ft}) = \pm 5.8 \text{ ft}$$

The \bar{y}_m to the displaced CM = $58.0 \text{ ft} \pm 5.8 \text{ ft} = 63.8 \text{ ft}$ or 52.2 ft

The total eccentricity is the distance between the displaced center of mass and the center of rigidity $\bar{y}_r = 54.5 \text{ ft}$

$$\therefore e_y = 63.8 - 54.5 = \underline{\underline{9.3 \text{ ft}}} \quad \text{or} \quad 52.2 - 54.5 = \underline{\underline{-2.3 \text{ ft}}}$$

Note that the distance is slightly different than in Design Example 2.

Note that in this Design Example, displacing the center of mass 5 percent can result in the CM being on either side of the CR and can produce added torsional shears to all walls.

Note that the 5 percent may not be conservative. The contents-to-structure weight ratio can be higher in light framed structures than in heavier types of construction. Also, the location of the calculated center of rigidity is less reliable for light framed structures than for other structural systems. Use engineering judgment when selecting the eccentricity e .

Forces in the north-south (y) direction:

The building is symmetrical about the y-axis. Therefore, the distance to the CM and CR is

$$\bar{x}_m = \frac{48.0}{2} = \underline{\underline{24.0 \text{ ft}}}$$

$$\text{min. } e'_x = (0.05)(48 \text{ ft}) = \pm 2.4 \text{ ft}$$

Because the CM and CR locations coincide,

$$e_x = e'_x$$

$$\therefore e_x = \underline{\underline{2.4 \text{ ft}}} \quad \text{or} \quad \underline{\underline{-2.4 \text{ ft}}}$$

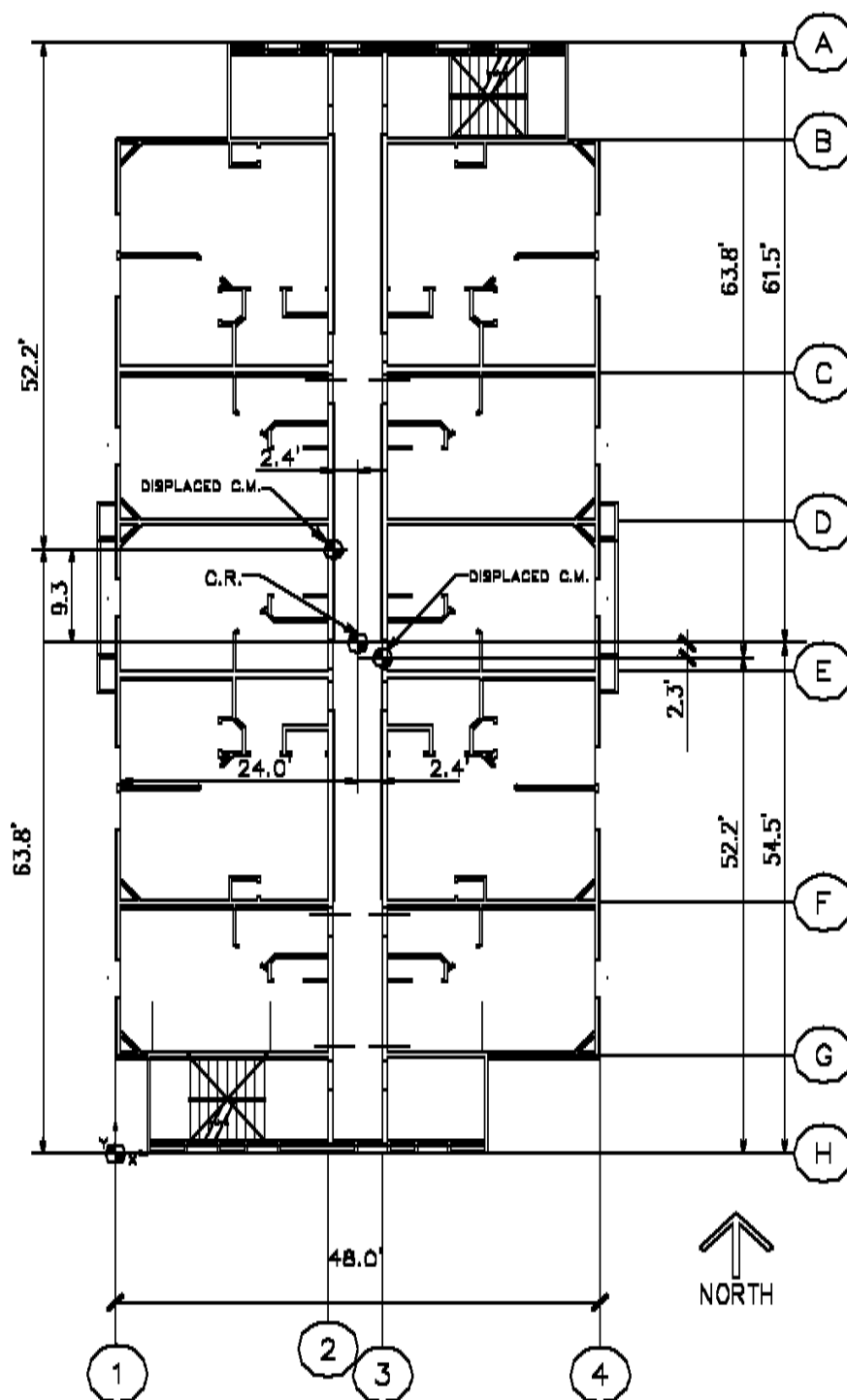


Figure 3-7. Center of rigidity and location of displaced centers of mass for diaphragms

3b.**Determine total shears on walls at roof level.**

The total shears on the walls at the roof level are the direct shears F_v and the shears due to torsion (combined actual and accidental torsion) F_{ti} .

Torsion on the roof diaphragm is computed as follows:

$$T_x = Fe_y = 44,500 \text{ lb} (9.3 \text{ ft}) = 413,850 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

$$\text{or } T_x = 44,500 \text{ lb} (2.3 \text{ ft}) = 102,350 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$$

$$T_y = Fe_x = 44,500 \text{ lb} (2.4 \text{ ft}) = 106,800 \text{ ft} \cdot \text{lb}$$

Since the building is symmetrical for forces in the north-south direction, the torsional forces can be subtracted for those walls located on the opposite side from the displaced center of mass. However, when the forces are reversed then the torsional forces will be additive. As required by the UBC, the larger values are used in this Design Example. The critical force is then used for the design of these walls. Table 3-6 summarizes the spreadsheet for determining combined forces on the roof level walls.

Table 3-6. Distribution of forces to shear walls below the roof level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	8.0			61.5	492.0	30,258	2,617	+908	3,525
	B	15.0			51.5	772.5	39,784	4,910	+1,426	6,336
	C	30.0			27.5	825.0	22,688	9,815	+1,523	11,338
	E	30.0			4.5	135.0	608	9,815	+62	9,877
	F	30.0			28.5	855.0	24,368	9,815	+390	10,205
	G	15.0			44.5	667.5	29,704	4,910	+305	5,215
	H	8.0			54.5	436.0	23,762	2,618	+199	2,817
	Σ	136.0					171,172	44,500		
North-South	1		46.0	24.0		1,104	26,496	22,250	+526	22,776
	4		46.0	-24.0		-1,104	26,496	22,250	-526	21,724
	Σ		92.0				52,992	44,500		
	Σ						224,164			

3C.

Determine center of rigidity, center of mass, and eccentricities for the third floor diaphragm.

Since the walls stack with uniform fasteners, it can be assumed that the center of rigidity for the third floor and the second floor diaphragms will coincide with the center of rigidity of the roof diaphragm.

Torsion on the third floor diaphragm is:

$$F = (44,500 + 42,700) = 87,200 \text{ lb}$$

$$T_x = Fe_y = 87,200 \text{ lb} (9.3 \text{ ft}) = 810,960 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

$$\text{or } 87,200 \text{ lb} (2.3 \text{ ft}) = 200,560 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$$

$$T_y = Fe_x = 87,200 \text{ lb} (2.4 \text{ ft}) = 209,280 \text{ ft} \cdot \text{lb}$$

Results for the third floor are summarized in Table 3-7.

Table 3-7. Distribution of forces to shear walls below the third floor level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	8.0			61.5	492	30,258	4,104	+1,467	5,571
	B	20.0			51.5	1030	53,045	10,258	+3,071	13,329
	C	38.0			27.5	1045	28,738	19,492	+3,116	22,608
	E	38.0			4.5	171	770	19,492	+126	19,618
	F	38.0			28.5	1083	30,865	19,492	+798	20,290
	G	20.0			44.5	890	39,605	10,258	+656	10,914
	H	8.0			54.5	436	23,762	4,104	+329	4,433
	Σ	170.0					207,043	87,200		
North-South	1		56.0	24		1,344	32,256	25,700	+1,034	26,734
	2		39.0	2.5		97.5	244	17,900	+76	17,976
	3		39.0	-2.5		-97.5	244	17,900	-76	17,824
	4		56.0	-24		-1,344	32,256	25,700	-1,034	24,666
	Σ		190.0				65,000	87,200		
	Σ						272,043			

3d.

Determine center of rigidity, center of mass, and eccentricities for the second floor diaphragm.

Torsion on the second floor diaphragm is.

$$F = (44,500 + 42,700 + 21,100) = 108,300 \text{ lb}$$

$$T_x = Fe_y = 108,300 \text{ lb} (9.3 \text{ ft}) = 1,007,190 \text{ ft} \cdot \text{lb} \text{ for walls A, B, and C}$$

$$\text{or } 108,300 \text{ lb} (2.3 \text{ ft}) = 249,090 \text{ ft} \cdot \text{lb} \text{ for walls E, F, G, and H}$$

$$T_y = Fe_x = 108,300 \text{ lb} (2.4 \text{ ft}) = 259,920 \text{ ft} \cdot \text{lb}$$

Results for the second floor are summarized in Table 3-8.

Table 3-8. Distribution of forces to shear walls below second floor level

	Wall	R_x	R_y	d_x	d_y	Rd	Rd^2	Direct Force F_v	Torsional Force F_t	Total Force $F_v + F_t$
East-West	A	8			61.5	492	30,258	4,444	+1,695	6,139
	B	22			51.5	1,133	58,350	12,218	+3,901	16,119
	C	45			27.5	1,238	34,031	24,992	+4,263	29,255
	E	45			4.5	203	911	24,992	+172	25,164
	F	45			28.5	1,283	36,551	24,992	+1,093	26,085
	G	22			44.5	979	43,565	12,218	+834	13,052
	H	8			54.5	436	23,762	4,444	+371	4,815
	Σ	195					227,428	108,300		
North-South	1		56.0	+24.0		1,344	32,256	31,920	+1,195	33,115
	2		39.0	+2.5		97.5	244	22,230	+87	22,317
	3		39.0	-2.5		-97.5	244	22,230	-87	22,143
	4		56.0	-24.0		-1,344	32,256	31,920	-1,195	30,725
	Σ		190.0				65,000	108,300		
	Σ						292,428			

3e.**Comparison of flexible vs. rigid diaphragm results.**

Table 3-9 summarizes wall forces determined under the separate flexible and rigid diaphragm analysis. Fastener requirements were established in Part 2 in Design Example 2. These determinations should be checked for results of the rigid diaphragm analysis and adjusted if necessary (also shown in Table 3-9).

Table 3-9. Comparison of loads on shear walls using flexible versus rigid diaphragm results and recheck of wall fastening

Wall	$F_{flexible}^{(3)}$ (lbs)	F_{rigid} (lbs)	Rigid/ Flexible ratio	b (ft)	$v = \frac{F_{max}}{(b)1.4}$ (plf)	Plywood 1 or 2 sides	Allowable Shear ⁽¹⁾ (plf)	Edge Nail Spacing (in.)
<i>Roof Level</i>								
A	1,430	3,525	+147%	12.5	205	1	310	6
B	6,280	6,336	+1%	22.0	205	1	310	6
C	11,310	11,338	0%	43.0	190	1	310	6
E	11,310	9,877	-13%	43.0	190	1	310	6
F	8,080	10,205	+26%	43.0	170	1	310	6
G	4,660	5,215	+11%	22.0	170	1	310	6
H	1,430	2,817	+97%	12.5	165	1	310	6
1	22,250	22,776	+2%	64.5	255	1	310	6
4	22,250	22,776 ⁽⁴⁾	-2%	64.5	255	1	310	6
<i>Third Floor</i>								
A	2,805	5,571	+99%	12.5	320	1	310	6 ⁽²⁾
B	12,305	13,329	+8%	22.0	435	1	400	4 ⁽²⁾
C	22,160	22,608	+2%	43.0	375	1	400	4
E	22,160	19,618	-11%	43.0	370	1	400	4
F	15,830	20,290	+28%	43.0	340	1	400	4
G	9,135	10,914	+19%	22.0	355	1	400	4
H	2,805	4,433	+58%	12.5	255	1	310	6
1	31,955	26,734	-16%	64.5	355	1	400	4
2	11,645	17,976	+54%	60.0	215	1	310	6
3	11,645	17,976 ⁽⁴⁾	+54%	60.0	215	1	310	6
4	31,955	26,734 ⁽⁴⁾	-16%	64.5	355	1	400	4
<i>Second Floor</i>								
A	3,485	6,139	+77%	12.5	350	1	310	6 ⁽²⁾
B	15,280	16,119	+5%	22.0	525	1	585	3
C	27,525	29,255	+6%	43.0	485	1	585	3
E	27,525	25,164	-9%	43.0	460	1	585	3
F	19,660	26,085	+33%	43.0	435	1	585	3
G	11,345	13,052	+15%	22.0	425	1	585	3
H	3,485	4,815	+38%	12.5	275	1	310	6
1	36,750	33,115	-10%	64.5	410	1	400	4 ⁽²⁾
2	17,400	22,317	+28%	60.0	265	1	310	6
3	17,400	22,317 ⁽⁴⁾	+28%	60.0	265	1	310	6
4	36,750	33,115 ⁽⁴⁾	-10%	64.5	410	1	400	4 ⁽²⁾

Notes:

1. Allowable shears are determined from UBC Table 22-VIII-C for 15/32-inch Structural I sheathing using nominal shear values divided by factor of safety (Ω) of 2.5. Sheathing may be either plywood or oriented strand board (OSB).
2. Screw spacing needs to be decreased from that required for Design Example 2 forces. See also discussion about building weight for the two example problems.
3. Forces taken from Design Example 2.
4. Designates the force used was the higher force for the same wall at the opposite side of the structure.

Comment: Wall rigidities used in this analysis are approximate. The initial rigidity R can be significantly higher than estimated due to the stiffening effects of stucco, drywall walls not considered, and areas over doors and windows. During an earthquake, some low stressed walls may maintain their stiffness and others may degrade in stiffness. Some walls and their collectors may attract significantly more lateral load than anticipated in either a flexible or rigid diaphragm analysis. It must be understood that the method of analyzing a structure using rigid diaphragms takes significantly more engineering effort. This rigid diaphragm analysis method indicates that some lateral resisting elements can attract significantly higher seismic demands than those determined under tributary area analysis methods.

4.

Reliability/redundancy factor ρ .

The reliability/redundancy factor penalizes lateral force resisting systems without adequate redundancy. In this Design Example, Part 1, the reliability/redundancy factor was previously assumed to be $\rho = 1.0$. This will now be checked:

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

where:

r_{max} = the maximum element-story shear ratio. For shear walls, the wall with the largest shear per foot at or below two-thirds the height of the building; or in the case of a three-story building, the ground level and the second level. See the SEAOC Blue Book Commentary §C105.1.1.1. The total lateral load in the wall is multiplied by $10/l_w$ and divided by the story shear.

l_w = length of wall in feet

A_B is the ground floor area of the structure.

$$r_i = \frac{V_{max}(10/l_w)}{F}$$

$$A_B = 5,288 \text{ sq ft}$$

For ground level.

For east-west direction:

Using strength level forces for wall B:

$$V_{max} = 16,119 \text{ lb applied to 2 walls.}$$

$$r_i = \frac{(16,119 \times 0.5)(10/11.0)}{108,300} = 0.068$$

$$\rho = 2 - \frac{20}{0.068\sqrt{5,288}} = -2.0 < 1.0 \text{ minimum } o.k.$$

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

Therefore, there is no increase in base shear due to lack of reliability/redundancy.

For north-south direction:

Using strength level forces for walls 1 and 4:

$$\text{Load to wall: } 36,750 \times 11.5 / 64.5 = 6,550 \text{ lbs}$$

$$r_i = \frac{(6,550)(10/11.5)}{108,300} = 0.053$$

Note that this is the same as using the whole wall.

$$r_i = \frac{(36,750)(10/64.5)}{108,300} = 0.053$$

$$\rho = 2 - \frac{20}{0.053\sqrt{5,288}} = -3.2 < 1.0 \text{ minimum } o.k.$$

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

For second level.

For east-west direction:

Using strength-level forces for wall B:

$$r_{max} = \frac{(13,329)(10/11.0)}{87,200} = 0.069$$

$$\rho = 2 - \frac{20}{0.069\sqrt{5,288}} = -1.9 < 1.0 \text{ minimum } o.k.$$

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

Therefore, there is no increase in base shear due to lack of reliability/redundancy.

For north-south direction:

Using strength-level forces for walls 1 and 4:

$$r_{max} = \frac{(31,955)(10/64.5)}{87,200} = 0.057$$

$$\rho = 2 - \frac{20}{0.057\sqrt{5,288}} = -2.8 < 1.0 \text{ minimum } o.k.$$

$$\therefore \rho = 1.0$$

Therefore, for both directions, there is no increase in base shear required due to lack of reliability/redundancy.

The SEAOC Seismology Committee added the sentence “The value of the ratio of $10/l_w$ need not be taken as greater than 1.0” in the 1999 Blue Book—which will not penalize longer walls, but in this Design Example has no effect.

5.

Tiedown forces for the shear wall at line C.

§2220.2

5a.

Determination of tiedown forces.

Tiedowns are required to resist the uplift tendency of shear walls caused by overturning moments. In this step, tiedown forces for the three-story shear wall on line C (Figure 3-8) are determined.

Since there are two identical shear walls on line C, forces from Table 3-7 must be divided by two. Computation of story forces for one of the two walls is shown below. Note that forces are on *strength design basis*.

$$F_{roof} = 11,338/2 = 5,669 \text{ lb/wall (two walls on line C)}$$

$$F_{third} = (22,608 - 11,338)/2 = 5,635 \text{ lb}$$

$$F_{second} = (29,255 - 22,608)/2 = 3,324 \text{ lb}$$

$\Omega_o = 2.8$ bearing wall system

Table 16-N

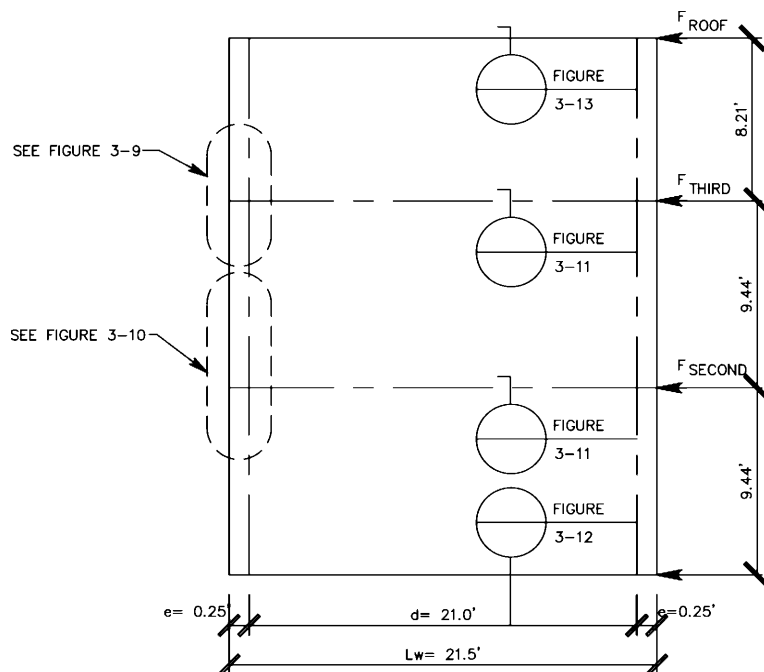


Figure 3-8. Typical shear wall C elevation

The distance between the centroid of the boundary forces that represent the overturning moment at each level must be estimated. This is shown below.

e = the distance to the center of tiedown and boundary studs or collectors studs (Figure 3-10)

$$e = 3 \text{ in.} = 0.25 \text{ ft}$$

d = the distance between centroids of the tiedowns and the boundary studs.

Note that it is also considered acceptable to use the distance from the end of the shear wall to the centroid of the tiedown.

$$d = 21.5 \text{ ft} - 2(0.25 \text{ ft}) = 21.0 \text{ ft}$$

The resisting moment M_R is determined from the following dead loads:

$$w_{\text{roof}} = 13.5 \text{ psf}(1.33 \text{ ft}) = 18.0 \text{ plf}$$

$$w_{\text{floor}} = 25.0 \text{ psf}(1.33 \text{ ft}) = 33.0 \text{ plf}$$

$$w_{\text{wall}} = 10.0 \text{ plf}$$

Overturning resisting moments are determined from simple statics. Calculations are facilitated by use of a spreadsheet. Table 3-10 summarizes the tiedown (i.e., uplift) forces for the shear walls on line C.

Table 3-10. Tiedown forces for shear wall C

Level	M_{OT} (ft-lb)	$\Omega_o M_{OT}$ (ft-lb)	M_R (ft-lb)	$0.85M_R^{(1)}$ (ft-lb)	Strength Uplift $\frac{\Omega_o M_{OT} - 0.85M_R}{d}$ (lbs)	ASD Uplift $\frac{\Omega_o M_{OT} - 0.85M_R}{d(1.4)}$ (lbs)
Roof	46,545	130,330	23,135	19,665	5,275	3,770
Third	153,255	429,115	52,580	44,695	18,315	13,080
Second	291,340	815,755	82,025	69,720	35,525	25,375

Notes:

1. The 0.85 dead load factor of §2213.5.1 is different from the 0.9 factor of §1612.4.

5b.**Load combinations using allowable stress design.**

The UBC has two special sections for shear walls with light framing in Seismic Zones 3 and 4. For metal framing, §2220 is used, and for wood framing, Section 2315.5.1. Section 2220.2 specifies requirements for steel stud wall boundary members and anchorage and refers to §2213.5.1 for load combinations. Section 2315.5.1 deals with wood stud walls and does not have any such special requirements. In the case of identical building types (as in Design Example 2 and Design Example 3 of this manual) this would give an apparent advantage to wood framing.

The basic load combinations of §1612.3.1 do not permit stress increases.

The alternate basic load combinations if §1612.3.2 permit stress increases.

Errata to the First Printing added Equation (12-16-1):

$$0.9D \pm \frac{E}{1.4} \text{ to the alternate basic load combinations} \quad (12-16-1)$$

Since this exact same load combination is listed in the basic load combinations the code is in contradiction and confusing (to say at least). This Design Example will use one-third stress increase of §1612.3.2.

6.**Allowable shear and nominal shear strength of No. 10 screws.**

Tiedown connections for the line C shear wall will utilized 12-gauge straps at the third floor. This part shows determination of the shear strength of the No. 10 screws that will be used to connect the tiedown straps to the 18-gauge boundary studs.

There are two basic ways of determining the shear strength of the screws. The first is to use the values established in an ICBO Evaluation Report with appropriate conversion to strength design. The second is to compute the shear strength of a screw using the '96 AISI specification. Both methods are shown below.

6a.**Nominal shear strength determined from ICBO Evaluation Report.**

The Metal Stud Manufacturers' Association provides ICBO ER No. 4943. Shear values on an ASD basis are provided for various gauge studs having a minimum yield strength of 33 ksi and a minimum ultimate strength of 45 ksi.

For No. 10 screws in an 18-gauge stud, the allowable shear is given as 258 lbs per screw. This must be increased as shown below to convert to the strength design basis used in this example.

$$P_{ns} = \Omega P_{as}$$

where:

$$P_{ns} = \text{nominal shear strength per screw}$$

$$P_{as} = \text{allowable shear strength per screw}$$

$$\Omega = 3.0$$

96AISI E4

$$P_{ns} = 3.0(258 \text{ lb}) = 774 \text{ lb per screw}$$

Note that ER No. 4943 also specifies a minimum edge distance and a minimum on center spacing of 9/16 inch for No. 10 screws.

6b.**Calculation of nominal shear strength using strength design.**

The nominal shear strength is the screw capacity without the appropriate reduction factors for allowable stress design (Ω) or load and resistance factor design (ϕ).

$$d = 0.190 \text{ in.}$$

$$F_{u1} = 45,000 \text{ psi}$$

Note: some connector straps and hardware have an $F_u = 65,000 \text{ psi}$, which will give higher screw capacities.

$$F_{u2} = 45,000 \text{ psi}$$

Case 1: Strap applied to stud flange (Figure 3-9).

Assume 12-gauge galvanized strap:

$$t_1 = 0.1017 \text{ in.}$$

With 18-gauge studs:

$$t_2 = 0.0451 \text{ in.}$$

$$t_2/t_1 = 0.0451/0.1017 = 0.44 < 1.0$$

$$P_{ns} = 4.2 \left(t_2^3 d \right)^{1/2} F_{u2} = 789 \text{ lb} \quad 96 \text{ AISI (E4.3.1-1)}$$

$$P_{ns} = 2.7 t_1 d F_{u1} = 2,348 \text{ lb} \quad 96 \text{ AISI (E4.3.1-2)}$$

$$P_{ns} = 2.7 t_2 d F_{u2} = 1,041 \text{ lb} \quad 96 \text{ AISI (E4.3.1-3)}$$

Using the smallest value of P_{ns} :

$$P_{ns} = 789 \text{ lb per screw}$$

Note how this value is almost equal to the 774 lb determined from Part 6a, above.

Case 2: Strap applied to double stud webs (Figure 3-10).

Assume 10-gauge galvanized strap:

$$t_1 = 0.138 \text{ in.}$$

With 18-gauge studs:

Since there are two stud webs, thickness t_2 is doubled.

$$t_2 = 0.0451 \times 2 = 0.0902 \text{ in.}$$

$$t_2/t_1 = 0.0902/0.138 = 0.65 < 1.0$$

$$P_{ns} = 4.2 \left(t_2^3 d \right)^{1/2} F_{u2} = 2,232 \text{ lb} \quad 96 \text{ AISI (E4.3.1-1)}$$

$$P_{ns} = 2.7 t_1 d F_{u1} = 3,186 \text{ lb} \quad 96 \text{ AISI (E4.3.1-2)}$$

$$P_{ns} = 2.7 t_2 d F_{u2} = 2,082 \text{ lb} \quad 96 \text{ AISI (E4.3.1-3)}$$

Using the smallest value of P_{ns} :

$$P_{ns} = 2,082 \text{ lb}$$

6C.

Calculation of allowable shear using ASD.

Case I: Strap applied to stud flange (Figure 3-9):

From Part 6b, above:

$$P_{ns} = 789 \text{ lb}$$

$$P_{as} = P_{ns} / \Omega = 789 / 3.0 = 263 \text{ lb per screw}$$

Case II: Strap applied to double stud webs (Figure 3-10):

From Part 6b, above:

$$P_{ns} = 2,082 \text{ lb}$$

$$P_{as} = P_{ns} / \Omega = 2,082 / 3.0 = 694 \text{ lb per screw}$$

7.

Tiedown connection at third floor for wall on line C.

Shown below is the strength design of the tiedown strap to be used for the shear walls on line C at the third floor. The configuration at the tiedown is shown on Figure 3-9.

$$\text{Uplift} = 3,770 \text{ lb}$$

Try a 12-gauge \times 3 inch strap and No. 10 screws:

$$P_{ns} = 789 \text{ lb per screw}$$

$$\text{LRFD design strength} = \phi P_{ns}$$

96 AISI (3.1)

where:

$$\phi = 0.50$$

$$\phi P_{ns} = 0.5(789) = 395 \text{ lb}$$

Number of screws required:

$$3,770/395 = 9.5$$

∴ Use 12 minimum

With 2 rows of #10 screws @ 3½ inches on center the length of strap required:

Strap is pre-manufactured, use half spacing for end distance or 1¾ inch. Net spacing is screws is 1.75 inches on center. Need to add in thickness of 1½ inch lightweight concrete and ¾-inch sheathing, plus the 12-inch depth for the floor joist:

$$(1.75 + (1 + 12)1.75 + 1.75)2 + (1.5 + 0.75 + 12) = 65.0$$

∴ Use 72-inch-long strap

Check capacity of strap for tension:

Strap to be used will be a pre-manufactured strap for which there is an ICBO Evaluation Report. The rated capacity, including 33 percent increase for wind or seismic loading, is given as 9,640 lb.

$$9,640 \text{ lb} > 3,770 \text{ lb} \quad o.k.$$

If the strap does not have an ICBO rated capacity, the manufacturer should be contacted to determine the strength of the steel used. It is probable that the steel used in the strap will have strengths that differ from the steel used in the studs. Generally, strengths differ from manufacturer to manufacturer.

Checking capacity of strap:

$$T_n = A_n F_y \quad 96 \text{ AISI (C2-1)}$$

$$b = 3.0 \text{ in. (strap width)}$$

$$t = 0.1046 \text{ in. (strap thickness)}$$

$$d = 0.171 \text{ in. (diameter of holes)}$$

Design Example 3 ■ Cold-Formed Steel Light Frame Three-Story Structure

$$A_n = 0.2987 \text{ in.}^2 \text{ (net area of strap)}$$

$$F_y = 45,000 \text{ psi (yield strength of particular manufacturer)}$$

$$T_n = 0.2987(45,000) = 13,443 \text{ lb (nominal strength of strap)}$$

For ASD:

Tie force = 3,770 lb (Table 3-10)

$$\frac{T_n}{\Omega_t} = \text{allowable tension} = \frac{13,443}{1.67} = 8,050 \text{ lb} > 3,770 \text{ lb} \quad o.k.$$

For LRFD:

Tie force = 5,275 lb (Table 3-10)

$$\phi T_n = \text{tension strength} = 0.95(13,443) = 12,770 \text{ lb} \Rightarrow 5,275 \text{ lb} \quad o.k.$$

Use 12-gauge $\times 3 \text{ in.} \times 72 \text{ in.}$ strap with 12 #10 screws @ $3\frac{1}{2}$ inches o.c. each end.

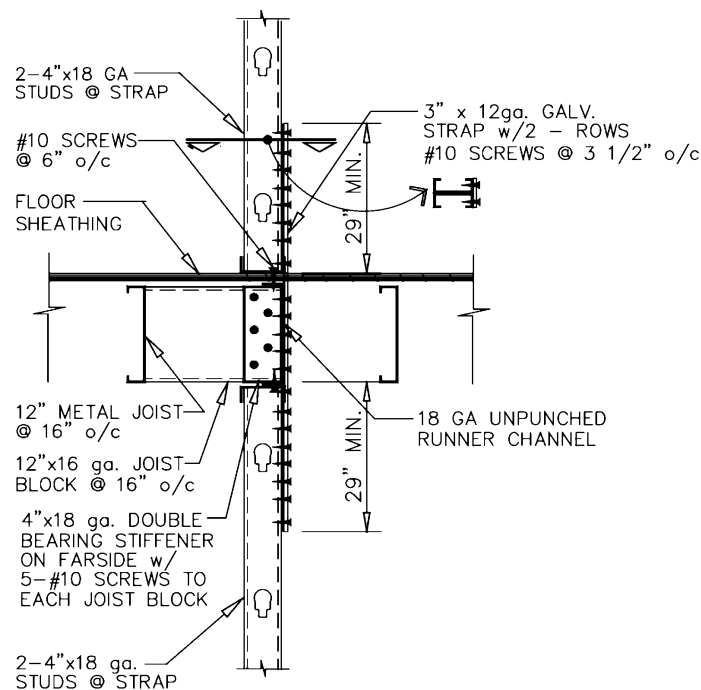


Figure 3-9. Typical tiedown connection at the third floor on line C.

8.**Tiedown connection at second floor for wall on line C.**

Design of the pre-manufactured tiedowns for the second floor shear walls on line C is shown below. Figure 3-10 shows the configuration of the tiedown.

Uplift = 13,080lb from Table 3-10

The connector is an ICBO approved, pre-manufactured holdown device. The rated capacity including the 33 percent increase for wind or seismic loading is 9,900lb.

Using two holdowns, one on each boundary stud, the capacity is:

$$2 \times 9,900 = 19,800\text{lb} > 13,080\text{lb} \quad o.k.$$

In general, when using pre-manufactured tiedowns, consult with ICBO Evaluation Service or the manufacturer for the necessary approvals for hardware selection.

9.

Boundary studs for first floor wall on line C.

The studs at each end of the shear walls on line C must be designed to resist overturning forces. In this example, double studs as shown in Figure 3-10 will be used at each end. The critical aspect of design is checking the studs for axial compression. This is shown below.

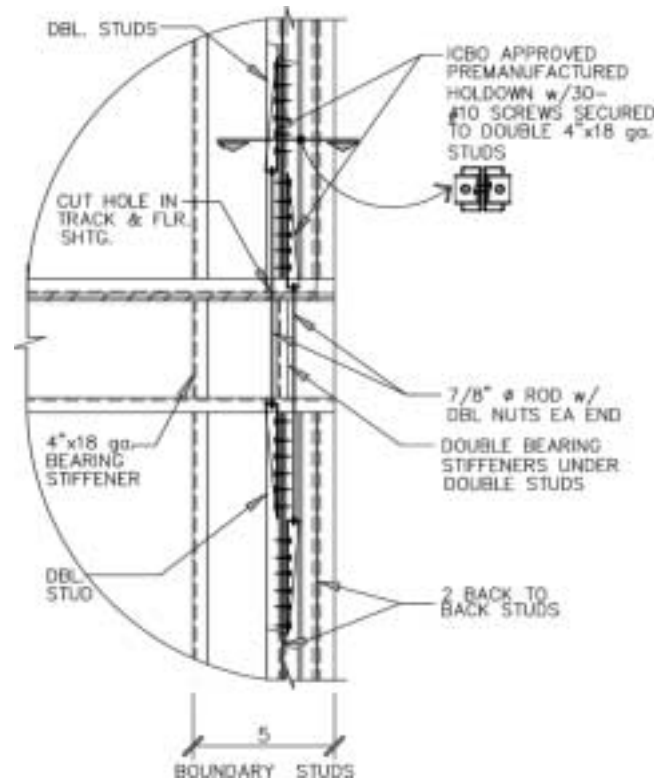


Figure 3-10. Typical tiedown connection at the second floor on line C

Note that §2220.2 of Division VII (Lateral Resistance of Steel Stud Wall Systems) requires use of the requirements of §2315.5.1. This includes use of the seismic force amplification factor Ω_o to account for structural overstrength. This requirement *does not* apply for boundary elements of wood stud shear walls.

For axial compression, the load combination to be used is:

$$1.0P_{DL} + 0.7P_{LL} + \Omega_o P_E \quad \text{\S 2213.5.1}$$

$$P_{LL} = [40 \text{ psf} + (16/12)(16/12)]2 = 145 \text{ lb}$$

$$P_{DL} = [13.5 \text{ psf} + (25.0)2](16/12) + [10 \text{ psf} (8 \text{ ft})(16/12)(3)] = 405 \text{ lb}$$

From Table 3-10:

$$\Omega_o M_{OT} = 815,696 \text{ ft} \cdot \text{lb}$$

$$\Omega_o P_E = 815,755 \text{ ft} \cdot \text{lb} / (21.0 \text{ ft})(1.4) = 27,745 \text{ lb}$$

Thus, the design load to boundary studs using the equation of §2213.5.1 is:

$$1.0(405) + 0.7(145) + 27,745 = 28,250 \text{ lb}$$

With a computer program using 1996 AISI Specifications, the allowable axial load for a 4"×18-gauge stud with 2-inch flanges is 4,042 lb with the flanges braced at mid-height.

$$\text{No. of studs required} = \frac{28,250}{4,042 \times 1.7} = 4.1$$

where:

1.7 is the allowable stress increase

Therefore, use 5 studs at ends of wall as follows:

Use two back-to-back studs, plus two back-to-back studs with additional stud (Figure 3-10).

10. Shear transfer at second floor on line C.

Shear forces in the second floor diaphragm are transferred to the shear walls below as shown in Figure 3-11.

From Table 3-9, the ASD shears in the wall are:

$$v = 485 \text{ lb/ft}$$

Try using #8 screws, 18-gauge metal side plates and Douglas Fir plywood:

$$Z = 119 \text{ lb/screws}$$

91NDS Table II.3B

$$C_D = 1.33$$

§1612.3.2

$$\text{Maximum spacing} = \frac{ZC_D}{v} = \frac{119(1.33)12}{485} = 3.9 \text{ in.}$$

∴ Use # 8 screws at 3 inches on center.

Capacity of the #8 screws in the 18-gauge tracks and runner channels are O.K. by inspection.

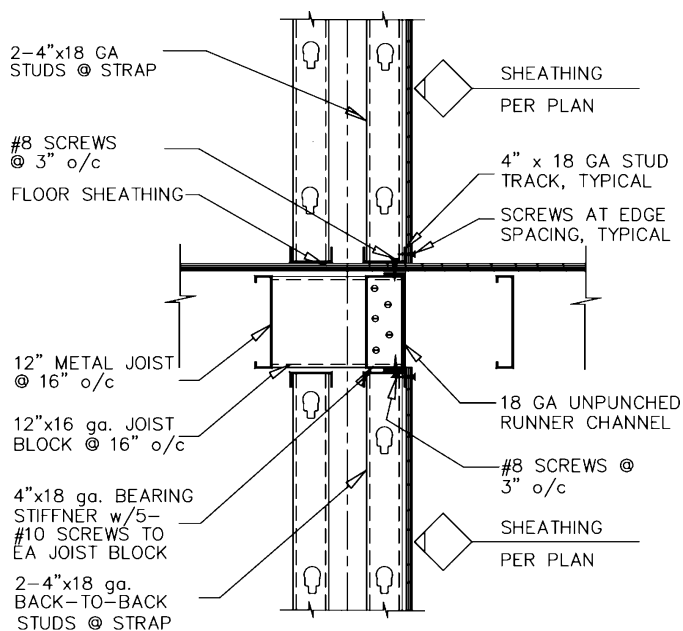


Figure 3-11. Typical detail for shear transfer through floor on line C

11.**Shear transfer at foundation for walls on line C.**

Shown below is the design of the connection to transfer the shear force in the walls on line C to the foundation. This detail is shown in Figure 3-12.

From Table 3-9:

$$v = 485 \text{ lb/ft}$$

Allowable load based on bolt bearing on track:

96 AISI (E3.3)

For 5/8" bolts and 18-gauge track:

$$P_n = 2.22 F_u d$$

96 AISI, Table E3.3-2

where:

P_n = nominal resistance

F_u = 45 ksi (minimum value)

d = 0.625 in.

t = 0.0451 in.

$$P_n = 2.22 (45)(0.625)(0.0451) = 282 \frac{\text{k}}{\text{bolt}}$$

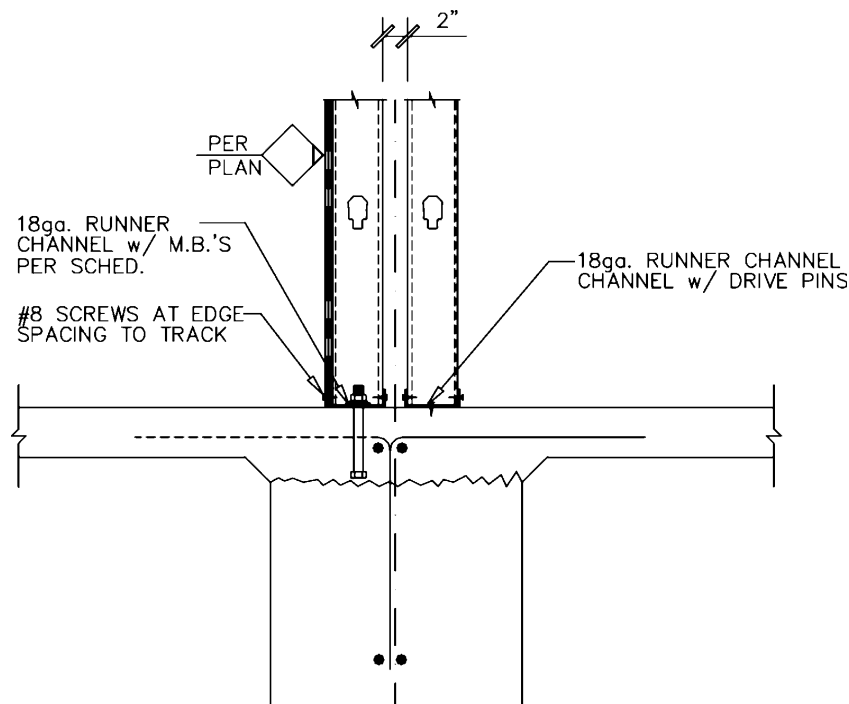


Figure 3-12. Detail for shear transfer at foundation on line C.

Allowable service load on embedded bolts in concrete is determined as follows.

For 5/8" bolts and 3000 psi concrete:

$$\text{Allowable shear} = 2,750 \frac{\text{lb}}{\text{bolt}}$$

Table 19-D

Therefore the bolt in concrete governs the required spacing:

$$\text{Maximum spacing} \frac{2,750}{485} = 5.67 \text{ ft o.c.}$$

∴ Use 5/8" diameter bolts at 4'-0" o.c. spacing

12.**Shear transfer at roof on line C.**

Shear forces in the roof diaphragm are transferred to the shear walls below as shown in Figure 3-13. From Table 3-9 are the ASD shears in the wall.

$$v = 190 \text{ lb/ft}$$

From manufacturer's catalog, allowable load for the 6-3/8-inch-long framing clip is 915 pounds.

With framing clips at 4.0 ft centers, the design ASD force is:

$$(190)(4) = 760 \text{ lb} < 915 \text{ lb} \quad o.k.$$

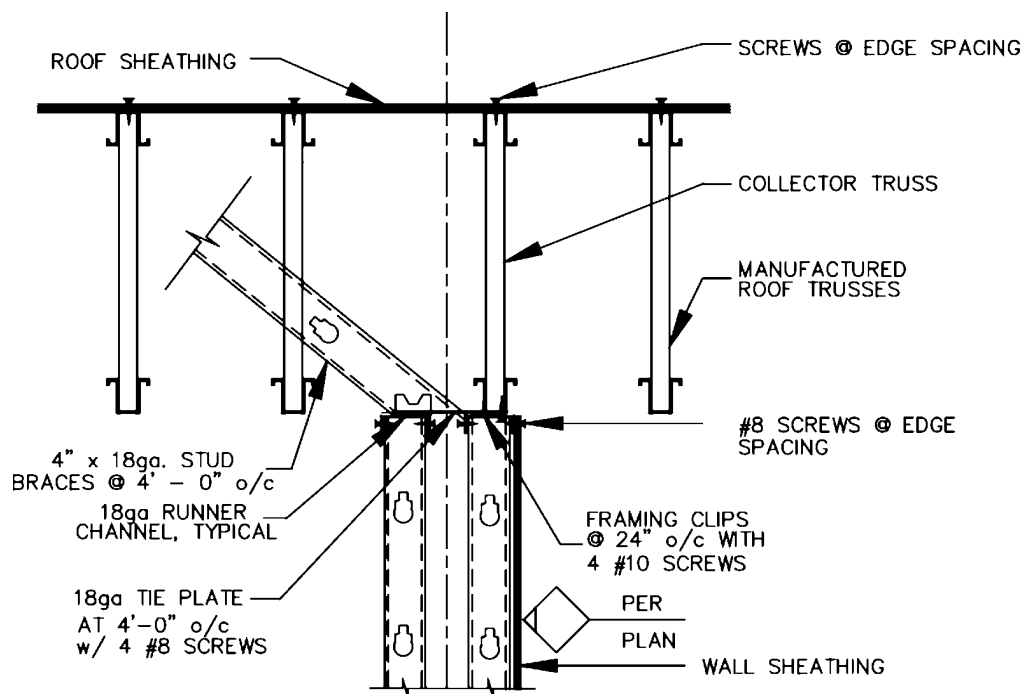


Figure 3-13. Shear transfer at roof at line C

Note that double studs are used for sound control, but that only one stud is considered in shear wall calculations.

Commentary

The code does not have conventional construction provisions for cold-formed steel similar to the conventional light frame construction provisions for wood. The 2000 International Residential Code (IRC) has included prescriptive provisions for cold-formed steel for one- and two-family dwellings. It should be noted that the structure shown in example could not use the IRC prescriptive provisions. Inasmuch as there is no one standard for the manufacturing of the studs, the process to design gravity load members is a tedious method and should not be done by prescriptive means.

The AISI Specification for Design of Cold-Formed Steel Structural Members has complex equations and is considered by most engineers too difficult to be readily used in design.

Due to the complex nature of the equations, in the AISI code it is recommended that engineers designing in cold-formed steel utilize computer software for design.

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Design Example 4

Masonry Shear Wall Building

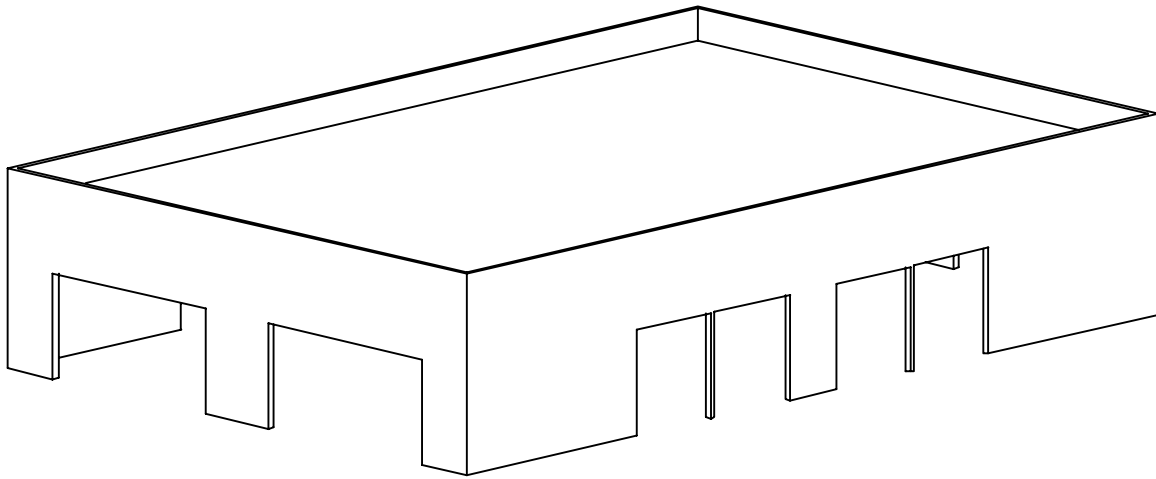


Figure 4-1. Schematic CMU building elevation

Overview

Reinforced concrete block masonry is frequently used in one-story and lowrise construction, particularly for residential, retail, light commercial, and institutional buildings. This type of construction has generally had a good earthquake performance record. However, during the 1994 Northridge earthquake, some one-story buildings with concrete masonry unit (CMU) walls and panelized wood roofs experienced wall-roof separations similar to that experienced by many tilt-up buildings.

This building in this Design Example 4 is typical of one-story masonry buildings with wood framed roofs. The building is characterized as a heavy wall and flexible roof diaphragm “box building.” The masonry building for this example is shown schematically in Figure 4-1. Floor and roof plans are given in Figure 4-2 and 4-3, respectively. The building is a one-story bearing wall building with CMU shear walls. Roof construction consists of a plywood diaphragm over wood framing. An

elevation of the building on line A is shown in Figure 4-4. A CMU wall section is shown in Figure 4-5, and a plan view of an 8'-0" CMU wall/pier is shown in Figure 4-6.

The design example illustrates the strength design approach to CMU wall design for both in-plane and out-of-plane seismic forces.

Outline

This example will illustrate the following parts of the design process.

- 1. Design base shear coefficient.**
- 2. Base shear in the transverse direction.**
- 3. Shear in wall on line A.**
- 4. Design 8'-0" shear wall on line A for out-of-plane seismic forces.**
- 5. Design 8'-0" shear wall on line A for in-plane seismic forces.**
- 6. Design 8'-0" shear wall on line A for axial and in-plane bending forces.**
- 7. Deflection of shear wall on line A.**
- 8. Requirements for shear wall boundary elements.**
- 9. Wall-roof out-of-plane anchorage for lines 1 and 3.**
- 10. Chord design.**

Given Information

Roof weights:

Roofing+ one re-roof	7.5 psf
½" plywood	1.5
Roof framing	4.5
Mech./elec.	1.5
Insulation	<u>1.5</u>
Total dead load	17.0 psf
Roof live load	20.0 psf

Exterior 8-inch CMU walls:

75 psf (fully grouted, light-weight masonry)
$f'_m = 2,500$ psi
$f_y = 60,000$ psi

Seismic and site data:

$Z = 0.4$ (Seismic Zone 4)
 $I = 1.0$ (standard occupancy)
 Seismic source type = A
 Distance to seismic source = 5 km
 Soil profile type = S_D

Table 16-I
Table 16-K

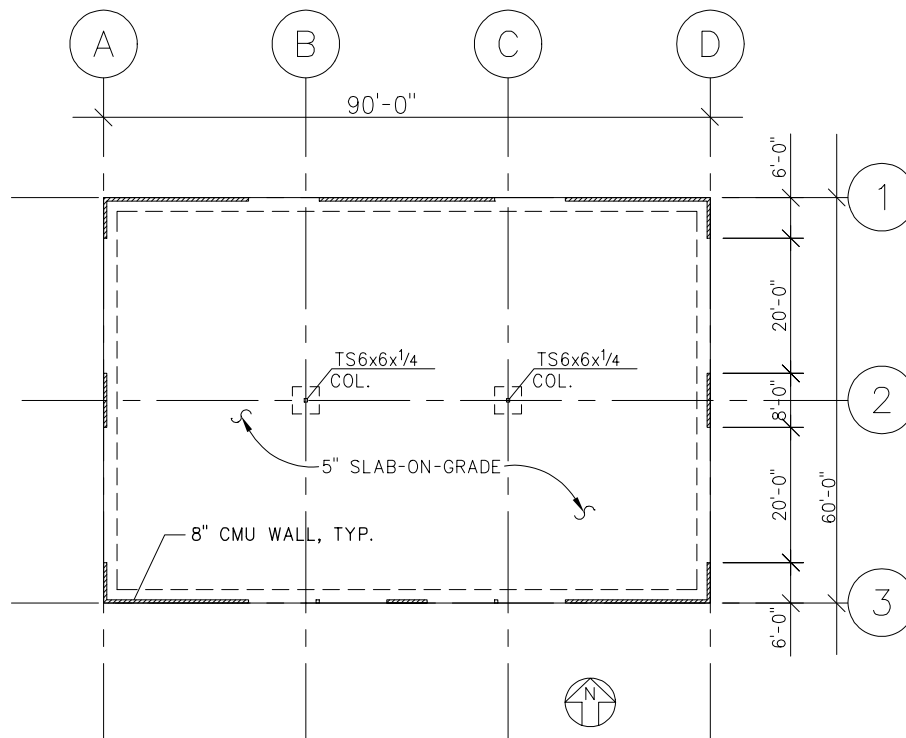


Figure 4-2. Floor plan

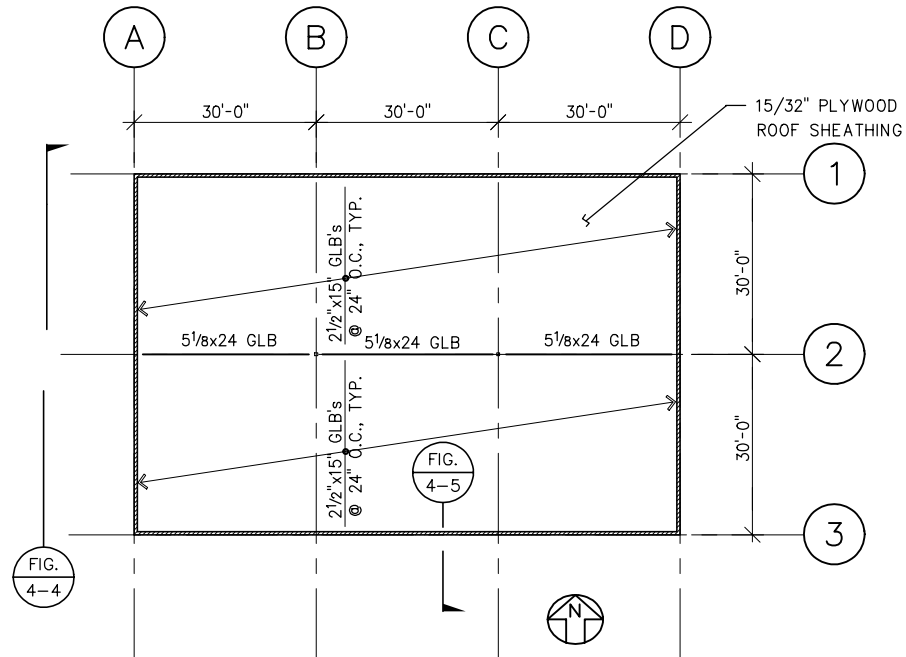


Figure 4-3. Roof plan

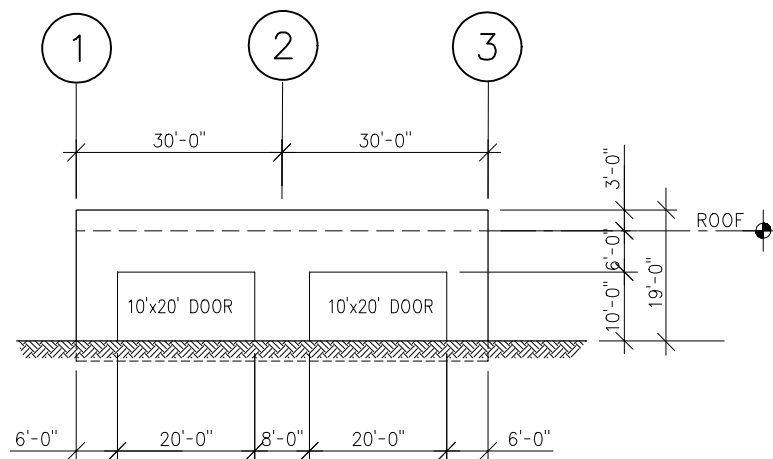


Figure 4-4. Elevation of wall on line A

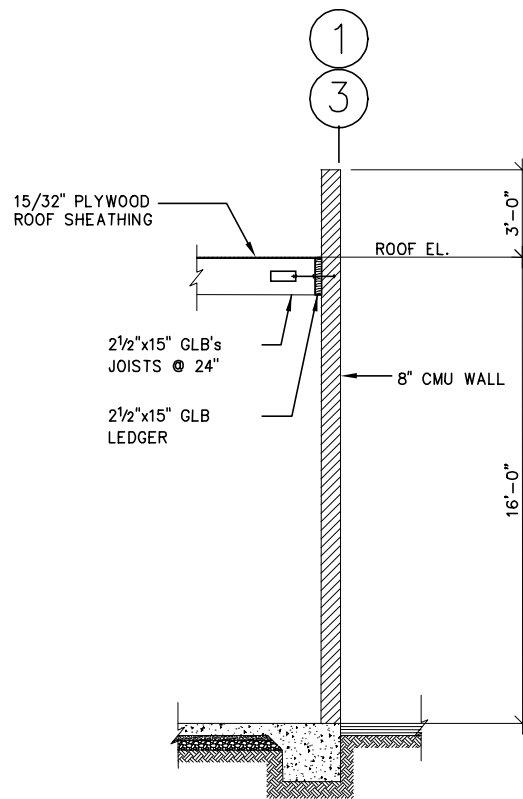


Figure 4-5. Section through CMU wall along lines 1 and 3

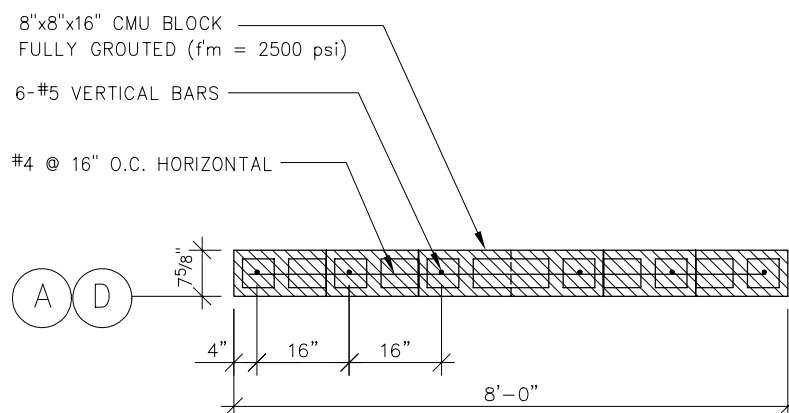


Figure 4-6. Reinforcement in 8'-0" CMU shear walls on lines A and D

Calculations and Discussion

Code Reference

1.**Design base shear coefficient.****§1630.2.2**

Period using Method A (see Figure 4-5 for section through structure):

$$T = C_t (h_n)^{3/4} = .020 (16 \text{ ft})^{3/4} = 0.16 \text{ sec} \quad (30-8)$$

Near source factors for seismic source type A and distance to source = 5 km

$$N_a = 1.2 \quad \text{Table 16-S}$$

$$N_v = 1.6 \quad \text{Table 16-T}$$

Seismic coefficients for Zone 4 and soil profile type S_D are:

$$C_a = 0.44 N_a = 0.53 \quad \text{Table 16-Q}$$

$$C_v = 0.64 N_v = 1.02 \quad \text{Table 16-R}$$

The R coefficient for a masonry bearing wall building with masonry shear walls is:

$$R = 4.5 \quad \text{Table 16-N}$$

Calculation of design base shear:

$$V = \frac{C_v I}{RT} W = \frac{1.02 (1.0)}{4.5 (0.16)} W = 1.417 W \quad (30-4)$$

but need not exceed:

$$V = \frac{2.5 C_a I}{R} W = \frac{2.5 (0.53) (1.0)}{4.5} W = 0.294 W \quad (30-5)$$

The total design shear shall not be less than:

$$V = 0.11 C_a I W = 0.11 (0.53) (1.0) W = 0.058 \quad (30-6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than:

$$V = \frac{0.8ZN_v I}{R} W = \frac{0.8(0.4)(1.60)(1.0)}{4.5} W = 0.114W \quad (30-7)$$

Therefore, Equation (30-5) controls the base shear calculation and the seismic coefficient is thus:

$$V = \underline{\underline{0.294W}}$$

2.

Base shear in transverse direction.

This building has a flexible roof diaphragm and heavy CMU walls (see Figure 4-3). The diaphragm spans as a simple beam between resisting perimeter walls in both directions and will transfer 50 percent of the diaphragm shear to each resisting wall. However, in a building that is not symmetric or does not have symmetric wall layouts, the wall lines could have slightly different wall shears on opposing wall lines 1 and 3 and also on A and D.

The building weight (mass) calculation is separated into three portions: the roof, longitudinal walls, and transverse walls for ease of application at a later stage in the calculations. The reason to separate the CMU wall masses is because masonry walls that resist ground motions parallel to their in-plane directions resist their own seismic inertia without transferring seismic forces into the roof diaphragm. This concept will be demonstrated in this example for the transverse (north-south) direction.

For the transverse direction, the roof diaphragm resists seismic inertia forces originating from the roof diaphragm and the longitudinal masonry walls (out-of-plane walls oriented east-west) on lines 1 and 3, which are oriented perpendicular to the direction of seismic ground motion. The roof diaphragm then transfers its seismic forces to the transverse masonry walls (in-plane walls oriented north-south) located on lines A and D. The transverse walls resist seismic forces transferred from the roof diaphragm and seismic forces generated from their own weight. Thus, seismic forces are generated from three sources: the roof diaphragm; in-plane walls at lines 1 and 3; and out-of-plane walls at lines A and D.

The design in the orthogonal direction is similar and the base shear is the same. However, the proportion of diaphragm and in-plane seismic forces is different. The orthogonal analysis is similar in concept, and thus is not shown in this example.

Roof weight:

$$W_{roof} = 17 \text{ psf} (5,400 \text{ sf}) = 92 \text{ kips}$$

For longitudinal wall weight (out-of-plane walls), note that the upper half of the wall weight is tributary to the roof diaphragm. This example neglects openings in the top half of the walls.

$$W_{walls, long} = 75 \text{ psf} (2 \text{ walls})(90 \text{ ft})(19) \left(\frac{19 \text{ ft}}{2} \right) \left(\frac{1}{16 \text{ ft}} \right) = 75 \text{ psf} (180 \text{ ft}) \frac{(19 \text{ ft})^2}{2(16 \text{ ft})} = 152 \text{ kips}$$

For forces in the transverse direction, seismic inertial forces from the transverse walls (lines A and D) do not transfer through the roof diaphragm. Therefore, the effective diaphragm weight in the north-south direction is:

$$W_{trans. diaph} = W_{roof} + W_{walls, long} = 92 \text{ k} + 152 \text{ k} = 244 \text{ kips}$$

The transverse seismic inertial force (shear force), which is generated in the roof diaphragm is calculated as follows:

$$V_{trans. diaph} = 0.294 W_{trans. diaph} = 0.294(244 \text{ kips}) = 72 \text{ kips}$$

The seismic inertial force (shear force) generated in the transverse walls (in-plane walls) is calculated using the full weight (and height) of the walls (with openings ignored for simplicity).

$$V_{trans. walls} = 0.294 (75 \text{ psf})(19 \text{ ft})(60 \text{ ft})(2 \text{ walls}) = 50 \text{ kips}$$

The design base shear in the transverse direction is the sum of the shears from the roof diaphragm shear and the masonry walls in-plane shear forces.

$$\therefore V_{trans.} = V_{trans. diaph} + V_{trans. walls} = 72 \text{ k} + 50 \text{ k} = \underline{\underline{122 \text{ kips}}}$$

3.

Shear wall on line A.

The seismic shear tributary to the wall on line A comes from the roof diaphragm (transferred at the top of the wall) and the in-plane wall inertia force:

$$V_A = \frac{V_{trans. diaph}}{2} + \frac{V_{trans. walls}}{2} = \frac{72 \text{ kips}}{2} + \frac{50 \text{ kips}}{2} = \underline{\underline{61 \text{ kips}}}$$

4.

Design 8'-0" shear wall on line A for out-of-plane seismic forces.

In this part, the 8'-0" shear wall on line A (Figure 4-4) will be designed for out-of-plane seismic forces. This wall is a bearing wall and must support gravity loads. It must be capable of supporting both gravity and out-of-plane seismic forces, and gravity plus in-plane seismic forces at different instants in time

depending on the direction of seismic ground motion. In this Part, the first of these two analyses will be performed.

The analysis will be done using the “slender wall” design provisions of §2108.2.4. The analysis incorporates static plus $P\Delta$ deflections caused by combined gravity loads and out-of-plane seismic forces and calculates an axial plus bending capacity for the wall under the defined loading.

4a.**Vertical loads.**

Gravity loads from roof framing tributary to the 8'-0" shear wall at line A:

$$P_{DL} = (17 \text{ psf}) \left(\frac{60 \text{ ft}}{2} \right) \left(\frac{30 \text{ ft}}{2} \right) = 7,650 \text{ lb}$$

Live load reduction for gravity loads:

$$R = r(A - 150) \leq 40 \text{ percent} \quad \S 1607.5$$

$$A = (30 \text{ ft})(15 \text{ ft}) = 450 \text{ sq ft}$$

$$R = 0.8(450 \text{ sq ft} - 150 \text{ sq ft}) = 24 \text{ percent}$$

$$R_{max} = 23.1 \left(1 + \frac{DL}{LL} \right) = 23.1 \left(1 + \frac{17}{20} \right) = 42.7 \text{ percent}$$

$$\therefore R = 24 \text{ percent}$$

The reduced live load is:

$$P_{RLL} = (20 \text{ psf}) \left(\frac{60 \text{ ft}}{2} \right) \left(\frac{30 \text{ ft}}{2} \right) (100 \text{ percent} - 24 \text{ percent}) = 6,840 \text{ lb}$$

Under §2106.2.7, the glulam beam reaction load may be supported by the bearing width plus four times the nominal wall thickness. Assuming a 12-inch bearing width from a beam hanger, the vertical load is assumed to be carried by a width of wall $12 \text{ in.} + 4(8 \text{ in.}) = 44 \text{ in.}$

$$P_{beamD+L} = \frac{(7,650 \text{ lb} + 6,840 \text{ lb})}{(44 \text{ in.}/12 \text{ in.})} = 3,952 \text{ plf}$$

$$P_{beamD} = \frac{7,650 \text{ lb}}{(44 \text{ in.}/12 \text{ in.})} = 2,086 \text{ plf}$$

Wall load on 8-foot wall (at wall mid-height):

$$P_{wall DL} = (75 \text{ psf})(8 \text{ ft}) \left(\frac{16 \text{ ft}}{2} + 3 \text{ ft} \right) = 6,600 \text{ lb}$$

$$w_{wall DL} = \frac{6,600 \text{ lb}}{8 \text{ ft}} = 825 \text{ plf}$$

Dead load from wall lintels:

$$P_{Lintel D} = (75 \text{ psf})(9 \text{ ft}) \left(\frac{20 \text{ ft}}{2} \right) = 6,750 \text{ lb}$$

$$l = (96 \text{ in.} - 44 \text{ in.})/2 = 26 \text{ in.}$$

$$w_{Lintel D} = \frac{6,750 \text{ lb}}{26 \text{ in.}/12 \text{ in.}} = 3,115 \text{ plf}$$

Since the lintel loads are heavier than the beam load, and since dead load combinations will control, the loads over the wall/pier length will be averaged.

The gravity loads on the 8'-0" wall from the weight of the wall, the roof beam, and two lintels are:

$$\sum P_{DL} = (6,600 \text{ lb} + 7,650 \text{ lb} + 6,750 \text{ lb} + 6,750 \text{ lb}) = \underline{\underline{27,750 \text{ lb}}}$$

$$\sum P_{RLL} = \underline{\underline{6,840 \text{ lb}}}$$

4b.

Seismic forces.

Out-of-plane seismic forces are calculated as the average of the wall element seismic coefficients at the base of the wall and the top of the wall. The coefficients are determined under the provisions of §1632.2 using Equation (32-2) and the limits of Equation (32-3).

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (32-2)$$

$$0.7 C_a I_p W_p \leq F_p \leq 4.0 C_a I_p W_p \quad (32-3)$$

At the base of the wall:

$$\begin{aligned}
 F_p &= \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \\
 &= \frac{(1.0) C_a I_p}{R_p} \left(1 + 3 \frac{0 \text{ ft}}{16 \text{ ft}} \right) W_p \\
 &= 0.133 C_a I_p W_p \leq 0.7 C_a I_p W_p \\
 \therefore \text{Use } 0.7 C_a I_p W_p \\
 F_p &= 0.7 (.53) (1.0) W_p = 0.37 W_p \\
 &= 0.37 (75 \text{ psf}) = 27.8 \text{ psf}
 \end{aligned}$$

At roof:

$$\begin{aligned}
 F_p &= \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \\
 &= \frac{(1.0) C_a I_p}{R_p} \left(1 + 3 \frac{0'}{16'} \right) W_p \\
 &= 1.33 C_a I_p W_p \leq 4.0 C_a I_p W_p \\
 \therefore \text{Use } 1.33 C_a I_p W_p \\
 F_p &= 1.33 (.53) (1.0) W_p = 0.37 W_p \\
 &= 0.70 (75 \text{ psf}) = 52.5 \text{ psf}
 \end{aligned}$$

Thus, use the average value of $F_p = (1/2)(27.8 \text{ psf} + 52.5 \text{ psf}) = \underline{\underline{40.2 \text{ psf}}}$

Calculation of wall moments due to out-of-plane forces is done using the standard beam formula for a propped cantilever. See Figure 4-7 for wall out-of-plane loading diagram and Figure 4-8 for tributary widths of wall used to determine the loading diagram.

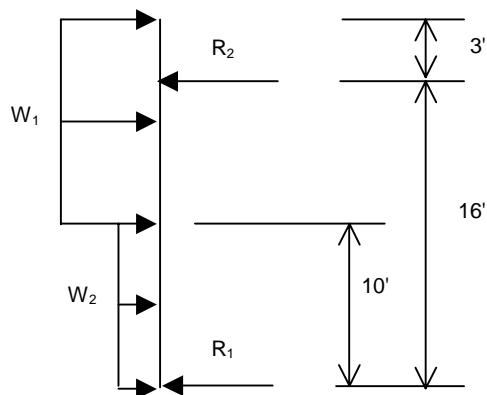


Figure 4-7. Propped cantilever loading diagram

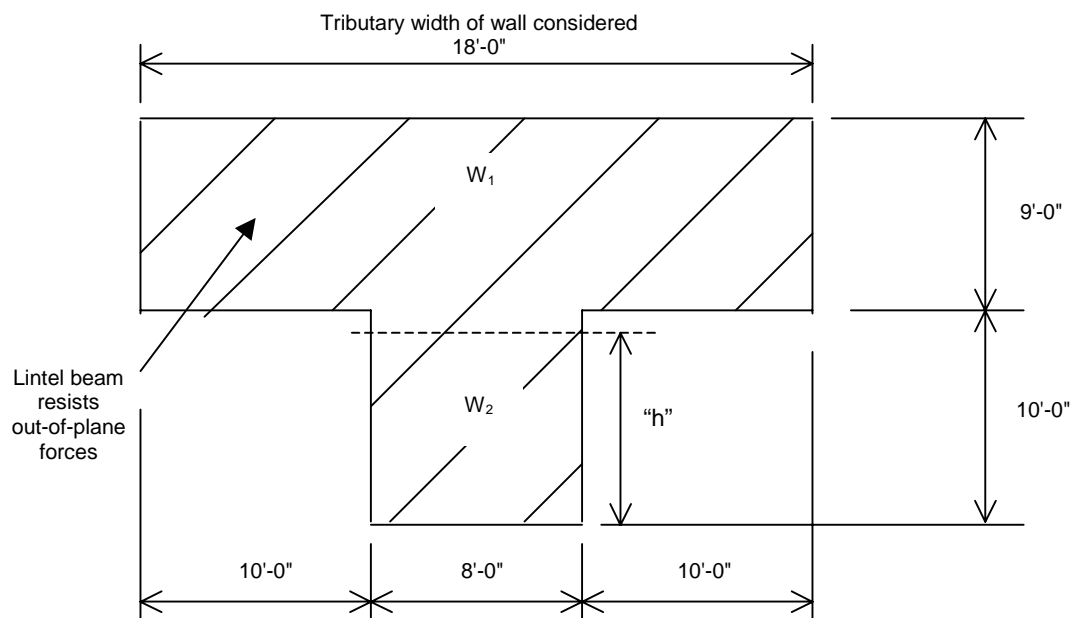


Figure 4-8. Tributary width of wall for out-of-plane seismic inertial force calculations

$$W_1 = (10 \text{ ft} + 8 \text{ ft} + 10 \text{ ft})(40.2 \text{ psf}) = 1,125 \text{ plf}$$

$$W_2 = 8 \text{ ft}(40.2 \text{ psf}) = 322 \text{ plf}$$

Using simple beam theory to calculate moment M_{oop} for out-of-plane forces, the location of maximum moment is at $h = 9.8$ feet:

$$M_{oop} = 15,530 \text{ lb-ft} = 186,360 \text{ lb-in.}$$

Comparison of seismic out-of-plane forces with wind (approximately 25 psf) indicate that seismic forces control the design.

4C.**Design for out-of-plane forces.****§1612.2.1**

The wall section shown in Figure 4-6 will be designed. The controlling load combinations for masonry are:

$$1.2D + 1.6L_r \quad (12-3)$$

$$1.1(1.2D + 1.0E) = 1.32D + 1.1(E_h + E_v) \quad (12-5)$$

$$1.1E_v = 1.1(0.5)C_a I D = 0.55(0.53)(1.0)D = 0.30D$$

Note: Exception 2 of §1612.2.1 requires that a 1.1 factor be applied to the load combinations for strength design of masonry elements including seismic forces. The SEAOC Seismology Committee has recommended that this factor be deleted. However; this example shows use of the factor because it is a present requirement of the code, thus:

$$P_{D+RLL} = 1.2(27,750 \text{ lb}) + 1.6(6840 \text{ lb}) = 44,244 \text{ lb} \quad (12-3)$$

$$\begin{aligned} P_u &= P_{D+L+E} = P_D + 1.1E_v \\ &= 1.32(27,750 \text{ lb}) + (0.30)(27,750 \text{ lb}) = 44,955 \text{ lb} \end{aligned} \quad (12-5)$$

The controlling load case by examination is Equation (12-5) for gravity plus seismic out-of-plane forces.

Slender wall design of masonry walls with an axial load of $0.04f'_m$ or less are designed under the requirements of §2108.2.4.4.

Check axial load vs. $0.04f'_m$ using unfactored loads:

$$\frac{P_w + P_f}{A_g} \leq 0.04f'_m$$

$$\frac{27,750 \text{ lb}}{(7.625 \text{ ft})(8 \text{ ft})(12 \text{ in.})} = 38 \text{ psi} \leq 0.04(2500 \text{ psi}) = 100 \text{ psi}$$

∴ o.k.

Calculate equivalent steel area A_{se} :

$$\begin{aligned} A_{se} &= \frac{A_s f_y + P_u}{f_y} \\ &= \frac{(0.31 \text{ in.}^2)(6 \text{ bars})(60,000 \text{ psi}) + 44,955 \text{ lb}}{60,000 \text{ psi}} = 2.61 \text{ in.}^2 \end{aligned} \quad (8-24)$$

Calculate I_{cr} :

$$a = \frac{(P_u + A_s f_y)}{.85 f'_m b} = \frac{44,955 \text{ lb} + (1.86 \text{ in.}^2)(60,000 \text{ psi})}{.85(2500 \text{ psi})(96 \text{ in.})} = 0.77 \text{ in.} \quad (8-25)$$

$$c = \frac{a}{.85} = 0.86 \text{ in.}$$

$$n = \frac{E_s}{E_m} = \frac{29,000,000 \text{ psi}}{1,875,000 \text{ psi}} = 15.46 \quad \text{§2106.2.12.1}$$

$$\begin{aligned} I_{cr} &= \frac{bc^3}{3} + nA_{se}(d - c)^2 \\ &= \frac{96 \text{ in.}(0.90 \text{ in.})^3}{3} + (15.46)(2.62 \text{ in.}^2)(3.81 \text{ in.} - 0.90 \text{ in.})^2 = 365.0 \text{ in.}^4 \end{aligned}$$

Calculate M_{cr} using the value for f_r from §2108.2.4.6, Equation (8-31):

$$M_{cr} = S_g f_r = \left(\frac{96 \text{ in.}(7.625 \text{ in.})^2}{6} \right) (4.0)(2,500)^{1/2} = 186,050 \text{ lb} \cdot \text{in.} \quad (8-30)$$

Calculate I_g :

$$I_g = \frac{(96 \text{ in.})(7.625 \text{ in.})^3}{12} = 3546.6 \text{ in.}^4$$

Calculate M_u based on Equation (8-20) of §2108.2.4.4:

First iteration for moment and deflection (note that eccentric moment at mid-height of wall is one-half of the maximum moment):

$$M_u = M_{\text{out-of-plane}} + M_{\text{eccentric}} = 1.1E + 1.1(1.2D) + 1.1(1.6)(L = 0)$$

$$M_u = M_{\text{out-of-plane}} + M_{\text{eccentric}} = 1.1(186,360 \text{ lb} \cdot \text{in.}) + 1.32(7,650 \text{ lb})(6 \text{ in.})/2 = 235,960 \text{ lb} \cdot \text{in.} \quad (8-20)$$

$$\Delta_u = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_u - M_{cr})h^2}{48E_mI_{cr}} \quad (8-28)$$

$$\Delta_u = \frac{5(186,050 \text{ lb} \cdot \text{in.})(192 \text{ in.})^2}{48(1,875,000 \text{ psi})(3,546.6 \text{ in.}^4)} + \frac{5(235,290 \text{ lb} \cdot \text{in.} - 186,050 \text{ lb} \cdot \text{in.})(192 \text{ in.})^2}{48(1,875,000 \text{ psi})(365.0 \text{ in.}^4)} = 0.11 \text{ in.} + 0.28 \text{ in.} = 0.38 \text{ in.}$$

Note: The deflection equation used is for uniform lateral loading, maximum moment at mid-height, and pinned-pinned boundary conditions. For other support and fixity conditions, moments and deflections should be calculated using established principals of mechanics. Beam deflection equations can be found in the AITC or AISC manuals or accurate methods can be derived.

Second iteration for moment and deflection:

$$M_u = 235,290 \text{ lb} \cdot \text{in.} + 44,955 \text{ lb}(0.38 \text{ in.}) = 252,540 \text{ lb} \cdot \text{in.}$$

$$\Delta_u = 0.11 \text{ in.} + \frac{5(252,540 \text{ lb} \cdot \text{in.} - 186,050 \text{ lb} \cdot \text{in.})(192 \text{ in.})^2}{48(1,875,000 \text{ psi})(365.0 \text{ in.}^4)}$$

$$= 0.11 \text{ in.} + 0.37 \text{ in.} = 0.48 \text{ in.}$$

Third iteration for moment and deflection:

$$M_u = 235,290 \text{ lb} \cdot \text{in.} + 44,955 \text{ lb} \cdot \text{in.}(0.48 \text{ in.}) = 256,891 \text{ lb} \cdot \text{in.}$$

$$\Delta_u = 0.11 \text{ in.} + \frac{5(256,891 \text{ lb} \cdot \text{in.} - 186,050 \text{ lb} \cdot \text{in.})(192 \text{ in.})^2}{48(1,875,000 \text{ psi})(365.0 \text{ in.}^4)}$$

$$= 0.11 \text{ in.} + 0.40 \text{ in.} = 0.51 \text{ in.}$$

Final moment (successive iterations are producing moments within 3 percent, therefore convergence can be determined):

$$M_u = 235,290 \text{ lb} \cdot \text{in.} + 44,955 \text{ lb}(0.51 \text{ in.}) = 258,217 \text{ lb} \cdot \text{in.}$$

Calculation of wall out-of-plane strength:

$$\phi M_u = \phi A_{se} f_y \left(d - \frac{a}{2} \right)$$

$$= 0.80(2.47 \text{ in.}^2)(60,000 \text{ psi}) \left(3.81 \text{ in.} - \frac{0.73 \text{ in.}}{2} \right)$$

$$= 408,439 \text{ lb} \cdot \text{in.} \geq 258,217 \text{ lb} \cdot \text{in.}$$

Since the wall strength is greater than the demand, the wall section shown in Figure 4-4 is okay.

Note that out-of-plane deflections need to be checked using same iteration process, but with service loads per §2108.2.4.6, (i.e., $P_D = 27,750 \text{ lbs}$). Since ultimate deflections are within allowable, there is no need to check service deflections in this example. The limiting deflection is $0.007h$ per §2108.2.4.6 is $0.007(16 \times 12) = 1.34"$. The deflection from this analysis is 0.50 inches. Thus the deflection is within allowable limits.

Check that the wall reinforcement is less than 50 percent of balanced reinforcement per §2108.2.4.2:

$$\rho_b = \frac{.85\beta_1 f'_m}{f_y} + \frac{87,000}{87,000 + f_y} = 0.0178$$

$$\rho = \frac{(6)(0.31 \text{ in.}^2)}{(3.81 \text{ in.})(96 \text{ in.})} = 0.0051 \leq 0.0089$$

∴ o.k.

Check the unbraced parapet moment:

$$a_p = 2.5$$

Table 16-0

$$R_p = 3.0$$

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p = \frac{(2.5)(.53)(1.0)}{(3.0)} \left(1 + 3 \left(\frac{16 \text{ ft}}{16 \text{ ft}} \right) \right) W_p$$

$$= 1.76 W_p = 1.76 (75 \text{ psf}) = 132.5 \text{ psf}$$

$$M_u (132.5 \text{ psf}) (3 \text{ ft})^2 / 8 = 596 \text{ lb} \cdot \text{ft} = 7,155 \text{ lb} \cdot \text{in.} \leq 408,439 \text{ lb} \cdot \text{in.}$$

∴ Wall section is okay at parapet.

5.

Design 8'-0" shear wall on line A for in-plane seismic forces.

5a.

Shear force distribution.

The shear force on line A must be distributed to three shear wall piers (6', 8', and 6' in width, respectively) in proportion to their relative rigidities. This can be accomplished by assuming that the walls are fixed at the tops by the 9-foot-deep lintel. Reference deflection equations are given below for CMU or concrete walls with boundary conditions fixed top or pinned top. For this Design Example, the fixed/fixed equations are used because the deep lintel at the wall/pier tops will act to fix the tops of wall piers.

$$\Delta_i = \frac{V_i h^3}{12 E_m I} + \frac{1.2 V_i h}{AG} \text{ for walls/piers fixed top and bottom}$$

$$\Delta_i = \frac{V_i h^3}{3 E_m I} + \frac{1.2 V_i h}{AG} \text{ for walls/piers pinned top and fixed at bottom}$$

$$G = 0.4 E_m \text{ for concrete masonry under §2106.2.12.13} \quad (6-6)$$

Relative rigidity is thus $\frac{1}{\Delta}$ where Δ is the deflection under load V_i . Using the fixed/fixed equation, the percentage shears to each wall are shown in Table 4-1.

Table 4-1. Distribution of line A shear to three shear walls.

Wall Length (ft)	Moment Deflection (in.)	Shear Deflection (in.)	Total Deflection (in.)	Rigidity (1/in.)	Distribution to Piers (%)	Wall Shear (k)
6	1.17E-05	3.50E-07	1.20E-05	83,28	26.6%	16.2
8	6.56E-06	2.62E-07	6.82E-06	146,63	46.8%	28.6
6	1.17E-05	3.50E-07	1.20E-05	83,28	26.6%	16.2
Totals				313,20	100%	61.0

The seismic shear force E_h to the 8-foot pier is $(0.468)61\text{ k} = 28.6\text{ k}$.

Calculation of reliability/redundancy factor ρ is shown below. For shear walls the maximum element story shear ratio r_i is determined as:

§1630.1.1

$$r_{i8} = (28.6\text{ k})(10) / 8\text{ ft} / 122\text{ k} = 0.29 \text{ for 8 ft segment}$$

$$r_{i6} = (16.2\text{ k})(10) / 6\text{ ft} / 122\text{ k} = 0.22 \text{ for 6 ft segment}$$

$$\therefore r_{max} = 0.29$$

$$\rho = 2 - \frac{20}{r_{max} \sqrt{A_B}} = 2 - \frac{20}{(0.29) \sqrt{5,400\text{ ft}^2}} \quad (30-3)$$

$$\therefore \rho = \underline{\underline{1.06}}$$

The strength design shear for the 8'-0" wall is:

$$\therefore V_{8'\text{ wall}} = 1.06(28.6\text{ k}) = \underline{\underline{30.3\text{ k}}}$$

5b.

Determination of shear strength.

The in-plane shear strength of the wall must be determined and compared to demand. The strength of the wall is determined as follows. Vertical reinforcement is #5@16 inches o.c. Try #4@16 inches o.c. horizontally. Note that concrete masonry cells are spaced at 8-inch centers, thus reinforcement arrangements must have spacings in increments of 8 inches (such as 8 inches, 16 inches, 24 inches, 32 inches, 40 inches, and 48 inches). Typical reinforcement spacings are 16 inches and 24 inches for horizontal and vertical reinforcement.

Calculate M/V_d :

$$\frac{M}{V_d} = \frac{151.5 \text{ k} \cdot \text{ft}}{(30.3 \text{ k})(8 \text{ ft})} = 0.625$$

From Table 21-K and by iteration, the nominal shear strength coefficient $C_d = 1.8$

$$V_n = V_m + V_s \quad (8-36)$$

$$V_m = C_d A_{mv} \sqrt{f_m} = (1.80)(7.625 \text{ in.})(96 \text{ in.})\sqrt{2,500 \text{ psi}} = 65.9 \text{ k} \quad (8-37)$$

$$V_s = A_{mv} \rho_n f_y \quad (8-38)$$

for $\phi = 0.80$, with #4 @ 16" o.c. horizontally:

$$\phi V_s = \phi A_{mv} \rho_n f_y = (0.80)(7.625 \text{ in.})(96 \text{ in.}) \left[\frac{(0.20 \text{ in.}^2)}{(7.625 \text{ in.})(16 \text{ in.})} \right] (60,000 \text{ psi}) = 57.6 \text{ k}$$

for $\phi = 0.60$, with #4 @ 16" o.c. horizontally:

$$\phi V_s = \phi A_{mv} \rho_n f_y = (0.60)(7.625 \text{ in.})(96 \text{ in.}) \left[\frac{(0.20 \text{ in.}^2)}{(7.625 \text{ in.})(16 \text{ in.})} \right] (60,000 \text{ psi}) = 43.2 \text{ k}$$

Thus, conservatively, using $\phi = 0.60$

$$\phi V_n = 0.6(65.9 \text{ k}) + 43.2 \text{ k} = 82.7 \text{ k}$$

The designer should check the failure mode. If failure mode is in bending, $\phi = 0.80$. If failure mode is in shear, $\phi = 0.60$. For this example, we will conservatively use $\phi = 0.60$. The method of checking the failure mode is to check how much moment M_u is generated when the shear force is equal to shear strength V_n with $\phi = 1.0$. Then that moment is compared with the wall P_n and M_n with a $\phi = 1.0$. If there is reserve moment capacity, there will be a shear failure. If not, there will be a bending failure. Later in the example this will be checked.

The reason the failure mode should be checked is to understand whether a brittle shear failure will occur or a ductile bending failure. Since the bending failure is more desirable and safer, the ϕ factor is allowed to be higher.

$$V_u = 1.1(30.3 \text{ k}) = 33.3 \text{ k} \leq \phi V_n = 82.7 \text{ k}, \text{ for } 0.60, \therefore \text{ o.k.}$$

\therefore Use #4 @ 16" horizontal reinforcement in the wall/pier.

6.**Design 8'-0" shear wall on line A for combined axial and in-plane bending actions.**

Part 5 illustrated the design of the wall for shear strength. This Part illustrates design for wall overturning moments combined with gravity loads. A free body diagram of the wall/pier is needed to understand the imposed forces on the wall.

The load combinations to be considered are specified in §1612.2.1. These are as follows (with the 1.1 factor of Exception 2 applied):

$$1.1(1.2D + 0.5L + 1.0E) \text{ (floor live load, } L = 0) \quad (12-5)$$

$$1.1(0.9D - 1.0E) \quad (12-6)$$

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

$$E_v = 0.5C_a ID = 0.5(0.53)(1.0)D = 0.27D \quad \S 1630.1.1$$

The resulting Equation (12-5) is:

$$1.1(1.2D + 0.27D + 1.0E_h) = 1.61D - 1.1E_h$$

The resulting Equation (12-6) is:

$$1.1(0.9D + 0.27D + 1.0E_h) = 0.63D - 1.1E_h$$

$$E_h = V_{8'-0" \text{ wall}} = 1.1(30.3 \text{ k}) = 33.3 \text{ k}$$

Axial loads P_u are calculated as P_{u1} and P_{u2} for load combinations of Equations (12-5) and (12-6):

$$P_{u1} = 1.61(27,750 \text{ lb}) = 44.7 \text{ kips}$$

$$P_{u2} = 0.63(27,750 \text{ lb}) = 17.5 \text{ kips}$$

By performing a sum of moments about the bottom corner at point A (Figure 4-9):

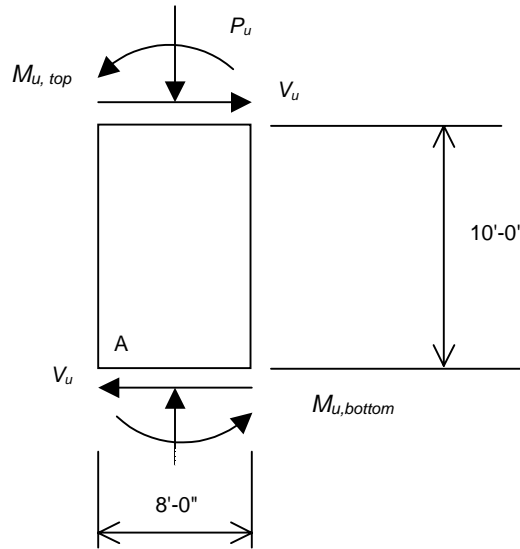


Figure 4-9. Free body diagram of 8'-0" shear wall

$$\sum M_A = 0 = 2M_u - V_u(10 \text{ ft})$$

$$M_{u,top} \approx M_{u,bottom} = \frac{(33.3 \text{ k})(10 \text{ ft})}{2} = 166.5 \text{ k} \cdot \text{ft}$$

The reader is referred to an excellent book for the strength design of masonry *Design of Reinforced Masonry Structures*, by Brandow, Hart, Verdee, published by Concrete Masonry Association of California and Nevada, Sacramento, CA, Second Edition, 1997. This book describes the calculation of masonry wall/pier strength design in detail.

The axial load vs. bending moment capacity (P-M) diagram for the wall must be calculated. For this, the designer must understand the controlling strain levels that define yielding and ultimate strength. At yield moment, the steel strain is the yielding strain (0.00207 in./in. strain) and the masonry strain must be below 0.002 in./in. (for under-reinforced sections). At ultimate strength, the masonry has reached maximum permissible strain (0.003 in./in.) and the steel strain is considered to have gone beyond yield strain level (see §2108.2.1.2 for a list of design assumptions). See Figure 4-10 for concrete masonry stress-strain behavior. A representation of these strain states is shown in Figures 4-11 and 4-12 (the pier width is defined as h).

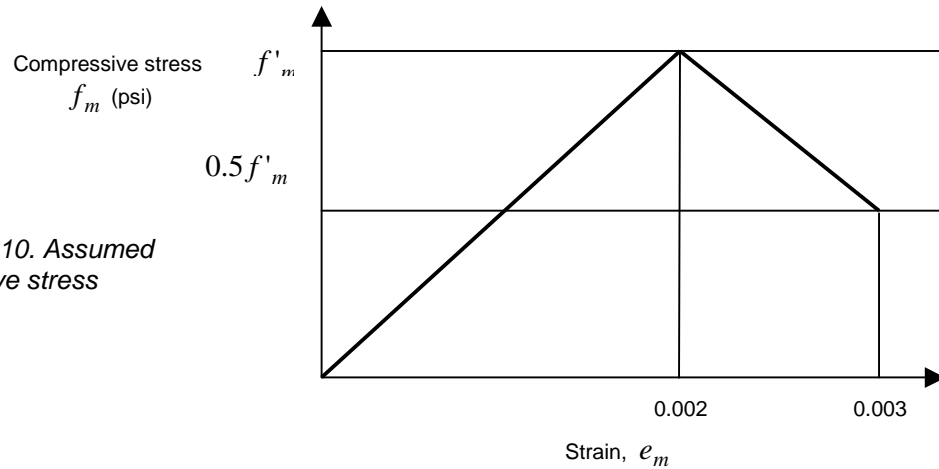


Figure 4-10. Assumed masonry compressive stress versus strain curve

Figure 4-11. Strain diagram at yield moment; steel strain = 0.00207 in./in.; masonry strain is less than yield for under-reinforced sections

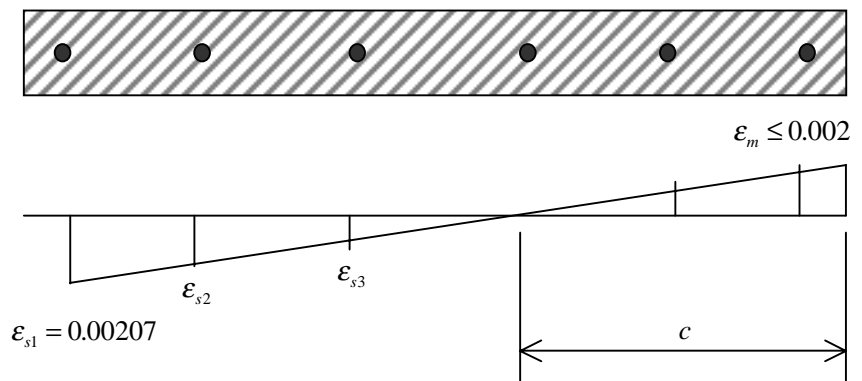
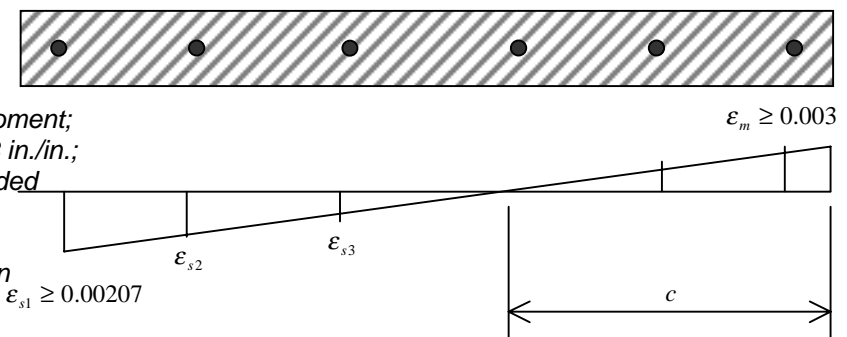


Figure 4-12. Strain diagram at ultimate moment; masonry strain = 0.003 in./in.; steel strain has exceeded 0.00207 in./in.; the Whitney stress block analysis procedure can be used to simplify calculations



Note that masonry strain may continue to increase with a decrease in stress beyond strains of 0.002 in./in. at which time stresses are at f'_m . At strains of 0.003, masonry stresses are $0.5f'_m$. With boundary element confinement, masonry strains can be as large as 0.006 in./in.

By performing a summation of axial forces F , the axial load in the pier is calculated as:

$$\sum F = P = C_1 = T_1 = T_2 = T_3$$

The corresponding yield moment is calculated as follows:

$$M_y = T_1 \left(d_1 - \frac{h}{2} \right) + T_2 \left(d_2 - \frac{h}{2} \right) + T_3 \left(d_3 - \frac{h}{2} \right) + C \left(\frac{h}{2} - \frac{c}{3} \right)$$

The ultimate moment is calculated as:

$$M_u = T_1 \left(d_1 - \frac{h}{2} \right) + T_2 \left(d_2 - \frac{h}{2} \right) + T_3 \left(d_3 - \frac{h}{2} \right) + C \left(\frac{h}{2} - \frac{a}{2} \right)$$

Strength reduction factors, ϕ , for in-plane flexure are determined by Equation (8-1) of §2108.1.4.1.1

$$\phi = 0.80 - \frac{P_u}{(A_e f'_m)}, 0.6 \leq \phi \leq 0.8 \quad (8-1)$$

Strength reduction factors for axial load, $\phi = 0.65$. For axial loads, ϕP_n , less than $0.10f'_m A_e$, the value of ϕ may be increased linearly to 0.85 as axial load, ϕP_n , decreases to zero.

The balanced axial load, P_b , is determined by Equations (8-2) and (8-3).

$$P_b = 0.85f'_m b a_b \quad (8-2)$$

$$a_b = 0.85d \left(\frac{e_m}{e_m + \frac{f_y}{E_s}} \right) \quad (8-3)$$

$$P_b = 0.85(2,500)(7.625 \text{ in.})(0.85)(92 \text{ in.})(0.003/0.00507) = 750 \text{ kips}$$

$$\phi P_b = 0.65(750 \text{ kips}) = 487 \text{ kips}$$

A P-M diagram can thus be developed. The P-M diagrams were calculated and plotted using a spreadsheet program. By observation, the design values P_u and M_u ($P_u = 43 \text{ k}$, $M_u = 167 \text{ k-ft}$) are within the nominal strength limits of ϕP_n , ϕM_n values shown in Figure 4-13. Plots for P_n vs. M_n can be seen in Figure 4-13 and for ϕP_n vs. ϕM_n in Figure 4-14.

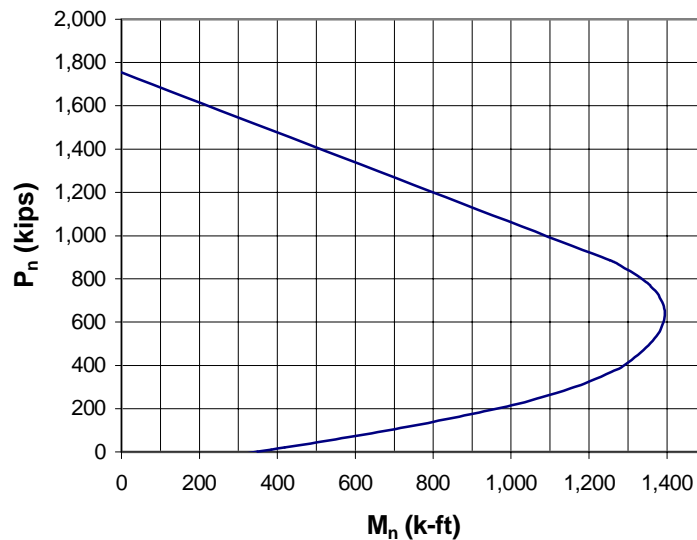


Figure 4-13. The P_n - M_n nominal strength curve with masonry strain at 0.003 in./in.

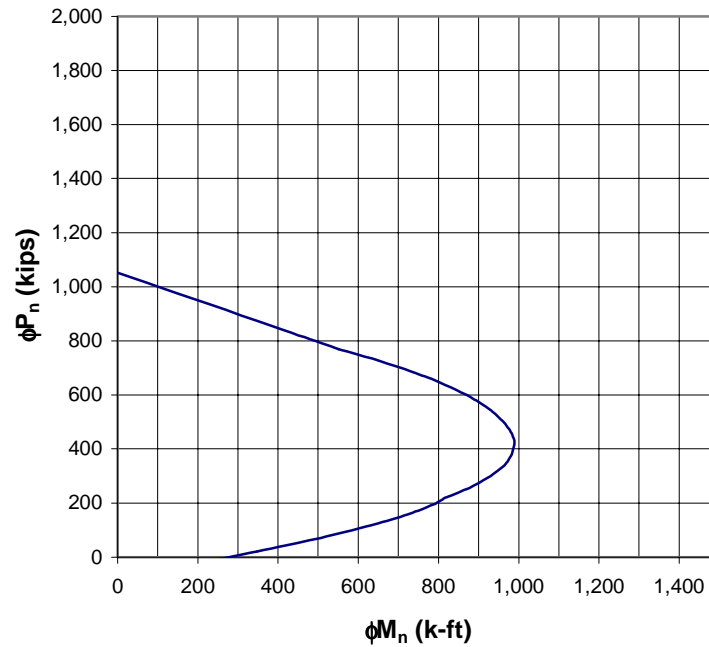


Figure 4-14. The ϕP_n - ϕM_n design strength curve with masonry strain at 0.003 in./in.

Check for type of wall failure by calculating wall moment at shear V_n :

$$M_u = \frac{V_n(10')}{2} = \frac{\left(\frac{82.7 \text{ k}}{0.60}\right)(10')}{2} = 689 \text{ k} \cdot \text{ft}$$

$$P_u = 43.7 \text{ k}$$

By looking at the $P_n - M_n$ curve, this $P_u - M_u$ load is just outside the P_n , M_n curve. The shear wall failure will likely be a bending failure. However, the designer might still consider a $\phi = 0.60$ for shear design to be conservative.

7.

Deflection of shear wall on line A.

§1630.10

In this part, the deflection of the shear wall on line A will be determined. This is done to check actual deflections against the drift limits of §1630.10.

Deflections based on gross properties are computed as:

$$\Delta_s = \frac{V_i h^3}{12E_m I} + \frac{1.2V_i h}{AG} \text{ for wall/piers fixed top and bottom}$$

$$\Delta_s = \frac{(28.6 \text{ k})(120 \text{ in.})^3}{12 (1,875 \text{ ksi})(562,176 \text{ in}^3)} + \frac{1.2 (28.6 \text{ k})(120 \text{ in.})}{(732 \text{ in}^2)(750 \text{ ksi})} = 0.011 \text{ in.}$$

Assume cracked section properties and $I_{cr} = 0.3I_g$ (approximately):

$$\Delta_s = \frac{(28.6 \text{ k})(120 \text{ in.})^3}{12 (1,875 \text{ ksi})(168,652 \text{ in}^3)} + \frac{1.2 (28.6 \text{ k})(120 \text{ in.})}{(732 \text{ in}^2)(750 \text{ ksi})} = 0.021 \text{ in.}$$

$$\Delta_m = 0.7R\Delta_s = 0.7(4.5)(0.021 \text{ in.}) = 0.066 \text{ in.} \quad (30-17)$$

Thus, deflections are less than $0.025h = 3.0 \text{ in.}$

\therefore o.k.

8.

Requirements for shear wall boundary elements.

§2108.2.5.6

Section §2108.2.5.6 requires boundary elements for CMU shear walls with strains exceeding 0.0015 in./in. from a wall analysis with $R = 1.5$. The intent of masonry boundary elements is to help the masonry achieve greater compressive strains (up to 0.006 in./in.) without experiencing a crushing failure.

The axial load and moment associated with this case is:

$$P_u = 44.7 \text{ kips}$$

$$M_u = \frac{4.5}{1.1} = \frac{(166.5 \text{ k} \cdot \text{ft})}{1.1} = 619 \text{ k} \cdot \text{ft}$$

This P-M point is not within the P-M curve using a limiting masonry strain of 0.0015 in./in. (see Figure 4-15). From an analysis it can be determined that the maximum c distance to the neutral axis is approximately 22 inches. For this example, boundary ties are required. Note that narrow shear wall performance is greatly increased with the use of boundary ties.

The code requires boundary elements to have a minimum dimension of $3 \times$ wall thickness, which is 24 inches due to yield moments. After yield moment capacity is exceeded, the c distance is reduced. Thus, if boundary element ties are provided at each end of the wall/pier extending 24 inches inward, the regions experiencing strain greater than 0.0015 in./in. are confined. Space boundary ties at 8-inch centers. The purpose of masonry boundary ties is not to confine the masonry for compression, but to support the reinforcement in compression to prevent buckling. Tests have been performed to show that masonry walls can achieve 0.006 in./in. compressive strains when boundary ties are present.

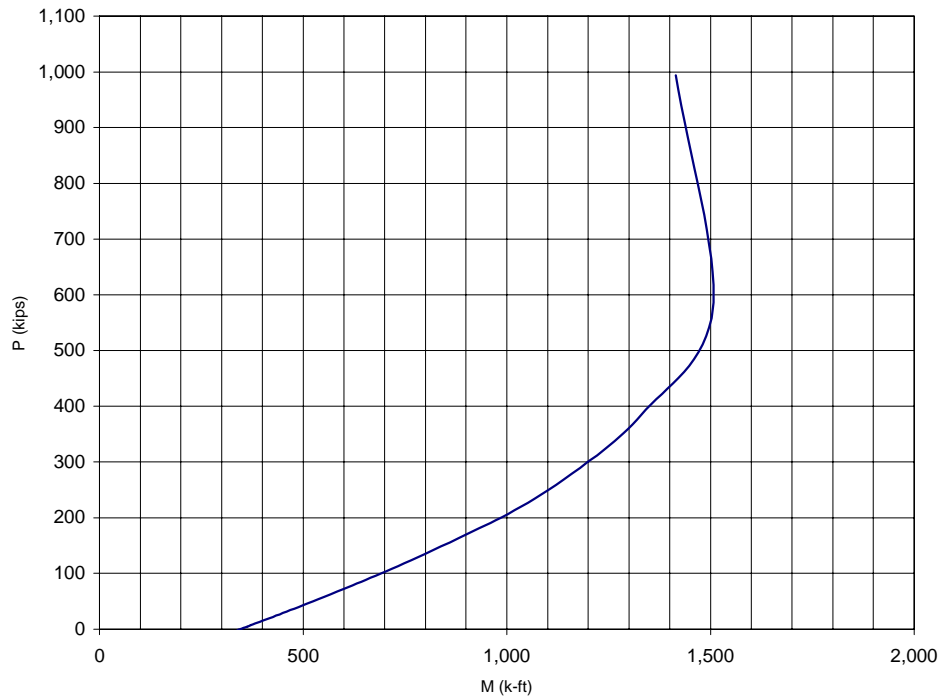


Figure 4-15. P-M curve for boundary element requirements; masonry strain is limited to 0.0015 in./in.

The P-M curve shown in Figure 4-15 is derived by setting masonry strain at the compression edge at 0.0015 in./in. and by increasing the steel tension strain at the opposite wall reinforcement bars. Moments are calculated about the center of the wall pier and axial forces are calculated about the cross-section. P-M points located at the outside of the denoted P-M boundary element curve will have masonry strains exceeding the allowable, and thus will require boundary element reinforcement or devices.

It can be seen that boundary reinforcement is required for the point ($P_u = 45\text{ k}$, $M_u = 619\text{ k}$). Boundary element confinement ties may consist of #3 or #4 closed reinforcement in 10-inch and 12-inch CMU walls. At 8-inch CMU walls pre-fabricated products such as the “masonry comb” are the best choice for boundary reinforcement because these walls are too narrow for reinforcement ties (even #3 and #4 bars). The boundary reinforcement should extend around three vertical #4 bars at the ends of the wall.

9.

Wall-roof out-of-plane anchorage for lines 1 and 3.

CMU walls should be adequately connected to the roof diaphragm around the perimeter of the building. In earthquakes, including the 1994 Northridge event, a common failure mode has been separation of heavy walls and roofs leading to partial collapse of roofs. A recommended spacing is 8'-0" maximum. However, 6'-0" or 4'-0" might be more appropriate and should be considered for many buildings. This anchorage should also be provided on lines A and D, which will require similar but different details at the roof framing perpendicular to wall tie tie condition. UBC §1633.2.9 requires that diaphragm struts or ties crossing the building from chord to chord be provided that transfer the out-of-plane anchorage forces through the roof diaphragm. Diaphragm design is presented in Design Example 5, and is not presented in this example.

Per §1633.2.8.1, elements of the wall out-of-plane anchorage system shall be designed for the forces specified in §1632 where $R_p = 3.0$ and $a_p = 1.5$.

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (32-2)$$

$$F_p = \frac{1.5(.53)(1.0)}{3.0} \left(1 + 3 \times \frac{16'}{16'} \right) W_p = 1.06 W_p$$

or:

$f_p = 1.06 w_p$, where w_p is the panel weight of 75 psf (see Figure 4-16) loading.

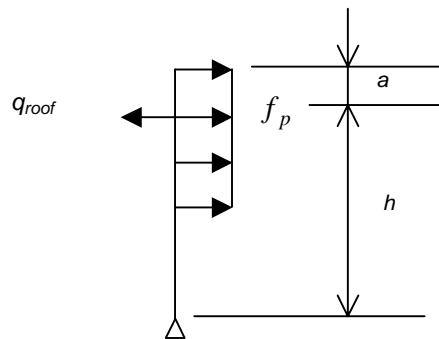


Figure 4-16. Wall-roof connection loading diagram

Calculation of the reaction at the roof level is:

$$q_{roof} = \frac{w_p(h+a)^2}{2h} = \frac{(1.06)(75 \text{ psf})(16 \text{ ft} + 3 \text{ ft})^2}{2(16 \text{ ft})} = 897 \text{ plf}$$

Section 1633.2.8.1 requires a minimum wall-roof anchorage of $q_{roof} = 420 \text{ plf}$

$$q_{roof} = 897 \text{ plf} \geq 420 \text{ plf}$$

$$\therefore \text{ use } q_{roof} = 897 \text{ plf}$$

The design anchorage reaction at different anchor spacings is thus:

$$\text{at 4'-0" centers, } q_{roof} = 3,588 \text{ lb}$$

$$\text{at 6'-0" centers, } q_{roof} = 5,382 \text{ lb}$$

$$\text{at 8'-0" centers, } q_{roof} = 7,175 \text{ lb}$$

Therefore, choose wall-roof anchors that will develop the required force at the chosen spacing. The roof diaphragm must also be designed to resist the required force with the use of subdiaphragms (or other means). The subject of diaphragm design is discussed in Design Example 5.

For this example, a double holdown connection spaced at 8'-0" centers will be used (see Figure 4-19). This type of connection must be secured into a solid roof framing member capable of developing the anchorage force.

First check anchor capacity in concrete block of Tables 21-E-1 and 21-E-2 of Chapter 21. Alternately, the strength provisions of §2108.1.5.2 can be used.

The required tension, T , for bolt embedment is $T = E/1.4 = 7,175 \text{ lbs}/1.4 = 5,125 \text{ lb}$. For $\frac{3}{4}$ -inch diameter bolts embedded 6 inches, $T = 2,830 \text{ lb}$ per Table 21-E-1 and $3,180 \text{ lb}$ per Table 21-E-2. These values are for use with allowable stress design (ASD).

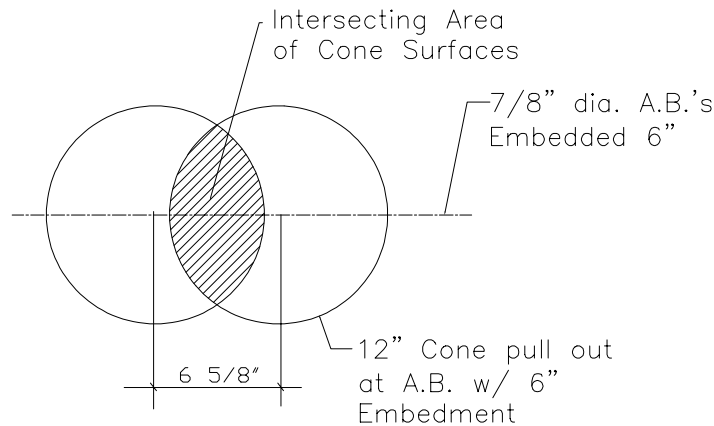


Figure 4-17. Intersection of anchor bolt tension failure cones

The anchor bolts are spaced at 6-5/8 inches center to center (considering purlin and hardware dimensions) and have 12-inch diameter pull-out failure cones. Thus, the failure surfaces will overlap (Figure 4-17). In accordance with §2108.1.5.2, the maximum tension of this bolt group may be determined as follows:

Calculate B_{tn} per bolt using the strength provisions of Equation (8-5):

$$B_{tn} = 1.04A_p\sqrt{f'_m} = 1.04(113\text{ in.}^2)(50\text{ psi}) = 5,876\text{ lb} \quad (8-5)$$

Calculate one-half the area of intersection of failure surfaces from two circles with radius 6 inches and centers $(2\text{--}1/16" + 2\frac{1}{2}" + 2\text{--}1/16")$ 6 5/8" apart. $A_p = 37.8\text{ in.}^2$ from Equations (8-7) and (8-8). Thus the bolt group tension can be calculated as:

$$(1.0)\left(2 \times 113\text{ in.}^2 - 2 \times 37.8\text{ in.}^2 / 2\right)(50\text{ psi}) = 9,410\text{ lb}$$

$$\phi B_m \geq B_{tu} \therefore 0.8(9,410\text{ lb}) = 7,528\text{ lb} \geq 7,175\text{ lb}$$

\therefore o.k.

By choosing a pair of pre-fabricated holdown brackets with adequate capacity for a double shear connection into a 2½-inch glued-laminated framing member, the brackets are good for $2 \times 3,685\text{ lb} = 7,307\text{ lb}$ (ASD) $> 7,175\text{ lb} \times 1.4$ steel element factor/1.4 ASD factor = 7,175 lb. Thus, the brackets are okay.

Also check bolt adequacy in the double shear holdown connection with metal side plates (2½-inch main member, 7/8-inch bolts) per NDS Table 8.3B.

$T = 2 \times 3,060 \text{ lb} \times 1.33 = 8,140 \text{ lb} > 7,175 \text{ lb}$, if the failure is yielding of bolt (Mode III_s or IV failure). If the failure is in crushing of wood (Mode I_m failure), the required force is $0.85 \times 5,125 \text{ lb} = 4,356 \text{ lb}$. Therefore, the double shear bolts and pre-fabricated holdown brackets can be used.

Thus, use two holdown brackets on each side of a solid framing member connecting the masonry wall to the framing member with connections spaced at 8'-0" centers.

Verify that the CMU wall can span laterally 8'-0" between anchors. Assume a beam width of 6'-0" (3' high parapet plus an additional three feet of wall below roof) spanning horizontally between wall-roof ties.

$$w = q_{\text{roof}} = 897 \text{ plf}$$

$$M_u = \frac{wl^2}{8} = \frac{(897 \text{ plf})(8 \text{ ft})^2}{8} = 7,176 \text{ lb} \cdot \text{ft}$$

The wall typically has #4@16-inch horizontal reinforcement, therefore a minimum 4-#4 bars in 6'-0" wall section.

$$a = \frac{A_s f_y}{.85 f'_m b} = \frac{4 (20 \text{ in.}^2) (60,000 \text{ psi})}{.85 (2,500 \text{ psi}) (72 \text{ in.})} = 0.314 \text{ in.}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$\phi M_n = 0.8 (4) (20 \text{ in.}^2) (60,000 \text{ psi}) \left(3.81 \text{ in.} - \frac{.314 \text{ in.}}{2} \right) \left(\frac{1}{12 \text{ in.}} \right) = 11,689 \text{ lb} \cdot \text{ft} \leq 7,176 \text{ lb} \cdot \text{ft}$$

$\therefore o.k.$

Per §1633.2.8.1, item 5, the wall-roof connections must be made with 2½-inch minimum net width roof framing members (2½-inch GLB members or similar) and developed into the roof diaphragm with diaphragm nailing and subdiaphragm design.

Anchor bolt embedment and edge distances are controlled by §2106.2.14.1 and §2106.2.14.2. Section 2106.2.14.1 requires that the shell of the masonry unit wall next to the wood ledger have a hole cored or drilled that allows for 1-inch grout all around the anchor bolt. Thus, for a 7/8-inch diameter anchor bolt, the core hole is 2-7/8-inch in diameter at the inside face masonry unit wall. Section 2106.2.14.2 requires that the anchor bolt end must have 1½ inches clearance to the outside face of masonry. The face shell thickness for 8-inch masonry is 1¼ inches, thus the anchor bolt end distance to the inside face of the exterior shell is 7-5/8"-1¼"-6" = 3/8". It is recommended that the minimum clear dimension is ¼-inch if fine grout is used and ½-inch if coarse pea gravel grout is used (Figure 4-18).

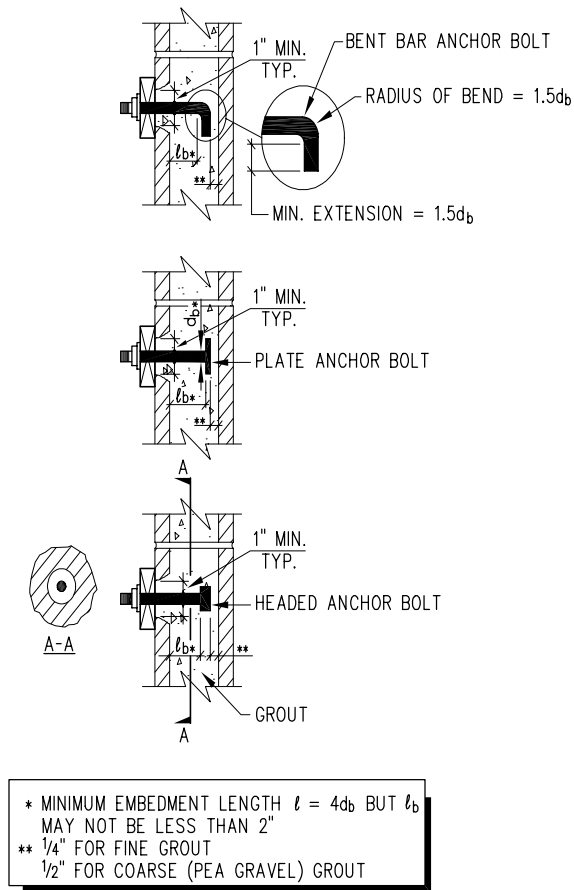


Figure 4-18. Embedment of anchor bolts in CMU walls (MIA, 1998)

10.**Chord design.**

Analysis of transverse roof diaphragm chords is determined by calculation of the diaphragm simple span moment ($wl^2/8$) divided by the diaphragm depth.

$$w_{diaph, trans.} = \frac{(72k + 50k)}{90'} = 1,356 \text{ plf}$$

Modify w for $R = 4.0$ by factor $(4.5/4.0) = 1.125$

§1633.2.9, Item 3

$$M_{diaph.} = wl^2/8 = 1.125(1,356 \text{ plf})(90 \text{ ft})^2/8 = 1,545 \text{ k} \cdot \text{ft}$$

$$T_u = C_u = 1,545 \text{ k} \cdot \text{ft} / 60 \text{ ft} = 25.7 \text{ kips}$$

Using reinforcement in the CMU wall for chord forces:

$$A_{s, required} = \frac{T_u}{\phi f_y} = \frac{25.7 \text{ k}}{(0.80)(60 \text{ ksi})} = 0.54 \text{ in.}^2$$

Thus 2-#5 chord bars ($A_s = 0.62 \text{ in.}^2$) are adequate to resist the chord forces. Place chord bars close to the roof diaphragm level. Since roof framing often is sloped to drainage, the chord placement is a matter of judgment.

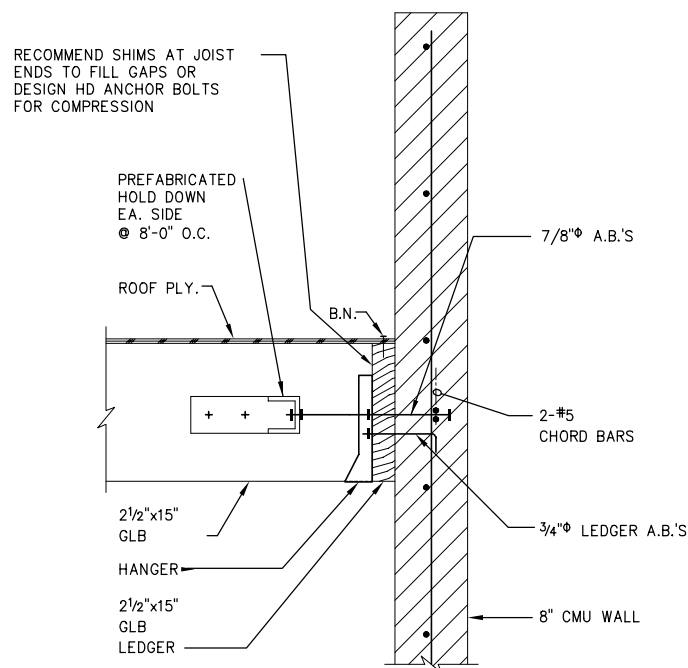


Figure 4-19. CMU wall section at wall-roof ties

References

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- MIA, 1998, *Reinforced Concrete Masonry Construction Inspector's Handbook: Conforming to the 1997 UBC*. Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, California 90057.
- Paulay, T. and Priestly, M.J.N., 1992, *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, Inc., New York.
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Design Example 5

Tilt-Up Building

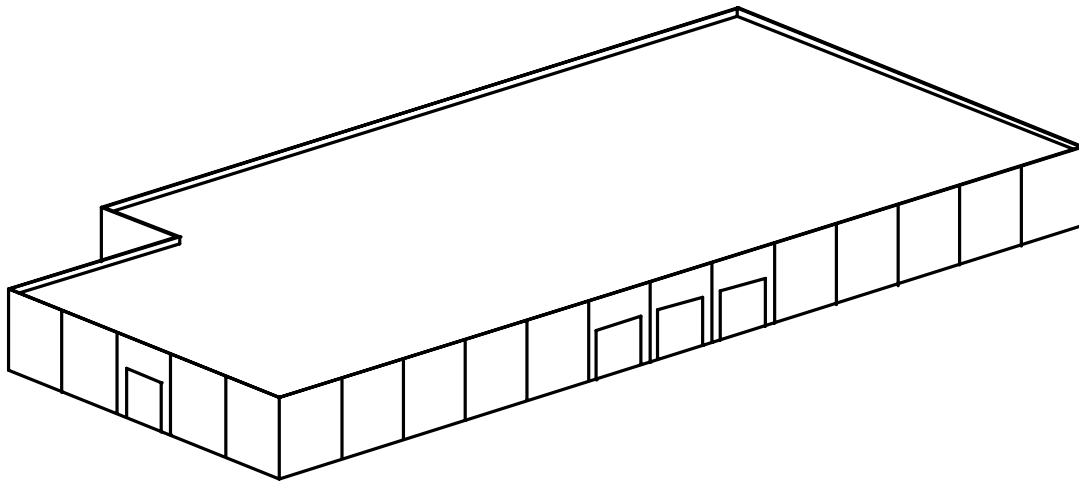


Figure 5-1. Tilt-up building of Design Example 5

Overview

In this example, the seismic design of major components of a tilt-up building are presented. Many tilt-up buildings have suffered severe structural damage in earthquakes, particularly during the 1971 San Fernando and 1994 Northridge events. The most common problem is wall-roof separation, with subsequent partial collapse of the roof. In the 1997 UBC, substantial improvements, including higher wall-roof anchorage forces, have been added to help prevent the problems that appeared in tilt-up buildings built to codes as recent as the 1994 UBC.

The example building is the warehouse shown in Figure 5-1. This building has tilt-up concrete walls and a panelized plywood roof system. The building's roof framing plan is shown in Figure 5-2, and a typical section through the building is given in Figure 5-3. The emphasis in this Design Example 5 is the seismic design of the roof diaphragm, wall-roof anchorage, and a major collector.

Outline

This example will illustrate the following parts of the design process:

1. Design base shear coefficient.
2. Design the roof diaphragm.
3. Design typical north-south subdiaphragm.
4. Design wall-roof ties for north-south subdiaphragm.
5. Design continuity ties for north-south direction.
6. Design of collector along line 3 between lines B and C.
7. Required diaphragm chord for east-west seismic forces.
8. Required wall panel reinforcing for out-of-plane forces.
9. Deflection of east-west diaphragm.
10. Design shear force for east-west panel on line 1.

Given Information

The following information is given:

Roof:

dead load = 14.0 psf

Walls:

thickness = 7.25"

height = 23'

normal weight concrete = 150 pcf

$f'_c = 4,000$ psi

A615, Grade 60 rebar ($f_y = 60$ ksi)

Roof sheathing:

Structural I plywood

Seismic and site data:

$Z = 0.4$ (Zone 4)

$I = 1.0$ (Standard occupancy)

seismic source type = B

distance to seismic source = 13km

soil profile type = S_D

$\rho_{NS} = 1.0$

$\rho_{E/W} = 1.5$ (due to short wall on line 3)

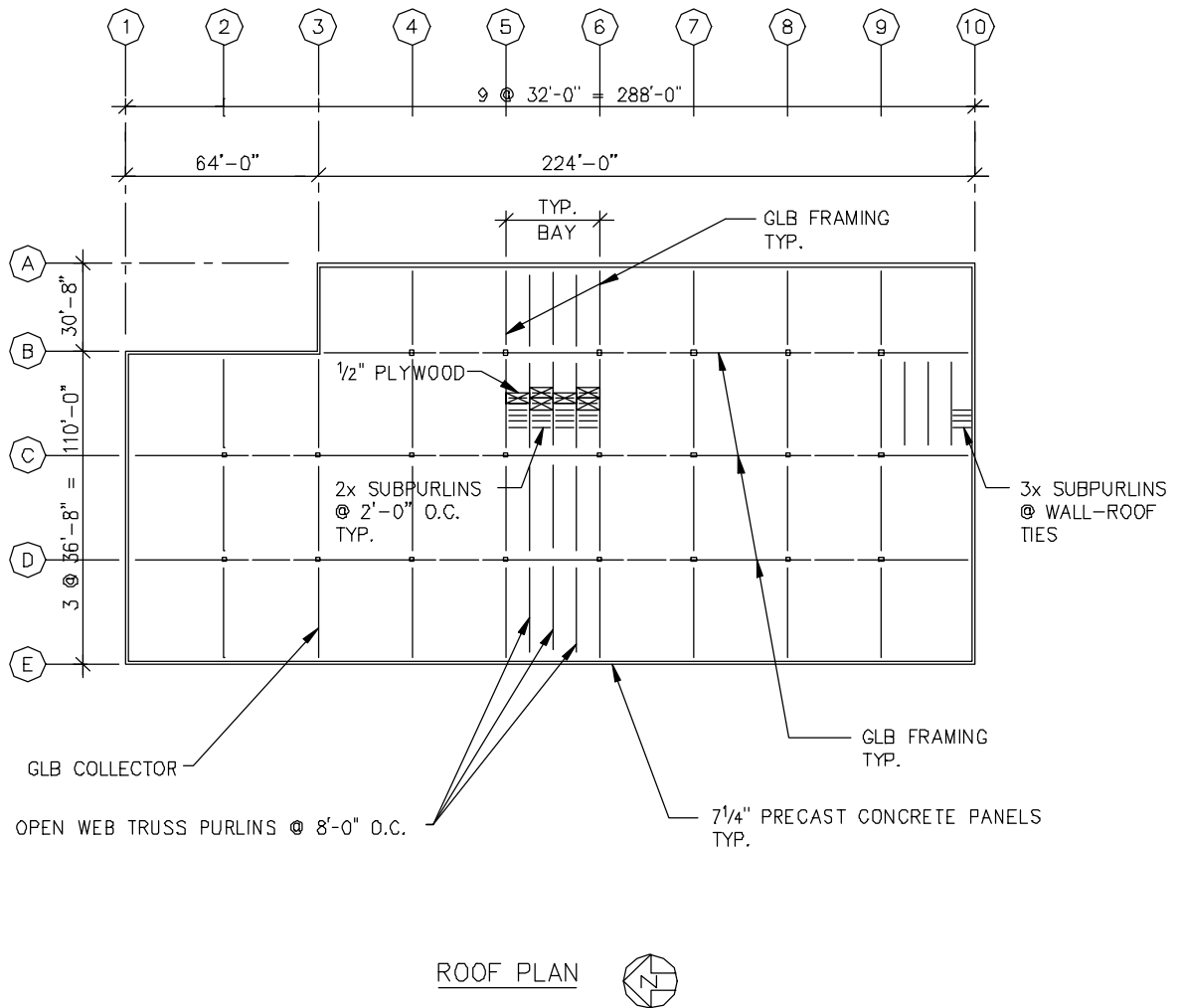


Figure 5-2. Roof framing plan of tilt-up building

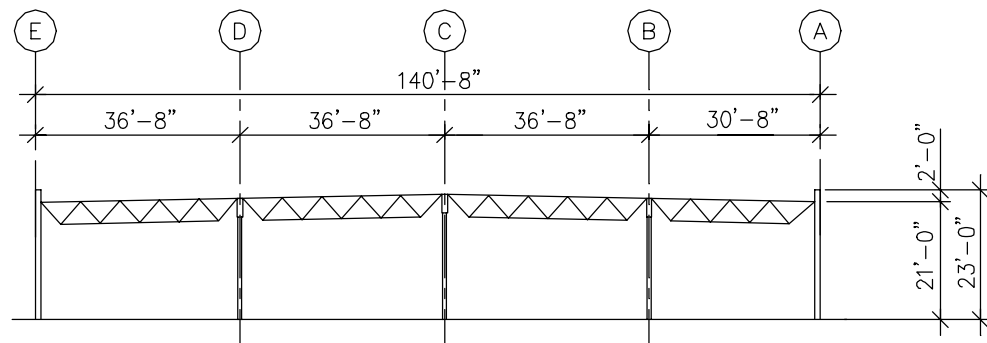


Figure 5-3. Typical cross-section

Calculations and Discussion

Code Reference

1.

Design base shear coefficient.**§1630.2.2**

Using Method A, the period is calculated as:

$$T = C_t(h_n)^{3/4} = .020(21)^{3/4} = .20 \text{ sec} \quad (30-8)$$

Comment: The building's lateral force-resisting system has relatively rigid walls and a flexible roof diaphragm. The code formula for period does not take into consideration that the real period of the building is highly dependent on the roof diaphragm construction. Consequently, the period computed above using Equation (30-8) is not a good estimate of the real fundamental period of the building, however it is acceptable for determining design base shear.

With seismic source type B and distance to source = 13 km

$$N_a = 1.0 \quad \text{Table 16-S}$$

$$N_v = 1.0 \quad \text{Table 16-T}$$

For soil profile type S_D and Z = .4

$$C_a = .44N_a = .44(1.0) = .44 \quad \text{Table 16-Q}$$

$$C_v = .64N_v = .64(1.0) = .64 \quad \text{Table 16-R}$$

Since tilt-up concrete walls are both shear walls and bearing walls:

$$R = 4.5 \quad \text{Table 16-N}$$

Design base shear is calculated from:

$$V = \frac{C_v I}{RT} W = \frac{.64(1.0)}{4.5(.21)} W = .677W \quad (30-4)$$

but base shear need not exceed:

$$V = \frac{2.5C_a I}{R} W = \frac{2.5(.44)(1.0)}{4.5} W = .244W \quad (30-5)$$

A check of Equations (30-6) and (30-7) indicate these do not control, therefore the base shear in both directions is

$$V = \underline{\underline{.244W}}$$

Note that the base shear is greater than that required under the 1994 UBC. The principal reason for this is that base shear under the 1997 UBC is determined on a strength design basis. If allowable stress design (ASD) is used, the base shear is divided by 1.4 according to §1612.3.

2.**Design the roof diaphragm.****2a.****Roof diaphragm weight.**

Seismic forces for the roof are computed from the weight of the roof and the tributary weights of the walls oriented perpendicular to the direction of the seismic forces. This calculation is shown below:

$$\text{roof area} = 110 \text{ ft } (64 \text{ ft}) + 140.67 \text{ ft } (224 \text{ ft}) = 38,550 \text{ sq ft}$$

$$\text{roof weight} = 38,550 \text{ sq ft } (14 \text{ psf}) = 539.7 \text{ kips}$$

$$\text{wall weight} = \frac{7.25}{12} \times 150 = 90.6 \text{ psf}$$

$$\text{north-south walls} = 90.6 \text{ psf } (2 \text{ ft} + 10.5 \text{ ft}) (140.67 \text{ ft}) (2) = 318.6 \text{ kips}$$

$$\text{east-west walls} = 90.6 \text{ psf } (2 \text{ ft} + 10.5 \text{ ft}) (288 \text{ ft}) (2) = 652.3 \text{ kips}$$

In this example, the effect of any wall openings has been neglected. This is considered an acceptable simplification because the openings usually occur in the bottom half of the wall.

2b.**Roof diaphragm shear.**

The roof diaphragm must be designed to resist seismic forces in both directions. The following formula is used to determine the total seismic force, F_{px} , on the diaphragm at a given level of a building.

In general, separate forces are computed for each direction.

$$F_{px} = \frac{F_t + \sum_{i=x}^n F_i}{\sum_{i=x}^n W_i} W_{px} \quad (33-1)$$

Base shear for this building is $V = .244W$. This was determined using $R = 4.5$ as shown in Part 1 above. For diaphragm design, however, §1633.2.9 requires that R not exceed 4. Since this is a one-story building with $F_t = 0$, and using $R = 4$, Equation (33-1) becomes the following:

$$F_{px} = \frac{4.5}{4} \left(\frac{V}{W} \right) W_{px} = \frac{4.5}{4} (.244) W_{px} = 0.275 W_{px}$$

$$F_{px} \text{ need not exceed } 1.0 C_a I W_{px} = 1.0 (.44)(1.0) W_{px} = .44 W_{px} \quad \S 1633.2.9$$

$$\text{but cannot be less than } 0.5 C_a I W_{px} = 0.5 (.44)(1.0) W_{px} = .22 W_{px} \quad \S 1633.2.9$$

Therefore, for diaphragm design use $F_{px} = \underline{\underline{.275 W_{px}}}$

Note: The reliability/redundancy factor ρ is not applied to horizontal diaphragms, except transfer diaphragms. (Refer to Examples 15 and 16 in Volume I of the *Seismic Design Manual* for a discussion of the ρ factor.)

North-south direction:

$$W_{px} = 539.7 \text{ k} + 318.6 \text{ k} = 858.3 \text{ kips}$$

$$F_{px} = .275 (858.3) = 236.0 \text{ kips}$$

The equivalent uniform load on the diaphragm can be computed as:

$$w = \frac{236.0 \text{ kips}}{140.67'} = 1,678 \text{ plf}$$

In this calculation, an approximation has been made that the uniform load between lines A and B is the same as that between B and E. The actual load on the A-B segment is less, and the load on the B-E segment is slightly greater than that shown. This has been done to simplify the computations.

Because the panelized wood roof diaphragm in this building is considered flexible (see §1630.6 for definition of flexible diaphragm), lines A, B and E are considered lines of resistance for the north-south seismic forces. A collector is needed along line B to drag the tributary north-south diaphragm forces into the shear wall on line B. The shear diagram is shown below.

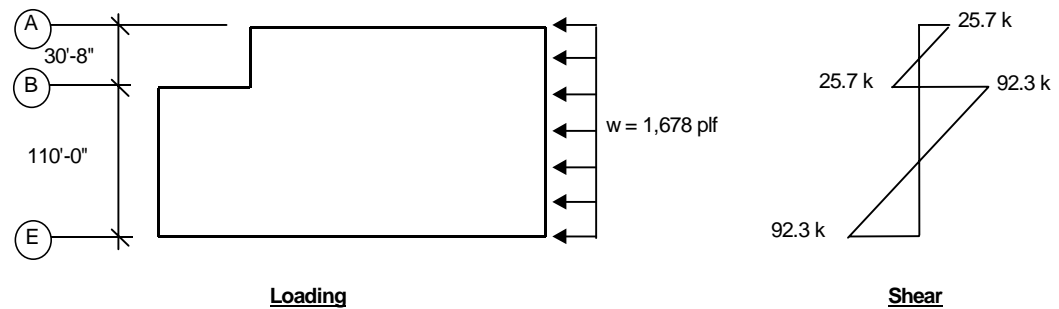


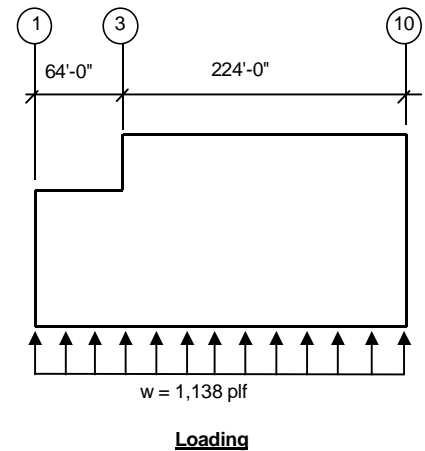
Figure 5-4. Seismic loading and shear diagram for north-south diaphragm

Diaphragm shear at line A and on the east side of line B is:

$$\frac{25,700\text{ lbs}}{224'} = 115 \text{ plf}$$

Diaphragm shear at the west side of line B and at line E is:

$$\frac{92,300 \text{ lb}}{288 \text{ ft}} = 320 \text{ plf}$$



East-west direction:

Diaphragm forces for the east-west direction are computed using the same procedure and assumptions as the north-south direction. The actual load on segment 1-3 is less than that shown, and the load on 3-10 slightly greater.

$$W_{px} = 539.7 \text{ k} + 652.3 \text{ k} = 1,192.0 \text{ kips}$$

$$F_{px} = .275 (1,192.0 \text{ k}) = 327.8 \text{ kips}$$

$$\text{Equiv. } w = \frac{327.8 \text{ k}}{288 \text{ ft}} = 1,138 \text{ plf}$$

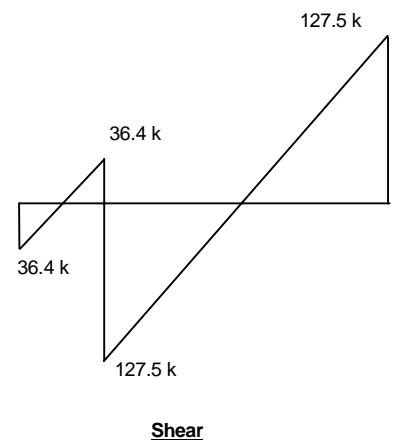


Figure 5-5. Seismic loading and shear diagram for east-west diaphragm

Diaphragm shear at line 1 and the north side of line 3 is:

$$\frac{36,400 \text{ lb}}{110 \text{ ft}} = 331 \text{ plf}$$

Diaphragm shear at the south side of line 3 and at line 10 is:

$$\frac{127,500 \text{ lb}}{140.67 \text{ ft}} = 906 \text{ plf}$$

2C.

Design of east-west diaphragm.

The east-west diaphragm has been selected to illustrate the design of a plywood roof diaphragm. Allowable stress design (ASD) will be used. The basic earthquake loading combination is given by Equation (30-1). When ASD is used, vertical effects need not be considered, and in this example of the diaphragm design, they would not come into use even if strength design was being used. As discussed earlier, the reliability/redundancy factor does not apply to the diaphragm, and $p=1$ in Equation (30-1).

$$E = \rho E_h + E_v = 1.0E_h + 0 = 1.0E_h \quad (30-1)$$

For ASD, the basic load combination to be used to combine earthquake and dead load is Equation (12-9). This simplifies to the following:

$$D + \frac{E}{1.4} = 0 + \frac{E}{1.4} = \frac{E}{1.4} \quad (12-9)$$

Assume the diaphragm is to be constructed with ½-inch Structural I plywood with all edges supported. Refer to use UBC Table 23-II-H for nailing requirements. Sheathing arrangement (shown in Figure 5-2) for east-west seismic forces is Case 4. Diaphragm shear forces must be divided by 1.4 to convert to ASD. Because open web truss purlins with double 2x4 chords are used in this direction, the framing width in the east-west direction is 3½ inches. However, in the north-south direction, the framing consists of 2× subpurlins, and strength is therefore limited by the 2-inch nominal width. Required nailing for panel edges for various zones of the roof (for east-west seismic only) is given in Table 5-1 below. Minimum field nailing is 10d @ 12 inches. A similar calculation (not shown) must be done for north-south seismic forces.

Table 5-1. Diaphragm nailing for east-west seismic forces

Zone	Boundary and East-West Edge Nailing ⁽¹⁾	North-South Edge Nailing ⁽²⁾	Allowable Shear	ASD Shear	Status
A	10d @ 2½"	4"	640 plf	906/1.4 = 647 plf	say o.k.
B	10d @ 4"	6"	425 plf	583/1.4 = 416 plf	o.k.
C	10d @ 6"	6"	320 plf	331/1.4 = 236 plf	o.k.

Notes:

1. The east-west running sheet edges are the "continuous panel edges parallel to load" mentioned in Table 23-II-H.
2. The north-south sheet edges are the "other panel edges" in Table 23-II-H. Note that the nailing for north-south running diaphragm boundaries is 10d @ 2½ inches.

The demarcation between nailing zones A and B is determined as follows. It was decided to use 10d at 2½-inch spacing in A and 4-inch spacing in B. The limiting shear for 10d at 4 inches (from Table 23-II-H) is 425 plf. Shear reduces from a maximum of 906 plf at lines 3 and 10 to 595 plf (i.e., $425 \text{ plf} \times 1.4 = 595 \text{ plf}$) at 38.4 feet from lines 3 and 10. Rounding to the nearest 8-foot increment because purlins are spaced at 8 feet o.c., zone A extends a distance of 40 feet from lines 3 and 10 as shown below.

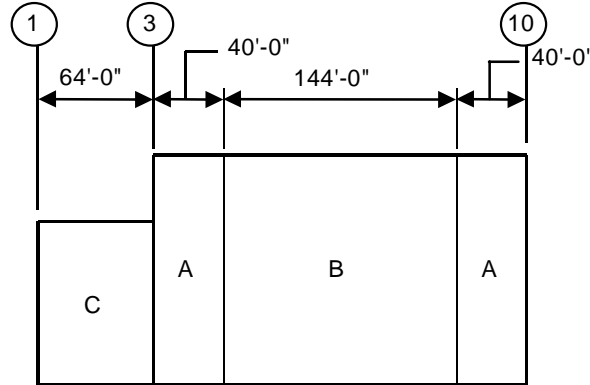


Figure 5-6. Nailing zones for east-west roof diaphragm

The above illustrates design of the east-west diaphragm for shear. Design of the chord for the east-west diaphragm is shown in Part 7 of this example. Design of ledger bolts, required to transfer the diaphragm shear to the wall panels, is not shown.

3.**Design typical north-south subdiaphragm.**

Subdiaphragms are used to transfer out-of-plane seismic forces from the tilt-up wall panels to the main diaphragm. Consequently, subdiaphragms are considered to be part of the wall anchorage system as defined in §1627. In the example below, design of a typical subdiaphragm for north-south seismic forces is shown. Design of subdiaphragm for east-west seismic forces is similar but not shown.

3a.**Check subdiaphragm aspect ratio.**

Maximum allowable subdiaphragm ratio is 2.5 to 1

§1633.2.9

From Figure 5-2, the maximum north-south subdiaphragm span = $\frac{110 \text{ ft}}{3} = 36.67 \text{ ft}$

Minimum subdiaphragm depth = $\frac{36.67 \text{ ft}}{2.5} = 14.67 \text{ ft}$

Typical roof purlin spacing = 8' – 0"

Minimum subdiaphragm depth = 16' – 0"

∴ Must use subdiaphragm at least = 16' – 0" deep

3b.**Forces on subdiaphragm.**

Because subdiaphragms are part of the out-of-plane wall anchorage system, they are designed under the requirements of §1633.2.8.1. Seismic forces on a typical north-south subdiaphragm are determined from Equation (32-2) with $R_p = 3.0$ and $a_p = 1.5$.

$$w_p = 90.6 \text{ psf}$$

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (32-2)$$

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.

The tributary wall weight is one-half of the weight between the roof and base *plus* all of the weight above the roof.

$$W_p = 90.6 \text{ psf} (2 \text{ ft} + 10.5 \text{ ft})(1 \text{ ft}) = 1,133 \text{ lb/ft}$$

$$F_p = \frac{1.5(.44)1.0}{3.0} \left(1 + 3 \times \frac{21}{21} \right) W_p = .88 W_p$$

Solving for the uniform force per foot, q , at the roof level

$$q = .88 W_p = .88(1,133) = 997 \text{ plf}$$

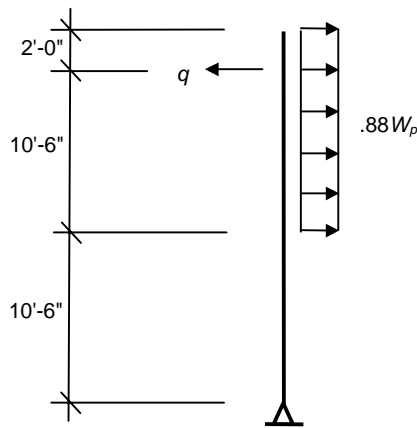


Figure 5-7. Loading diagram for wall-roof anchorage design

Check minimum wall-roof anchorage force

$$997 \text{ plf} > 420 \text{ plf} \quad o.k.$$

§1633.2.8.1(1)

$$\therefore q = \underline{\underline{997 \text{ plf}}}$$

3C.

Check subdiaphragm shear.

Assume a 32-foot deep subdiaphragm as shown below. This is done for two reasons. First, the GLB along Line 9 can be used as a chord. Second, the deeper than required subdiaphragm depth (32 feet vs. 16 feet) makes the subdiaphragm displacement more compatible with that of the main north-south diaphragm.

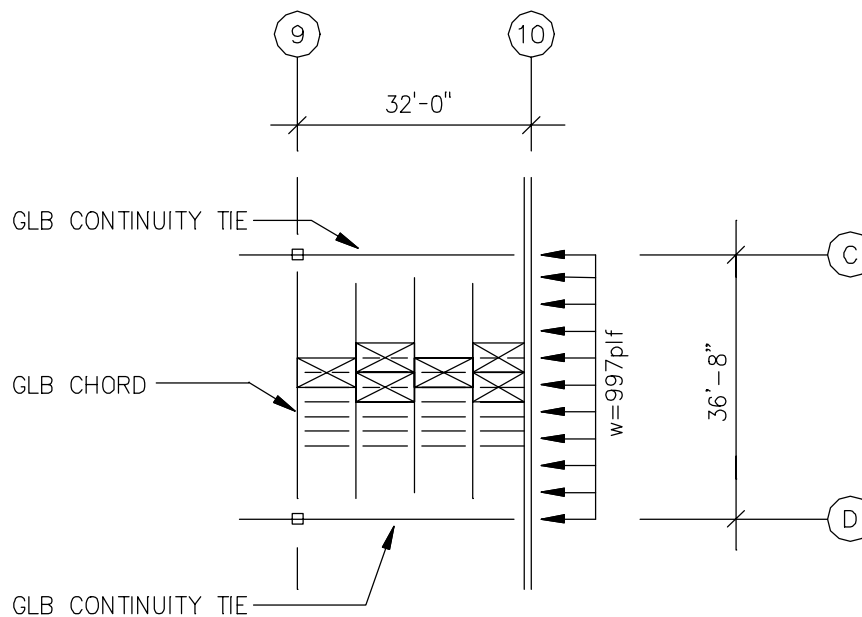


Figure 5-8. Typical north-south subdiaphragm

Shear reaction to glulam beams along lines C and D:

$$R = \frac{997 \text{ plf} (36.67 \text{ ft})}{2} = 18,280 \text{ lb}$$

$$\text{Maximum shear} = \frac{18,280 \text{ lb}}{32} = 571 \text{ plf}$$

From Table 5-1, the minimum nailing in Zone A (Figure 5-6) is 10d @ 4 in. along north-south edges, except at boundaries.

Load on an ASD basis with the 0.85 load factor of §1633.2.8.1(5) applied is

$$0.85 \frac{(571 \text{ plf})}{1.4} = 347 \text{ plf}$$

Check 10d @ 4 in. for Case 2, capacity = 640 plf > 347 plf *o.k.*

Table 23-II-H

∴ Use of Zone A nailing for subdiaphragm okay

3d.**Check GLB as subdiaphragm chord.**

Glulam beams (GLB) along lines 2 and 9, and the continuous horizontal reinforcement in panels along lines 1 and 10, act as chords for the subdiaphragms. Check to see if the GLB can carry additional seismic force within incremental one-third allowable tension increase using ASD. Note that 0.85 load factor of §1633.2.8.1(5) is applied to the chord force when checking the tension stress in the GLB.

$$\text{Chord force} = \frac{997 \text{ plf} (36.67)^2}{8(32)} = 5,237 \text{ lb}$$

Assume GLB $6\frac{3}{4} \times 24$ with 24F-V4 DF/DF

$$A = 162 \text{ in.}^2$$

$$F_t = 1,150 \text{ psi}$$

Table 5A, 91 NDS

$$f_t = \frac{0.85(5,237 \text{ lb})}{1.4 \times 162 \text{ in.}^2} = 20 \text{ psi} < \frac{1}{3} \times 1,150 \text{ psi} = 383 \text{ psi } o.k.$$

Comment: In reality, the GLB along line 9 may not act in tension as a subdiaphragm chord as shown above. It will be loaded in tension only when compressive wall anchorage forces act on the diaphragm. Under this loading, the seismic forces probably do not follow only the subdiaphragm path shown above but are also transmitted through the wood framing to other parts of the diaphragm. Even if subdiaphragm action does occur, the subdiaphragm may effectively be much deeper than shown. However, because it is necessary to demonstrate that there is a system to resist the out-of-plane forces on the diaphragm edge, the subdiaphragm system shown above is provided.

3e.**Determine minimum chord reinforcement at exterior concrete walls.**

This Design Example 5 assumes that there is continuous horizontal reinforcement in the walls at the roof level that acts as a chord for both the main diaphragm and the subdiaphragms. The 1.4 load factor of §1633.2.8.1(4) must be applied to the reinforcement.

$$\text{Subdiaphragm chord force} = P = 5,237 \text{ lb}$$

$$A_s = \frac{P}{\phi f_y} = \frac{1.4(5,237)}{0.9(60,000)} = \underline{\underline{0.14 \text{ in.}^2}}$$

This is a relatively small amount of reinforcement. Generally, the main diaphragm chord reinforcement exceeds this amount. In present California practice, the subdiaphragm chord steel requirement is not added to the chord steel requirement for the main diaphragm. Determination of the main chord reinforcement is shown in Part 7.

4.**Design wall-roof ties for north-south subdiaphragm.****§1633.2.8.1**

The key elements in the wall anchorage system, defined in §1627, are the wall-roof ties. Wall-roof ties are used to transfer out-of-plane seismic forces on the tilt-up wall panels to the subdiaphragms. Requirements for connection of out-of-plane wall anchorages to flexible diaphragms are specified in §1633.2.8.1.

4a.**Seismic force on wall-roof tie.**

Seismic forces are determined using Equations (32-1) or (32-2). Values of R_p and a_p are:

$$R_p = 3.0 \quad a_p = 1.5 \quad \text{§1633.2.8.1(1)}$$

Forces on the anchorage were computed above in Part 3, using the same values of R_p and a_p , and are $q = \underline{\underline{997 \text{ plf}}}$.

4b.**Design typical wall-roof tie.**

Minimum required thickness of a subpurlin used as wall-roof tie = 2½ inches §1633.2.8.1(5)

Try ties at 8 ft-0 in. spacing, and determine F_p

$$F_p = 8 \text{ ft} \times 997 \text{ plf} = 7,976 \text{ lb}$$

Comment: When tie spacing exceeds 4 feet, the SEAOC Blue Book (§108.2.6) recommends that walls be designed to resist bending between anchors.

Try prefabricated metal holdowns with two ¾-inch bolts in subpurlin and two ¾-inch bolts connecting the subpurlin to the wall panel. This connection (Figure 5-9) is designed to take both tension and compression as recommended by the SEAOSC/COLA Northridge Tilt-up Building Task Force and the SEAOC Blue Book (§C108.2.8.1). Design of the holdown hardware not shown. Consult ICBO Evaluation Reports for allowable load capacity of pre-manufactured holdowns. Note that if a one-sided holdown is used, eccentricities in the subpurlin must be considered, as specified in §1633.2.8.1(2). Generally, one-sided wall-roof anchorage is not recommended.

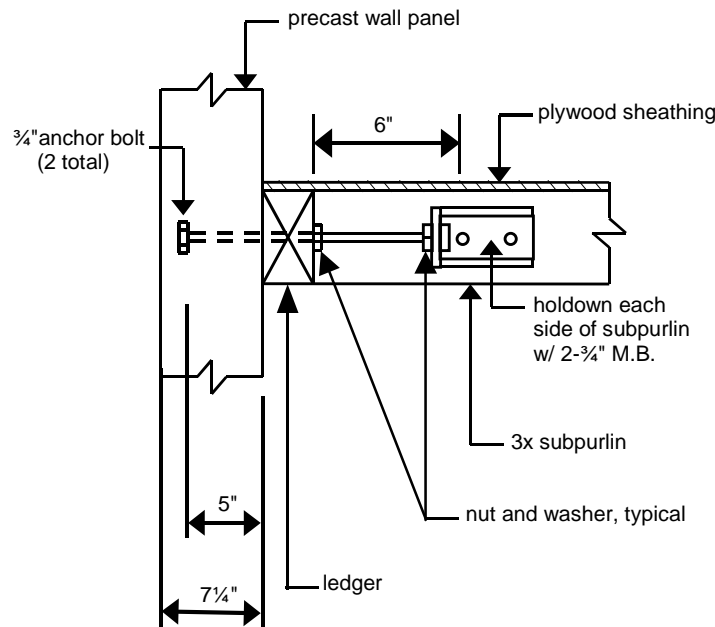


Figure 5-9. Typical subpurlin wall-roof tie

Check capacity of the two $\frac{3}{4}$ -inch bolts in DF-L subpurlin using ASD: Table 8.3B, 91 NDS

$$(2,630)(2 \text{ bolts})(1.33) = 6,996 \text{ lb} > \frac{0.85(7,976 \text{ lb})}{1.4} = 4,843 \text{ lb} \quad o.k.$$

Note that the .85 load factor of §1633.2.8.1(5) is used to reduce the seismic force. This applies to forces on nails and bolts connecting brackets or strips to the wood framing because these are considered “wood elements” under the code (see SEAOC Blue Book §C108.2.8.1).

Comment: The Blue Book (§C108.2.8.1) makes a recommendation for the minimum length to diameter ratio of the through-bolts connecting the holdowns to the subpurlin. In this case, the l/d ratio is $2.5/0.75 = 3.3$. The minimum recommended value is 4.5. This ratio is necessary to maintain a ductile failure mode (e.g., bending of the through-bolts). To satisfy the Blue Book recommendation, a 4x subpurlin would be required in this situation.

Minimum required end distance = $7D = 7(.75) = 5.25 \text{ in.}$ Table 8.5.4, 91 NDS

A distance of 6 inches from the through-bolt in the holdown to the ledger will be used. Often, there is a gap of $\frac{1}{8}$ -inch or more between the end of the subpurlin and the side of the ledger due to panelized roof erection methods, and the use of a 6-inch edge distance will ensure compliance with the 7D requirement. A larger

distance can be used to ensure that through-bolt tear out does not occur in the 3× subpurlin.

Check tension capacity of two ¾-inch A307 anchor bolts using ASD:

$$F_t = 20.0 \text{ ksi}$$

Table 1-A, AISC-ASD

$$P = F_t A_B (2 \text{ bolts})(1.33)$$

$$P = (20.0 \text{ ksi})(0.4418 \text{ in.}^2) (2 \text{ bolts})(1.33) = 23.5 \text{ k} > \frac{1.4(7,976)}{1.4} \text{ lb} = 8.0 \text{ kips} \quad o.k.$$

As specified in §1633.2.8.1(4), the 1.4 steel factor has been used to increase the seismic force.

Check compression capacity of two ¾-inch A307 anchor bolts using ASD:

$$\text{Radius of gyration of } \frac{3}{4}\text{-inch rod} = 0.75\text{-inch}/4 = 0.1875\text{-inch}$$

$$\text{Assume } L = 4\frac{1}{2}\text{-inch}$$

$$\frac{L}{r} = \frac{4.5"}{0.1875"} = 24, \quad F_a = 20.35 \text{ ksi}$$

Table C-36, AISC-ASD

$$P = F_a A_B (2 \text{ rods})(1.33)$$

$$P = (20.35 \text{ ksi})(0.4418 \text{ in.}^2) (2 \text{ rods})(1.33) = 23.9 \text{ k} > 8.0 \text{ kips} \quad o.k.$$

Check tension capacity of anchor bolts in wall panel for concrete strength:

The tilt-up panels are exterior wall elements, but the requirements of §1633.2.4.2 do not apply. This is because the tilt-up panels are both bearing walls and shear walls. The requirements of §1633.2.8 are the appropriate design rules in this situation. This section requires that wall anchorage using straps be attached or hooked so as to transfer the forces to the reinforcing steel. In this case, we are using cast-in-place bolts instead of straps, and the bolts are not required to be “hooked” around the wall reinforcement. In fact, headed anchor bolts have been shown to be more effective than L-bolts in resisting pull-out forces [Shipp and Haninger, 1982].

Try anchor bolts with a 5-inch embedment. Although this embedment is considered shallow anchorage under §1632.2, R_p is 3.0 regardless of whether the anchorage has shallow embedment because §1633.2.8.1 is applicable. The material specific load factors of §1633.2.8.1 (1.4 for steel and 0.85 for wood) are intended to provide the nominal overstrength necessary to resist brittle failure of the wall anchorage system when subjected to the maximum anticipated roof accelerations of flexible diaphragms. Section 1633.2.8.1 is intended as a stand-alone section, and

the more restrictive requirements on R_p of §1632.2 do not apply (see Blue Book §C108.2.8.1).

$$F_p = 7,976 \text{ lb}$$

Actual bolt spacing is:

$$\begin{aligned} &2\frac{1}{2} \text{ in.} \quad (\text{width of } 3\times \text{ subpurlin}) \\ &+4\frac{1}{4} \text{ in.} \quad (2 \text{ times bolt edge distance of holdown flange}) \\ &6\frac{3}{4} \text{ in.} \end{aligned}$$

From Table 19-D, required spacing for full capacity is 9 inches. Minimum spacing is 50 percent of this, or $4\frac{1}{2}$ in. Interpolation for $6\frac{3}{4}$ in. spacing is shown below with $f'_c = 4,000$ psi and assuming Special Inspection. Alternately, using strength design, the requirements of §1923.2 could be used with computation for overlapping pull-out cones. If §1923.2 is used, a load multiplier of 1.3 and a strength reduction factor of 0.65 would be used:

<u>Tension Capacity (w/Special Inspection)</u>	<u>Bolt Spacing</u>
6,400 lb/bolt	9 in.
4,800	$6\frac{3}{4}$ in.
3,200	$4\frac{1}{2}$ in.

$$\frac{F_p}{2} = \frac{7,976}{2} = 3,988 \text{ lb/bolt}$$

$$\text{Allowable} = 4,800 \text{ lb} (1.33) = 6,384 \text{ lb} > \frac{3,988}{1.4} = 2,849 \quad o.k.$$

Comment: The code in §1633.2.8.1 requires that material-specific load factors be applied in the design of elements of the wall anchorage system. These factors are 1.4 for steel, 1.0 for concrete, and 0.85 for wood. They are applied to the anchorage force determined from Equation (32-2). A background discussion on this is given in the Blue Book Commentary §C108.2.8.1, where the load factors are shown to provide a connection having nominal overstrength of approximately 2.0. This is required to meet the maximum expected roof acceleration of four times the peak ground acceleration. The latter is also discussed in §C108.2.8.1 and is shown to be equivalent to doubling the design anchorage force F_p . Thus, an anchorage connection designed under §1633.2.8.1 should have the overstrength that just meets the maximum expected demand of $2F_p$. This overstrength approach was selected, in lieu of a ductility approach, after wall anchorage failures were observed in steel strap connectors with limited yield and deformation range.

Because anchor bolt pull-out is a critical and brittle failure mode, it must be prevented by having sufficient embedment strength. The nominal factor of two

overstrength for concrete anchorage just meets the expected maximum demand. This is based on dividing the 1.3 load factor by a ϕ -factor of 0.65 as discussed in §C108.2.8.1 of the Blue Book. Shown below is the calculation of the strength of the anchorage shown in Figure 5-9 using the method of §1923.3.2 (an alternate method is given in Cook, 1999). In this calculation, a ϕ -factor of 0.65 is used to provide an additional margin of safety beyond the code minimum. If the overstrength desired was only 2.0, then $\phi=1.0$ would be used. Note that the capacity ϕP_c is greater than $2F_p$.

$$\phi P_c = \phi \lambda 4 A_p \sqrt{f'_c} \quad \text{§1923.3.2}$$

For $\frac{3}{4}$ in. bolts with hex heads, the width across the flats is $1\frac{1}{8}$ in., and A_p is computed as follows.

$$A_p = 0.785(10 + 1.125)^2 + 6.75(10 + 1.125) = 172 \text{ in.}^2 < 2(0.785)(11.125)^2 = 194 \text{ in.}^2$$

$$\phi = 0.65 \quad \text{§1923.3.2}$$

$$\lambda = 1.0 \quad \text{§1923.3.2}$$

$$\phi P_c = (0.65)(1.0)4(172 \text{ in.}^2)\sqrt{4,000} = 28.3 \text{ kips} > 2F_p = 16.0 \text{ kips} \quad o.k.$$

Therefore, the anchorage in Figure 5-9 is strong enough to resist the expected pull-out forces for code-level ground motions. In general, it is recommended that the concrete pull-out strength exceed the bolt yield strength. If this is not possible, it is recommended that the concrete pull-out strength exceed the code minimum by a substantial margin (as shown above).

An alternate wall-roof tie connection is in Figure 5-10. However, this connection, which utilizes a heavy-gauge strap, does not offer the same compression resistance as the bolt scheme (Figure 5-9). Compression forces in the subpurlin generally must be carried by the strap and/or plywood sheathing because subpurlins are typically not installed snugly against the ledgers. Often there is a $\frac{1}{8}$ -inch to $\frac{1}{4}$ -inch gap at each end. Providing both tension and compression capability in wall-roof ties protects the diaphragm edge nailing under the reversible seismic forces. In this case, the strap is hooked around a reinforcing bar to meet the requirements of §1633.2.8.

The code requires that different loads be applied to the various materials involved in the wall anchorage system. However, most hardware manufacturer's catalogs provide only a single allowable stress capacity for the component, which often includes concrete, steel, and wood elements. To properly apply code requirements, the design engineer must compute the capacity of each element separately.

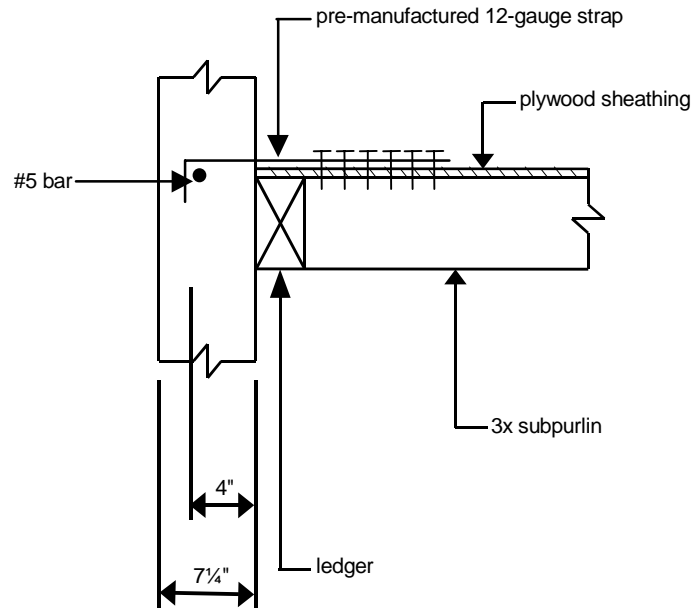


Figure 5-10. Alternate wall-roof tie

4C.**Design connection to transfer seismic force across first roof truss purlin.**

Under §1627, continuity ties in the subdiaphragms are considered part of the wall anchorage system. Consequently, the forces used to design the wall-roof ties must also be used to design the continuity ties within the subdiaphragm.

$$F_p = \text{wall-roof tie load} = 7,976 \text{ lb}$$

If the subdiaphragm is 32-foot deep and roof truss purlins are spaced at 8 feet, then the connection at the first roof truss purlin must carry three-quarters of the wall-roof tie force.

Comment: Some engineers use the full, unreduced force, but this is not required by rational analysis.

$$\frac{(32-8)}{32} \times F_p = \frac{3}{4} \times 7,976 = 5,982 \text{ lb}$$

At the second and third roof truss purlins, the force to be transferred is one-half and one-fourth, respectively, of the wall-roof tie force.

$$\frac{1}{2} \times 7,976 = 3,988 \text{ lb}$$

$$\frac{1}{4} \times 7,976 = 1,994 \text{ lb}$$

Try 12-gauge metal strap with 10d common nails. Design of strap not shown. Consult ICBO Evaluation Reports for allowable load capacity of pre-manufactured straps. Note that the 1.4 load factor of §1633.2.8.1(4) applies to the strap design and that the 0.85 load factor of §1633.2.8.1(5) applies to the nails. Tension on the gross and net areas of the strap must be checked separately. The tensile capacity of the strap, which is generally not indicated in the ICBO Evaluation Report, is usually controlled by the nails. Consult with the strap manufacturer for appropriate values of F_y and F_u .

The following calculation shows determination of the number of 10d common nails required at the first connection:

$$\frac{0.85(5,982 \text{ lb})}{120 \text{ lb}(1.4)(1.33)} = 22.8$$

Table 23-III-C-2
Table 12-3F, 91 NI

∴ Use 12-gauge metal strap with 24-10d nails each side

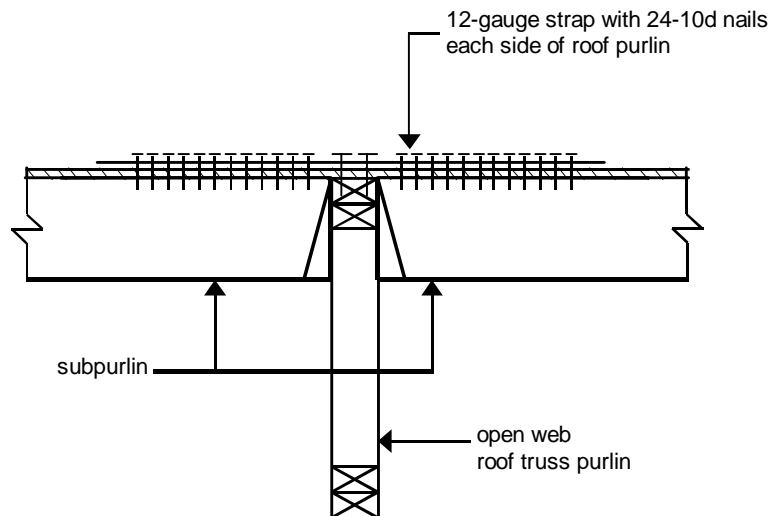


Figure 5-11. Subpurlin continuity tie at first purlin

Note that both subpurlins in Figure 5-11 would be 3× members because of the heavy strap nailing.

Design of the second and third connections is similar to that shown above.

5.

Design continuity ties for north-south direction

In a tilt-up building, continuity ties have two functions. The first is to transmit the subdiaphragm reactions (from out-of-plane seismic forces on the wall panels) and distribute these into the main roof diaphragm. The second function is that of “tying” the interior portions of the roof together. In this example, the continuity ties on lines C and D will be designed.

5a.

Seismic forces on continuity ties on lines C and D.

Force in the continuity tie at line 10 is the wall-roof tie force:

$$P_{10} = (997 \text{ plf})(8 \text{ ft}) = 7,976 \text{ lb}$$

Force in continuity tie at the glulam beam splice north of line 9 is the sum of both subdiaphragm reactions.

$$P_9 = \frac{997 \text{ plf}(36.67 \text{ ft})}{2}(2 \text{ subdiaph.}) = 36,560 \text{ lb}$$

The splice near line 9 must also be checked for the minimum horizontal tie force of §1633.2.5. Assume the splice is at fifth point of span as shown on the roof plan of Figure 5-2. This requirement imposes a minimum tie force on the GLB connections and is based only on the dead and live loads carried by the beams.

$$F_p = 0.5C_aIW_{D+L} \quad \text{§1633.2.5}$$

$$W_{DL} = 14 \text{ psf}, \quad W_{LL} = 12 \text{ psf}$$

$$W_{D+L} = (14 \text{ psf} + 12 \text{ psf})(36.67 \text{ ft}) \left(32 \text{ ft} - \frac{32 \text{ ft}}{5} - \frac{32 \text{ ft}}{5} \right) = 18,306 \text{ lb}$$

$$F_p = 0.5(.44)(1.0)(18,306 \text{ lb}) = 4,027 \text{ lb} < 36,560 \text{ lb}$$

∴ Subdiaphragm reaction controls

5b.**Design glulam beam (continuity tie) connection to wall panel.**

In this example, walls are bearing walls, and pilasters are not used to vertically support the GLBs. Consequently, the kind of detail shown in Figure 5-12 must be used. This detail provides both vertical support for the GLB and the necessary wall-roof tie force capacity. The tie force is the same as that for wall-roof tie of Part 5a ($P_{10} = 7,976 \text{ lb}$). The detail has the capacity to take both tension and compression forces. Details of the design are not given. The horizontal force design is similar to that shown in Part 4.

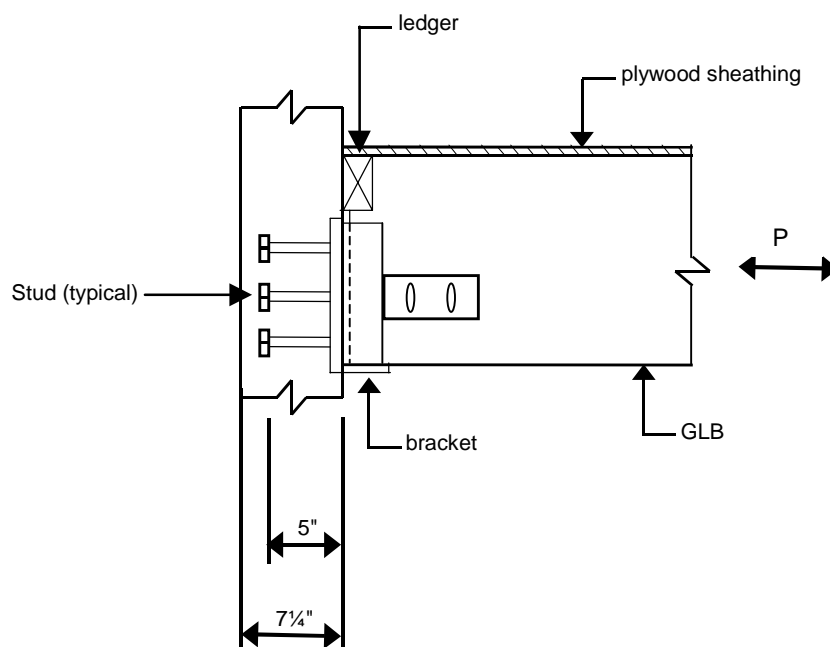


Figure 5-12. Bracket for wall-roof anchorage at GLB

It should be noted that the alternate wall-roof tie of Figure 5-10 is not acceptable in this situation because the strap cannot resist compression.

Comment: Although not required by code, some designers design the wall-GLB tie to take all of the tributary wall-roof forces (assuming the subpurlin wall ties carry none) and carry this force all across the building as the design force in the continuity ties. In this example, this force is $P_9 = 36,560 \text{ lb}$. This provides for a much stronger “tie” between the wall and the GLB for buildings without pilasters (the usual practice today) to help prevent loss of support for the GLB and subsequent local collapse of the roof under severe seismic motions.

5c.**Design continuity tie across glulam splice.**

$$P_9 = 36,560 \text{ lb at splice near line 9}$$

The ASD design force for the continuity tie is computed below. Note that the 0.85 wood load factor of §1633.2.8.1(5) is used for bolts in wood (see discussion in Blue Book §C108.2.8.1).

$$P = \frac{0.85(36,560 \text{ lb})}{1.4} = 22,197 \text{ lb}$$

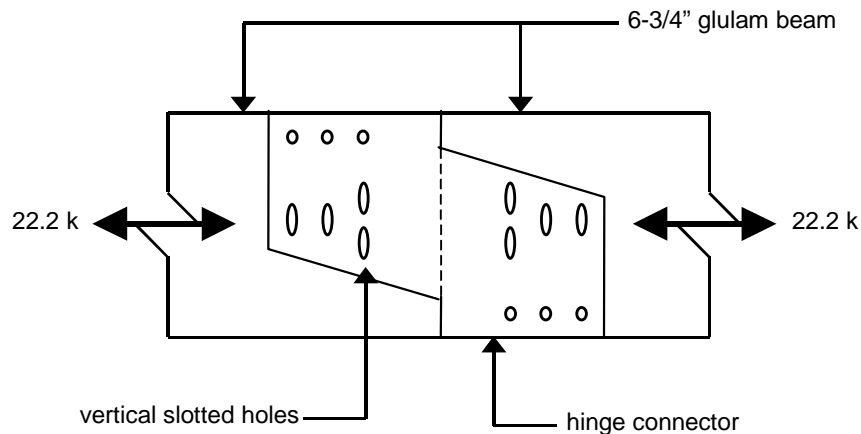


Figure 5-13. Typical continuity tie splice

Try four 7/8-inch bolts in vertical slotted holes at center of hinge connector. Design of hinge connector hardware not shown. Consult ICBO Evaluation Reports for allowable load capacity of pre-manufactured hinge connectors. Note that the bolt capacity is based on the species of the inner laminations (in this case DF-L).

$$4(4,260 \text{ lb})(1.33) = 22,663 \text{ lb} > 22,197 \text{ lb } o.k.$$

Table 8.3D, 91 NDS

5d.**Check GLB for continuity tie force.**

The glulam beams along lines C and D must be checked for the continuity tie axial force. See Part 6 for an example of this calculation. Note that use of the amplified force check of §1633.2.6 is not required for continuity ties that are not collectors.

6.**Design collector along line 3 between lines B and C.**

The collector and shear wall ledger along line 3 carry one-half of the east-west roof diaphragm seismic force. The force in the collector is “collected” from the tributary area between lines B and E and transmitted to the shear wall on line 3.

6a.**Determine seismic forces on collector.**

From diaphragm shear diagram for east-west seismic forces, the maximum collector load on at line 3 is:

$$R = 36.4 \text{ k} + \left(\frac{110.0 \text{ ft}}{140.67 \text{ ft}} \right) 127.5 \text{ k} = 136.1 \text{ kips tension or compression}$$

Uniform axial load in collector can be approximated as the total collector load on line 3 divided by the length of the collector (110'-0") in this direction.

$$q = \frac{R}{L} = \frac{136,100 \text{ lb}}{110.00 \text{ ft}} = \underline{\underline{1,237 \text{ plf}}}$$

6b.**Determine the collector force in GLB between lines B and C.**

Assume the collector is a GLB $6\frac{3}{4} \times 21$ with 24F-V4 DF/DF and it is adequate to support dead and live loads. $A = 141.8 \text{ in.}^2$, $S = 496 \text{ in.}^2$, and $w = 34.5 \text{ plf}$. Calculate seismic force at mid-span. Tributary length for collecting axial forces is

$$l = 110.00 \text{ ft} - \frac{36.67 \text{ ft}}{2} = 91.67 \text{ ft}$$

$$P = ql = 1.237 \text{ klf} (91.67 \text{ ft}) = \underline{\underline{113.4 \text{ kips tension or compression in beam}}}$$

6c.**Check GLB for combined dead and seismic load as required by §1612.3.2.**

$$D + L + S + \frac{E}{1.4} \tag{12-16}$$

$$w_{DL} = 8 \text{ ft} (14 \text{ psf}) + 34.5 \text{ plf} = 146.5 \text{ plf}$$

$$M_{DL} = \frac{0.147 \text{ k/ft} (36.67 \text{ ft})^2}{8} = 24.7 \text{ kip-ft}$$

$$F_b^* = 2,088 \text{ psi}$$

Table 5A, 91 NDS

$$f_b = \frac{24.7 \text{ k-ft} (12,000)}{496 \text{ in.}^3} = 598 \text{ psi}$$

$$P = \frac{113.4}{1.4} = 81.0 \text{ kips tension or compression on ASD basis}$$

$$F_t = 1,150 \text{ psi}$$

Table 5A, 91 NDS

$$F_c = 1,650 \text{ psi}$$

Table 5A, 91 NDS

$$f_t = f_c = \frac{81,000 \text{ lb}}{141.8} = 571 \text{ psi}$$

Because there is a re-entrant corner at the intersection of lines B and 3, a check for Type 2 plan irregularity must be made. Requirements for irregular structures are given in §1629.5.3.

North-south direction check:

$$.15 \times (288) = 43.2 \text{ ft} < 64' - 0''$$

Table 16-M

East-west direction check:

$$.15 \times (110.0 + 30.67) = 21.1' < 30' - 8''$$

Table 16-M

Since both projections are greater than 15 percent of the plan dimension in the direction considered, a Type 2 plan irregularity exists. The requirements of Item 6 of §1633.2.9 apply, and the one-third allowable stress increase *cannot* be used.

Checking combined bending and axial tension using Equation (3.9-1) of NDS:

$$\frac{f_b}{F_b^*} + \frac{f_t}{F_t} \leq 1.00$$

3.9.1, 91 NDS

$$\frac{598}{2,088} + \frac{571}{1,150} = 0.29 + 0.50 = 0.79 < 1.00 \text{ o.k.}$$

Equation (3.9-2) of NDS o.k. by inspection.

Checking combined bending and axial compression using Equation (3.9-3) of NDS and considering the weak axis of the GLB laterally braced by the roof:

$$\left[\frac{f_c}{F'_c} \right]^2 + \frac{f_b}{F'_b \left(1 - \frac{f_c}{F_{cE}} \right)} \leq 1.0 \quad 3.92, 91 \text{ NDS}$$

Find F'_c by first calculating the column stability factor C_p .

$$l_e = k_e l = 1.0(36.67) = 36.67 \text{ ft} \quad 3.7.1.2, 91 \text{ NDS}$$

$$F_{cE} = \frac{K_{cE} E'}{(l_e/d)^2} = \frac{0.418(1,600,000)}{(36.67 \times 12/21)^2} = 1,523 \text{ psi} \quad 3.7.1.5, 91 \text{ NDS}$$

$$F_c^* = F_c = 1650 \text{ psi} \quad \text{Table 5A, 91 NDS Supplement}$$

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c} \right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad \text{Eq. 3.7-1, 91 NDS}$$

$$C_p = \frac{1 + (1,523/1,650)}{2(0.9)} - \sqrt{\left[\frac{1 + (1,523/1,650)}{2(0.9)} \right]^2 - \frac{1,523/1,650}{0.9}}$$

$$C_p = 0.73$$

$$F'_c = F_c (C_p) = 1,650 (0.73) = 1,205 \text{ psi} \quad \text{Table 2.3.1, 91 NDS}$$

$$\left[\frac{571}{1,205} \right]^2 + \frac{598}{2,088 \left(1 - \frac{571}{1,523} \right)} = 0.22 + 0.46 = 0.68 < 1.0 \quad o.k.$$

6d.

Check GLB collector for amplified force requirements.

The GLB must also be checked for the special collector requirements of §1633.2.6.

Using ASD, an allowable stress increase of 1.7 may be used for this check. The relevant equations are:

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

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

$$E_m = \Omega_o E_h \quad (30-2)$$

E_m is an estimate of the maximum force transmitted by the collector elements in the seismic event. Unless a more refined analysis is done and the maximum force that the diaphragm, or the shear wall, can transmit to the collector determined, the seismic force E_h is scaled by the amplification factor Ω_o for estimating E_m .

$$\Omega_o = 2.8$$

Table 16-N

$$E_h = 113.4 \text{ kips from Part 6b, above}$$

$$E_m = 2.8(113.4) = 317.5 \text{ kips tension or compression in beam}$$

Comment: The axial force $E_m = 317.5$ kips in the above calculations is 1.4 times greater than that which would be obtained using the $3R_w/8$ factor applied to collector forces obtained under the 1994 UBC provisions. This is because forces in the 1997 UBC are strength based and were established to be 1.4 times greater than those of the 1994 UBC. Unfortunately, the 1997 UBC does not first reduce the forces by the 1.4 ASD factor when increasing the axial force by the $\Omega_o = 2.8$ factor. This appears to result in an unnecessarily conservative design for elements like the GLB collector in this example.

Under both §1612.2.1 and §1612.4, roof live load is not included in the seismic design load combinations. Generally, Equation (12-17) controls over Equation (12-18). Because the $6\frac{3}{4} \times 21$ GLB will not work, a $6\frac{3}{4} \times 27$ beam will be tried.

$$A = 182 \text{ in.}^2, S = 820 \text{ in.}^3, \text{ and } w = 44.3 \text{ plf.}$$

Dead load bending stress at mid-span is (neglecting small increase in beam weight):

$$M_{DL} = 24.7 \text{ kip-ft}$$

$$f_b = \frac{24,700 \text{ lb-ft}(12)}{820} = 361 \text{ psi}$$

$$F_b = 0.85(2,400 \text{ psi}) = 2,040 \text{ psi}$$

Table 5A, 91 NDS

$$f_t = f_c = \frac{317,500 \text{ lb}}{182} = 1,745 \text{ psi}$$

Check combined dead plus tension and compression seismic stresses using Equation (12-17). The load factors are 1.2 on dead load and 1.0 on seismic forces, and the allowable stress increase is 1.7.

Check tension using NDS Equation (3.9-1):

$$\left(\frac{1.2f_b}{1.7F_b^*} \right) + \left(\frac{1.0f_t}{1.7F_t'} \right) \leq 1.0$$

$$\left(\frac{1.2(361)}{1.7(2,040)} \right) + \left(\frac{1,745}{1.7(1,150)} \right) = 0.12 + 0.89 = 1.01 \approx 1.0 \quad \text{say } o.k.$$

NDS Equation (3.9-2) is *o.k.* by inspection.

Check compression using NDS Equation (3.9-3) as modified below:

$$\left[\frac{1.0f_c}{1.7F_c'} \right]^2 + \frac{1.2f_b}{1.7F_b' \left(1 - \frac{1.0f_c}{F_{cE}} \right)} \leq 1.0$$

$$F_{cE} = \frac{K_{cE}E'}{(l_e/d)^2} = \frac{0.418(1,600,000)}{(36.67 \times 12/27)^2} = 2,518 \text{ psi} \quad 3.7.1.5, 91 \text{ NDS}$$

$$C_p = 0.88$$

$$F_c' = F_c (C_p) = 1,650(0.88) = 1,452 \text{ psi} \quad \text{Table 2.3.1, 91 NDS}$$

$$\left[\frac{1,745}{1.7(1,452)} \right]^2 + \frac{1.2(361)}{1.7(2,040) \left(1 - \frac{1,745}{2,518} \right)} = 0.50 + 0.41 = 0.91 < 1.0 \quad o.k.$$

\therefore Use GLB $6 \frac{3}{4} \times 27$

Note that the special collector requirement of §1633.2.6 has necessitated that the size of the GLB be increased from $6 \frac{3}{4} \times 21$ to $6 \frac{3}{4} \times 27$.

6e.

Collector connection to shear wall.

The design of the connection of the GLB to the shear wall on line 3 is not given. This is an important connection because it transfers the large “collected” seismic force into the shear wall. The connection must be designed to carry the same seismic forces as the beam, including the amplified collector force of §1633.2.6. Because there is also a collector along line B, there is similarly an important connection of the GLB between lines 3 and 4 to the shear wall on line B. Having to carry two large tension (or compression) forces through the intersection of lines B and 3 (but not simultaneously) requires careful design consideration.

7.

Required diaphragm chord reinforcement for east-west seismic forces.

Chords are required to carry the tension and compression forces developed by the moments in the diaphragm. In this building, the chords are continuous reinforcement located in the wall panels at the roof level as shown in Figure 5-14. (These must be properly spliced between panels.)

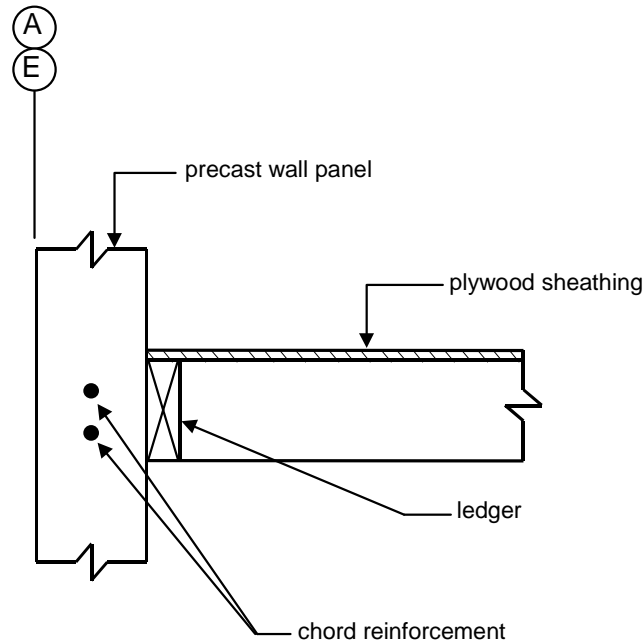


Figure 5-14. Diaphragm chord

The east-west diaphragm spans between lines 1 and 3 and lines 3 and 10. The plywood diaphragm is considered flexible, and the moments in segments 1-3 and 3-10 can be computed independently assuming a simple span for each segment. In this example, the chord reinforcement between lines 3 and 10 will be determined. This reinforcement is for the panels on lines A and E.

Equiv. $w = 1,138$ plf from Part 2

$$M = \frac{wl^2}{8} = \frac{1.14 \text{ klf} (224)^2}{8} = 7,150 \text{ kip-ft}$$

The chord forces are computed from

$$T = C = \frac{7,150 \text{ k-ft}}{140.67 \text{ ft}} = 50.8 \text{ kips}$$

The chord will be designed using strength design with Grade 60 reinforcement. The load factor of Equations (12-5) and (12-6) is 1.0 for seismic forces.

$$A_s = \frac{T}{\phi f_y} = \frac{50.8 \text{ k}}{0.9(60 \text{ ksi})} = 0.94 \text{ in.}^2$$

∴ Use minimum 2-#7 bars, $A_s = 1.20 \text{ in.}^2 > 0.94 \text{ o.k.}$

Comment: The chord shown above consists of two #7 bars. These must be spliced at the joint between adjacent panels, typically using details that are highly dependent on the accuracy in placing the bars and the quality of the field welding. Alternately, chords can also be combined with the ledger such as when steel channels or bent steel plates are used, and good quality splices can be easier to make.

8.

Required wall panel reinforcing for out-of-plane forces.

In this part, design of a typical solid panel (no door or window openings) is shown. The panel selected is for lines 1 and 10, and includes the reaction from a large GLB. The wall spans from floor to roof, and has no pilaster under the GLB. There are no recesses or reveals in the wall.

8a.

Out-of-plane seismic forces.

Requirements for out-of-plane seismic forces are specified in §1632.2. Equation (32-2) is used to determine forces on the wall.

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (32-2)$$

$$F_{p \min} = 0.7 C_a I_p W_p \quad (32-3)$$

$$F_{p \max} = 4.0 C_a I_p W_p \quad (32-3)$$

$$R_p = 3.0 \text{ and } a_p = 1.0$$

Table 16-0, Item 1.A.(2)

$$C_a = 0.44$$

F_p can be determined by calculating the equivalent seismic coefficient at the ground and roof levels. The average of the two values is used to determine the uniform out-of-plane seismic force applied over the height of the wall.

At the ground level, $h_x = 0$, and the effective seismic coefficient from Equation (32-2) is:

$$\frac{a_a C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) = \frac{1.0(.44)(1.0)}{3.0} \left(1 + 3 \frac{0}{21} \right) = 0.147$$

Check minimum value from Equation (32-3):

$$0.7 C_a I_p = 0.7(.44)(1.0) = 0.308 > 0.147$$

∴ Use 0.308

At the roof level, $h_x = h_r$, and the effective seismic coefficient from Equation (32-2) is:

$$\frac{a_a C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) = \frac{1.0(.44)(1.0)}{3.0} \left(1 + 3 \frac{21}{21} \right) = 0.587$$

Check maximum value from Equation (32-3):

$$4.0 C_a I_p = 4.0(.44)(1.0) = 1.76 > 0.587$$

∴ Use 0.587

The average force over the height of the wall is:

$$F_p = \frac{1}{2} (0.308 + 0.587) W_p = 0.448 W_p$$

Design of the wall for moments from out-of-plane seismic forces is done by assuming the force F_p to be uniformly distributed over the height of the wall as shown in Figure 5-15.

Solving for the uniform force per foot f_p :

$$f_p = .448 (90.6 \text{ psf}) = 40.6 \text{ psf}$$

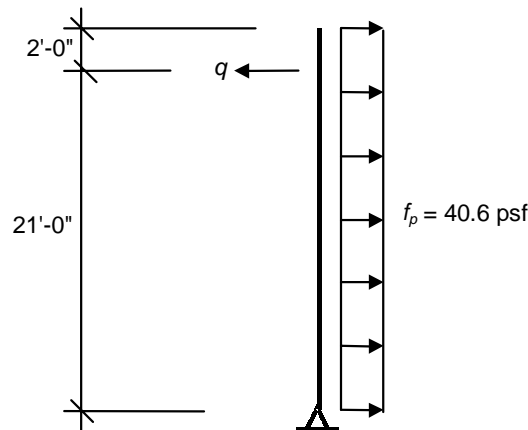


Figure 5-15. Loading diagram for out-of-plane wall design

8b.**Check applicability of alternate slender wall design criteria.**

The panel to be designed is shown in Figure 5-16. The section at mid-height carries the maximum moment from out-of-plane seismic forces. At the same time, this section also carries axial load, from the weight of the panel and the GLB, as well as bending moments due to the eccentricity of the GLB reaction on the wall and $P\Delta$ effects.

The tributary width of wall for support of the vertical loads of the GLB was determined as follows. The GLB is supported on the wall as shown in Figure 5-12. The vertical reaction on the wall is assumed to be at the bottom of the GLB, and the wall is assumed to span from finished floor to roof in resisting out-of-plane forces. These are conservative assumptions made for the convenience of the analysis. Other assumptions can be made. For example, the center of the stud group (see Figure 5-12) can be assumed to be the location of the GLB reaction on the wall. This assumption would result in a wider effective width of wall to carry vertical loads. The mid-depth of the beam could be assumed to be the point to which the wall spans for out-of-plane forces. This assumption would result in a lower moment in the wall due to the out-of-plane forces.

Assume $6\frac{3}{4} \times 25\frac{1}{2}$ GLB bearing on wall

$$\text{Tributary width} = t_{GLB} + H/2 - d_{GLB} = \left(\frac{6.75}{12} \right) + \left(\frac{21.0}{2} \right) - \left(\frac{25.5}{12} \right) = 8.94 \text{ ft} \quad \S 1914.8.2(4)$$

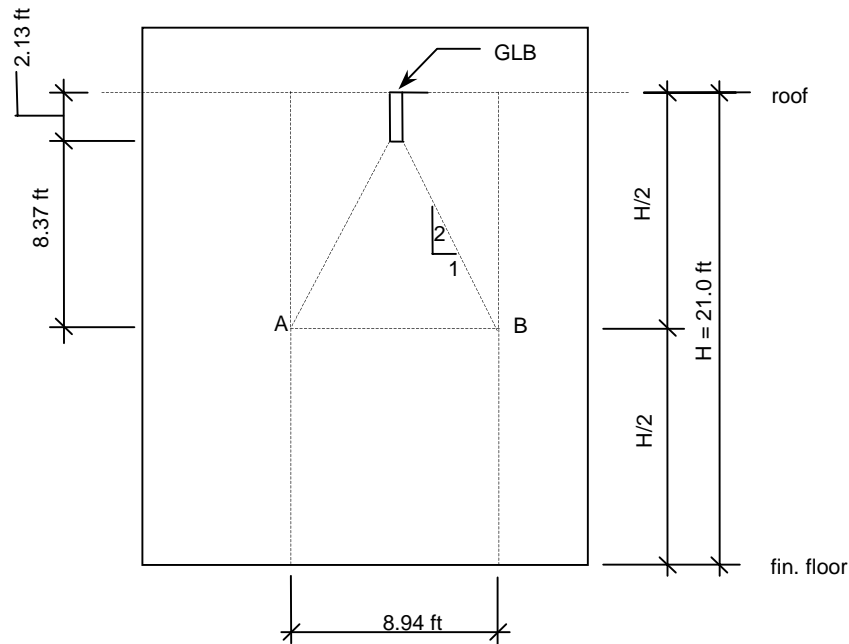


Figure 5-16. Typical panel supporting a GLB; line A-B denotes the tributary width of wall to be checked for the vertical load of the GLB and the moment due to out-of-plane seismic forces

Generally, it is advantageous to use the alternate design slender wall criteria of §1914.8. This will be shown below. As a first step, check the limitations on the use of this criteria. These are indicated in §1914.8.2.

1. Check that vertical service load is less than $0.04 f'_c A_g$: §1914.8.2(1)

$$P_{roof} = \frac{14 \text{ psf} (36.7)(32/2)}{8.94} = 0.92 \text{ kip/ft}$$

$$P_{wall} = 90.6 \text{ psf} \left(\frac{21.0}{2} + 2.0 \right) = 1.13 \text{ kip/ft}$$

$$P = P_{roof} + P_{wall} = 0.92 + 1.13 = 2.05 \text{ kip/ft}$$

$$0.04 f'_c A_g = 0.04 (4,000)(12)(7.25) = 13.9 \text{ kip/ft} > 2.05 \text{ kip/ft}$$

∴ Vertical service load is less than $0.04 f'_c A_g$

2. Check that the reinforcement does not exceed $0.6\rho_b$. §1914.8.2(2)

Assume vertical #4 @ 12 inches o.c. in center panel:

$$\rho = \frac{A_s}{bd} = \frac{0.20}{(12)(3.63)} = 0.00459$$

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y} \left(\frac{87,000}{87,000 + f_y} \right) \quad (8-1)$$

$$\rho_b = \frac{0.85(0.85)(4,000)}{60,000} \left(\frac{87,000}{87,000 + 60,000} \right) = 0.0285$$

$$0.6\rho_b = 0.6(0.0285) = 0.0171 > 0.00459 \text{ o.k.}$$

∴ Reinforcement does not exceed $0.6\rho_b$

3. Check that $\phi M_n > M_{cr}$: §1914.8.2(3)

Before ϕM_n is calculated, ϕ must be determined. Calculate ϕ based on requirements of §1909.3.2.2. The axial load considered to determine M_n is the factored vertical load, and this is also used in determining ϕ .

Because strength design is being used, the load effect of vertical motion, E_v , must be added to the vertical load.

$$E_v = 0.5 C_a ID = 0.5(0.44)(1.0)D = 0.22D \quad §1630.1.1$$

E_v has the effect of increasing the dead load by 0.22 D to a total of 1.42 D. The load factors of Equation (12-5) must be multiplied by 1.1 for concrete as required by Exception 2 of §1612.2.1. The net effect of this is shown below.

$$P_u = 1.1(1.2D + 0.22D) = 1.56D \quad §1612.2.1$$

$$P_u = 1.56(P_{roof} + P_{wall}) = 1.56(2.05) = 3.20 \text{ kip/ft}$$

Section 1909.3.2.2 states that ϕ may be increased up to 0.9 as ϕP_n decreases from the smaller of ϕP_b and $0.1f_c' A_g$ to zero. Calculate ϕP_b and $0.1f_c' A_g$:

$$P_b = b f_y b d - A_s f_y = (0.0285)(60)(12)(3.63) - (0.20)(60) = 62.4 \text{ kip/ft} \quad §1910.3.2$$

$$\phi P_b = (0.7)62.4 = 43.7 \text{ kip/ft}$$

$$0.1 f'_c A_g = 0.1(4.0)(12)(7.25) = 34.8 \text{ kip/ft} < \phi P_b$$

∴ Use $0.1 f'_c A_g$ in calculating ϕ

$$\phi = 0.9 - \frac{0.2(P_u)}{0.1 f'_c A_g} = 0.9 - \frac{0.2(3.20)}{34.8} = 0.882 \quad \text{§1909.3.2.2}$$

Calculate M_n for the given axial load of 2.05 kip/ft. Note that values of A_{se} and a are taken from Part 8c below.

$$M_n = A_{se} f_y \left(d - \frac{a}{2} \right) = 0.253(60) \left(3.63 - \frac{0.372}{2} \right) = 52.3 \text{ kip-in.}$$

$$\phi M_n = 0.882(52.3) = 46.1 \text{ kip-in.}$$

Calculate the cracking moment M_{cr} .

$$I_g = \frac{bh^3}{12} = \frac{12(7.25)^3}{12} = 381 \text{ in.}^4$$

$$M_{cr} = \frac{5\sqrt{f'_c} (I_g)}{y_t} = \frac{5\sqrt{4000} (381)}{3.63} = 33.2 \text{ kip-in.} \quad \text{§1914.0}$$

$$M_{cr} < \phi M_n \text{ o.k.}$$

4. A 2:1 slope may be used for the distribution of the concentrated load throughout the height of the panel (Figure 5-16). §1914.8.2(4)

∴ Slender wall criteria may be used

8C.**Check wall strength.**

Combine factored moment due to out-of-plane seismic forces with moment due to roof vertical load eccentricity and the moment due to $P\Delta$ effects. Calculate $P\Delta$ moment using the maximum potential deflection, Δ_n .

$$E_c = 57,000 \sqrt{f'_c} = 3,605 \text{ ksi} \quad \S 1908.5.1$$

$$E_s = 29,000 \text{ ksi} \quad \S 1908.5.2$$

$$n = \frac{E_s}{E_c} = \frac{29,000}{3,605} = 8.04$$

$$A_{se} = \frac{P_u + A_s f_y}{F_y} = \frac{3.20 + 0.20(60)}{60} = 0.253 \text{ in.} \quad \S 1914.8.4$$

$$a = \frac{A_{se} f_y}{0.85 f'_c b} = \frac{0.253(60)}{0.85(4.0)12} = 0.372 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{0.372}{0.85} = 0.438 \text{ in.}$$

$$d = 3.63 \text{ in.}$$

$$\begin{aligned} I_{cr} &= n A_{se} (d - c)^2 + \frac{bc^3}{3} \\ &= 8.04 (0.253)(3.63 - 0.438)^2 + \frac{12(0.438)^3}{3} = 21.1 \text{ in.}^4 \end{aligned} \quad \S 1914.8.4$$

Maximum potential deflection is:

$$\Delta_n = \frac{5 M_n l_c^2}{48 E_c I_{cr}} = \frac{5(52.3)(21 \times 12)^2}{48(3,605)(21.1)} = 4.55 \text{ in.}$$

Assuming the GLB reaction is 2 in. from the face of wall

$$e = 2.0 + \frac{t_{wall}}{2} = 2.0 + \frac{7.25}{2} = 5.63 \text{ in.}$$

$$P_{u,roof} = 1.56 \quad P_{roof} = 1.56(0.92) = 1.44 \text{ kip / ft}$$

$$P_{u,wall} = 1.56 \quad P_{wall} = 1.56(1.13) = 1.76 \text{ kip/ft}$$

Required factored moment at mid-height of the wall is:

$$M_u = \frac{f_p l_c^2}{8} + \frac{P_{u,roof}(e)}{2} + P_u \Delta_n$$

$$M_u = \frac{40.6(21.0)^2}{8(1,000)} + \frac{1.44(5.63)}{(2)(12)} + \frac{3.20(4.55)}{12}$$

$$M_u = 2.24 + 0.34 + 1.21$$

$$M_u = 3.79 \text{ kip-ft} = 45.5 \text{ kip-in.}$$

$$\phi M_n = 46.1 \text{ kip-in.} > M_u \quad o.k. \quad \S 1914.8.3$$

Required factored shear is:

$$V_u \approx \frac{f_p l_c}{2} + \frac{P_{u,roof}(e)}{h} = \frac{40.6(21.0)}{2(1,000)} + \frac{1.44(5.63)}{12(21.0)} = 0.458 \text{ kip/ft}$$

$$\phi V_c = 0.85(2.0)(\sqrt{f_c'})bd = 0.85(2.0)(\sqrt{4,000})(12)(3.63) = 4.68 \text{ kips/ft.} \gg V_u \quad o.k.$$

∴ Wall strength is *o.k.*

8d.

Check service load deflection.

§1914.8.4

The mid-height deflection under service lateral and vertical loads cannot exceed the following:

$$\Delta_s = \frac{l_c}{150} = \frac{21.0(12)}{150} = 1.68 \text{ in.} \quad (14-3)$$

The service level moment M_s is determined as follows:

$$M_s = \frac{f_p l_c^2}{(1.4)8} + \frac{P_{roof}(e)}{2} + P \Delta_s = \frac{40.6(21.0)^2}{(1.4)(8)(1,000)} + \frac{0.92(5.63)}{2(12)} + \frac{2.05(1.68)}{12}$$

$$M_s = 2.10 \text{ kip-ft} = 25.2 \text{ kip-in.}$$

Note $M_s < M_{cr}$

$$\Delta_s = \frac{5M_s I_c^2}{48 E_c I_g} = \frac{5(25.2)(21 \times 12)^2}{48(3,605)(381)} = 0.12 \text{ in.} < 1.68 \text{ in.}$$

∴ Use #4 @ 12 in. o.c. vertical reinforcing in wall.

8e.**Additional comments.**

1. The parapet must be checked as a separate structural element for seismic forces determined from Equation (32-2) with $R_p = 3.0$ and $a_p = 2.5$. This check is not shown.
2. Attention must be given to the location of panel joints and wall openings. These can change the tributary width of wall available to resist combined axial loads and moments.
3. An iterative approach to the calculation of M_u and M_s may allow for a less conservative analysis.
4. The effective depth of the wall must be modified for architectural reveals, if these are used.

9.**Deflection of east-west diaphragm.**

Diaphragm deflections are estimated primarily to determine the displacements imposed on attached structural and nonstructural elements. Columns and walls connected to the diaphragm must satisfy the deformation compatibility requirements of §1633.2.4.

An acceptable method of determining the horizontal deflection of a plywood diaphragm under lateral forces is given in §23.222 of 1997 UBC Standard 23-2. The following equation is used:

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

The deflection of the diaphragm spanning between lines 3 and 10 will be computed. Values for each of the parameters in the above equation are given below:

$$v = \frac{wl}{2b} = \frac{(1,138 \text{ plf})(224 \text{ ft})}{2(140.67 \text{ ft})} = 906 \text{ plf}$$

$$L = 224'-0"$$

$$E = 29 \times 10^6 \text{ psi}$$

$$A = 2 \text{ \#7 bars} = 2 \times .60 = 1.20 \text{ in.}^2$$

$$b = 140.67 \text{ ft}$$

$$G = 90,000 \text{ psi}$$

Table 23-2-J

$$t = 0.54$$

Table 23-2-I

$$e_n = \text{see Table 5-2, below.}$$

Table 23-2-K

$$\Delta_c = 0 \text{ (Assume no slip in steel chord.)}$$

Table 5-2. Determination of e_n

Zone	L	Nails	s	Shear per nail	e_n
A	80'-0"	10d	2½"	906(2.5/12) = 189 lb	.042
B	144'-0"	10d	4"	583(4.0/12) = 194 lb	.044

Substituting the above parameters into the deflection equation, the deflection (in inches) at mid-span of the diaphragm is determined.

$$\Delta = \frac{5(906)(224)^3}{8(29 \times 10^6)(1.20)(140.67)} + \frac{(906)(224)}{4(90,000)(0.54)} + 0.188(80)(0.042) + 0.188(144)(0.044) + 0$$

$$\Delta = 1.30 + 1.04 + 0.63 + 1.19 = \underline{\underline{4.16 \text{ in.}}}$$

Under §1633.2.4, all structural framing elements and their connections that are part of the lateral force-resisting system and are connected to the roof must be capable of resisting the “expected” horizontal displacements. The “expected” displacements are amplified displacements taken as the greater of Δ_M or a story drift of 0.0025 times the story height. In this example, the “expected” displacement is:

$$\Delta_M = 0.7R\Delta_S = 0.7(4)(4.16 \text{ in}) = 11.6 \text{ in.} \quad (30-17)$$

Note that the R value used above is $R = 4$. This is the R value used to determine the shear in the diaphragm in Part 2b under the requirements of §1633.2.9(3).

Comment: The diaphragm deflection calculation shown above is based on strength design seismic forces. Under the 1994 UBC, seismic forces are based on ASD loads, and a smaller deflection would be calculated.

10.**Design shear force for east-west panel on line 1.**

In this part, determination of the in-plane shear force on a typical wall panel on line 1 is shown. There are a total of five panels on line 1 (Figure 5-1). The panel with the large opening is assumed not effective in resisting in-plane forces, and four panels are assumed to carry the total shear.

From Part 2, the total shear on line 1 is 36.4 kips. This force is on a strength basis and was determined using $R = 4$ for the diaphragm. Except for the diaphragm, the building is designed for $R = 4.5$, and an adjustment should be made to determine in-plane wall forces.

Earthquake loads on the shear walls must also be modified by the reliability/redundancy factor ρ . This factor varies between a minimum of 1.0 and a maximum of 1.5. Because the shear wall on line 3 (not shown) has large openings for a truck dock, the maximum element-story shear ratio, r_{max} of §1630.1.1, is large and the resulting reliability/redundancy factor for the east-west direction is the maximum value of 1.5. This requires that shear forces in individual east-west panels, determined from the analysis shown in Part 2, be increased by a factor of 1.5 as shown below.

Finally, seismic forces due to panel weight must also be included. These are determined using the base shear coefficient (.244) from Part 1. The panel seismic force is determined as follows:

Panel weight:

$$\text{width} = \frac{110 \text{ ft}}{5} = 22 \text{ ft}$$

$$W_p = 0.15 \left(\frac{7.25}{12} \right) (23 \text{ ft})(22 \text{ ft}) = 45.9 \text{ kips}$$

Seismic force due to panel weight:

$$F_p = 0.244 W_p = 0.244 (45.9 \text{ k}) = 11.2 \text{ kips}$$

The total seismic force on the panel, E , is the horizontal shear transferred from the diaphragm and the horizontal seismic force due to the panel weight, both adjusted for the reliability/redundancy factor. This calculation is shown below:

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

$$E_h = \frac{1}{4} \left(\frac{4}{4.5} \right) (36.4 \text{ k}) + (11.2 \text{ k}) = 19.3 \text{ kips}$$

$$E_v = 0$$

$$\therefore V_{\text{panel}} = \rho E_h + E_v = 1.5(19.3) + (0) = \underline{\underline{29.0 \text{ kips}}} \text{ per panel}$$

Comment: 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. A redundancy factor is computed for each principal direction. In general, they are not applied to diaphragms, except transfer diaphragms.

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Design Example 6

Tilt-Up Wall Panel With Openings

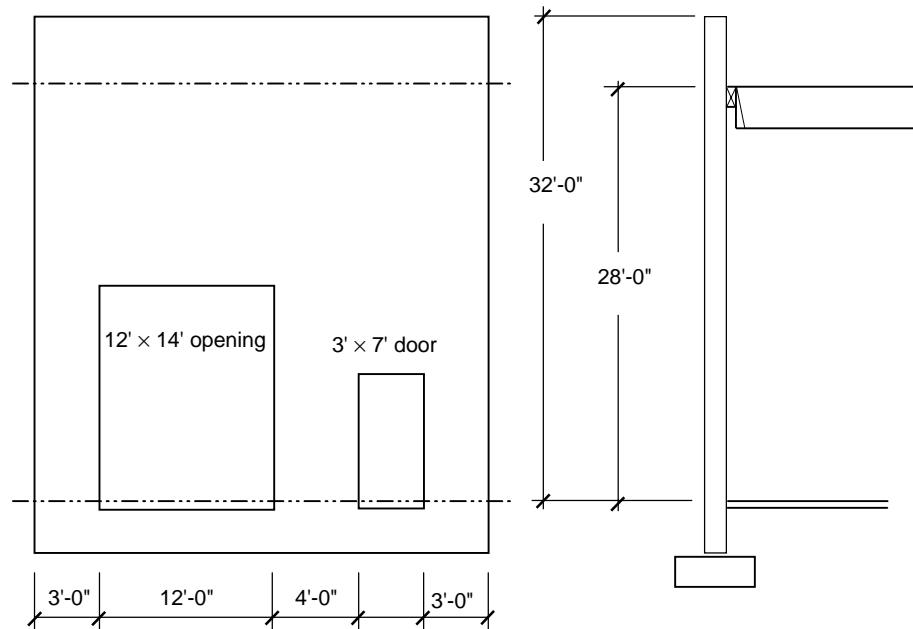


Figure 6-1. Wall elevation and section

Overview

Walls designed under the alternative slender wall method of UBC §1914.8, are typically tilt-up concrete panels that are site-cast, cured, and tilted into place. They are designed to withstand out-of-plane forces and carry vertical loads at the same time. These slender walls differ from concrete walls designed under the empirical design method (UBC §1914.5) in that there are greater restrictions on axial loads and reinforcement ratios. In addition, secondary effects of eccentricities and p-delta moments play an important role in analysis and design of these slender tilt-up panels.

In this example, the out-of-plane lateral design forces for a one-story tilt-up concrete slender wall panel with openings are determined, and the adequacy of a proposed reinforced concrete section is checked. The example is a single-story tilt-up concrete wall panel with two openings, site-cast, and tilted up into place. The pier between the two openings is analyzed using the slender

wall design method (UBC §1914.8). Analysis of the wall panel for lifting stresses or other erection loads is not a part of this example.

Outline

This example will illustrate the following parts of the design process:

1. Out-of-plane lateral design forces.
2. Basic moment from the out-of-plane forces.
3. Vertical design forces acting on the pier.
4. Nominal moment strength ϕM_n .
5. Factored moment including eccentricity and p-delta effects.
6. Service load out-of-plane deflection.
7. Special horizontal reinforcing.

Given Information

Wall material: $f'_c = 3000$ psi normal weight concrete

Reinforcing steel material: $f_y = 60,000$ psi

Wall thickness = 9¼ inches with periodic ¾-inch narrow reveals.

Reinforcing steel area = 7 #5 each face at wall section between openings.

Reinforcing depth based on 1-inch minimum cover per UBC §1907.7.1 item 4.

Loading data:

Roof loading to wall = uniform loading; 40-foot span of 12 psf dead load; no snow load.

Roof loading eccentricity = 4 inches from interior face of panel.

Seismic Zone = Zone 4

Near-source influence = more than 10 km to any significant seismic source ($N_a = 1$).

Soil profile = S_D

Seismic importance factor = 1.0

Wind does not govern this wall panel design.



Calculations and Discussion

Code Reference

1.

Out-of-plane lateral design forces.

The wall panel is subdivided into a design strip. Typically, a solid panel is subdivided into one-foot-wide design strips for out-of-plane design. However, where wall openings are involved, the entire pier width between openings is generally used as the design strip for simplicity. The distributed loading accounts for the strip's self-weight, as well as the tributary loading from above each opening.

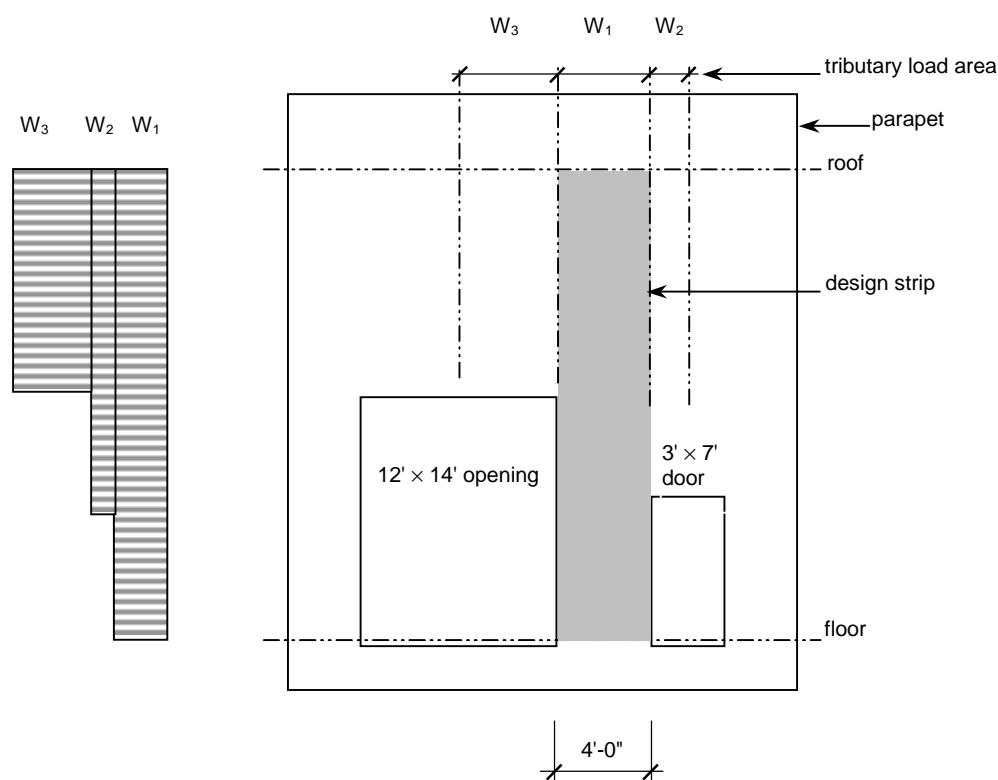


Figure 6-2. Design strip and distributed out-of-plane loading

1a.**Seismic coefficient of wall element.**

The wall panel is considered an element of a structure, thus §1632.2 applies in determining the lateral seismic force. UBC Equations 32-2 and 32-3 are used to determine forces for design.

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + 3 \frac{h_x}{h_r} \right) W_p \quad (32-2)$$

$$\text{Except: } F_p \text{ is limited by } 0.7 C_a I_p W_p \leq F_p \leq 4 C_a I_p W_p \quad (32-3)$$

$$a_p = 1.0 \quad \text{Table 16-O}$$

$$R_p = 3.0 \quad \text{Table 16-O}$$

$$C_a = 0.44 \quad \text{Table 16-Q}$$

$$I_p = 1.0 \quad \text{Table 16-K}$$

Therefore, the limits on F_p are: $0.308 W_p \leq F_p \leq 1.76 W_p$

h_x is defined as the attachment height above grade level. Since the wall panel is connected at two different heights, an equivalent lateral force will be obtained using the average of the roof F_p and the at-grade F_p [ref. 1999 SEAOC Blue Book Commentary §C107.2.3].

$$F_{p_{\text{roof}}} = \frac{(1.0)(0.44)(1.0)}{3.0} \left(1 + 3 \frac{h_r}{h_r} \right) W_p = 0.587 W_p$$

$$F_{p_{\text{grade}}} = \frac{(1.0)(0.44)(1.0)}{3.0} \left(1 + 3 \frac{0}{h_r} \right) W_p = 0.147 W_p,$$

but $F_{p_{\text{min}}} = 0.308 W_p$ governs.

$$F_{p_{\text{wall}}} = \frac{F_{p_{\text{grade}}} + F_{p_{\text{roof}}}}{2} = \frac{0.587 + 0.308}{2} = 0.448 W_p$$

Note: The seismic coefficient 0.448 is virtually the same as the 1994 UBC coefficient 0.30 when adjusted for strength design and the different seismic zone coefficient C_a defaults:

$$F_p(\text{1994 UBC equivalent}) = \frac{0.448 W_p}{1.4} \left(\frac{0.40}{0.44} \right) = 0.291 \approx 0.30$$

1b.**Load combinations for strength design.**

For this example, the use of load combination (12-5) of §1612.2.1 is applicable, and governs for concrete strength design under seismic loading.

$$1.2D + 1.0E + (f_1L + f_2S) \quad (12-5)$$

where:

D = self weight of wall and dead load of roof

L = 0 (floor live load)

S = 0 (snow load)

$$E = \rho E_h + E_v \text{ where } \rho = 1.0 \text{ (§1632.2) and } E_v = 0.5C_aID \quad (30-1)$$

Load combination (12-5) reduces to:

$$(1.2 + 0.5C_aI)D + 1.0E_h \text{ or } (1.2 + 0.22)D + 1.0E_h \\ \text{or } 1.42D + 1.0E_h$$

Note: Exception 2 under §1612.2.1, which multiplies strength design load combinations by 1.1, has been determined to be inappropriate by SEAOC and others, and has not been included in the 1999 SEAOC Blue Book, *Recommended Lateral Force Requirements and Commentary*. For the purposes of this example, the 1.1 multiplier has been included in order to conform to the 1997 UBC as originally published. For additional information, see “Design of Reinforced Concrete Buildings under the 1997 UBC,” by S.K. Ghosh, published in *Building Standards*, May-June 1998, ICBO.

Load combination (12-5) increases to:

$$1.1(1.42D + 1.0E_h) = 1.56D + 1.1E_h$$

1c.**Lateral out-of-plane wall forces.**

The lateral wall forces E_h are determined by multiplying the wall's tributary weight by the lateral force coefficient. Three different distributed loads are determined due to the presence of two door openings of differing heights. See Figure 6-2.

$$\text{Wall weight} = \frac{9.25}{12} 150 \text{ pcf} = 116 \text{ lb/ft}^2$$

$$F_{p_{wall}} = 0.448 (116 \text{ lb/ft}^2) = 52 \text{ lb/ft}^2$$

$$W_1 = 52 \text{ lbs/ft}^2 \times 4 \text{ ft} = 208 \text{ plf}$$

$$W_2 = 52 \text{ lbs/ft}^2 \times 3/2 \text{ ft} = 78 \text{ plf}$$

$$W_3 = 52 \text{ lbs/ft}^2 \times 12/2 \text{ ft} = 312 \text{ plf}$$

2.

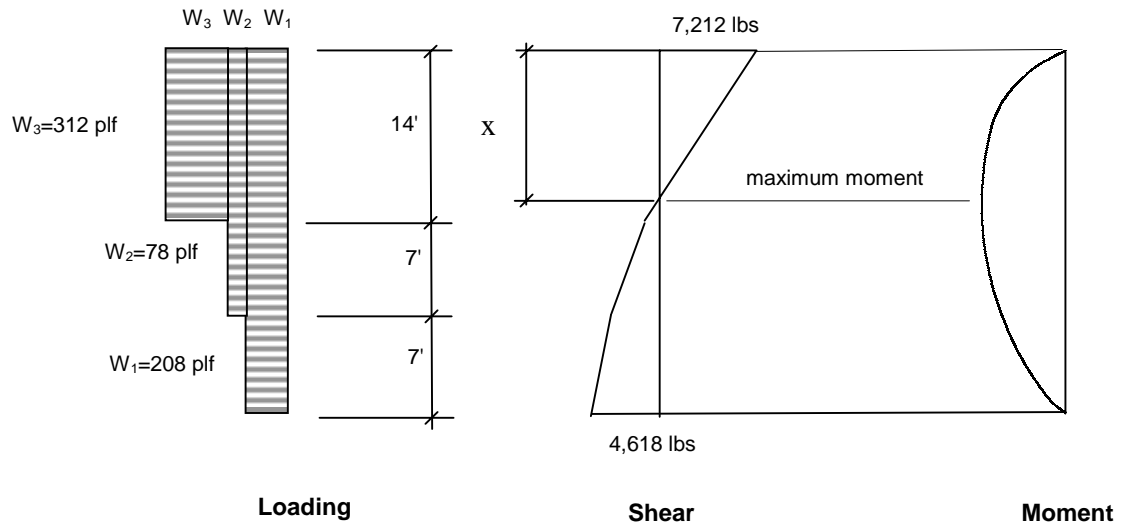
Basic moment from out-of-plane forces.

Figure 6-3. Corresponding loading, shear, and moment diagrams

Locate the point of zero shear for maximum moment. Ignore the parapet's negative moment benefits in reducing the positive moment for simplicity of analysis. If the designer decides to use the parapet's negative moment to reduce the positive moment, special care should be taken to use the shortest occurring parapet height. For this analysis, the seismic coefficient for the parapet shall be the same as that for the wall below ($a_p = 1.0$, not 2.5). The parapet should be checked separately later, but is not a part of this example.

This example conservatively assumes the maximum moment occurs at a critical section width of 4'-0". In cases where the maximum moment occurs well above the doors, a more comprehensive analysis could consider several critical design sections, which would account for a wider design section at the location of maximum moment and for a narrower design section with reduced moments near the top of the doors.

2a.**Determine the shear reactions at each support.** R_{grade} = shear reaction at grade level for design strip R_{roof} = shear reaction at roof level for design strip

$$R_{grade} = \left[208 \frac{(28)^2}{2} + 78 \frac{(21)^2}{2} + 312 \frac{(14)^2}{2} \right] \frac{1}{28} = 4,618 \text{ lbs}$$

$$R_{roof} = [208(28) + 78(21) + 312(14)] - 4618 = 7,212 \text{ lbs}$$

Determine the distance of the maximum moment from the roof elevation downward (Figure 6-3):

$$X = \frac{7212}{(208 + 78 + 312)} = 12.1 \text{ feet to point of zero shear (maximum moment)}$$

2b.**Determine $M_{u \text{ basic}}$**

This is the primary strength design moment, *excluding* p-delta effects and vertical load eccentricity effects, but *including* the 1.1 load factor (see the earlier discussion of this load factor in Step 1b, above):

$$M_{u \text{ basic}} = 1.1 \left[7212(12.1) - (208 + 78 + 312) \frac{(12.1)^2}{2} \right] = 47,837 \text{ lb-ft}$$

$$M_{u \text{ basic}} = 47.8 \text{ k-ft}$$

3.**Vertical design forces acting on the center pier.**

The pier's vertical loads are comprised of a roof component P_{roof} and a wall component P_{wall} . The applicable portion of the wall component is the top portion $P_{wall \text{ top}}$ above the design section.

 P_{roof} = gravity loads from the roof acting on the design strip

The appropriate load combinations using strength or allowable stress design do not include roof live load in combination with seismic loads. However, strength designs considering wind loads must include a portion of roof live loads per §1612.2.1.

$P_{roof} = (\text{roof dead load}) \times (\text{tributary width of pier}) \times (\text{tributary width of roof})$

$$P_{roof} = (12 \text{ psf}) \left(4 + \frac{3}{2} + \frac{12}{2} \right) \frac{40}{2} = 2,760 \text{ lb}$$

Note: When concentrated gravity loads, such as from a girder, are applied to slender walls, the loads are assumed to be distributed over an increasing width at a slope of 2 vertical to 1 horizontal down to the flexural design section height (§1914.8.2.4).

$P_{wall\ top}$ = the portion of the wall's self weight above the flexural design section. It is acceptable to assume the design section is located midway between the floor and roof levels

$$P_{wall\ top} = (116 \text{ psf}) \left(4 + \frac{3}{2} + \frac{12}{2} \right) \left(\frac{28}{2} + 4 \right) = 24,012 \text{ lbs}$$

$$P_{total} = P_{roof} + P_{wall\ top} = 2760 + 24012 = 26,772 \text{ lbs}$$

Check the vertical service load stress for applicability of the slender wall design method (UBC §1914.8.2 item 1). Use the net concrete section considering the reveal depth:

$$\text{stress} = \frac{P_{total}}{A_{conc}} = \frac{26772}{48(9.25 - 0.75)} = 66 \text{ psi} < 0.04f'_c = 0.04(3000) = 120 \text{ psi} \quad o.k.$$

The compressive stress is low enough to use the alternative slender wall method; otherwise a different method, such as the empirical design method (§1914.5), would be required along with its restrictions on wall height.

4.

Nominal moment strength ϕM_n .

The nominal moment strength ϕM_n is given by the following equation:

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2} \right)$$

where:

$$\phi = 0.9 - \frac{0.2P_u}{0.10f'_c A_{conc}} = 0.9 - \frac{0.2(1.56)(26772)}{0.10(3000)(48)(9.25 - 0.75)} = 0.83 \quad \text{§1909.3.2.2}$$

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{1.56(26772) + 7(0.31)(60000)}{60000} = 2.87 \text{ in.}^2$$

$$a = \frac{P_u + A_s f_y}{0.85f'_c b} = \frac{1.56(26772) + 7(0.31)(60000)}{0.85(3000)(48)} = 1.40 \text{ in.}$$

Reinforcing depth is based on new tilt-up cover provision §1907.7.1 item 4.

$$d = \text{thickness} - \text{reveal} - \text{cover} - \text{tie diameter} - \frac{1}{2} \text{ bar diameter}$$

$$d = 9 \frac{1}{4} - \frac{3}{4} - 1 - \frac{3}{8} - \left(\frac{1}{2}\right)\left(\frac{5}{8}\right) = 6.8 \text{ in.}$$

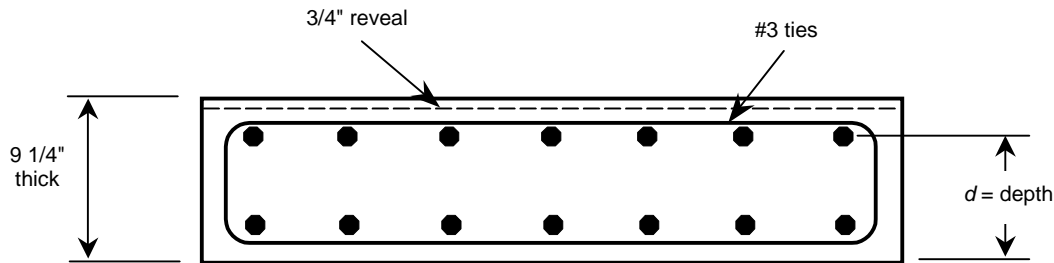


Figure 6-4. Design section

Thus:

$$M_n = 2.87 (60000) \left(6.8 - \frac{1.40}{2} \right) = 1050 \text{ k} - \text{in} = 87.5 \text{ k} - \text{ft}$$

$$\phi M_n = 0.83 (87.5) = 72.6 \text{ k} - \text{ft}$$

Verify that $M_{cr} < \phi M_n$ to determine the applicability of the slender wall design method (UBC §1914.8.2 item 3). M_{cr} is defined uniquely for slender walls in UBC §1914.0.

$$M_{cr} = 5\sqrt{f'_c} \frac{I_g}{y_t} = \frac{5\sqrt{3000} (48) \frac{(9.25)^3}{12}}{\frac{9.25}{2}} = 187,458 \text{ lb} - \text{in.} = 15.6 \text{ k} - \text{ft} \quad \text{§1914.0}$$

$$M_{cr} = 15.6 \text{ k} - \text{ft} < \phi M_n = 72.6 \text{ k} - \text{ft} \quad o.k.$$

Sufficient reinforcing is provided to use the alternative slender wall method, otherwise the empirical design method of UBC §1914.5 would be necessary.

Note: For the purposes of §1914.8.2 item 3, I_g and y_t are conservatively based on the gross thickness without consideration for reveal depth. This approach creates a worst-case comparison of M_{cr} to ϕM_n . In addition, the exclusion of the reveal depth in the M_{cr} calculation produces more accurate deflection values when reveals are narrow.

Verify the reinforcement ratio $\rho \leq 0.6\rho_b$ to determine the applicability of the slender wall design method (§1914.8.2 item 2):

$$0.6\rho_b = 0.6 \frac{0.85\beta_1 f'_c}{f_y} \frac{87000}{87000 + f_y} = 0.6 \frac{0.85(0.85)3000}{60000} \frac{87000}{(87000 + 60000)} = 0.0128 \quad (8-1)$$

$$\rho = \frac{A_s}{bd} = \frac{7(0.31)}{48(6.8)} = 0.0066 < 0.0128 \quad o.k.$$

Therefore, the slender wall method is applicable.

5.

Factored moment, including eccentricity and p-delta effects.

Determine the design moment including the effects from the vertical load eccentricity and p-delta ($P\Delta$):

$$M_u = M_{u \text{ basic}} + M_{u \text{ eccentricity}} + M_{u P\Delta}$$

Use the figures below to determine $M_{u \text{ eccentricity}}$ and $M_{u P\Delta}$:

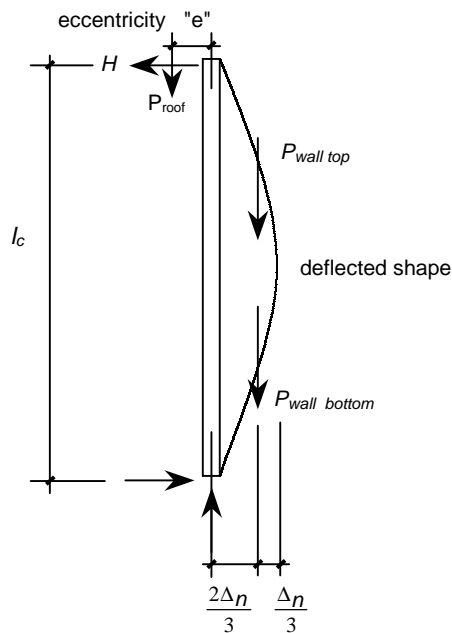


Figure 6-5. Vertical loading

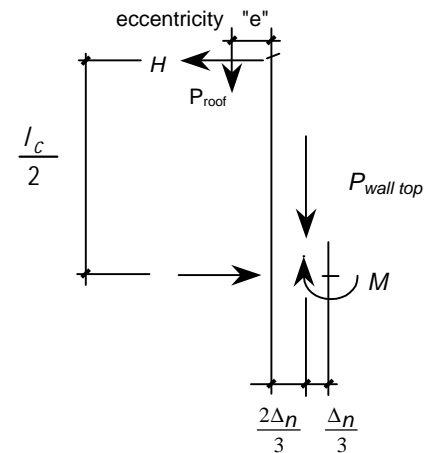


Figure 6-6. Freebody of upper half

5a. Determine force component H from statics (moment about base of wall).

From Figure 6-5, assuming a parabolic deflected shape:

$$H = \frac{(P_{wall\ top} + P_{wall\ bottom}) \frac{2\Delta_n}{3} - P_{roof}e}{l_c}$$

Since the panel's openings are not positioned symmetrically with the panel's mid-height, $P_{wall\ bottom}$ will be less than $P_{wall\ top}$. For ease of calculation, conservatively assume $P_{wall\ bottom} = P_{wall\ top}$, as is similar to panels without openings.

$$H = \frac{4P_{wall\ top}\Delta_n}{3l_c} - \frac{P_{roof}e}{l_c}$$

5b. Determine moment component M from statics using Figure 6-6 to account for eccentricity and $P\Delta$ effects:

$$M = P_{roof}(\Delta_n + e) + P_{wall\ top} \frac{\Delta_n}{3} + H \frac{l_c}{2}$$

$$M = P_{roof} \frac{e}{2} + (P_{wall\ top} + P_{roof})\Delta_n$$

5c. Determine the wall's deflection at full moment capacity Δ_n .

$$\Delta_n = \frac{5M_n l_c^2}{48E_c I_{cr}} \quad \S 1914.8.4$$

where:

M_n is from Step 4.

$$E_c = 57\sqrt{f'_c} = 3122 \text{ ksi} \quad \S 1908.5.1$$

$$I_{cr} = nA_{se}(d - c)^2 + \frac{bc^3}{3}; \quad \text{where } c = \frac{a}{0.85} = \frac{1.40}{0.85} = 1.65 \text{ in.}$$

$$I_{cr} = \frac{29,000}{3,122} 2.87(6.8 - 1.65)^2 + \frac{48(1.65)^3}{3} = 779 \text{ in.}^4$$

$$\Delta_n = \frac{5(87.5)(28)^2(12)^3}{48(3122)(779)} = 5.1 \text{ in.}$$

Section 1914.8.3 requires the maximum potential deflection Δ_n be assumed in the calculation of the $P\Delta$ moment, unless a more comprehensive analysis is used. An iterative approach or use of a moment magnifier are examples of acceptable “more comprehensive” analyses, but are beyond the scope of this example.

5d.**Determine and check the total design moment M_u .**

$$\begin{aligned} M_u &= M_{u \text{ basic}} + M_{u \text{ eccentricity}} + M_{u P\Delta} \\ M_u &= 47.8 + P_{u \text{ roof}} \frac{e}{2} + (P_{u \text{ wall top}} + P_{u \text{ roof}}) \Delta_n \\ M_u &= 47.8 + 1.56(2.76) \frac{1}{2} \left(4 + \frac{9.25 - 0.75}{2} \right) \frac{1}{12} + 1.56(24.0 + 2.76)(5.1) \frac{1}{12} \\ M_u &= 47.8 + 1.5 + 17.7 \\ M_u &= 67.0 \text{ k} - \text{ft} < \phi M_n = 72.6 \text{ k} - \text{ft} \quad o.k. \end{aligned} \quad (14-2)$$

Therefore, the design section's strength is acceptable.

6.**Service load out-of-plane deflection.****6a.****Determine if the wall's cross-section is cracked.**

The service load moment M_s is determined with the following formula where the denominators are load factors to convert from load combination (12-5) to load combination (12-13):

$$M_s = \frac{M_{u \text{ basic}}}{1.1(1.4)} + \frac{M_{u \text{ eccentricity}}}{1.56} + M_{s P\Delta}$$

Assume the service load deflection is the maximum allowed $\frac{l_c}{150}$:

$$\Delta_{s \text{ Maximum}} = \frac{l_c}{150} = \frac{28(12)}{150} = 2.24 \text{ in.} \quad (14-3)$$

$$M_{s P\Delta} = (P_{\text{wall}} + P_{\text{roof}}) \Delta_s = (24.0 + 2.76) 2.24 = 59.9 \text{ k} - \text{in.} = 5.00 \text{ k} - \text{ft}$$

$$M_s = \frac{M_{u \text{ basic}}}{1.1(1.4)} + \frac{M_{u \text{ eccentricity}}}{1.56} + M_{s P\Delta}$$

$$M_s = \frac{47.8}{1.1(1.4)} + \frac{1.5}{1.56} + 5.00 = 37.0 \text{ k} - \text{ft}$$

$$M_{cr} = 15.6 \text{ k} - \text{ft} < M_s$$

Therefore, section is cracked and Equation (14-4) is applicable for determining Δ_s .
If the section is uncracked, Equation (14-5) is applicable.

6b.**Determine the deflection at initiation of cracking Δ_{cr} .**

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} = \frac{5(15.6)(28)^2(12)^3}{48(3122)\frac{(48)(9.25)^3}{12}} = 0.22 \text{ in.} \quad \text{\S 1914.8.4}$$

I_g is based on gross thickness, without consideration for the architectural reveal depth, since this produces more accurate results when the reveals are narrow.

6c.**Determine and check the service load deflection Δ_s .**

$$\Delta_s = \Delta_{cr} + \left(\frac{M_s - M_{cr}}{M_n - M_{cr}} \right) (\Delta_n - \Delta_{cr}) \quad (14-4)$$

$$\Delta_s = 0.22 + \left(\frac{37.0 - 15.6}{87.5 - 15.6} \right) (5.1 - 0.22) = 1.67 \text{ in.}$$

$$\Delta_s = 1.67 \text{ in.} < \frac{l_c}{150} = 2.24 \text{ in.} \quad o.k. \quad \text{\S 1914.8.4}$$

Therefore, the proposed slender wall section is acceptable using the alternative slender wall method.

7.**Special horizontal reinforcing.****7a.****Determine the horizontal reinforcing required above the largest wall opening for out-of-plane loads.**

The portion of wall above the twelve-foot-wide door opening spans horizontally to the vertical design strips on each side of the opening. This wall portion will be designed as a one-foot unit horizontal design strip and subject to the out-of-plane loads computed in this example earlier.

$$F_{p_{wall}} = 0.448(116 \text{ lbs/ft}^2) = 52 \text{ lb/ft}^2$$

The moment is based on a simply supported horizontal beam with the 1.1 multiplier per Exception 2 under §1612.2.1:

$$\begin{aligned} M_u &= 1.1 \left(F_p \frac{(\text{opening width})^2}{8} \right) = 1.1 \left(52 \frac{12^2}{8} \right) \\ &= 1030 \text{ lb-ft} = 1.03 \text{ k-ft} \end{aligned}$$

Try using #5 bars at 18-inch spacing to match the same bar size as being used vertically at the maximum allowed spacing for wall reinforcing.

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

where:

$$\phi = 0.9 \text{ and } A_s = 0.31 \left(\frac{12}{18} \right) = 0.21 \text{ in.}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.21)(60000)}{0.85(3000)(12)} = 0.41 \text{ in.}$$

Assume the reinforcing above the opening is a single curtain with the vertical steel located at the center of the wall's net section. The horizontal reinforcing in concrete tilt-up construction is typically placed over the vertical reinforcing when assembled on the ground.

$$d = \frac{1}{2} (\text{thickness} - \text{reveal}) - \text{bar diameter}$$

$$d = \frac{1}{2} \left(9 \frac{1}{4} - \frac{3}{4} \right) - \frac{5}{8} = 3.63 \text{ in.}$$

$$\phi M_n = 0.9 (.21) (60) \left(3.63 - \frac{0.41}{2} \right) = 38.8 \text{ k} - \text{in.}$$

$$= 3.24 \text{ k} - \text{ft} \geq 1.03 \text{ k} - \text{ft} \quad o.k.$$

$$\phi M_n \geq M_u \quad o.k.$$

Therefore, the horizontal reinforcing is acceptable.

7b.

Typical reinforcing around openings.

Two #5 bars are required around all window and door openings per §1914.3.7. The vertical reinforcing on each face between the openings provides two bars along each jamb of the openings, and thus satisfies this requirement along vertical edges. Horizontally, two bars above and below the openings are required to be provided. In addition, it is common to add diagonal bars at the opening corners to assist in limiting the cracking that often occurs due to shrinkage stresses (Figure 6-7).

7c.

Required horizontal (transverse) reinforcing between the wall openings.

The style and quantity of horizontal (transverse) reinforcing between the wall openings is dependent on several factors relating to the in-plane shear wall design of §1921.6. Sections conforming to “wall piers,” as defined in §1921.1, shall be reinforced per §1921.6.13. Wall pier reinforcing has special spacing limitations and is often provided in the form of closed ties. In narrow piers, these ties are often preferred so as to assist in supporting both layers of reinforcing during construction, even if not required by the special wall pier analysis (Figure 6-7).

Configurations not defined as wall piers, but which have high in-plane shears, also have special transverse reinforcing requirements per §1921.6.2.2. In these situations, the transverse reinforcing is required to be terminated with a hook or “U” stirrup.

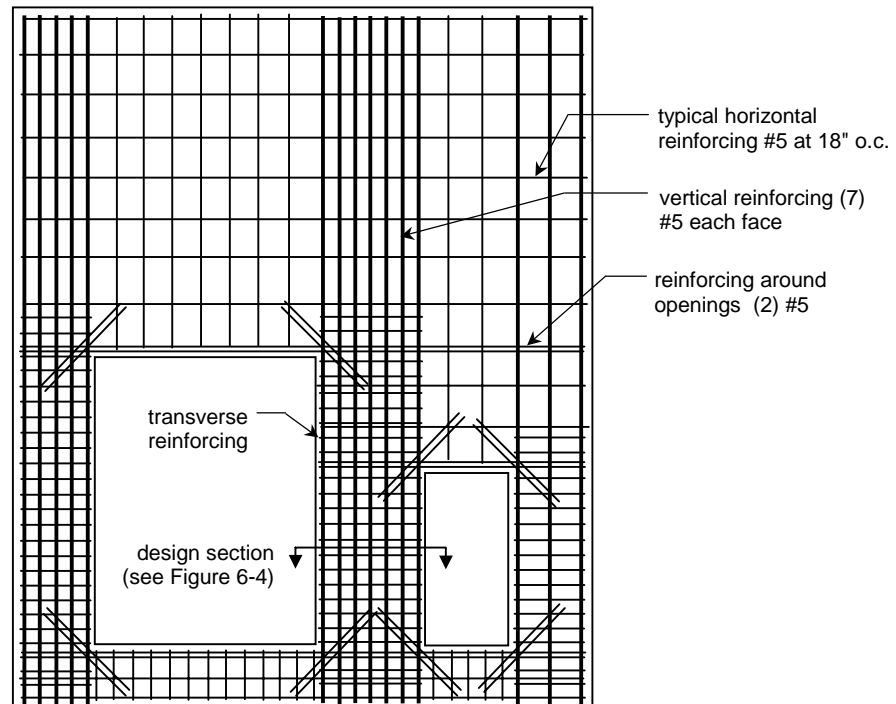


Figure 6-7. Typical wall reinforcing

Commentary

The UBC section on the alternative slender wall method made its debut in the 1988 edition. It is largely based on the equations, concepts, and full-scale testing developed by the Structural Engineers Association of Southern California and published in the *Report of the Task Committee on Slender Walls* in 1982. The American Concrete Institute (ACI) has incorporated similar provisions for slender wall design in their publication ACI 318-99.

Tilt-up wall construction has become very popular due to its versatility and its erection speed. However, wall anchorage failures at the roofline have occurred during past earthquakes. In response to these failures, the 1997 UBC anchorage design forces and detailing requirements are significantly more stringent than they have been under past codes (see Design Example 5).

References

- Recommended Tilt-up Wall Design*, Structural Engineers Association of Southern California, 1979. 5360 Workman Mill Road, Whittier, CA 90601 (562) 908-6131.
- Report of the Task Committee on Slender Walls*, Southern California Chapter American Concrete Institute and the Structural Engineers Association of Southern California, 1982.
- Brooks, Hugh, *The Tilt-up Design and Construction Manual*, HBA Publications, Fourth Edition, 1990. 2027 Vista Caudal, Newport Beach, CA 92660.
- Tilt-Up Concrete Structures Reported by ACI Committee 551*, American Concrete Institute, 1997. P.O. Box 9094, Farmington Hills, MI 48333 (248) 848-3700.
- “Design of Reinforced Concrete Buildings under the 1997 UBC,” *Building Standards*. S.K. Ghosh, ICBO, May-June 1998. 5360 Workman Mill Road, Whittier, CA 90601

Example 1**Seismic Zone 4 Near-Source Factor****§1629.4.2**

The 1997 UBC introduced the concept of near-source factors. Structures built within 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.



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

Calculations and Discussion**Code Reference**

Determine N_a and N_v .

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** (this is discussed on page vii of the UBC fault book). Thus, the distance from the site to the source is $6 \text{ km} + 2 \text{ km} = 8 \text{ km}$.

Distance from site to fault zone: 8 km.

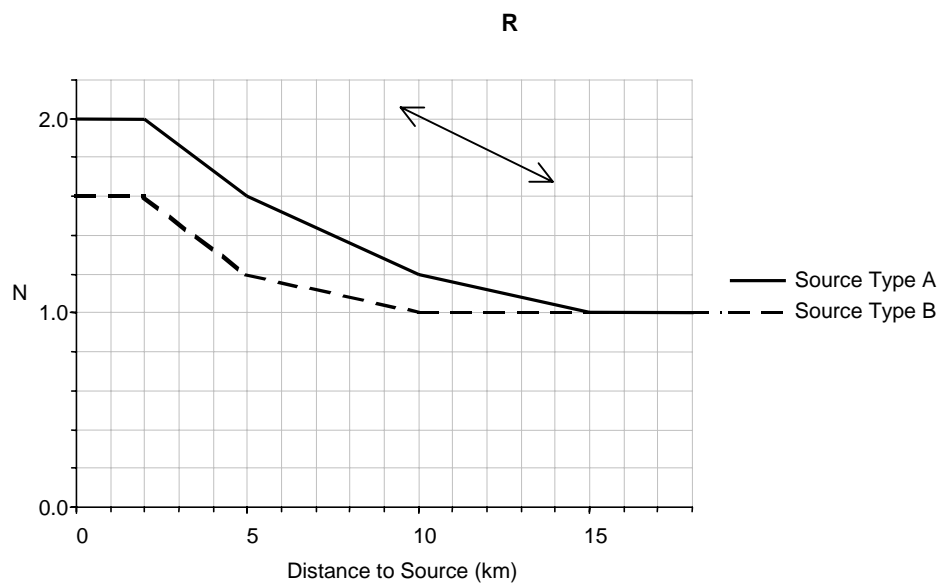
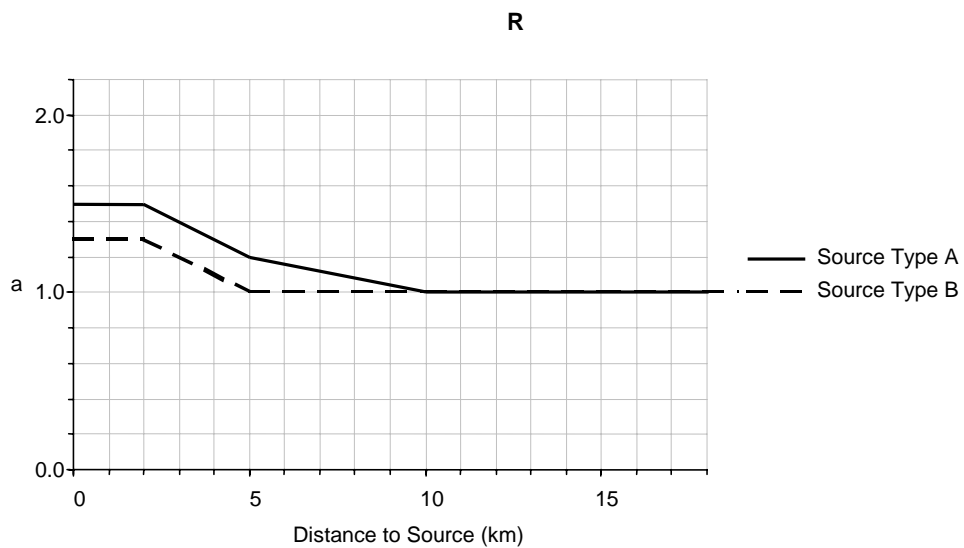
Values of N_a and N_v are given in Tables 16-S and 16-T for distances of 2, 5, 10, and 15 km. For other distances, interpolation must be done. N_a and N_v have been plotted below. For this site, N_a and N_v can be determined by entering the figures at a distance 8 km. and using the source type A curves.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

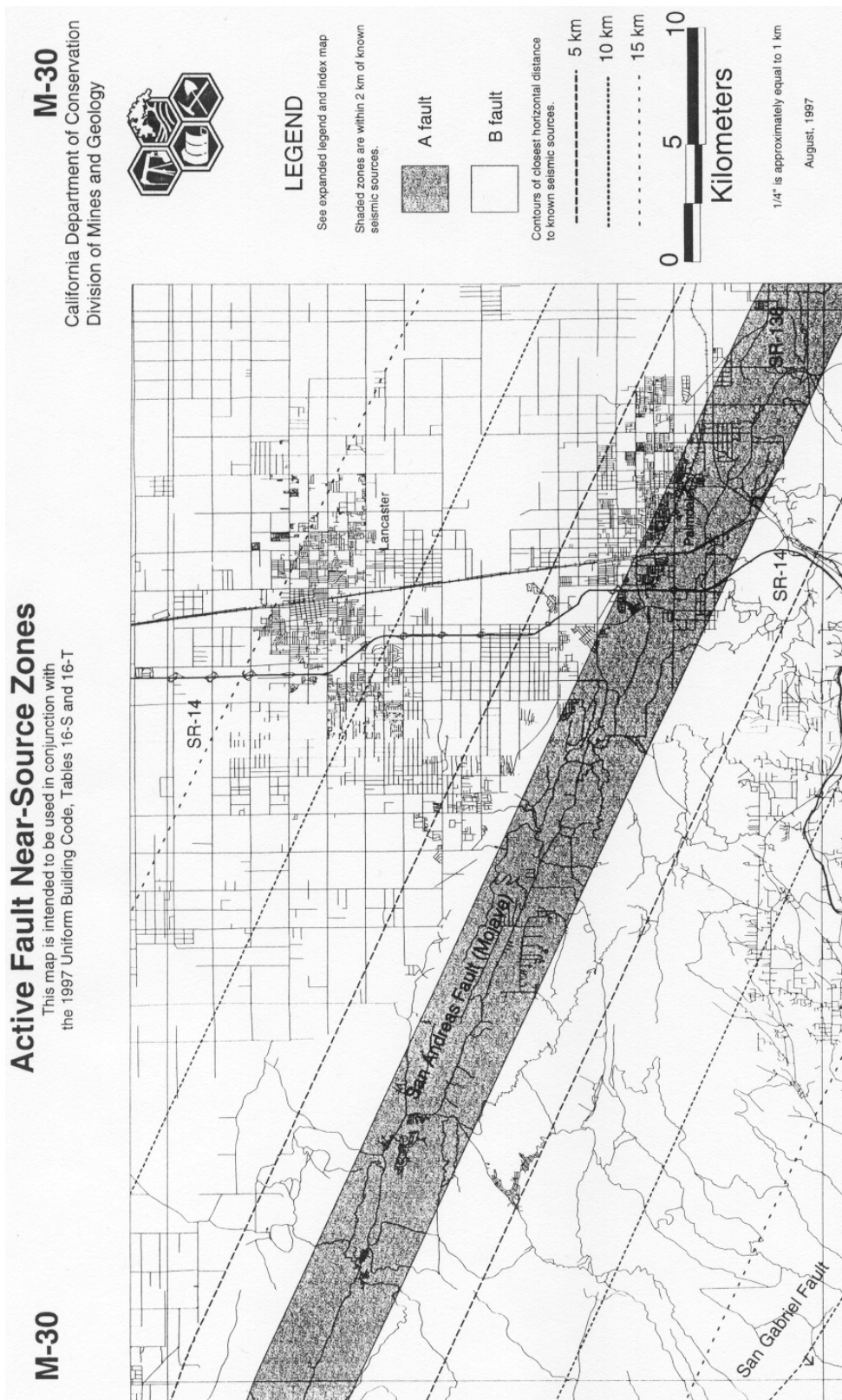
$$N_a = \underline{\underline{1.08}}$$

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

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.





Introduction to Vertical Irregularities

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 are 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.

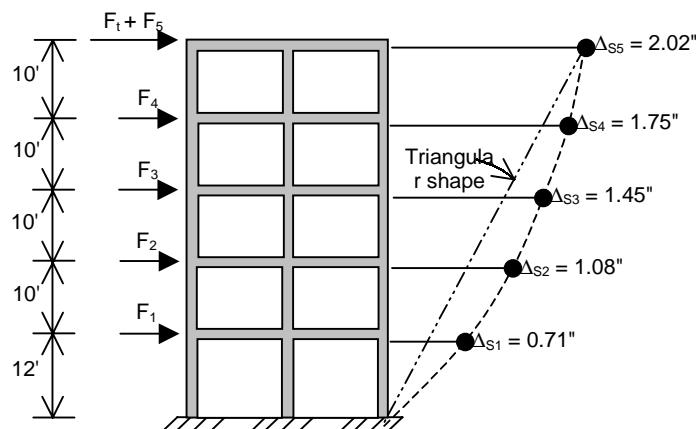
Regular structures are assumed to have a reasonably uniform distribution of inelastic behavior in elements throughout the lateral force resisting system. When vertical irregularity Types 4 and 5 exist, there is the possibility of having localized concentrations of excessive inelastic deformations due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

Example 2

Vertical Irregularity Type 1

§1629.5.3

For example: 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.



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

Calculations and Discussion

Code Reference



To determine if this is a Type 1 vertical irregularity, stiffness irregularity—soft story, there 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 these 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 the 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 inter-story 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 $\frac{\Delta_{s1}}{h_1}$ exceeds $\frac{\Delta_{s2} - \Delta_{s1}}{h_2}$., or
2. When 80 percent of $\frac{\Delta_{s1}}{h_1}$ 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:

$$0.70\left(\frac{\Delta_{s1}}{h_1}\right) = 0.70(0.00493) = 0.00345 > 0.00308$$

∴ Soft story exists

Checking the 80 percent requirement:

$$0.80\left(\frac{\Delta_{s1}}{h_1}\right) = 0.80(0.00493) = 0.00394 > 0.00289$$

∴ Soft story exists

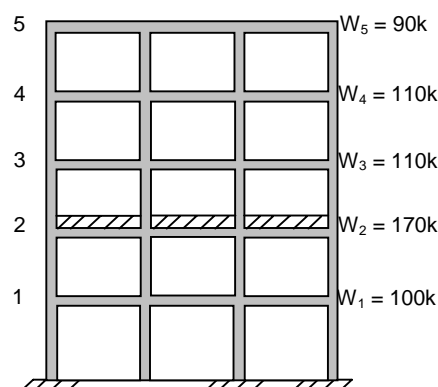
Commentary

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 displacements 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 3**Vertical Irregularities 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:

**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

$$1.5 \times W_1 = 1.5(100k) = 150k$$

At Level 3

$$1.5 \times W_3 = 1.5(110k) = 165k$$

$$W_2 = 170k > 150k$$

\therefore Weight irregularity exists

Commentary

As in the case of 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 4**Vertical Irregularity Type 3****§1629.5.3****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

$$1.5 \times W_1 = 1.5(100k) = 150k$$

At Level 3

$$1.5 \times W_3 = 1.5(110k) = 165k$$

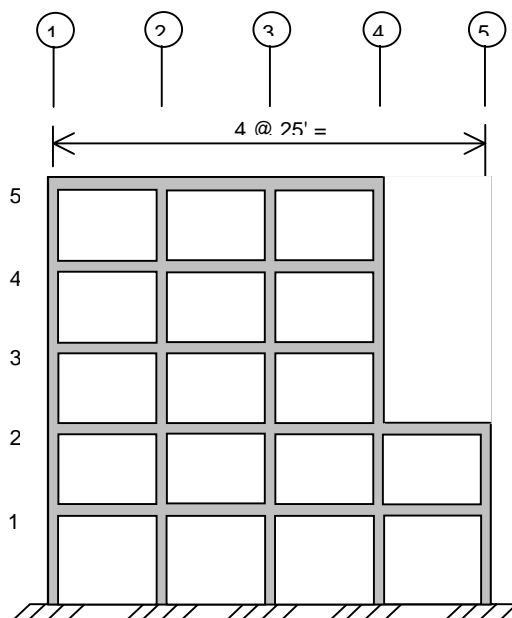
$$W_2 = 170k > 150k$$

\therefore Weight irregularity exists

Commentary

As in the case of 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).

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.



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 set-back 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

∴ Vertical geometric irregularity exists

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.

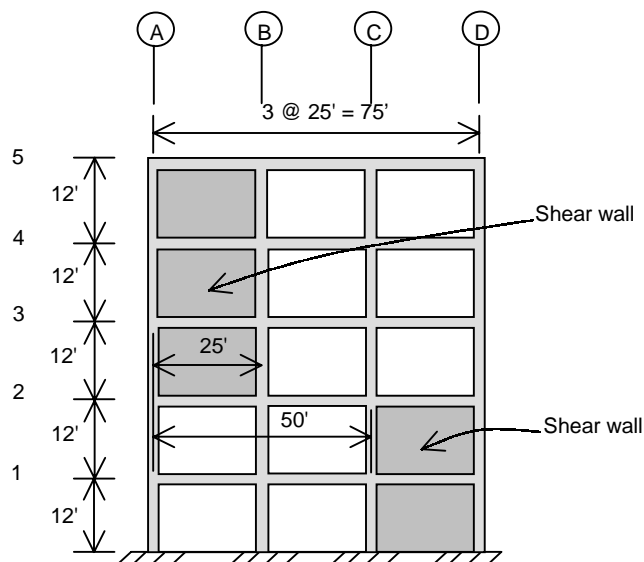
Example 5**Vertical Irregularity Type 4**

§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.



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.

∴ 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.