2006 IBC®

STRUCTURAL/SEISMIC DESIGN MANUAL

2

BUILDING DESIGN EXAMPLES FOR LIGHT-FRAME, TILT-UP AND MASONRY

SECOND EDITION

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Structural Engineers Association of California (SEAOC)
1414 K Street, Suite 260
Sacramento, CA 95814
Telephone: (916) 447-1198; Fax: (916) 443-8065
E-mail: lee@seaoc.org; Web address: www.seaoc.org

The Structural Engineers Association of California (SEAOC) is a professional association of four regional member organizations (Southern California, Northern California, San Diego, and Central 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

International Code Council

Disclaimer

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Preface

This is the second volume in the three-volume 2006 *IBC Structural/Seismic Design Manual.* It has been developed and funded by the Structural Engineers Association of California (SEAOC). It is intended to provide guidance on the interpretation and use of the seismic requirements in the 2006 *International Building Code (IBC),* published by the International Code Council, Inc.

The 2000 *IBC Structural/Seismic Design Manual* was developed to fill a void that exists between the commentary of SEAOC's Blue Book, which explained the basis for the code provisions, and everyday structural engineering design practice. The 2006 *IBC Structural/Seismic Design Manual* illustrates how the provisions of the code are used. *Volume 1: Code Application Examples,* provides step-by-step examples for using individual code provisions, such as computing base shear or building period. *Volumes 2 and 3: Building Design Examples,* furnish examples of seismic design ofcommontypes of buildings. In Volumes 2 and 3, 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.

The examples in the 2006 *IBC Structural/Seismic Design Manual* do not necessarily illustrate the only appropriate methods of design and analysis. Proper engineering judgment should always be exercised when applying these examples to real projects. The 2006 *IBC Structural/Seismic Design Manual* is not meant to establish a minimum standard of care but, instead, presents reasonable approaches to solving problems typically encountered in structural/seismic design.

The example numbers used in the prior Seismic Design Manuals—1997 UBC and 2000 IBC Volume 2 building design example problems have been retained herein to provide easy comparison to revised code requirements.

SEAOC, NCSEA, and ICC intend to update the 2006 *IBC Structural/Seismic Design Manual* with each new edition of the building code.

Acknowledgments

Authors

The 2006 *IBC Structural/Seismic Design Manual—Volume 2* was written by a group of highly qualified structural engineers. 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 1, 2, and 3 are:

Jon P. Kiland, *Co-Project Manager*
Rafael Sabelli, *Co-Project Manager*
Douglas S. Thompson
Dan Werdowatz
Matt Eatherton
John W. Lawson
Joe Maffei
Kevin Moore
Stephen Kerr

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 review of code interpretations as well as detailed checking of the numerical computations.

Seismology Committee

Close collaboration with the SEAOC Seismology Committee was maintained during the development of the document. The 2004-2005 and 2005-2006 committees reviewed the document and provided many helpful comments and suggestions. Their assistance is gratefully acknowledged.

Production and Art

ICC

How to Use This Document

ASCE/SEI 7-05 notation is generally used throughout. Some other notation is also defined in the following pages, or in the examples.

Throughout the document, reference to specific code provisions and equations is given in the right-hand margin under the category Code Reference. For example, "ASCE/SEI 7-05 Section 12.3" is given as §12.3 with ASCE/SEI 7-05 being understood. "Equation (12-4-1)" is designated Eq 12.4-1. The phrase "T 15.2.1" is understood to be Table 15.2.1 and Figure 22-1 is designated F 22-1.

The 2006 *IBC Structural/Seismic Design Manual—Volume 2* primarily references the ASCE/SEI 7-05, unless otherwise indicated. References to IBC sections, tables, and equations are enclosed in parentheses. Occasionally, reference is made to other codes and standards (e.g., ACI 318-99 or 1997 NDS). When this is so, these documents are clearly identified.

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" that 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 "Foreword" and/or "Factors Influencing Design" section that contains remarks on salient points about the design.

Design Example 5
Tilt-up Building

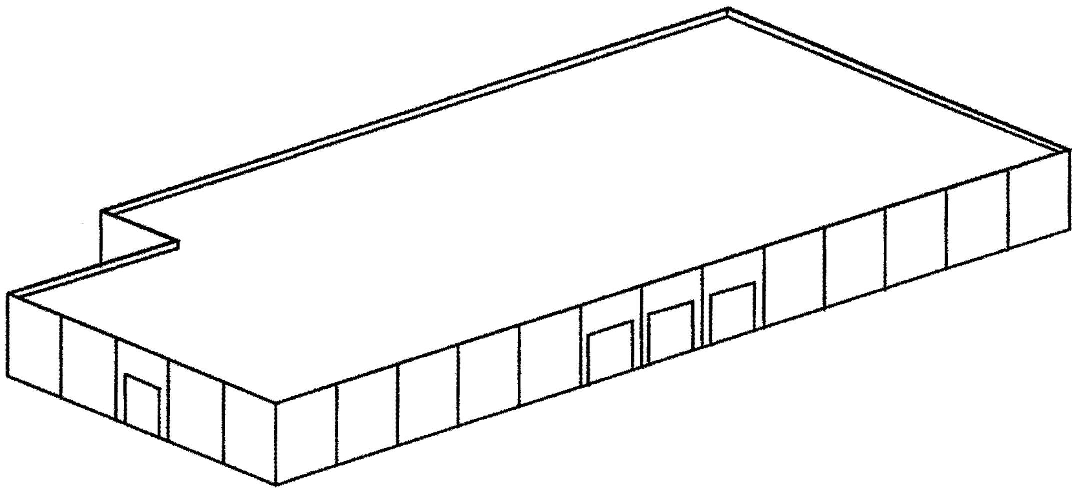

Figure 5-1. Tilt-up building

Overview

This example presents the seismic design of major components of a tilt-up building. 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 was wall-roof separation, with subsequent partial collapse of the roof. Since those events, the building codes have significantly improved, yet a major earthquake has yet to test the current tilt-up code provisions.

The example building is a warehouse, shown in Figure 5-1, which has tilt-up concrete walls and a panelized hybrid roof system. The hybrid roof, common in California and Nevada, consists of a panelized plywood system supported on open web steel joists. 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 is on 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. **Required diaphragm chord for north-south seismic forces**
4. **Design of collector along line 3 between lines B and C**

5. **Diaphragm deflection**

6. **Design shear force for north-south panel on line 1**

7. **Design wall-roof anchorage for north-south loads**

8. **Design wall-roof anchorage for east-west loads**

9. **Design typical east-west loaded subdiaphragm**

10. **Design continuity ties for east-west direction**

Given Information

Roof

dead load = 14 psf

live load (roof) = 20 psf (reducible) (T 1607.1)

Walls

thickness = 7.25 inches

height = 23 feet

normal weight concrete = 150 pcf

f'_c = 4000 psi

A615, Grade 60 rebar (F_y = 60 ksi)

Roof sheathing

Structural-I sheathing (wood structural panel)

Roof structure

Pre-engineered/pre-manufactured open-web steel joists and joist-girders with full-width nailers. All wood is Douglas-fir.

Seismic force-resisting system

Bearing wall system consisting of intermediate precast shear walls.

Seismic and site data

Mapped spectral accelerations for the site

S_s = 1.5 (Short period)

S_1 = 0.6 (1-second period)

Occupancy Category = II

Site Class = D

Wind

Assumed not to govern

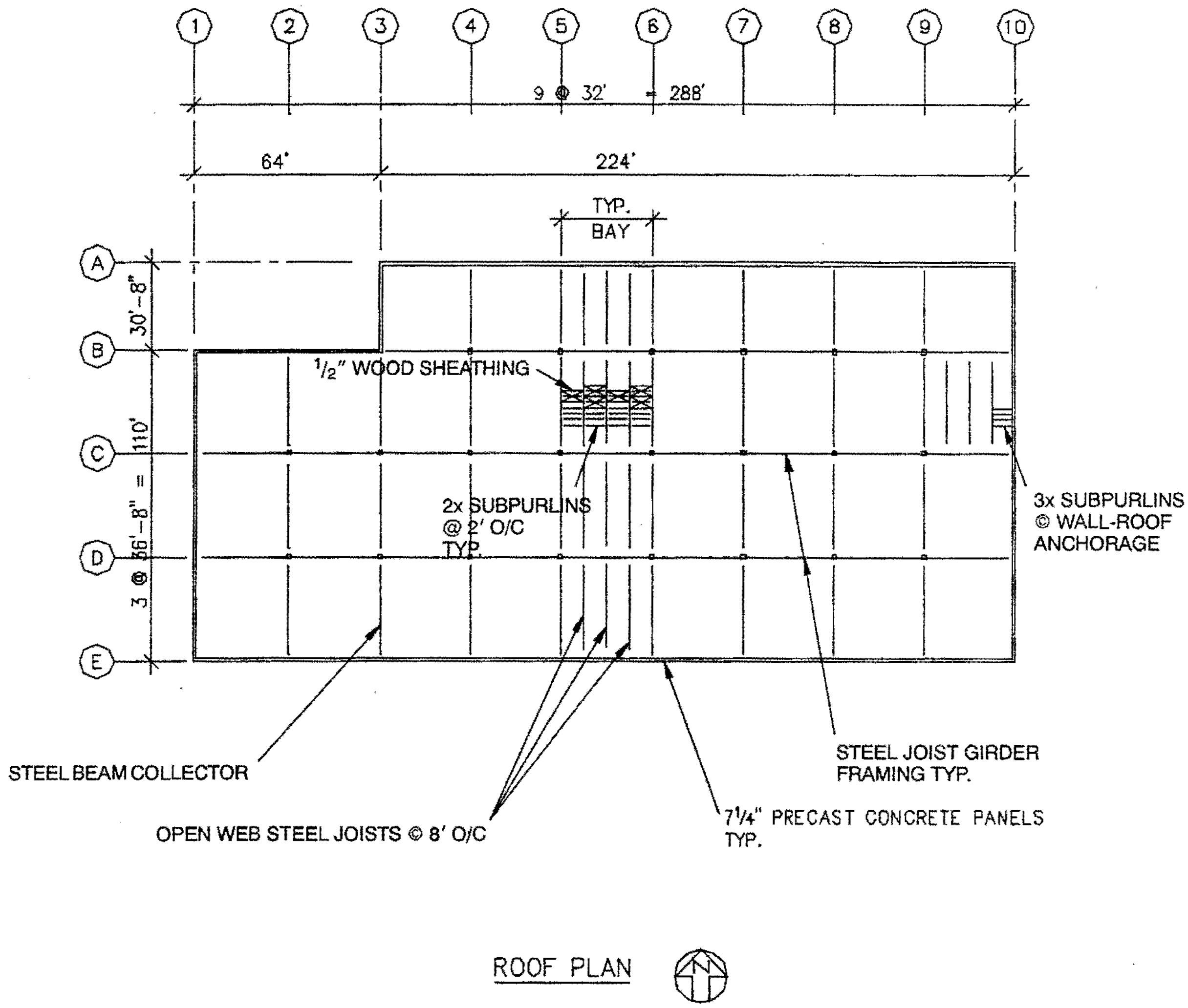

Figure 5-2. Roof framing plan of tilt-up building

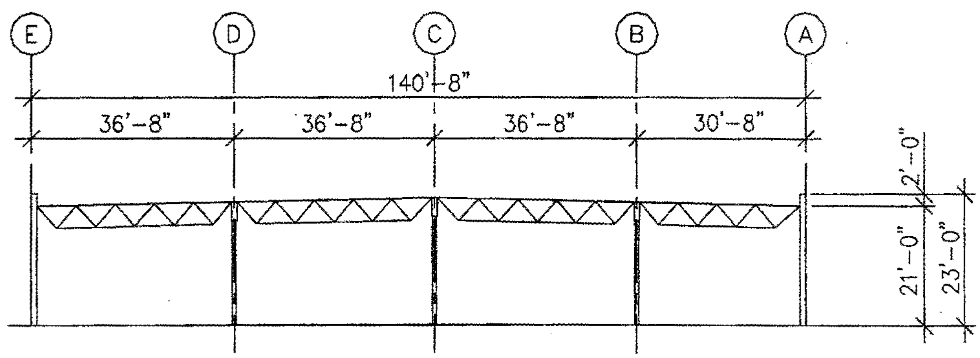

Figure 5-3. Typical cross-section

Calculations and Discussion | **Code Reference**

1. Design base shear coefficient

1a. Design spectral response accelerations S_{DS} and S_{D1}

The site coefficients F_a, F_v are used to modify the mapped spectral accelerations. Using the given spectral accelerations $S_s = 1.5$, $S_1 = 0.6$, and site class D, the following site coefficients are determined from IBC Tables 1613.5.3

$F_a = 1$ (short period)

$F_v = 1.5$ (1-second period)

Using these site coefficients, the site-adjusted spectral accelerations are determined

$S_{MS} = F_a S_s = 1.0(1.5) = 1.5$ (short period) (Eq 16-37)

$S_{M1} = F_v S_1 = 1.5(0.6) = 0.9$ (1-second period) (Eq 16-38)

The design spectral response accelerations are obtained as follows

$S_{DS} = ⅔{*}S_{MS} = 1.0$ (short period) (Eq 16-39)

$S_{D1} = ⅔{*}S_{M1} = 0.6$ (1-second period) (Eq 16-40)

Using the design spectral response accelerations and the occupancy category, the next step is to determine the appropriate seismic design category (SDC) from IBC Tables 1613.5.6. Both the short period and 1-second period design categories are level D, thus SDC D governs.

short period category = D (T 1613.5.6(1))

1-second period category = D (T 1613.5.6(2))

governing SDC = D

The appropriate analysis procedure is obtained using ASCE/SEI 7-05 §12.6 in conjunction with Table 12.6-1. Use the equivalent lateral-force procedure of §12.8 to determine the seismic base shear coefficient. For this concrete shear wall building, the approximate fundamental period T is obtained using ASCE/SEI 7-05 Equation 12.8-7 (or 12.8-9) with a $C_T = 0.020$ and an average roof height $h_n = 21$ feet.

$T_a = C_T h_n^{3/4} = 0.2$ seconds Eq 12.8-7

If this example involved a regular structure five stories or fewer in height, having a period T less than 0.5 seconds, the design spectral response acceleration, S_{DS}, need not exceed the value calculated using a value of 1.5 for S_s (§12.8.1.3). The design spectral response accelerations and SDC remain as originally calculated.

$S_{DS\,design} = 1.0$ (short period)

$S_{D1\,design} = 0.6$ (1-second period)

But this structure has a re-entrant corner irregularity per ASCE/SEI 7-05 Table 12.3-1, item 2.

1b. Base shear using the equivalent lateral-force procedure

ASCE/SEI 7-05 §12.8.1 defines the seismic base shear as

$$V = C_s W \qquad \text{where } C_s = \frac{S_{DS}}{R/I} \qquad \text{Eq 12.8-1 \& 12.8-2}$$

Because these tilt-up concrete walls will be considered load-bearing walls and intermediate precast shear walls

$R = 4$ Response modification factor T 12.2-1

In addition, the importance factor is defined by Occupancy Category II:

$I = 1.0$ T 11.5-1

Therefore

$$C_s = \frac{S_{DS}}{R/I} = 1.0/(4) = 0.25 \qquad \text{Eq 12.8-2}$$

Checking the maximum limit for C_s where $T \le T_L$

$$C_{s\,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = 0.75 > 0.25 \;\ldots\; o.k. \qquad \text{Eq 12.8-3}$$

Checking the minimum allowed value for C_s, Equations 12.8-5 and 12.8-6 are applicable. In this example, S_1 is equal to $0.6g$, therefore Equation 12.8-6 is valid to check the minimum allowed C_s.

$$C_{s\,min} = 0.01 < 0.25 \;\ldots\; o.k. \qquad \text{Eq 12.8-5}$$

$$C_{s\,min} = \frac{0.5S_1}{R/I} = 0.075 < 0.25 \;\ldots\; o.k. \qquad \text{Eq 12.8-6}$$

The calculated value for $C_s = 0.25$ is between the maximum and minimum allowed values.

$$C_{s\,governs} = 0.25$$

Substituting into Equation 12.8-1

$$V = C_s W = 0.25W$$

1c. Base shear using the simplified alternative structural design criteria

Instead of the lengthy seismic analysis shown above, simple buildings that meet the twelve limitations of §12.14.1.1 may use the simplified analysis procedure in §12.14. Using §12.1.1, the simplified analysis procedure of §12.14 is allowed as an

alternative method for designing this example's structure to resist seismic forces. This example will not follow the simplified alternative method.

2. Design the roof diaphragm

2a. Roof diaphragm shear coefficient

The roof diaphragm must be designed to resist seismic forces in each direction. The following formula is used to determine the total seismic force F_{px} on the diaphragm at a given level of a building.

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

Base shear for this building is $V = 0.25w$. Because it is a one-story building, Equation 12.10-1 becomes the following

$$F_{px} = 0.25w_{px}$$

F_{px} need not exceed

$$0.4S_{DS}Iw_{px} = 0.4(1.00)(1.0)w_{px} = 0.4w_{px} \qquad \S 12.10.1.1$$

but shall not be less than

$$0.2S_{DS}Iw_{px} = 0.2(1.00)(1.0)w_{px} = 0.2w_{px} \qquad \S 12.10.1.1$$

Based on the criteria given in §12.10.1.1, $F_{px} = 0.25w_{px}$

Therefore, for diaphragm design use $F_p = 0.25w_p$

2b. Roof diaphragm shears

The wood structural panel roof system is permitted to be idealized as a flexible diaphragm per §12.3.1.1 and IBC 1613.6.1. Seismic forces for the roof are computed from the tributary weight of the roof and the walls oriented perpendicular to the direction of the seismic forces. Uniform loading will be computed in each direction.

East-west direction

Because the the panelized wood roof diaphragm in this building is idealized as flexible, lines A, B, and E are considered lines of resistance for the east-west seismic

forces. A collector is needed along line B to drag the tributary east-west diaphragm forces into the shear wall on line B. The loading and shear diagrams are shown below

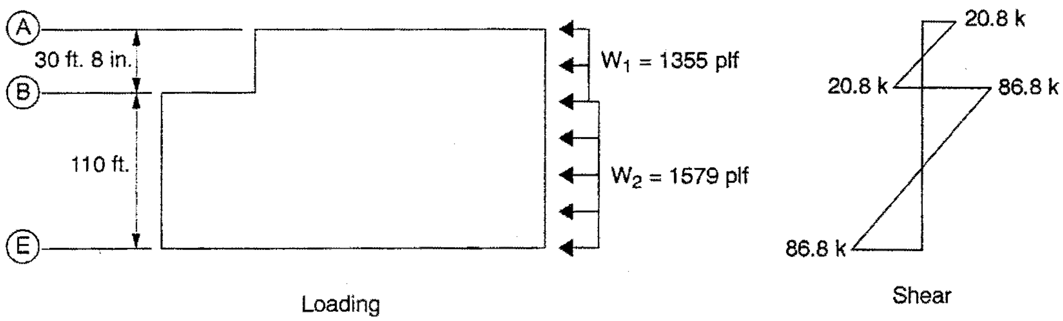

Figure 5-4. Seismic loading and shear diagrams for east-west diaphragm

The uniform loads W_1 and W_2 in the east-west direction are computed using the diaphragm lengths and wall heights.

Roof dead load = 14 psf

$$\text{Wall dead load} = \frac{7.25}{12}150 = 90.6\ \text{psf}$$

Roof height $= 21$ feet average

Parapet height $= 2$ feet average

$$W_1 = 0.25(14\ \text{psf})(224\ \text{ft}) + \left[0.25(90.6\ \text{psf})(23)\left(\frac{23}{2}\right)\left(\frac{1}{21}\right)\right]2 = 1355\ \text{plf}$$

$$W_2 = 0.25(14\ \text{psf})(288\ \text{ft}) + \left[0.25(90.6\ \text{psf})(23)\left(\frac{23}{2}\right)\left(\frac{1}{21}\right)\right]2 = 1579\ \text{plf}$$

In this example, the effect of any wall openings reducing the wall weight has been neglected. This is considered an acceptable simplification because the openings usually occur in the bottom half of the wall. In addition, significant changes in parapet height should also be considered if they occur.

Diaphragm shear at line A and on the north side of line B is

$$\frac{20{,}800\ \text{lb}}{224\ \text{ft}} = 93\ \text{plf}$$

Diaphragm shear at the south side of line B and at line E is

$$\frac{86{,}800\ \text{lb}}{288\ \text{ft}} = 301\ \text{plf}$$

North-south direction

Diaphragm forces for the north-south direction are computed using the same procedure and assumptions as the east-west direction

$$W_3 = 0.25(14)(110) + \left[0.25(90.6)(23)\left(\frac{23}{2}\right)\left(\frac{1}{21}\right)\right]2$$

$$W_3 = 956 \text{ plf}$$

$$W_4 = 0.25(14)(140.67) + \left[0.25(90.6)(23)\left(\frac{23}{2}\right)\left(\frac{1}{21}\right)\right]2$$

$$W_4 = 1{,}063 \text{ plf}$$

Diaphragm shear at line 1 and the west side of line 3 is

$$\frac{30{,}600 \text{ lb}}{110 \text{ ft}} = 278 \text{ plf}$$

Diaphragm shear at the east side of line 3 and at line 10 :

$$\frac{119{,}000 \text{ lb}}{140.67 \text{ ft}} = 846 \text{ plf}$$

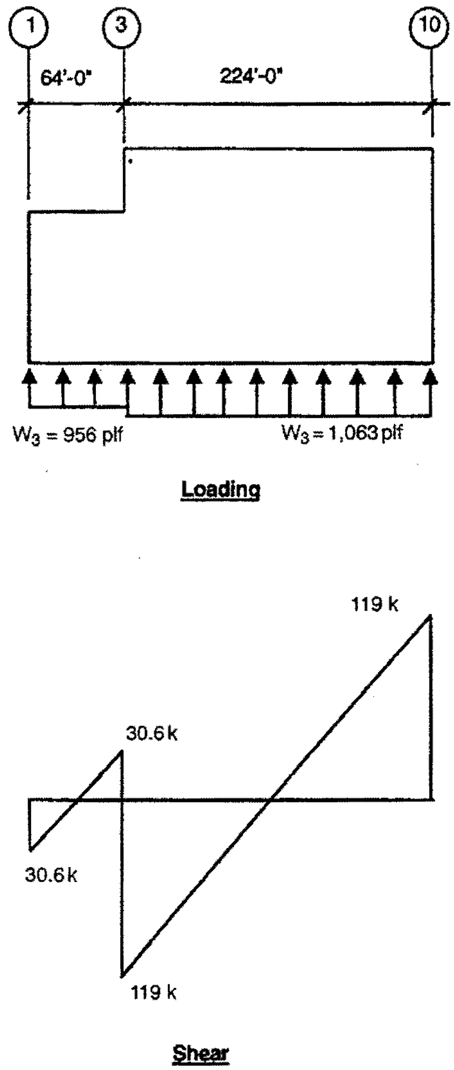

Figure 5-5. Seismic loading and shear diagram for north-south diaphragm

2c. Design of north-south diaphragm

The north-south diaphragm has been selected to illustrate the design of a wood structural panel roof diaphragm. Allowable stress design (ASD) will be used. The basic earthquake loading combinations are given in ASCE/SEI 7-05 §12.4.2.3.

The governing seismic load combination for allowable stress design is (5)

$$(1.0 + 0.14S_{DS})D + H + F + 0.7\rho Q_E \qquad \S 12.4.2.3$$

When designing the structural diaphragm, the vertical loading need not be considered in conjunction with the lateral diaphragm shear stresses. Therefore the dead load $D = 0$ in the load combinations. Additionally, $H = 0$, $F = 0$ and $L = 0$ for this example.

The redundancy factor $\rho = 1.0$ for typical diaphragms per §12.3.4.1. In unique situations where the diaphragm is acting to transfer forces horizontally between offsets, the redundancy factor ρ will conform to §§12.3.4 and 12.10.1.1. In this example, $\rho = 1.0$ for the diaphragm design. Thus, the applicable basic load combination reduces to simply $0.7Q_E$.

Assume the diaphragm is to be constructed with $^{15}/_{32}$-inch Structural-I sheathing (wood structural panel) with all edges supported (blocked). Refer to IBC Table 2306.3.1 for nailing requirements. Sheathing arrangement (shown in Figure 5-2) for north-south seismic forces is Case 4. Because open web steel joist purlins with full-width wood nailers are used in this direction, the framing width in the north-south direction is greater than 3-inch nominal. However, in the east-west direction, the framing consists of 2x subpurlins, and strength is therefore limited by the 2-inch nominal width. Required nailing for panel edges for various zones of the roof (for north-south seismic only) is given in Table 5-1. Minimum intermediate (field) nailing is 10d @ 12 inches and 10d nails require 1½-inch member penetration. A similar calculation (not shown) must be done for east-west seismic forces.

Table 5-1. Diaphragm nailing capacities

Zone	Boundary and North-South Edge Nailing[1] (in)	East-West Edge Nailing[2] (in)	ASD Allowable Shear (plf)
A	10d @ 2½	4	640
B	10d @ 4	6	425
C	10d @ 6	6	320

Notes:

1. The north-south running sheet edges are the "continuous panel edges parallel to load" mentioned in IBC 2306.3.1.
2. The east-west sheet edges are the "other panel edges" in IBC 2306.3.1. Note that the nailing for east-west running diaphragm boundaries is per the tighter boundary spacing.

The diaphragm boundaries at lines 3 and 10 have a shear demand of $v = 846$ plf (see Part 2a). Converting to allowable stress design, $v_{ASD} = 0.7(846) = 592$ plf, which is less than nailing zone A's allowable stress of 640 plf.

At some location, nailing zone B (425 plf) will become acceptable as the diaphragm shears reduce farther from the diaphragm boundary. The demarcation between nailing zones A and B may be located as follows using allowable stress design:

$$\text{Shear demand (ASD)} \quad = \text{shear capacity (ASD)}$$

$$0.7[119{,}000 \text{ lb} - (1063 \text{ plf})x] = 425 \text{ plf}(140.67 \text{ ft})$$

where

x = the demarcation distance from the diaphragm boundary.

Solving for x obtains

$$x = 31.6 \text{ ft}$$

Because a panelized wood roof system typically consists of 8-foot-wide panel modules, the demarcation is increased to the next 8-foot increment or to $x = 32$ feet.

A similar process is undertaken to determine the demarcation between zones B and C. In this situation, $x = 51.5$ ft and the demarcation is increased to 56 feet from the diaphragm boundary. The resulting diaphragm shears at these demarcation boundaries are as follows:

Table 5-2. Diaphragm nailing zone shear checks between lines 3 and 10

Nailing Zone	Distance from boundary	Maximum Shear	ASD Shear	Allowable Shear Capacity
A	0 feet	$v_{max} = 846$ plf	$v_{ASD} = 592$ plf	640 plf
B	32 feet	$v_{max} = 604$ plf	$v_{ASD} = 423$ plf	425 plf
C	56 feet	$v_{max} = 423$ plf	$v_{ASD} = 296$ plf	320 plf

The resulting nailing zones for the north-south loading are shown in Figure 5-6.

These demarcation calculations assume the full depth of the diaphragm is available for shear capacity. However, typical warehouse construction contains skylights and smoke vents that can substantially perforate the structural diaphragm. In these situations, the designer must account for these diaphragm interruptions resulting in larger shear stresses.

Comment: Plywood and other structural wood panels are common diaphragm materials in the west and parts of the south. Other parts of the nation commonly use metal deck for diaphragms in conjunction with steel roof framing. Metal deck diaphragms are approached in the same manner with a similar diaphragm table assigning various deck gauges and attachments to specific diaphragms zones depending on the shear demands.

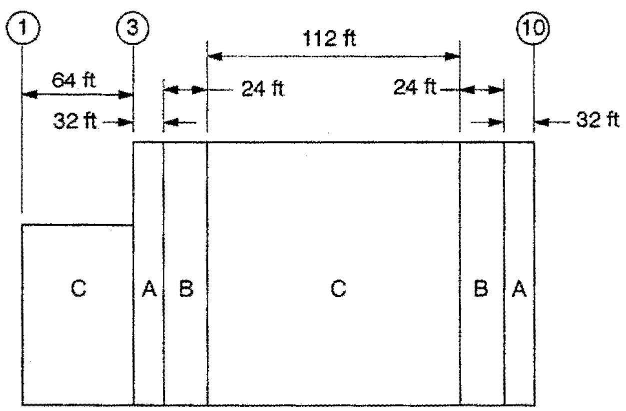

Figure 5-6. Nailing zones for north-south diaphragm

This wood diaphragm resisting seismic forces must have its aspect ratio checked against the limitations in IBC Table 2305.2.3. For blocked diaphragms of wood structural panels the maximum aspect ratio is $L/W = 4{:}1$.

For this example, $L/W = 224/140.67 = 1.6 < 4$ . . . *o.k.*

Comment: Aspect ratio limitations for metal deck diaphragms are found under the specific deck manufacturer's ICC-ES Evaluation Report. Within these reports, a table titled "Diaphragm Flexibility Limitation" provides guidance on limiting diaphragm flexibility in conjunction with diaphragm aspect ratios.

Because there is a re-entrant corner at the intersection of lines B and 3, a check for Type 2 horizontal structural irregularity must be made. Requirements for horizontal structural irregularities are given in ASCE/SEI 7-05 Table 12.3-1.

East-west direction check

$$0.15 \times 288 \text{ ft} = 43.2 \text{ ft} < 64 \text{ ft}$$

North-south direction check

$$0.15 \times (110.0 + 30.67) = 21.1 \text{ ft} < 30.67 \text{ ft}$$

Because both projections are greater than 15 percent of the plan dimension in the direction considered and the structure is SDC D or higher, a Type 2 horizontal structural irregularity exists. The requirements of ASCE/SEI 7-05 §12.3.3.4 apply, resulting in a 25-percent increase in seismic forces for connections of diaphragms to the vertical elements, and connections of diaphragms to collectors.

This 25-percent force increase is on ASCE/SEI 7-05 Equation 12.8-1, which results in diaphragm forces via Equation 12.10-1. Using the information obtained from Part 2a, the diaphragm connection forces are increased to $F_{px} = 1.25\ (0.25w_{px}) = 0.313w_{px}$. This still falls between the upper bound $0.4w_{px}$ and lower bound $0.2w_{px}$ found in Part 2a, thus $F_{px} = 0.313w_{px}$, which is a direct 25-percent increase to diaphragm connection forces.

This force increase applies to situations involving ledger and/or wood nailer bolting to shear walls, wood nailer bolting to collectors, and the row of diaphragm nailing that transfers the diaphragm shears directly to walls and collectors. The design of these elements is not a part of this example. This irregularity also affects the collector design, as will be shown in Part 4. The 25-percent force increase is not applied to out-of-plane wall anchorage forces connected to the diaphragms.

3. Required diaphragm chord for north-south seismic forces

Chords are required to carry the tension forces developed by the moments in the diaphragm. In this building, the chords are continuous reinforcement located in the wall panels at or near the roof level as shown in Figure 5-7. In this example, the chord reinforcement is below the roof ledger to facilitate the chord splice connection at the panel joint.

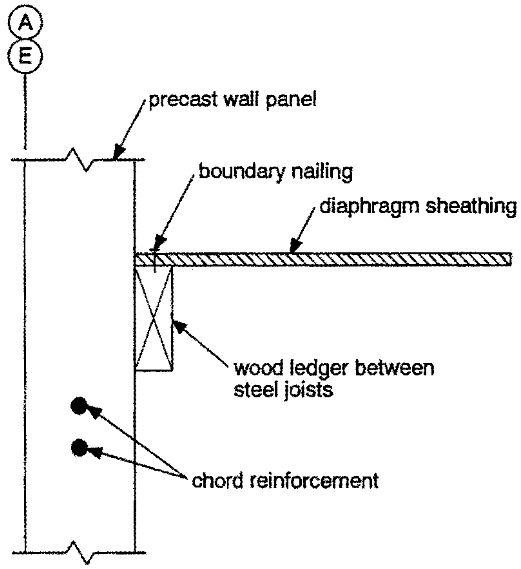

Figure 5-7. Diaphragm chord

The north-south diaphragm spans between lines 1 and 3 and lines 3 and 10. The diaphragm is idealized as 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.

$w = 1063$ plf from Part 2

$$M = \frac{w\ell^2}{8} = \frac{1.063\text{ klf}(224)^2}{8} = 6667\text{ kip-ft}$$

The chord forces are computed from

$$T_u = \frac{6667\text{ k-ft}}{140.67\text{ ft}} = 47.4\text{ kips}$$

The chord will be designed using strength design with ASTM A706 Grade 60 reinforcement. A706 reinforcing is used in anticipation that the steel will be welded at the panel joint splice. (See ACI §3.5.2.) The load factor is 1.0 for seismic forces. (ASCE/SEI 7-05 §2.3.2.)

$$A_s = \frac{T_u}{\phi f_y} = \frac{47.4\text{ k}}{0.9(60\text{ ksi})} = 0.877\text{ in}^2$$

$\therefore$ Use minimum two #6 bars, $A_s = 0.88\text{ in}^2 > 0.877 \ldots$ o.k.

Comment: The chord shown above consists of two #6 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. The welded reinforcing splice connection must develop at least 125 percent f_y per ACI 318 §12.14.3.4. Alternately, chords can also be combined with the ledger when steel channels or angles are used, and good quality splices can be easier to make.

4. Design collector along line 3 between lines B and C

The collector and shear wall ledger along line 3 carry one-half of the north-south 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.

4a. Determine seismic forces on the collector

From the diaphragm shear diagram for north-south seismic forces (Figure 5-5), the maximum collector load along line 3 is

$$R = 30.6\text{ k} + \left(\frac{110.0\text{ ft}}{140.67\text{ ft}}\right)119\text{ k} = 124\text{ kips tension or compression}$$

The uniform axial load that accumulates in the collector can be approximated as the total collected load on line 3 divided by the length of the collector (110 ft) in this direction.

$$q = \frac{R}{L} = \frac{124{,}000\text{ lb}}{110\text{ ft}} = 1127\text{ plf}$$

4b. Determine the collector force in steel beam between lines B and C

Assume the collector, a W18 × 50 with wood nailer, is adequate to support dead and live loads. ASTM A992, $F_y = 50$ ksi. Calculate the seismic force at mid-span. Tributary length for collecting axial forces is

$$\ell = 110.00\text{ ft} - \frac{36.67\text{ ft}}{2} = 91.67\text{ ft}$$

$$P = q\ell = 1{,}127\text{ klf }(91.67\text{ ft}) = 103\text{ kips tension or compression in beam}$$

4c. Check steel beam collector for load combinations as required by §12.4.2.3

The governing seismic load combination for LRFD under ASCE/SEI 7-05 §12.4.2.3 is

$$(5)\quad (1.2 + 0.25\,S_{DS})D + \rho Q_E + L + 0.25$$

For this example, $L = 0$, $S = 0$, and $S_{DS} = 1.0$. Because collectors are considered a part of the diaphragm system, the redundancy factor $\rho = 1.0$ was discussed previously in Part 2c for diaphragms. Thus, the applicable basic load combination for LRFD reduces to the following:

$$(5)\quad 1.4D + Q_E$$

The unfactored gravity loads and moments are as follows:

$$w_D = 8\text{ ft }(14\text{ psf}) + 50\text{ plf} = 162\text{ plf}$$

$$M_D = \frac{162\text{ plf}(36.67\text{ ft})^2}{8} = 27{,}230\text{ lb-ft or }27.2\text{ kip-ft}$$

$$L_r = L_O R_1 R_2 = (20\text{ psf})(0.91)(1.0) = 18.2\text{ psf} \qquad \text{(Eq 16-27)}$$

$$w_{Lr} = 8\text{ ft }(18.2\text{ psf}) = 146\text{ plf}$$

$$M_{Lr} = \frac{146\text{ plf}(36.67\text{ ft})^2}{8} = 24{,}541\text{ lb-ft or }24.5\text{ kip-ft}$$

As shown in Part 2c, this building contains a Type 2 horizontal structural irregularity, and the requirements of ASCE/SEI 7-05 §12.3.3.4 apply. This results in a 25-percent increase in seismic forces for collectors and their connections except where designed for load combinations with the overstrength factor of §12.4.3.2. The collector's axial seismic force becomes $Q_E = 1.25 \times 103$ kips = 129 kips.

AISC §H1 contains the equations for combined axial compression and bending. Because the bending is not biaxial, AISC §H1.3 is advantageous to use by checking failure about each axis independently. First, compute the available strengths P_c and M_c for use in the equations. P_c is a function of the collector's unbraced length. In this example, the lateral bracing to the collector's bottom flange is provided at the member's equal third points with use of an angle brace (design not shown) for an unbraced length of $\ell_y = 36.67/3 = 12.22$ ft. The strong axis unbraced length is simply the span $\ell_x = 36.67$ ft.

$$\frac{k\ell_x}{r_x} = \frac{1.0(36.67)12}{7.38} = 60$$

$$\frac{k\ell_y}{r_y} = \frac{1.0(12.22)12}{1.65} = 89$$

Because failure will be checked separately about each axis per AISC §H1.3, P_c corresponding with each axis will be determined:

X-axis:

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{k\ell_x}{r_x}\right)^2} = \frac{\pi^2(29\times10^3)}{(60)^2} = 79.5\text{ ksi} \qquad \text{AISC Eq E3-4}$$

Because $F_e \geq 0.44\, F_y$, AISC Equation E3-2 is applicable.

$$F_{cr} = [0.658^{\frac{F_y}{F_e}}]F_y = [0.658^{\frac{50}{79.5}}]50 = 38.4\text{ ksi}$$

$$P_{nx} = F_{cr} A_g = 38.4(14.7) = 564\text{ kips} \qquad \text{AISC Eq E4-1}$$

$$P_{cx} = \phi_c P_{nx} = 0.90(564) = 508\text{ kips} \qquad \text{AISC §E1}$$

Y-axis:

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{k\ell_y}{r_y}\right)^2} = \frac{\pi^2(29\times 10^3)}{(89)^2} = 36.1 \text{ ksi}$$

Because $F_e \geq 0.44\ F_y$, AISC Equation E3-2 is applicable.

$$F_{cr} = [0.658^{\frac{F_y}{F_e}}]F_y = [0.658^{\frac{50}{36.1}}]50 = 28.0 \text{ ksi}$$

$$P_{ny} = F_{cr}A_g = 28.0(14.7) = 412 \text{ kips}$$

$$P_{cy} = \phi_c\, P_{ny} = 0.90(412) = 371 \text{ kips}$$

With the top flange fully supported laterally:

$$M_c = \phi_b\, M_n = \phi_b F_y Z_x = 0.90(50 \text{ ksi})(101) = 4545 \text{ in-kips}$$
$$= 379 \text{ ft-kips}$$

Determine factored P_r and M_r using basic load combination (5) 1.4D + Q_E:

$$P_r = Q_E = 129 \text{ kips (includes increase for plan irregularity)}$$

$$M_r = 1.4M_D = 1.4(27.2) = 38.1 \text{ kip-ft}$$

Per AISC §H1.3(a), the in-plane stability check uses AISC Equations H1-1. P_{cx} is the appropriate in-plane bucking strength.

$$\frac{P_r}{P_{cx}} = \frac{129}{508} = 0.25 \geq 0.20.$$

Therefore, AISC Equation H1-1a is applicable for checking combined forces.

$$\frac{P_r}{P_{cx}} + \frac{8}{9}\left(\frac{M_r}{M_{cx}}\right) = 0.25 + \frac{8}{9}\left(\frac{38.1}{379}\right) = 0.34 \leq 1.0 \ldots\ o.k.$$

Per AISC §H1.3(b), the out-of-plane buckling check uses AISC Equation H1-2. $P_{co} = P_{cy}$.

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 = \frac{129}{371} + \left(\frac{38.1}{379}\right)^2 = 0.36 \leq 1.0 \ldots\ o.k.$$

Evaluating the W18 × 50 collector for combined axial tension and bending per AISC §H1.2 is not necessary, because P_c will be less and more critical for compression than for tension.

4d. Check steel beam collector for load combinations with overstrength factor per §12.4.3.2

As required by ASCE/SEI 7-05 §12.10.2.1 the steel beam (W18 × 50) must also be checked for the special load combinations of §12.4.3.2. The relevant strength design equations are

$$(5)\ (1.2 + 0.2\,S_{DS})\,D + \Omega_O\,Q_E + L + 0.25$$

$$(7)\ (0.9 - 0.2\,S_{DS})\,D + \Omega_O\,Q_E + 1.6\,H$$

$\Omega_O\,Q_E$ is an estimate of the maximum force transmitted by the collector elements in the seismic event. The horizontal seismic force Q_E is scaled by the amplification factor Ω_O for estimating E_m. The amplification factor Ω_O may be reduced by subtracting 0.5 for structures with flexible diaphragms; however, Ω_O shall not be reduced below 2.0.

$$\Omega_O = 2.5 - 0.5 = 2.0 \qquad \text{T 12.2-1}$$

Because the dead load component D is detrimental to the analysis, load combination (7) will not govern. Simplifying the remaining load combination for this example we obtain:

$$(5)\ 1.14D + 1.75Q_E\ 1.4D + 2.0Q_E$$

With this special load combination, re-analyze the W18 × 50 steel beam collector for combined axial and bending loads.

As determined earlier in Part 4, $M_D = 27.2$ kip-ft and $Q_E = 103$ kips. Notice that Q_E does not include a 1.25 factor increase for irregular buildings when considering special load combinations with overstrength per §12.3.3.4.

Because collector bending is not biaxial, AISC §H1.3 is advantageous to use by checking failure about each axis independently. Recall from Part 4c:

$$P_{cx} = 508 \text{ kips}$$

$$P_{cy} = 371 \text{ kips}$$

$$M_c = 379 \text{ ft-kips}$$

Evaluating the special load combinations with overstrength:

$$P_r = \Omega_O\,Q_E = 2.0(103 \text{ kips}) = 206 \text{ kips}$$

$$M_r = 1.4\,M_D = 1.4(27.2 \text{ kips}) = 38.1 \text{ kip-ft}$$

Per AISC §H1.3(a), the in-plane stability check uses AISC Equations H1-1. P_{cx} is the appropriate in-plane bucking strength.

$$\frac{P_r}{P_{cx}} = \frac{206}{508} = 0.41 \geq 0.20.$$

Therefore, AISC Equation H1-1a is applicable for checking combined forces.

$$\frac{P_r}{P_{cx}}+\frac{8}{9}\left(\frac{M_r}{M_{cx}}\right)=0.41+\frac{8}{9}\left(\frac{38.1}{379}\right)=0.50\leq 1.0\ldots\ o.k.$$

Per AISC §H1.3(b), the out-of-plane buckling check uses AISC Equation H1-2. $P_{co}=P_{cy}$.

$$\frac{P_r}{P_{co}}+\left(\frac{M_r}{M_{cx}}\right)^2=\frac{206}{371}+\left(\frac{38.1}{379}\right)^2=0.57\leq 1.0\ldots\ o.k.$$

Evaluating the W18 × 50 collector for combined axial tension and bending per AISC §H1.2 is not necessary, because P_c will be less and more critical for compression than for tension.

Thus, W18 × 50 steel beam collector is acceptable.

4e. Collector connection to shear wall

The design of the connection of the steel beam 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 seismic forces from the beam, including the load combinations with overstrength per §12.10.2.1. A plan irregularity can increase the connection forces for the collector and diaphragm by 25 percent when the overstrength factor is not included. As shown in Part 2b, this building has a Type 2 horizontal structural irregularity.

Because there is also a collector along line B, there is similarly an important connection of the girder 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.

5. Diaphragm deflection

Diaphragm deflections are estimated to determine the displacements imposed on attached structural and nonstructural elements, and to evaluate the significance of the P-delta effects. Under IBC §2305.2.2, diaphragm deflections are limited to the amount that will permit the attached elements to maintain structural integrity and to continue supporting their prescribed loads. For structural elements, the intent here is to ensure structural stability by avoiding formation of collapse mechanisms in the vertical support system and avoiding excessive P-delta loading effects. For nonstructural elements, the intent of this section is to prevent failure of connections or self-integrity that could result in a localized falling hazard.

5a. Deflection of north-south diaphragm

An acceptable method of determining the horizontal deflection of a blocked wood structural panel diaphragm under lateral forces is given in AF&PA SDPWS §4.2.2. The following equation is used

$$\delta_{\text{dia}}=\frac{5vL^3}{8EAW}+\frac{0.25vL}{1000\,G_a}+\frac{\Sigma(x\Delta_c)}{2W}$$

AF&PA SDPWS Eq 4.2-1

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 = 846 plf (see Part 2b)

L = 224 ft

E = 29×10^6 psi

A = 2 #6 bars = $2 \times 0.44 = 0.88$ in^2

W = 140.67 ft

G_a = 20.0 k/in Zone A (see part 2b for nailing zones) AF&PA SDPWS T 4.2A

15.0 k/in Zone B

24.0 k/in Zone C

Δ_c = 0 (Assume no slip in steel chord connections)

The flexural deformation portion of the equation $\dfrac{5vL^3}{8EAW}$ assumes a uniformly loaded diaphragm and is computed as follows:

$$\delta_{\substack{\text{diaphragm}\\\text{flexure}}} = \frac{5vL^3}{8EAW} = \frac{5(846\text{ plf})(224\text{ ft})^3}{8(29 \times 10^6\text{ psi})0.88(140.64\text{ ft})} = 1.66\text{ in}$$

The shear deformation portion of the equation $\dfrac{0.25vL}{1000\,G_a}$ is derived from a uniformly loaded diaphragm with uniform shear stiffness. Because our example has various nailing zones, and the apparent shear stiffness G_a varies by nailing zone, we will have to modify this portion of the equation. Using virtual work methods, the shear deformation of a uniformly loaded diaphragm with various shear stiffness zones is

$$\delta_{\substack{\text{diaphragm}\\\text{flexure}}} = \Sigma \frac{0.5 v_{i\,\text{ave}} L_i}{1000\, G_{ai}}$$

where

$v_{i\,\text{ave}}$ = the average diaphragm shear within each shear stiffness zone.

L_i = the length of each stiffness zone measured perpendicular to loading.

G_{ai} = the apparent shear stiffness of each shear stiffness zone being considered.

Working across the diaphragm from grid 3 to 10, the following table is helpful using information from Part 2c:

Table 5-3. Shear deformation of various nailing zones

Zone	v_{left}	v_{right}	$v_{i\,ave}$	L_i	G_a	$\frac{0.5v_{i\,ave}L_i}{1000\,G_{ai}}$
A	846	604	725	32 ft	20	0.58 in
B	604	423	514	24 ft	15	0.41 in
C	423	0	212	56 ft	24	0.25 in
C	0	423	212	56 ft	24	0.25 in
B	423	604	514	24 ft	15	0.41 in
A	604	846	725	32 ft	20	0.58 in
						$\Sigma = 2.48$ in

$$\delta_{\substack{\text{diaphragm}\\\text{shear}}} = 2.48 \text{ in}$$

Because the chord reinforcing bars are directly welded together at their splice, no chord slip is assumed to occur.

$$\delta_{\substack{\text{chord}\\\text{slip}}} = \frac{\Sigma(x\Delta_c)}{2W} = 0.00 \text{ in}$$

$$\delta_{dia} = \delta_{\substack{\text{diaphragm}\\\text{flexure}}} + \delta_{\substack{\text{diaphragm}\\\text{shear}}} + \delta_{\substack{\text{chord}\\\text{slip}}} = 1.66 + 2.48 + 0.00 = 4.14 \text{ in}$$

To compute the maximum expected diaphragm deflection δ_x, Equation 12.8-15 is used

$$\delta_x = \frac{C_d\delta_{xe}}{I}$$

δ_{xe} = 4.14 in (using an elastic analysis under strength forces, δ_{dia})

$C_d = 4$ T 12.2-1

$$\delta_x = \frac{4(4.14)}{1.0} - 16.6 \text{ in}$$

Note: The deflection amplification factor C_d is primarily associated with reversing the effects of applied response modification coefficient R used in determining the base shear $V = 0.25W$ and diaphragm shear coefficient $F_{px} = 0.25w$ (see Parts 1b and 2a).

Instead of using the AF&PA equation, the designer could use IBC §2305.2.2. Although the IBC method is a little more complex, it has the ability to be more accurate if properly applied. Additional information is available in the AF&PA SDPWS commentary and Skaggs, 2004.

5b. Limits on diaphragm deflection

Limits are placed on diaphragm deflection primarily for two reasons. The first reason is to separate the building from adjacent structures and property lines in accordance with §12.12.3. In this situation, d_x is computed for the shear walls and diaphragm and added together to obtain the overall deflection. Because the concrete shear wall drift is insignificant compared with the diaphragm deflection, the shear wall deformation is ignored in this example. In addition, out-of-plane wall deformaton does not need to be included.

The second reason for limiting diaphragm deflection is to maintain structural integrity under design load conditions. Diaphragm deflections are limited by IBC §2305.2.2, ASCE/SEI 7-05 §12.12.2, and AF&PA SDPWS §4.2.1.

> "Permissible deflection shall be that deflection up to which the diaphragm and any attached load distributing or resisting element will maintain its structural integrity under design load conditions, such that the resisting element will continue to support design loads without danger to occupants of the structure."

The language of this section is intentionally ambiguous, with the approach left much to the engineer's own rational judgment. The 1999 SEAOC Blue Book (§C108.2.9) states, "In lowrise concrete or masonry buildings, deflections that can cause secondary failures in structural and nonstructural walls should be considered."

The diaphragm's deflection results in the columns and perpendicular walls rotating about their bases because of the diaphragm's translation at the top. Assuming the columns and walls were modeled with pinned bases during their individual design, this base rotation is permitted to occur even if some unintentional fixity exists.

Unintentional fixity may be the result of standard column base plate anchorage or wall-to-slab anchorage. The assumption of plastic hinges forming at the base is acceptable, provided that these hinges do not result in an unstable condition.

A possible source of instability is the *P*-delta effect resulting from added diaphragm loading due to a horizontal thrust component from the axially loaded gravity columns and walls.

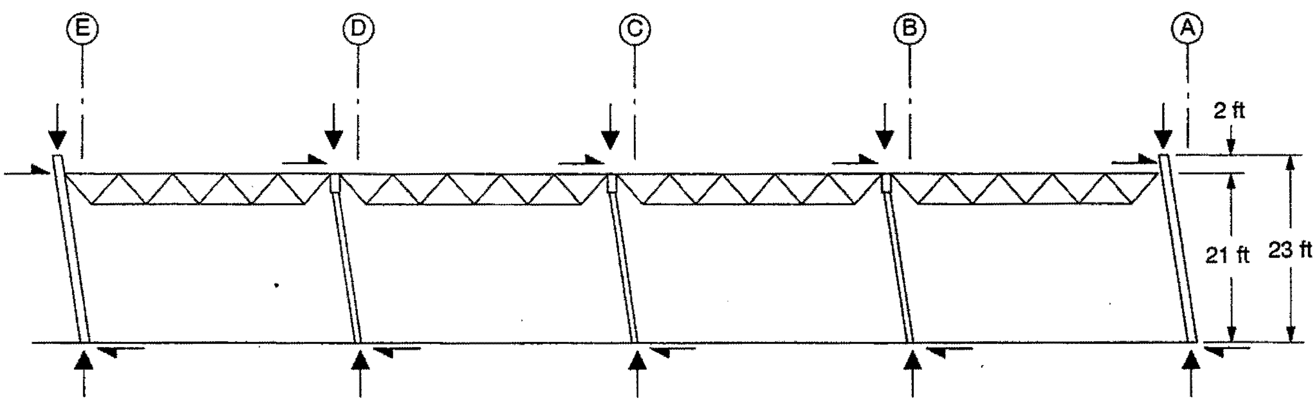

Figure 5-8. Deflected building section

Although it was not originally intended to be used to evaluate diaphragm deformations, §12.8.7 can be used as a guide to investigate stability of the roof system under diaphragm P-delta effects. The stability coefficient θ is defined as

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(Eq 12.8-16)}$$

P_x is the vertical load acting on the translating system and has two components in this example. $P_{x\,roof}$ is the translating roof load, and because load combination (5) of §12.4.2.3 is applicable, no roof live load is considered. $P_{x\,wall}$ is the translating concrete wall dead load and comprises the upper half of the wall plus parapet. Load factors need not exceed 1.0.

$P_{x\,roof}$ = 14 psf (224 ft)(140.67 ft) = 441 kips

$$P_{x\,wall} = \frac{7.25\text{ in}}{12}(150\text{ pcf})\left(\frac{21\text{ ft}}{2} + 2\text{ ft}\right)224\text{ ft}(2\text{ sides}) = 507\text{ kips}$$

P_x $= P_{x\,roof} + P_{x\,wall} = 441 + 507 = 948$ kips

Δ = the average horizontal translation. Because this is a flexible diaphragm with an approximately parabolic deflected shape, the average translation is

$$\frac{2}{3}\delta_x = \frac{2}{3}(16.6) = 11.1\text{ in}$$

V_x = the seismic shear force acting on the translating system under consideration

V_x = 1063 plf (224 feet) = 238 kips

h_{sx} = 21 ft × 12 = 252 in

C_d = 4 T 12.2-1

Therefore:

$$\theta = \frac{948(11.1)}{238(252)4} = 0.04 < 0.10$$

Thus: P-delta effects on story shears, moments, and story drifts are not required to be considered.

Note: The story drift limitations of §12.12.1 are not intended to apply to flexible diaphragm deflections, but instead are intended to apply to the acting lateral-resisting wall or frame systems. These limitations on building drift were primarily developed for the classic flexible frame system with rigid diaphragm. Story drift limits are designed to ensure that the frames and walls do not excessively distort in plane. Similarly, the P-delta limitations of §12.8.7 are also intended to restrict in-plane movements of the vertical seismic resisting system, especially in flexible frames resisting vertical and lateral forces together while subjected to potentially large secondary moments (Tilt-up buildings generally have stiff concrete shear walls that are not impacted by secondary moments from in-plane P-delta effects).

6. Design shear force for north-south 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 five panels on line 1 (Figure 5-1). The panel with the large opening is assumed to be not effective in resisting in-plane forces, and the four panels remaining are assumed to carry the total shear.

From Part 2, the total diaphragm shear on line 1 is 30.6 kips. This force is on a strength basis and was determined by using $F_p = 0.25w_p$ for the diaphragm. The building's main lateral-force-resisting system (shear walls) is designed for a base shear of $V = 0.25W$ also (see Part 1b), thus an adjustment is not necessary to determine in-plane wall forces.

Earthquake loads on the shear walls must be modified by the redundancy factor p. For buildings of Seismic Design Category D, E, or F, this factor is either 1.0 or 1.3 depending on how much redundancy exists within the vertical lateral-force-resisting system as evaluated by §12.3.4.2. Because this building contains a horizontal structural irregularity as described in Part 2c, Table 12.3-3 must be satisfied in order to use $\rho = 1.0$. An example illustrating the computation of the redundancy factor can be found in Volume 1 of this publication's series. For the purposes of this example, it is assumed the redundancy factor $\rho = 1.0$.

Finally, seismic forces caused by panel self-weight must also be included. These are determined using the base shear coefficient 0.25 from Part 1. The panel seismic force is determined as follows:

Panel self-weight

length = 110 ft

$$W_p = 0.15\left(\frac{7.25}{12}\right)(23 \text{ ft})(110 \text{ ft}) = 229 \text{ kips}$$

Seismic force due to panel self-weight

$$V_{\text{panel}} = 0.25W_p = 0.25(229 \text{ k}) = 57.3 \text{ kips}$$

The total horizontal seismic shear force on line 1 shear wall is the horizontal shear force transferred from the diaphragm and the horizontal seismic force due to the panel self-weight, both adjusted for the redundancy factor.

The wall line's horizontal shear force $V = \rho Q_E$ may be computed as

$$V_{\text{line 1}} = \rho Q_E = 1.0[30.6 + 57.3] = 87.9 \text{ kips}$$

Assuming the four solid panels on line 1 have equal relative stiffnesses and the panel with the large opening is not effective, the shear force per panel is

$$\therefore\ V_{\text{panel}} = 87.9/4 = 22.0 \text{ kips per panel}$$

Comment: Distribution of lateral forces along a line of resistance must consider the relative stiffnesses of the individual wall and pier elements. Unlike a masonry building

or a cast-in-place concrete building, a tilt-up building has numerous panel joints that can significantly affect the force distribution within a particular wall line. The stiffnesses are affected by both flexural rigidity and shear rigidity. Flexural rigidity considers the pier element's fixity top and bottom. The shear rigidity is proportional to the wall's length and is proportionally more significant on longer solid walls.

In situations where significantly different stiffnesses occur along a wall line, the chord steel may also be required to act as a strut for distribution of forces. It is important to determine whether chord steel is governed by diaphragm chord forces or by the distribution forces.

7. Design wall-roof anchorage for north-south loads

From a historical perspective, the most critical element in tilt-up engineered buildings is the wall anchorage. Prior to the 1971 San Fernando, California earthquake, engineers in the west typically provided no positive direct tie anchoring the perimeter concrete wall panels to the supporting wood roof structure. Instead, the roof plywood sheathing was simply nailed to a wood ledger that was bolted to the inside face of the wall panels. The roof's glue-laminated beams (glulams) were supported on top of concrete pilasters and had tie connections with minimal capacity. This indirect tie arrangement relied on the wood ledger in cross-grain bending, a very weak material property of wood.

In the 1971 San Fernando earthquake, tilt-up buildings performed poorly. Many wood ledgers split in half due to cross-grain bending loads, and plywood edge nailing pulled through plywood panel edges as the result of tension loads. Partial roof collapses and wall collapses were common in the areas of strong ground motion.

Beginning with the 1973 UBC, cross-grain bending in wood was expressly prohibited and specific wall anchorage requirements were established. Over the years since then, the wall anchorage design forces have increased in response to continuing poor performance of wall anchorage during earthquakes and additional information learned from instrumented tilt-up buildings.

The current wall anchorage code requirements are a result of the 1994 Northridge earthquake. The unexpected wall anchorage damage to newer buildings was primarily attributed to inadequate connection overstrength for the roof accelerations. Research has shown that roof top accelerations may be three to four times the ground acceleration. ASCE/SEI 7-05 §§12.11.2.1 and 12.11.2.2 govern wall anchorage design for most of the tilt-up buildings in seismically active areas (Seismic Design Category C and higher for structural walls). The wall tie force of $F_p = 0.8S_{DS}IW_p$ for flexible diaphragms is double the normal wall design force in §12.11.2 and three to four times the typical tilt-up building base shear to account for the expected roof top amplification associated with flexible diaphragms.

The requirements of §13.4.2 associated with anchorage of nonstructural concrete components do not apply because all bearing walls and shear walls are classified as structural walls under §11.2. In addition, all non-structural walls supported by flexible diaphragms are also anchored per §12.11.2 per Table 13.5-1 footnote b. The design forces associated with the concrete and masonry wall anchorage at structural walls have already been factored up to maximum expected levels in comparison with material overstrengths.

7a. Forces on wall anchorage ties

In this example, the structural concrete wall anchorage forces to the flexible diaphragm are governed by Equation 12.11-1 with $S_{DS} = 1.0$ and $I = 1.0$

$$W_p = 90.6 \text{ psf}$$

$$F_p = 0.8S_{DS}IW_p = 0.80W_p \qquad \text{Eq 12.11-1}$$

Using statics to sum moments about the wall's base, the following calculation includes the cantilever effects of the parapet in determining the wall anchorage force.

$$W_p = 90.6 \text{ psf}(23 \text{ ft})\left(\frac{23 \text{ ft}}{2}\right)\frac{1}{21 \text{ ft}} = 1141 \text{ plf}$$

Solving for the uniform force per foot (q) at the roof level

$$F_p = 0.8W_p = 0.8(1141) = 913 \text{ plf}$$

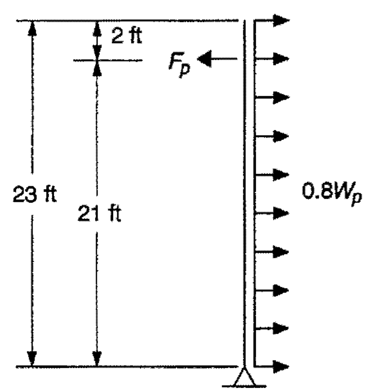

Figure 5-9. Loading diagram for wall-roof anchorage design

Check minimum wall-roof anchorage force per §12.11.2 and IBC §1604.8.2

913 plf > 280 plf . . . *o.k.* §1620.1.7

913 plf > $400S_{DS}I$ . . . *o.k.*

$F_p = 913 \text{ plf} \times 8 \text{ ft} = 7304 \text{ lb}$

Comment: When tie spacing exceeds 4 feet, §12.11.2 and IBC §1604.8.2 require that structural walls be designed to resist bending between anchors.

7b. Check concrete anchorage of typical wall-roof tie

Concrete anchorage design is in accordance with Appendix D of ACI 318, as referenced by IBC §1912.1 and modified by IBC §1908.1.16. The allowable service

loads on embedded bolts in IBC Table 1911.2 are not allowed for seismic design as stated under IBC §1911.1.

The wall-roof anchorage along the north and south walls consists of a steel joist seat welded to an embedded plate with headed weld studs. (See Figure 5-10). Because the embed resists both the wall tie force and the vertical gravity reaction of the steel joist, several loads must be combined.

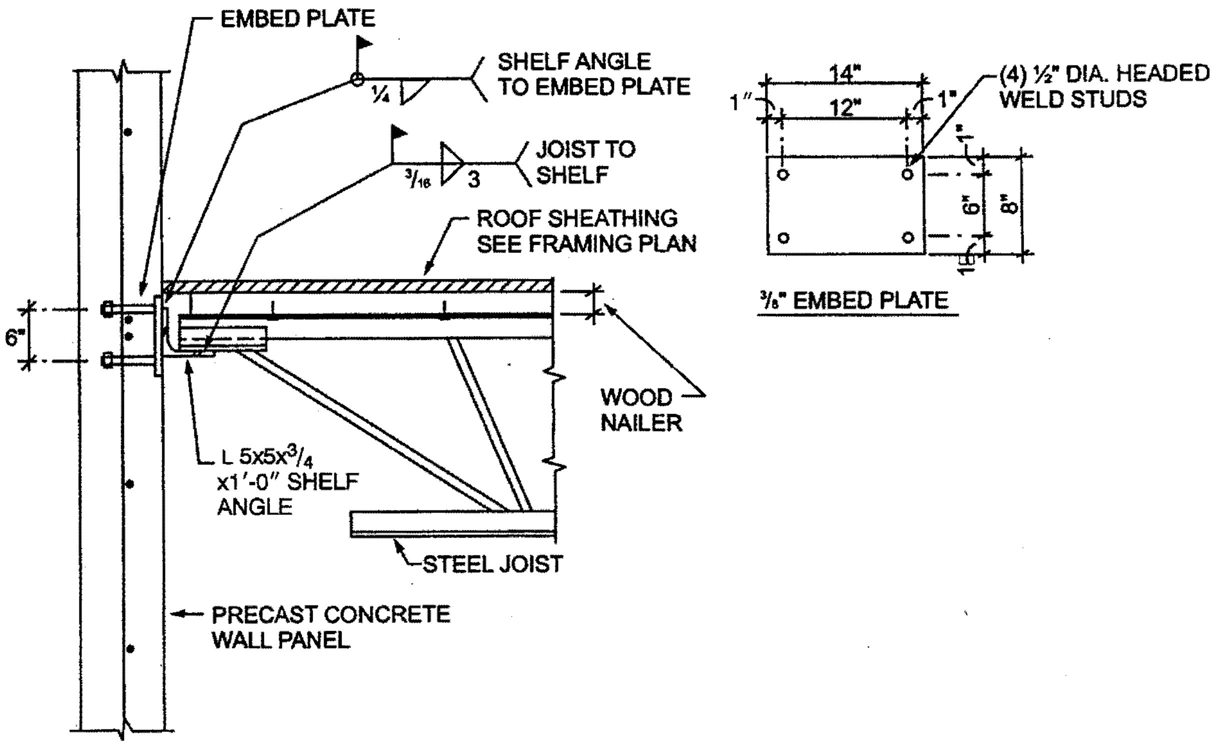

Figure 5-10. Typical steel joist wall-roof tie

The vertical gravity end reaction from the steel joist creates a prying force on the embedded plate's anchors. It will be assumed a force couple at the headed weld studs will resist the eccentric gravity load.

Calculate the joist end reaction R

$$R = (14\text{ psf} + 20\text{ psf})(8\text{ ft})\left(\frac{36.67\text{ ft}}{2}\right) = 2054\text{ lb (dead)} + 2934\text{ lb (live)}$$

Assuming the vertical joist reaction is acting at the edge of the shelf angle, the reaction eccentricity is 5 inches. With the 6-inch vertical spacing between the two pairs of headed weld studs, the following stud forces are determined using the load combinations of IBC §1605.2.1 and ASCE/SEI 7-05 §12.4.2.3:

Load Combination (3) (Eq 16-3)

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$$

Given $S = 0$, $R = 0$, $L = 0$, and wind is not being considered, load combination (3) reduces to

$$1.2D + 1.6L_r$$

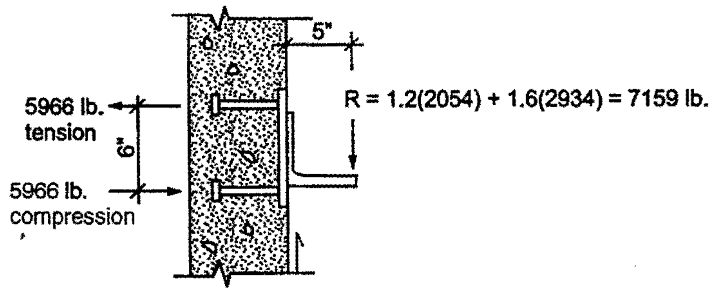

Figure 5-11. Load combination (3) force distribution

Load Combination (5) §12.4.2.3

$$(1.2D + 0.2S_{DS})D + \rho Q_E + L + 0.2S$$

Given $S_{DS} = 1.0$, $L = 0$, $S = 0$, and $\rho = 1.0$, load combination (5) reduces to

$$1.4D + Q_E$$

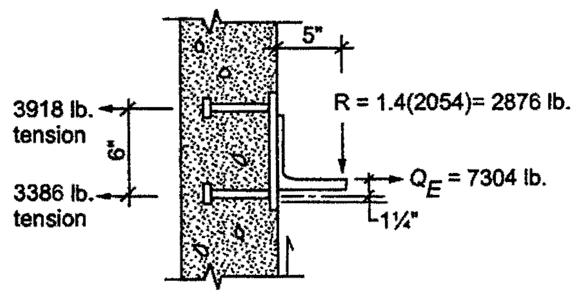

Figure 5-12. Load combination (5) force distribution

Load Combination (7) §12.4.2.3

$$(0.9\text{-}0.2S_{DS})D + \rho Q_E + L + 1.6H$$

Given $S_{DS} = 1.0$, $H = 0$ and $\rho = 1.0$, load combination (7) reduces to

$$0.7D + Q_E$$

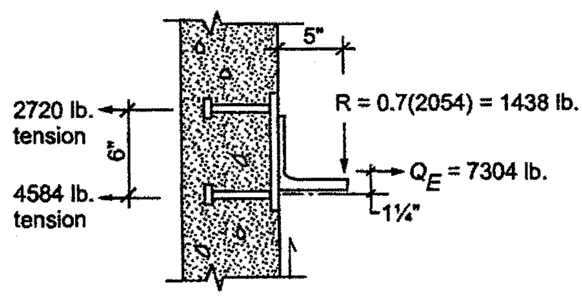

Figure 5-13. Load combination (7) force distribution

Load combination (3) results in only two weld studs loaded in tension, while load combinations (5) and (7) result in all four weld studs tension loaded. Load combination (3) is considered first

Load Combination (3) Analysis

Steel strength in tension N_{sa} — ACI §D.5.1

The nominal steel strength for two ½-inch-diameter ASTM A108 headed weld studs is computed using ACI Equation D-3.

$$N_{sa} = nA_{se}f_{uta}$$

n = 2 bolts in tension

$A_{se} = 0.196 \text{ in}^2$ (½-in-diameter shaft)

f_{uta} = 65,000 psi (AWSD1.1, Type B)

Thus, N_{sa} = 25.5 kips

Concrete breakout strength in tension N_{cbg} — ACI §D.5.2

The two top embedded weld stud anchors are spaced close enough to be considered group action. The ½-inch-diameter studs have an after-weld length of 5 inches, and with their ⁵⁄₁₆-inch-thick head have an effective embedment of h_{ef} = 4.688 inches. The plate's thickness may be added to h_{ef}, resulting in h_{ef} = 4.688 + 0.375 = 5.06 in. Say, h_{ef} = 5 inches.

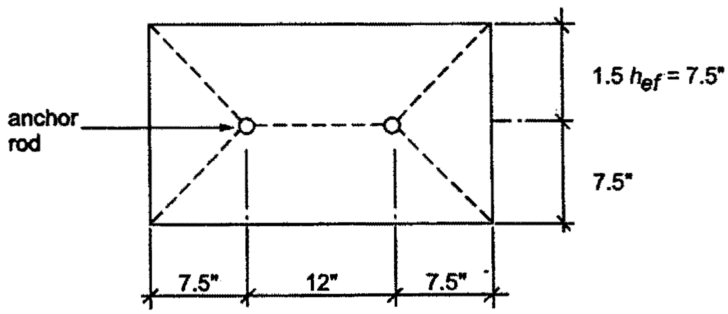

Figure 5-14. Projected failure area A_{Nc} for Load Combination (3)

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}(\psi_{ec,N})(\psi_{ed,N})(\psi_{c,N})(\psi_{cp,N})N_b \qquad \text{ACI Eq D-5}$$

$$A_{Nc} = 2(7.5 \text{ in})\,[2(7.5 \text{ in}) + 12 \text{ in}] = 405 \text{ in}^2 \qquad \text{ACI Eq D-6}$$

Per ACI Section D.5.2.1, A_{Nc} shall not exceed nA_{Nco}

$$nA_{Nco} = 2(3\,h_{ef})^2 = 450 \text{ in}^2 > A_{Nc} \ldots o.k.$$

$$\psi_{ec,N} = 2(225)\,\frac{1}{\left(1 + \dfrac{2e'_N}{3h_{ef}}\right)} \qquad \text{ACI Eq D-9}$$

where e'_N is the eccentricity of the resultant tensile force from the centroid of the bolt group acting in tension. Because there is only one row of bolts acting in tension in this load combination, the bolt group's resultant tension force aligns with the row and thus e'_N is zero.

$$e'_N = 0 \text{ in}$$

$$\psi_{ec,N} = \frac{1}{\left(\dfrac{1+2(0.0)}{3(5.0)}\right)} = 1.0$$

$\psi_{ed,N} = 1.0$ (no adjacent edge effects)

$\psi_{c,N} = 1.25$ (uncracked section due to short parapet)

$\psi_{cp,N} = 1.0$ (cast-in-place anchor)

$$N_b = k_c\sqrt{f'_c}h_{ef}^{1.5} = 24\sqrt{4000}(5)^{1.5} = 17.0 \text{ kips}$$ ACI Eq D-7

$$N_{cbg} = \frac{405}{225}(1.0)(1.0)(1.25)(1.0)(17.0) = 38.3 \text{ kips}$$

Pullout strength in tension — ACI §D.5.3

$$N_{pn} = \psi_{c,p}N_p$$ ACI Eq. D-14

$\psi_{c,P} = 1.4$ (assume uncracked section due to short parapet height)

$N_p = 8A_{brg}f'_c$ (where headed studs or bolts are used) ACI Eq D-15

A_{brg} = (head area) – (shank area) = 0.785 – 0.196 = 0.589 in^2

$N_{pn} = 1.4[8(0.589)(4000 \text{ psi})] = 26.4$ kips

$nN_{pn} = 2(26.4) = 52.8$ kips

Concrete side-face blowout strength in tension — ACI §D.5.4

Because it is assumed that this concrete anchorage is not located near an edge, N_{sb} will not govern the design.

Governing tensile strength

Comparing N_{sa}, N_{cbg}, N_{pn}, and N_{sb}, the governing strength in tension is the steel strength $N_{sa} = 25.5$ kips. Checking ACI Equation D-1 modified by ACI §D.3.3.3

$$0.75\phi N_n = 0.75(0.75)25.5 \text{ kips} = 14.3 \text{ kips} \geq 5.97 \text{ kips} \ldots \textit{o.k.}$$

where $\phi = 0.75$ for anchorage governed by ductile steel element strength per ACI §D.4.4 (Weld studs conforming to ASTM A108 Type B qualify as a ductile steel element).

Steel strength in shear V_{sa} — ACI §D.6.1

The nominal steel strength for four ½-inch-diameter ASTM A108 Type B headed weld studs is computed using ACI Equation D-19.

$$V_{sa} = nA_{se}f_{uta}$$

$n = 4$ bolts

$A_{se} = 0.196 \text{ in}^2$ (½-in-diameter shaft)

$f_{uta} = 65{,}000$ psi

Thus, $V_{sa} = 51.0$ kips

Concrete breakout strength in shear V_{cb} — ACI §D.6.2

As previously mentioned, it is assumed in this example that the embed plate is not located near an edge of the panel. In this situation, V_{cb} will not govern (ACI §RD.6.2.1). Often, the purlin layout is not well coordinated with the concrete panel joint layout and thus conflicts are likely to occur. Where purlin embeds are located in close proximity to panel joints, V_{cb} must be evaluated. This is also true for wall panels with no parapet.

Concrete pryout strength in shear V_{cpg} — ACI §D.6.3

The nominal pryout strength for anchors in shear V_{cpg} is a function of the concrete breakout strength N_{cbg} determined earlier.

$$V_{cpg} = k_{cp}N_{cbg} \qquad \text{ACI Eq D-30}$$

$k_{cp} = 2.0$ for anchor embedments $h_{ef} \geq 2.5$ in

$N_{cbg} = 38.3$ kips

$V_{cpg} = 2(38.3) = 76.6$ kips

Governing shear strength

Comparing V_{sa}, V_{cb}, and V_{cpg} the governing strength in shear is the steel strength $V_{sa} = 51.0$ kips. Checking ACI Equation D-2 modified by ACI §D.3.3.3

$$0.75\phi V_n = 0.75(0.65)5\ 1.0 \text{ kips} = 24.9 \text{ kips} \geq 7.16 \text{ kips} \ldots \textit{o.k.}$$

where $\phi = 0.65$ for shear anchorage governed by ductile steel strength per ACI §D.4.4.

Interaction of tensile and shear forces — ACI §D.7

Interaction equation check required if $V_{ua} < 0.2\phi V_n$. However, in Seismic Design Categories C and higher the design strength is multiplied by 0.75 per ACI §D.3.3.3. Thus in this seismic example, an interaction equation check is required if $V\upsilon\alpha < 0.2(0.75)\phi V_n$.

$$7.16 \text{ kips} > 0.2(0.75)(0.65)\ 51.0 = 4.97 \text{ kips}$$

Thus, interaction equation (D-31) is required to be checked. As stated in ACI §D.3.3.3, the design strength is multiplied by 0.75 in Seismic Design Categories C and higher.

$$\frac{N_{ua}}{0.75\phi N_n}+\frac{V_{ua}}{0.75\phi V_n}\le 1.2 \qquad \text{ACI Eq D-31 and ACI §D.3.3.3}$$

For the four weld stud anchorage configuration

$$\frac{5.97}{075(0.75)(25.5)}+\frac{7.16}{0.65(0.75)(51.0)}=0.42+0.29=0.71<1.2\ldots \textit{o.k.}$$

In summary, the weld studs under the gravity load combination (3) are acceptable.

Load Combinations (5) and (7) Analysis

Steel strength in tension N_{sa} — ACI §D.5.1

The nominal steel strength for four ½-inch-diameter ASTM A108 headed weld studs is computed using ACI Equation D-3.

$N_{sa}=nA_{se}f_{uta}$

$n\ =4$ bolts in tension

$A_{se}=0.196\text{ in}^2$ (½-in-diameter shaft)

$f_{uta}=65{,}000$ psi (AWS D1.1, Type B)

Thus, $N_{sa}=51.0$ kips

Concrete breakout strength in tension N_{cbg} — ACI §D.5.2

The four embedded weld stud anchors are spaced close enough to be considered group action. The ½-inch-diameter studs have an after-weld length of 5 inches, and with their 5⁄16-inch-thick head have an effective embedment of $h_{ef}=4.688$ inches. The plate's thickness may be added to h_{ef} resulting in $h_{ef}=4.688+0.375=5.06$ in. Say $h_{ef}=5$ inches.

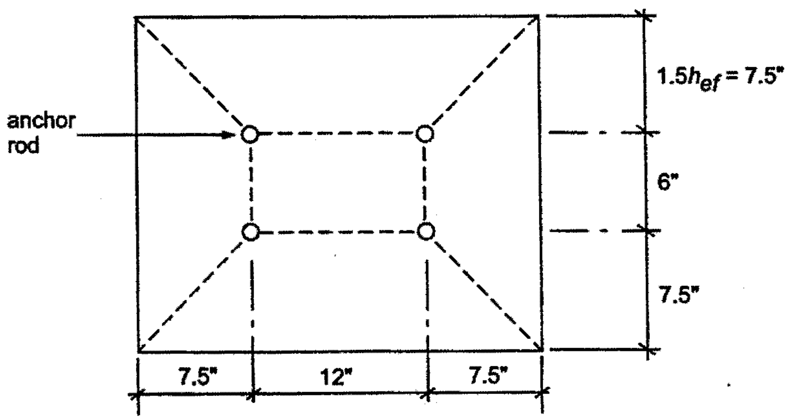

Figure 5-15. Projected failure area A_{Nc} for Load Combinations (5) and (7)

Because load combinations (5) and (7) result in all four stud anchors in tension, a larger concrete breakout projected area is used.

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}(\psi_{ec,N})(\psi_{ed,N})(\psi_{c,N})(\psi_{cp,N})N_b \qquad \text{ACI Eq D-5}$$

$$A_{Nc} = (2(7.5\text{ in}) + 6\text{ in})[2(7.5\text{ in}) + 12\text{ in}] = 567\text{ in}^2$$

$$A_{Nco} = 9h_{ef}^2 = 9(5)^2 = 225\text{ in}^2 \qquad \text{ACI Eq D-6}$$

Per ACI §D.5.2.1, A_{Nc} shall not exceed nA_{Nco}

$$nA_{Nco} = 4(225) = 900\text{ in}^2 > A_{Nc} \ldots o.k.$$

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{2e'_N}{3h_{ef}}\right)} \qquad \text{ACI Eq D-9}$$

where e'_N is the eccentricity of the resultant tensile force from the centroid of the bolt group. Using statics, e'_N is computed for both load combinations

$$e'_N = \frac{6\text{ in}}{2} - \frac{6\text{ in}(3918\text{ lb})}{7304\text{ lb}} = .022\text{ in} \qquad \text{Comb. (5)}$$

$$e'_N = \frac{6\text{ in}}{2} - \frac{6\text{ in}(2720\text{ lb})}{7304\text{ lb}} = 0.77\text{ in} \qquad \text{Comb. (7) [Governs]}$$

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{2(0.77)}{3(5.0)}\right)} = 0.91$$

$$\psi_{ed,N} = 1.0 \text{ (no adjacent edge effects)}$$

$$\psi_{c,N} = 1.25 \text{ (uncracked section due to short parapet)}$$

$$\psi_{cp,N} = 1.0 \text{ (cast-in-place anchor)}$$

$$N_b = k_c\sqrt{f'_c}h_{ef}^{1.5} = 24\sqrt{4000}(5)^{1.5} = 17.0\text{ kips} \qquad \text{ACI Eq D-7}$$

$$N_{cbg} = \frac{567}{225}(0.91)(1.0)(1.25)(1.0)(17.0) = 48.7\text{ kips}$$

Pullout strength in tension — ACI §D.5.3

$$N_{pn} = \psi_{c,p}N_p \qquad \text{ACI Eq D-14}$$

$$\psi_{c,p} = 1.4 \text{ (assume uncracked section due to short parapet height)}$$

$$N_p = 8A_{brg}f'_c \text{ (where headed studs or bolts are used)} \qquad \text{ACI Eq D-15}$$

$$A_{brg} = \text{(head area)} - \text{(shank area)} = 0.785 - 0.196 = 0.589\text{ in}^2$$

$$N_{pn} = 1.4[8(0.589)(4000\text{ psi})] = 26.4\text{ kips}$$

$$nN_{pn} = 4(26.4) = 105.6\text{ kips}$$

Concrete side-face blowout strength in tension — ACI §D.5.4

Because it is assumed that this concrete anchorage is not located near an edge, N_{sb} will not govern the design.

Governing tensile strength

Comparing N_{sa}, N_{cbg}, N_{pn}, and N_{sb}, the governing strength in tension is the concrete breakout $N_{cbg} = 48.7$ kips. Checking ACI Equation D-1 modified by ACI §D.3.3.3

$$0.75\phi N_n = 0.75(0.70)48.7 \text{ kips} = 25.6 \text{ kips} \geq 7.3 \text{ kips} \ldots \textit{o.k.}$$

where $\phi = 0.70$ for anchorage governed by concrete breakout per ACI §D.4.4.

Per ACI §D.3.3.4 as modified by IBC §1908.1.16, structures with SDC C or higher must show that the behavior of the anchorage or attachment is ductile or have a design strength of at least 2.5 times the connection's factored forces under seismic conditions. Because concrete breakout strength (brittle) governs over the steel strength (ductile), we must check ACI Equation D-1 modified by ACI §D.3.3.3 with the 2.5 overstrength factor

$$N_{ua} = 2.5F_p = 2.5(7.3) = 18.3 \text{ kips} \leq 25.6 \text{ kips} \ldots \textit{o.k.}$$

Because the weld stud anchorage forces are not distributed evenly among all four studs, separate checks for the steel strength N_{sa} and pullout strength N_{pn} are recommended for the heaviest loaded pair (the breakout strength equation already accounts for the uneven distribution). In load combination (7), the lower pair is the most heavily loaded.

$$N_{ua} = 2.5F_p = 2.5(4584 \text{ lb}) = 11{,}460 \text{ lb}$$

For two weld studs

$$0.75\phi N_{sa} = 0.75(0.75)51.0/2 = 14.3 \text{ kips} > 11.46 \text{ kips} \ldots \textit{o.k.}$$

$$0.75\phi N_{pn} = 0.75(0.75)2(26.4) = 29.7 \text{ kips} > 11.46 \text{ kips} \ldots \textit{o.k.}$$

Steel strength in shear V_{sa} — ACI §D.6.1

The nominal steel strength for four ½-inch-diameter ASTM A108 Type B headed weld studs is computed using ACI Equation D-19.

$$V_{sa} = nA_{se}f_{uta}$$

n = 4 bolts

$A_{se} = 0.196 \text{ in}^2$ (½-in-diameter shaft)

$f_{uta} = 65{,}000$ psi

Thus, $V_{sa} = 51.0$ kips

Concrete breakout strength in shear V_{cb} — ACI §D.6.2

As previously mentioned, it is assumed in this example that the embed plate is not located near an edge of the panel. In this situation, V_{cb} will not govern (ACI §RD.6.2.1). Often, the purlin layout is not well coordinated with the concrete panel joint layout and thus conflicts are likely to occur. Where purlin embeds are located in close proximity to panel joints, V_{cb} must be evaluated.

Concrete pryout strength in shear V_{cpg} — ACI §D.6.3

The nominal pryout strength for anchors in shear V_{cpg} is a function of the concrete breakout strength N_{cbg} determined earlier.

$$V_{cpg} = k_{cp} N_{cbg} \qquad \text{ACI Eq D-30}$$

$k_{cp} = 2.0$ for anchor embedments $h_{ef} \geq 2.5$ in

$N_{cbg} = 48.7$ kips

$$V_{cpg} = 2(48.7) = 97.4\text{kips}$$

Governing shear strength

Comparing V_{sa}, V_{cb}, and V_{cpg}, the governing strength in shear is the steel strength $V_{sa} = 51.0$ kips. Checking ACI Equation (D-2) modified by ACI §D.3.3.3

$$0.75\phi V_n = 0.75(0.65)\ 51.0\text{ kips} = 24.9\text{ kips} \geq 2.88\text{ kips} \ldots o.k.$$

where $\phi = 0.65$ for shear anchorage governed by ductile steel strength per ACI §D.4.4.

Per ACI §D.3.3.4 as modified by IBC §1908.1.16, structures in SDC C or higher must show that the behavior of the anchorage or attachment is ductile or have a design strength of at least 2.5 times the connection's factored forces. Checking ACI Equation D-2 modified by ACI §D.3.3.3 with this limitation is

$$V_{ua} = 2.5(2.88\text{ kips}) = 7.2\text{ kips} \leq 24.9\text{ kips}\ldots o.k.$$

Interaction of tensile and shear forces — ACI §D.7

Interaction equation check is required if $V_{ua} < 0.2\phi V_n$. However, in Seismic Design Categories C and higher the design strength is multiplied by 0.75 per ACI §D.3.3.3. Thus in this seismic example, an interaction equation check is required if $V_{ua} < 0.2(0.75)\phi V_n$.

$$7.2\text{ kips} > 0.2(0.75)(0.65)51.0 = 4.97\text{ kips}$$

Thus, interaction Equation D-31 is required to be checked. As stated in ACI §D.3.3.3, the design strength is multiplied by 0.75 in Seismic Design Categories C and higher.

$$\frac{N_{ua}}{0.75\phi N_n} + \frac{V_{ua}}{0.75\phi V_n} \leq 1.2 \qquad \text{ACI Eq D-31 and ACI §D.3.3.3}$$

For the four weld stud anchorage configuration:

$$\frac{18.3}{0.75(0.70)(48.7)}+\frac{7.2}{0.75(0.65)(51.0)}=0.72+0.29=1.01<1.2\ldots o.k.$$

As discussed when checking tensile strength previously, the bottom pair of weld studs is more critically loaded under load combination (7) than the group of four weld studs under any load combination. However, $V_{ua} < 0.2(0.75)\phi V_n$ for the weld studs under load combination (7) and thus a separate interaction check is not required in this example.

Check requirements to preclude splitting failure — ACI §D.8

For the cast-in-place headed studs, the following limits are checked:

Minimum center-to-center spacing = 4 diameters = 2 inches < 6 inches
Minimum edges distance = Concrete cover per ACI Section 7.7 . . . *o.k.*

In summary, the four ½-inch-diameter × 5-inch headed weld studs are acceptable.

7c. Check shelf angle at typical wall-roof tie

In this example, the steel joist purlin sits on a steel shelf angle (L5 × 5 × ¾-in × 1 ft). Without additional information, it is assumed the load acts at the tip of the leg. The horizontal leg is subject to bending and seismic tension stresses. Evaluating the array of load combinations for strength design (ASCE §§2.3.2 and 12.4.2.3), combinations (3) and (5) potentially govern.

Simplified load combination (3) — (Eq 16-3)

$$1.2D + 1.6L_r$$

Joist reaction = 1.2(2,054 lb) + 1.6(2,934 lb) = 7159 lb

Moment arm to critical section = leg − k dimension = 5 − 1.25 = 3.75 in

$$M_r = 7{,}159 \text{ lbs } (3.75 \text{ in}) = 26{,}846 \text{ in-lb}$$

$$\text{Plastic section modulues } Z=\frac{12 \text{ in } (0.75 \text{ in})^2}{4}=1.69 \text{ in}^3$$

Per AISC Section F11.1, the nominal flexural strength, M_n, may be checked as follows:

$$M_n = M_p = F_y Z \le 1.6M_y \qquad \text{AISC Eq F11-1}$$

$$M_p = 36{,}000 \text{ ksi } (1.69 \text{ in}^3) = 60{,}840 \text{ in-lb}$$

$$1.6M_y = 1.6F_y S = 1.6(36{,}000)\left(\frac{12 \text{ in}(0.75 \text{ in})^2}{6}\right)=64{,}800 \text{ in-lb}$$

Thus, M_n = 60,840 in-lb

The design flexural strength is checked as follows:

$$\phi_b M_n = 0.90(60{,}840) = 54{,}756 \text{ in-lb} \geq 26{,}846 \text{ in-lb} \ldots \textit{o.k.}$$

Simplified load combination (5) — ASCE 7 §12.4.2.3

$$1.4D + Q_E$$

A combination of gravity forces with horizontal tie forces will be evaluated.

Joist gravity reaction = 1.4(2054 lb) = 2876 lb (dead load)

Moment arm to critical section = leg – k dimension = 5 – 1.25 = 3.75 in

$$M_r = 2876 \text{ lb}(3.75 \text{ in}) = 10{,}785 \text{ in-lb}$$

$$Z = \frac{12 \text{ in}\,(0.75 \text{ in})^2}{4} = 1.69 \text{ in}^3$$

$$M_n = M_p = F_y Z \leq 1.6 M_y \qquad \text{AISC Eq F11-1}$$

$$M_p = 36{,}000 \text{ ksi}(1.69 \text{ in}^3) = 60{,}840 \text{ in-lb}$$

$$1.6M_y = 1.6F_y S = 1.6(36{,}000)\left(\frac{12 \text{ in}(0.75 \text{ in})^2}{6}\right) = 64{,}800 \text{ in-ft}$$

Thus, M_n = 60,840 in-lb

The design flexural strength is checked as follows:

$$\phi_b M_n = 0.90(60{,}840) = 54{,}756 \text{ in-lb} \geq 10{,}785 \text{ in-lb} \ldots \textit{o.k.}$$

Joist horizontal tie force = 7304 lb (from Part 7a)

Per ASCE §12.11.2.2.2, steel elements of the structural wall anchorage system (SDC C and above) are designed for strength forces with an additional 1.4 multiplier. This material-specific multiplier is based on the observed poor performance of steel straps during the Northridge earthquake. It was determined that an inadequate overstrength range existed in various steel elements to accommodate the maximum expected roof top accelerations. This 1.4 force multiplier is applied to all steel elements resisting the wall anchorage forces of §12.11 (SDC C and above) including wall connectors, subdiaphragm strapping, continuous ties and their connections. Concrete reinforcing steel, concrete anchor rods and headed weld studs, wood bolting and nailing are not subject to this force multiplier.

Required tie force P_r = 1.4(7304 lb) = 10,226 lb — ASCE §12.11.2.2.2

Tensile area A_g = 12 in × 0.75 in = 9 in^2

Design tensile strength for checking combined forces per AISC §H1.2:

$$P_c = \phi_t P_n = \phi_t F_y A_g = 0.90(36{,}000)9 = 291{,}600 \text{ lb} \qquad \text{AISC Eq D2-1}$$

$\frac{P_r}{P_c} = \frac{10,226}{291,600} = 0.04 < 0.2$, therefore AISC Equation H1-1b is applicable for checking the combined forces of tension and bending flexure.

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad \text{AISC Eq H1-1b}$$

$$\frac{0.04}{2} + \frac{10,785}{54,576} = 0.22 \le 1.0 \ldots o.k.$$

Therefore, the shelf angle support is adequate.

7d. Check the shelf angle weld to the embed plate

Check the use of a ¼-inch fillet weld all around the shelf angle's perimeter. Per AISC Table J2.4, the ¼-inch fillet weld meets the minimum weld size limitations for the thinner plate joined (⅜-inch embed plate), and per AISC §J2.2b the ¼-inch fillet weld meets the maximum weld size limitations for the ¾-inch edge thickness of the shelf angle.

Similar to the process in Part 7b, the force distribution to the shelf angle's upper and lower welds is shown in Figure 5-16 for the various potentially governing load combinations.

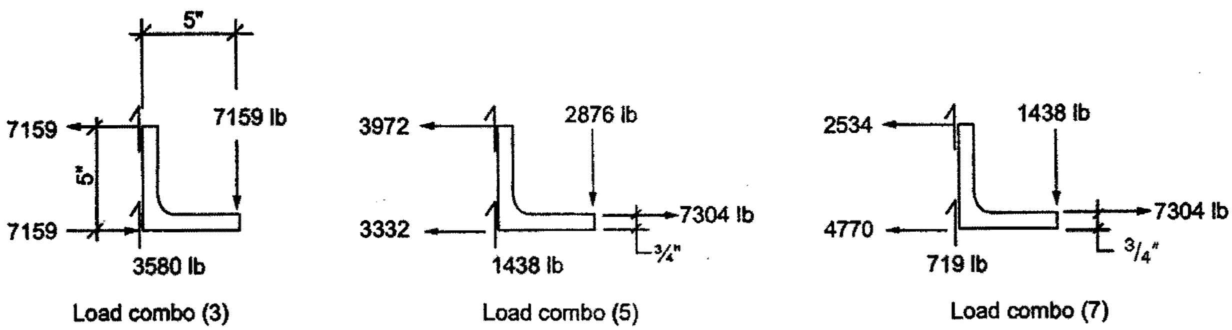

Figure 5-16. Load combination force distributions

Because load combinations (5) and (7) involve seismically induced wall anchorage force to the weld, they are subject to the 1.4 force multiplier of ASCE §12.11 (SDC C and above). Note that the dead load component of the seismic load combinations contains S_{DS} and thus both the vertical and horizontal acting forces are multiplied by 1.4. The following lists the effective results of the vertical and horizontal force vectors:

$$P_r = \sqrt{7159^2 + 3580^2} = 8004 \text{ lb} \qquad \text{Comb. (3)}$$

$$P_r = \sqrt{(1.4 \times 3972)^2 + (1.4 \times 1438)^2} = 5914 \text{ lb} \qquad \text{Comb. (5)}$$

$$P_r = \sqrt{(1.4 \times 4770)^2 + (1.4 \times 719)^2} = 6753 \text{ lb} \qquad \text{Comb. (7)}$$

In this example, the strictly gravity load combination governs at 8004 lb because the gravity load offsets a portion of the seismic anchorage force. Where larger wall anchorage loads occur, often the other load combinations govern.

Checking the strength of the ¼-inch × 12-inch-long fillet weld gives

$$\phi R_n = \phi F_w A_w = 0.75(0.6 \times 70 \text{ ksi})\left(\frac{0.25 \text{ in}}{\sqrt{2}} \times 12 \text{ in}\right) = 66.8 \text{ kips} > 8.0 \text{ kips} \ldots \textit{o.k.}$$

Therefore, the shelf angle weld to the embed plate is adequate.

7e. Check joist seat weld at typical wall-roof tie

The connection of the joist to the embed's shelf angle is through a fillet weld. Given its orientation, the steel shelf angle (L5 × 5 × ¾ in × 1 ft) has a flat run-out distance of 3¾-inches suitable for joist seat bearing.

Per the Steel Joist Institute's Standard Specification (2005), the minimum weld at the joist seat attachments is a ¼ × 2-inch-long fillet or equivalent each side of seat (LH-series joists). Because the seats in these roof systems are typically thinner than ¼ inch it is desirable to specify an equivalent 3⁄16 × 3-inch-long fillet weld. For seats of 3⁄16-inch or thicker material, this fillet weld meets maximum weld size limitations of AISC §J2.2b and the minimum weld size limitations of AISC Table J2.4.

Checking the strength of the two rows of 3⁄16 × 3-inch-long fillet weld is as follows:

$$\phi R_n = \phi F_w A_w = 0.75(0.6 \times 70 \text{ ksi})\left(\frac{0.1875 \text{ in}}{\sqrt{2}} \times 3 \text{ in}\right)2 = 25.1 \text{ kips} \qquad \text{AISC §J2.4}$$

Required tie force $P_r = 1.4(7304 \text{ lb}) = 10{,}226 \text{ lb} < 25{,}100 \text{ lb}$ . . . *o.k.*

Therefore, the joist seat weld to the shelf angle support is adequate.

7f. Design steel joist for typical wall-roof anchorage forces

Whether using a panelized wood sheathed roof or a metal deck roof, steel trusses or joists are the most common roof framing system now in tilt-up buildings. In the West, this trend began in the early 1990s when speculative timber prices disrupted the costs of traditional glulam wood roof systems. Specialty engineers in association with the joist manufacturer typically design the steel joist members. As required by IBC §2206.2, the building's design engineer is responsible for providing axial wall tie and continuity tie forces to the manufacturer along with information stating which load factors if any have already been applied.

In this example, it should be reported to the joist manufacture that the unfactored wall tie axial force (tension and compression) acting on the joist top chord is $Fp = 7304$ lb increased by the steel material overstrength factor 1.4 per §12.11.2.2.2 resulting in $F_p = 7304 \times 1.4 = 10{,}226$ lb. It is necessary to indicate to the joist manufacturer that this tie force is from seismic effects so that the joist's specialty engineer is able to apply the proper load combinations of §12.4.2.3.

Though not shown in this example, the top chord axial effects of wind W must also be considered if it could lead to a governing design of the joist. Because the load combinations of §2.3.1 (strength design) and §2.4.1 (allowable stress design) contain

very different formulas when considering seismic E and wind W, the design engineer cannot simply compare E and W to determine which governs. Currently, the joist industry is largely based on allowable stress design, but it is expected to transition to strength design in the future.

In conditions where axial loads are transferred through the joist seat at either the wall tie or at interior splices, it must be made clear to the manufacturer so that the seat strength will be checked also. There are limits to the amount of load that manufacturers can transfer through these joist seats, so check with the manufacturer's specialty engineer.

In Part 4 of this example, the collector member was a steel wide-flange beam. In some situations, the steel joist can resist lighter collector loads. In these situations, the building's engineer must specify an E_m collector load as well as an E wall tie load. The joist manufacturer's specialty engineer will have to check both the basic load combinations of §12.4.2.3 for$E$ as well as the basic load combinations with overstrength factor of §12.4.3.2 for E_m.

For this example, the following is the type of information to be placed on the drawings for the steel joist manufacturer to properly design his joists for lateral loadings. Note that the wall anchorage force E shown should already include the 1.4 multiplier for steel elements.

Joist Axial Forces E = 10.2 kips (unfactored)

E_m = 0.0 kips (unfactored) *Applicable only at collectors.*

W = 5.0 kips (unfactored)

Forces shall be checked in both tension and compression. Axial force shall be transferred through the joist seats where noted.

7g. Check joist-to-joist splice at the girder lines

Interconnection of elements within the building is required per ASCE/SEI 7-05 §§12.1.3 and 12.1.4. In addition, the joist axial load from the wall anchorage must be distributed across the building's main diaphragm from chord to chord per §12.11.2.2.1 using continuous ties (SDC C and above). Seismic loading in the north-south direction utilizes the steel joists as the continuous ties, and thus the joist axial load must be spliced across the interior girder lines. In Part 7c, the wall anchorage force and thus continuous tie force for the steel joists is P_r = 1.4(7,304 lb) = 10,226 lb.

Per §12.1.3, the minimum interconnection force is $0.133S_{DS}W = 0.133W$, but not less than $0.05W$, where W is the dead load of the smaller portion of the building being connected together. Unlike the wall anchorage force, Win this case includes the diaphragm weight and thus could govern at the interior of buildings. The worst-case value for W is at grid line C with the following result:

$$P_r\,(\text{min}) = 0.133(14\text{ psf})(8\text{ ft})(30.67\text{ ft} + 36.67\text{ ft}) + 0.133(90.6\text{ psf})(8\text{ ft})(23)(23/2)/21$$

$$= 2217\text{ lb}$$

Per §12.1.4, the minimum support connection force is 5 percent of the dead and live load reaction.

$$P_r\,(\text{min}) = 0.05(14\text{ psf} + 20\text{ psf})(8\text{ ft})(36.67\text{ ft}/2) = 249\text{ lb}$$

Thus, the wall anchorage continuous tie force $P_r = 1.4(7304\text{ lb}) = 10{,}226\text{ lb}$ governs.

The splice can be accomplished with a welded cover plate from joist top chord to joist top chord (see Figure 5-17). Check the use of a ¼ × 3-in-wide cover plate with $\frac{3}{16}$-in fillet welds:

Check the design tensile strength per AISC §D2

$$\phi_t P_n = \phi_t F_y A_g = 0.90(36{,}000)(0.25)(3) = 24{,}300\text{ lbs} \qquad \text{AISC Eq D2-1}$$

Required tie force $P_r = 10{,}226$ lbs < 24,300 lbs . . . *o.k.*

Using two lines of $\frac{3}{16}$ × 2-inch-long fillet welds, check the design weld strength per AISC §J2.4

$$\phi R_n = \phi F_w A_w = 0.75(0.6 \times 70\text{ ksi})\left(\frac{0.1875\text{ in}}{\sqrt{2}} \times 2\text{ in}\right)2 = 16.7\text{ kips}$$

Required tie force $P_r = 10{,}226$ lb < 16,700 lb . . . *o.k.*

Therefore, the steel joist splice across the interior girders is adequate.

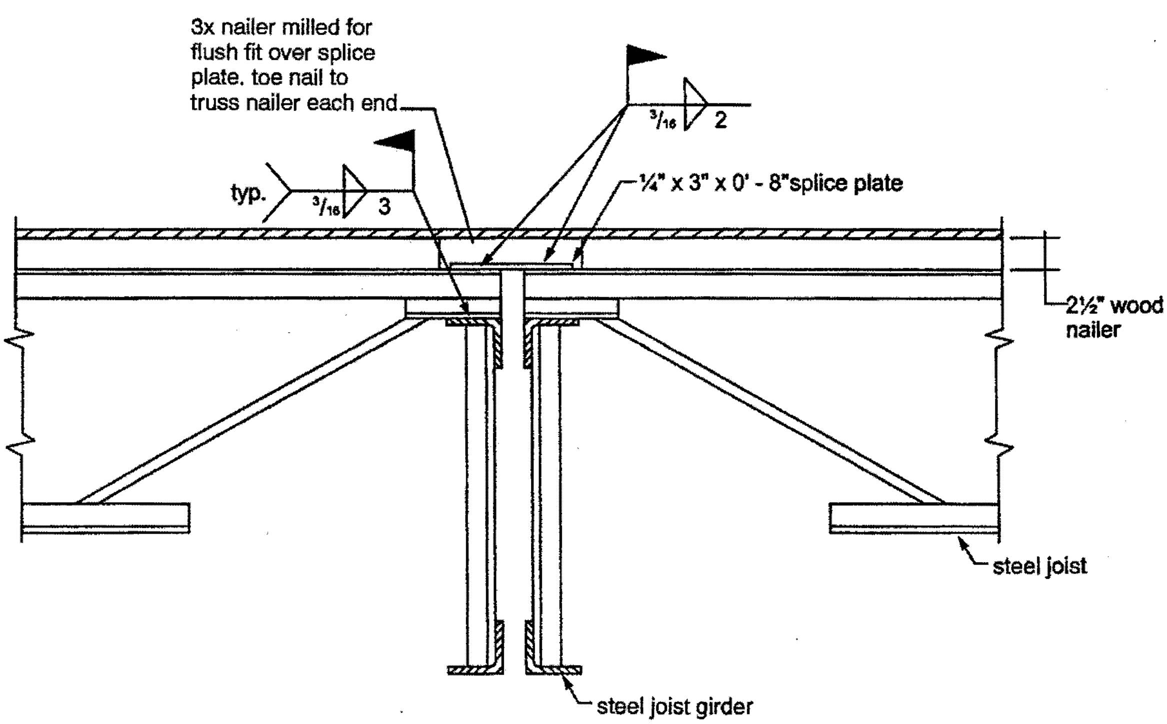

Figure 5-17. Joist-to-joist splice at joist girder

Comment: It is possible to splice the joist axial loads across the interior girders through their joist seats as is done at the wall anchorage joist end. However, this means added joist seat costs and requires the joist girder double-angle top chords to be joined together for this perpendicular force. If this is the design engineer's intent, it must be made clear to the joist manufacturer that the joist seats and joist-girders top chords are to be designed for these forces including the 1.4 overstrength factor.

8. Design wall-roof anchorage for east-west loads

On the east and west wall elevations, wall-roof ties are used to transfer out-of-plane seismic forces on the tilt-up wall panels to the subdiaphragms. Applicable requirements for connection of out-of-plane wall anchorages to flexible diaphragms are specified in §12.11.2.1.

8a. Seismic force on wall-roof tie

Seismic forces are determined using Equation 12.11-1. These are the same forces as those determined in Part 7 for the north and south walls.

$$F_p = 0.8S_{DS}IW_p = 0.8W_p = 913 \text{ plf} \qquad \text{(Eq 12.11-1)}$$

8b. Design typical wall-roof tie

Try ties at 8-foot spacing, and determine F_p

$$F_p = 8 \text{ ft} \times 913 \text{ plf} = 7304 \text{ lb}$$

Comment: When tie spacing exceeds 4 feet, §12.11.2 and IBC §1604.8.2 require that walls be designed to resist bending between anchors.

Try prefabricated metal hold-downs with two ¾-inch bolts into a 3x subpurlin and two ⅝-inch anchor rods connecting the hold-downs to the wall panel. This connection, illustrated in Figure 5-18, is designed to take both tension and compression as recommended by the SEAOS C/COLA Northridge Earthquake Tilt-up Building Task Force and the 1999 SEAOC Blue Book (§C108.2.8.1). Design of the hold-down hardware is not shown. Consult ICC-ES Evaluation Reports for the allowable load capacity of pre-manufactured hold-downs. Note that if a one-sided hold-down is used, eccentricities in the subpurlin should be considered per §12.11.2.2.6. Generally, one-sided wall-roof anchorage is not recommended in SDC C and above.

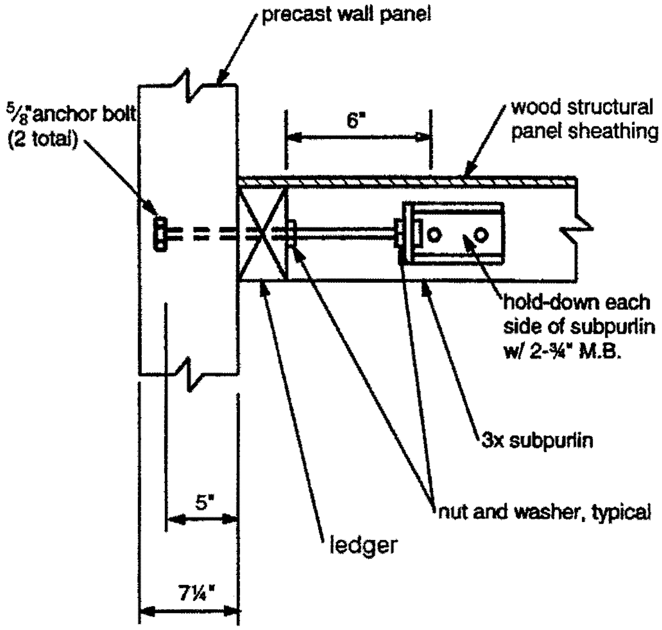

Figure 5-18. Typical subpurlin wall-roof tie

Check capacity of the two ¾-inch bolts in the Douglas Fir-Larch 3x subpurlin using 2005 AF&PA NDS Table 11G, where $C_d = 1.6$ and $C_g = 0.97$

$$(2630)(2 \text{ bolts})(1.6)(0.97) = 8164 \text{ lb} > 7304(0.7) = 5113 \text{ lb} \ldots o.k.$$

Minimum required end distance = $7D = 7(0.75) = 5.25$ in 2005 NDS T 11.5.1B

A distance of 6 inches from the through-bolt in the hold-down to the ledger will be used. Often, there is a gap of ⅛ inch or more between the end of the subpurlin and the side of the ledger caused by panelized roof erection methods, and the use of a 6-inch edge distance will ensure compliance with the 7*D* requirement. A larger distance can be used to ensure that through-bolt tear-out does not occur in the 3x subpurlin.

Check tension capacity of two ⅝-inch ASTM F1554 (grade 36) anchor rods using LRFD

$F_t = 0.75\,F_u = 0.75(58) = 43.5$ ksi AISC T J3.2

$\phi R_n = \phi F_t A_b = 0.75(43.5)(2)(0.307) = 20.0 \text{ kips} \ldots o.k.$ AISC Eq J3-1

$R_u = F_p = 7304 \text{ lb} < 20.0 \text{ kips} \ldots o.k.$

Note: The 1.4 factor normally applied to steel elements of the wall anchorage system is not applied to anchor rods per §12.11.2.2.2.

Check compression capacity of two ⅝-inch ASTM F1554 Grade 36 anchor rods using LRFD

$$P_n = F_{cr}A_g \qquad \text{AISC Eq E3-1}$$

$$A_g = A_b = 0.307 \text{ in}^2$$

$$\text{Radius of gyration of ⅝-in rod} = \frac{0.625 \text{ in}}{4} = 0.1563$$

Assume $L = 4\frac{1}{2}$ inches and $K = 1.0$

$$\frac{KL}{r} = \frac{1.0(4.5)}{0.1563} = 28.8$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = 345{,}074 \text{ psi} \geq 0.44\, F_y \text{ thus AISC Equation E3-2 is applicable}$$

$$F_{cr} = \left[0.658\frac{F_y}{F_e}\right]F_y = \left[0.658\frac{36}{345}\right]36{,}000 = 34{,}462 \text{ psi} \qquad \text{AISC Eq E3-2}$$

$$P_n = 34{,}462(0.307)(2 \text{ rods}) = 21{,}160 \text{ lb}$$

$$\phi_c P_n = 0.90(21{,}160) = 19{,}044 \text{ lb} \geq 7304 \text{ lb} \ldots \textit{o.k.}$$

Check tension capacity of anchor rods in wall panel for concrete strength.

The tilt-up panels are exterior wall elements, but the requirements of §§13.4.2 and 13.5.3 do not apply. This is because the tilt-up panels are structural walls instead of nonstructural architectural cladding. The requirements of §12.11 are the appropriate design rules in this situation. Section 12.11.2.2.5 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.

Recall that for wall anchorage, $F_p = 7304$ lb. Try a ⅝-inch-diameter ASTM F1554 Grade 36 hex headed bolt embedded in the concrete panel with 5 inches of embedment ($h_{ef} = 5$ inches). Assume that the bolt embedment is not near an edge and that the vertical shear load is negligible.

The wall's concrete anchorage needs to be checked using strength design under ACI 3 18-05 Appendix D. The vertical shear load on the anchor is very low because of the small subpurlin tributary roof load. ACI §D.7.1 allows the full tension strength to be used without reduction when the factored shear load is less than 20 percent of the nominal shear capacity of the anchorage as in this case.

ACI Equation D-1 normally requires $\phi N_n > N_{ua}$, but for structures in SDC C and above, IBC §1908.1.16 requires $0.754\ \phi N_n > N_{ua}$ when resisting seismic loads. N_n is the nominal tension strength of the anchorage. It is determined by checking the steel strength in tension N_{sa} (ACI §D.5.1), the concrete breakout strength in tension N_{cbg} (ACI §D.5.2), the pullout strength in tension N_{pn}(ACI §D.5.3), and the concrete side-face blowout strength in tension N_{sb} (ACI §D.5.4). An additional requirement for structures in SDC C and above is $N_{cbg} > N_{sa}$ (§D.3.3.4) to reduce the likelihood of brittle concrete failure. However, this may also be satisfied by providing anchors with a minimum design strength of 2.5 times the attachment's factored forces (IBC §1908.1.16).

Steel strength in tension N_{sa} — ACI §D.5.1

The nominal steel strength for ⅝-inch-diameter ASTM F1554 Grade 36 headed anchor rods is as follows. Equation D-3 is applicable

$$N_{sa} = nA_{se}f_{uta} \qquad \text{Eq D-3}$$

$n = 2$ bolts

$A_{se} = 0.226\ \text{in}^2$ (net tensile area) — AISC T 7-18

$f_{uta} = 58$ ksi — AISC T 2-5

$N_{sa} = 26.2$kips

Concrete breakout strength in tension N_{cb} — ACI §D.5.2

The two embedded anchors (one each side of subpurlin) are spaced close enough to be considered group action

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}(\psi_{ec,N})(\psi_{ed,N})(\psi_{c,N})(\psi_{cp,N})N_b \qquad \text{Eq D-5}$$

$$A_{Nc} = 2(7.5)\times(7.5+7.0+7.5)$$

$$= 330\ \text{in}^2 < nA_{Nco}\ldots o.k.$$

$$A_{Nco} = 9h_{ef}^2 = 9(5)^2 = 225\ \text{in}^2 \qquad \text{ACI Eq D-6}$$

$\psi_{ec,N} = 1.0$ (no eccentric loading)

$\psi_{ed,N} = 1.0$ (no adjacent edge effects)

$\psi_{c,N} = 1.25$ (uncracked section due to short parapet)

$\psi_{cp,N} = 1.0$

$$N_b = 24\sqrt{f'_c}h_{ef}^{1.5} \qquad \text{ACI Eq D-7}$$

$$= 24\sqrt{4000}\,5^{1.5} = 17.0\ \text{kips}$$

$$N_{cbg} = \frac{330}{225}\times(1.0)(1.0)(1.25)17.0 = 31.2\ \text{kips}$$

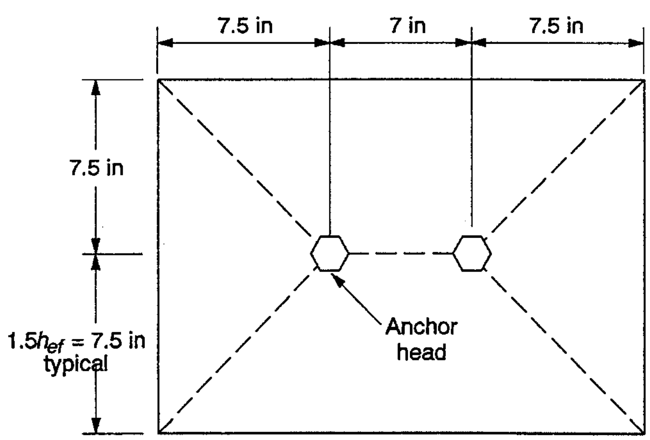

Figure 5-19. Projected failure area A_{Nc}

Pullout strength in tension §ACI D.5.3

$N_{pn} = \psi_{c,p} N_p = \psi_{c,p}\, 8 A_{brg} f'_c$ ACI Eq D-14, D-15

$\psi_{c,p} = 1.4$ (Assume uncracked section due to short parapet height)

A_{brg} = Bearing area of head = (Head area) – (shank area)

$$= \frac{3F^2}{2\sqrt{3}} - \text{(shank area)} = 0.761 - 0.307 = 0.454$$

$N_{pn} = 1.4\ (0.454)8\ (4000)(2\text{ bolts}) = 40.7\text{ kips} > N_{sa} \ldots o.k.$

Concrete side-face blowout strength in tension ACI §D.5.4

Since it is assumed that this concrete anchor is not located near an edge, N_{sb} will not govern the design.

Governing strength

The governing strength in tension is the steel strength $N_{sa} = 26.2$ kips. Checking ACI Equation D-1 modified by ACI §D.3.3.3 gives

$0.75\phi N_n = 0.75(0.75)26.2\text{ kips} = 14.7\text{ kips} \geq 7.3\text{ kips} \ldots o.k.$

where $\phi = 0.75$ for anchorage governed by ductile steel strength per ACI §D.4.4.

Therefore, the proposed two ⅝-inch-diameter anchor rods embedded 5 inches are acceptable.

It is interesting to note that the steel rod's tensile strength obtained from the ACI procedure is lower than the tensile strength obtained earlier using the AISC-LRFD procedure. This is because ACI uses the net tensile area of the threaded fastener while AISC-LRFD uses the nominal area.

Per ACI §D.3.3.4 and IBC §1908.1.16 structures in SDC C or higher must show that the behavior of the anchorage or attachment is ductile or provide an anchorage with a minimum design strength of 2.5 times the attachment's factored forces. Because the more brittle failing N_{cbg} (31.2 kips) and N_{pn} (39.0 kips) are greater than the more ductile failing N_{sa} (26.2 kips), §D.3.3.4 is satisfied here.

Compression

Wall anchorage forces act in compression as well as tension. Panelized wood roof systems by their very nature are not erected tight against the perimeter wall ledger, leaving a small gap to potentially close during seismic compression forces. Strap-type wall anchors that may have yielded and stretched under tensile forces are vulnerable to buckling and low-cycle fatigue as the gaps close. Cast-in-place anchor rods used in connectors can be checked for compression, but it is important to provide an additional nut against the interior wall surface to prevent the anchor punching through the wall. A common wall-roof tie connection shown in Figure 5-20 does not offer the same compression resistance as the anchor rod scheme presented in this example. Although there have been no failures of wall panels collapsing into the building, consideration of compressive forces will maintain the integrity of the wall anchorage tie and protect the diaphragm edge nailing under the reversible seismic forces.

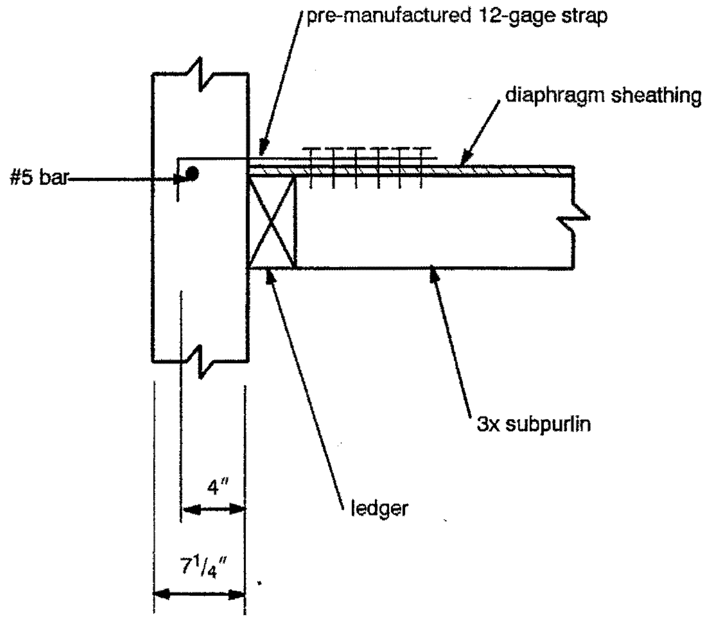

Figure 5-20. Common wall-roof strap tie

Comments about Anchorage Deformation

No prescriptive deformation limits of the wall tie system have been introduced into the IBC or ASCE/SEI 7-05, however the compatibility of the anchorage system's flexibility and the diaphragm shear nailing should be considered. Wall anchorage systems with too much flexibility will inadvertently load the wood sheathing edge nailing and either pull the nails through the sheathing edge or place the wood ledgers in cross-grain bending or tension. Pre-manufactured strap-type wall ties are designed to limit the maximum deformation to ⅛ inch at their rated allowable load, and pre-manufactured hold-down devices using anchor rods could allow even greater deformation (contact the device manufacturer for additional deformation information). This reported hold-down device flexibility is solely within the steel component itself and is additive to other sources of deformation. Additional deformation can be contributed by other anchorage components (e.g., bolts and nails) and installation practices (e.g., oversized holes).

8c. Design connection to transfer seismic force across first roof truss purlin

Under §12.11.2.2.1 for SDC C and higher, continuity ties are provided in diaphragms and subdiaphragms to distribute wall anchorage loads. Consequently, the forces used to design the wall-roof ties must also be used to design the continuity ties within the subdiaphragm. From Part 8b

$$F_p = \text{wall-roof tie load} = 7304 \text{ lb}$$

If the subdiaphragm is modeled as 32 feet deep and steel joist purlins are spaced at 8 feet, the connection at the first 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.

$$\frac{(32-8)}{32}\times F_p = \frac{3}{4}\times 7304 = 5478 \text{ lb}$$

At the second and third purlins, the force to be transferred is one-half and one-fourth, respectively, of the wall-roof tie force.

$$\frac{1}{2}\times 7{,}304 = 3652 \text{ lb}$$

$$\frac{1}{4}\times 7{,}304 = 1826 \text{ lb}$$

Try 12-gage metal strap with 10d common nails. Consult ICC-ES Evaluation Reports for allowable load capacity of pre-manufactured straps and ties.

The following calculation shows determination of the number of 10d common nails into Douglas Fir-Larch required at the first connection using allowable stress design

$$\frac{(0.7)5478}{127 \text{ lb}(1.60)} = 18.9 < 19 \text{ nails}$$ 2005 NDS T 11P and T 2.3.2

∴ Use 12-gage metal strap with 19 10d nails each side

The design of the 12-gage metal strap is not presented here, but the design is based on forces increased by 1.4 times the forces otherwise required under §12.11. This requirement of §12.11.2.2.2 is a result of the early strap failures observed in the Northridge Earthquake. It was found that many steel components lacked sufficient

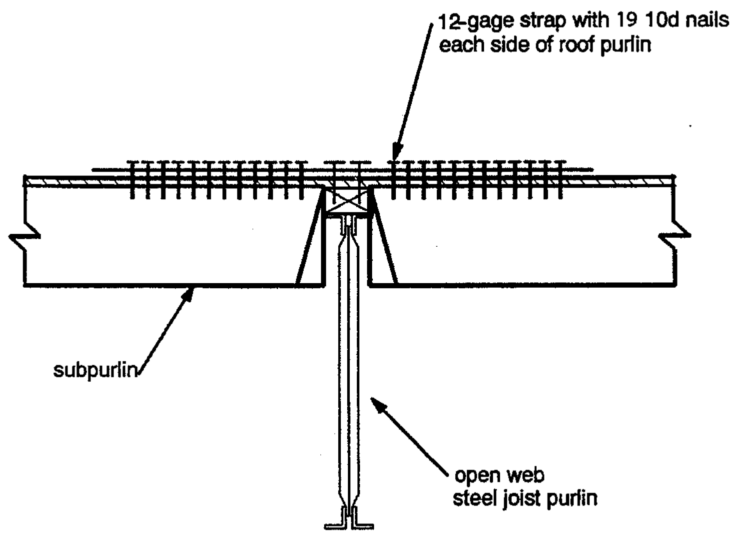

Figure 5-21. Subpurlin continuity tie at first purlin

ductility and overstrength to adequately accommodate seismic overloads. It is the intent of the 1.4 steel-material multiplier to provide sufficient overstrength to resist maximum anticipated wall anchorage forces.

Where pre-manufactured and pre-engineered straps and ties are utilized using capacity values published in ICC-ES Evaluation Reports, the engineer should compare the published capacity with the 1.4 steel increased force unless sufficient information is available to determine steel material values independently of other components.

Note that both subpurlins in Figure 5-21 likely would be 3x members because of the heavy strap nailing.

Design of the second and third connections is similar to that shown above.

Note: Additional requirements for eccentric wall anchorage and walls with pilasters are contained under §§12.11.2.2.6 and 12.11.2.2.7.

9. Design typical east-west subdiaphragm

In the 1976 UBC, the concept of subdiaphragms was introduced as an analytical device for transferring forces from the individual wall anchorage ties to the main diaphragm's continuous crossties. To transfer seismic forces from the heavy perimeter walls into the main roof diaphragm, continuous ties or crossties are necessary to drag the load uniformly across the diaphragm depth. Instead of creating a continuous tie at every wall anchorage location, continuous crossties can be placed at wider spacings using subdiaphragms. Subdiaphragms are portions of the main diaphragm that span between the continuous crossties and gather the wall anchorage loads and transfer these loads to the crossties. Once the load is collected into the continuous crossties it is distributed across the main diaphragm for further distribution to the building's shear walls and frames.

Subdiaphragms are provided for under ASCE/SEI 7-05 §12.11.2.2.1 as an analytical device to provide a rational load path for wall anchorage. Consequently, subdiaphragms are considered part of the wall anchorage system and are subject to loads per §12.11. For SDC C and above, subdiaphragm aspect ratios are limited to 2½ to 1, and this provides sufficient stiffness that the independent deflection between the subdiaphragm and the main diaphragm may be ignored.

9a. Check subdiaphragm aspect ratio

Maximum allowable subdiaphragm ratio is 2.5 to 1 §12.11.2.2.1

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 ft

Minimum subdiaphragm depth = 16 ft

$\therefore$ 32-foot-depth assumed . . . *o.k.*

9b. Forces on subdiaphragm

Because subdiaphragms are part of the out-of-plane wall anchorage system, they are designed under the requirements of §12.11.2.1, assuming the overall main diaphragm is flexible. Seismic forces on a typical east-west subdiaphragm are determined from Equation 12.11-1 with $S_{DS} = 1.0$ and $I = 1.0$

$$F_p = 0.8S_{DS}IW_p = 0.80W_p \qquad \text{Eq 12.11-1}$$

As shown in Part 7, $F_p = 913$ plf

9c. Check subdiaphragm shear

Assume a 32-foot-deep subdiaphragm as shown below. This is done for two reasons. First, the steel joist purlin along line 9 can be used as a subdiaphragm chord. Second, the deeper-than-required subdiaphragm depth (32 feet vs. 16 feet) reduces the subdiaphragm shear to manageable levels.

Shear reaction to continuity tie along lines C and D 913 plf (36.67 ft)

$$R = \frac{913 \text{ plf } (36.67 \text{ ft})}{2} = 16{,}740 \text{ lb}$$

$$\text{Maximum shear} = \frac{16{,}740 \text{ lb}}{32} = 523 \text{ plf}$$

Applying the ASD load combination

$$\text{ASD shear} = 0.7(523 \text{ plf}) = 366 \text{ plf}$$

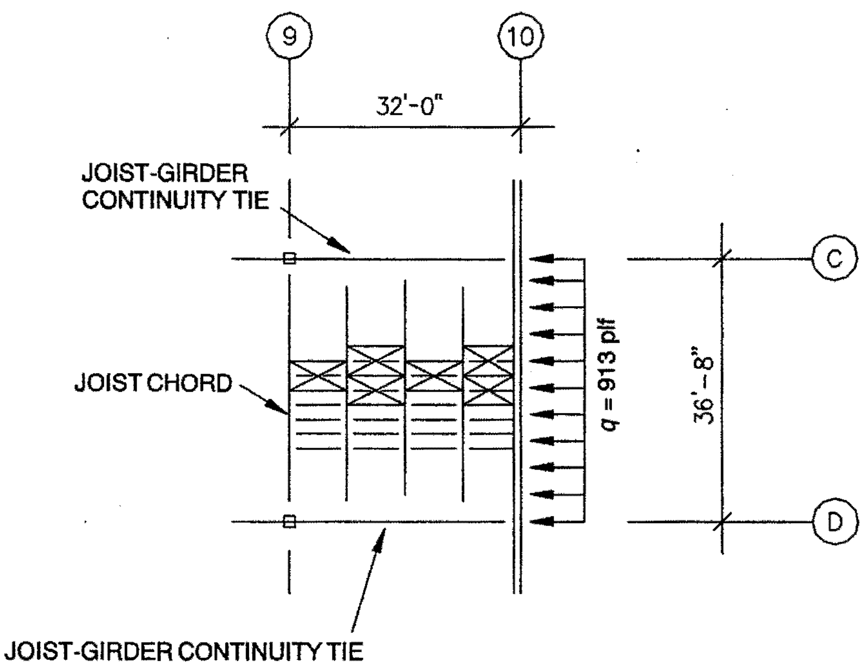

Figure 5-22. Typical subdiaphragm

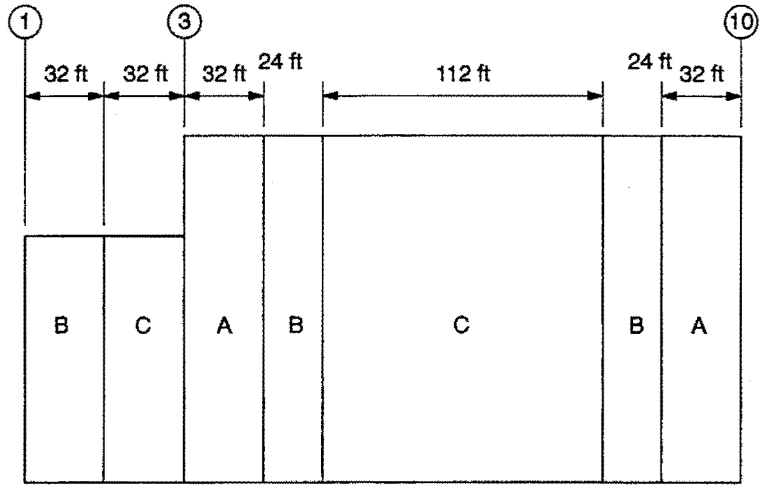

Figure 5-23. Revised nailing zones for north-south diaphragm

The 32-foot-deep subdiaphragm consists of zone A nailing (See Figure 5-6). The diaphragm's ASD shear strength 640 plf (Table 5-1) is adequate to resist the 366 plf load. On the west side of the building along line 1, a similar subdiaphragm situation exists, except the diaphragm design currently consists of the weaker zone C nailing. The first 32-feet will be revised to the stronger zone B nailing at 425 plf for purposes of the subdiaphragm.

Given the nailing of Figure 5-23, check the subdiaphragm shear.
ASD shear of the subdiaphragm = 366 plf < 425 plf. Thus, zone B nailing . . . *o.k.*

9d. Check steel joist as subdiaphragm chord

The steel joists 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 steel joist can carry additional seismic force.

$$\text{Chord force} \frac{913 \text{ plf } (36.67)^2}{8(32)} = 4796 \text{ lb}$$

Because the subdiaphragm chord is a steel element of the wall anchorage system, it is subject to a 1.4 force increase per §12.11.2.2.2.

$$\text{Chord Force (steel)} = 4796(1.4) = 6714 \text{ lbs}$$

This chord force is less than the wall anchorage force found in Part 7a, and thus does not govern.

Comment: In reality, the steel joist 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 the subdiaphragm path shown above but are 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.

9e. Determine minimum chord reinforcement at exterior concrete walls

This design example 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.

Subdiaphragm chord force = P = 4796 lb

$$A_s = \frac{P}{\phi f_y} = \frac{4796}{0.9(60{,}000)} = 0.09 \text{ in}^2$$

This is a relatively small amount of reinforcement. Generally, the main diaphragm chord reinforcement exceeds this amount. In present 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 3.

10. Design continuity ties for east-west direction

In a tilt-up building, continuous ties have two functions. The first is to transmit the heavy out-of-plane wall loads into the main 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.

10a. Seismic forces on continuity ties along lines C and D

A minimal interconnection of elements within the building is required per ASCE/SEI 7-05 §§12.1.3 and 12.1.4. Additionally, continuous ties or crossties are required per §12.11.2.2.1 (SDC C and above) to transfer seismic forces from the heavy perimeter walls into the main diaphragm. In the east/west load direction, the subdiaphragm load is collected into the continuous crossties and then distributed across the main diaphragm for further distribution to the building's shear walls and frames.

The continuous tie axial force at line 9 is the sum of both subdiaphragm reactions. Because the continuous ties are considered part of the wall anchorage system, their design force is subject to the steel material overstrength multiplier 1.4 per §12.11.2.2.2

$$P_9 = \frac{913 \text{ plf}(36.67 \text{ ft})}{2}(2)1.4 = 46{,}872 \text{ lb}$$

Per §12.1.3, the minimum interconnection force is $0.133 S_{DS} W = 0.133W$, but not less than $0.05W$, where W is the dead load of the smaller portion of the building being connected together. Unlike the wall anchorage force, W in this case includes the diaphragm weight and thus could govern at the interior of buildings. The worst-case value for W for the continuous tie is at grid line 6 with the following result:

$$P_r\text{(min)} = 0.133(14 \text{ psf})(36.67 \text{ ft})(4)(32 \text{ ft}) + 0.133(90.6 \text{ psf})(36.67 \text{ ft})(23)(23/2)/21$$
$$= 14{,}305 \text{ lb}$$

Per §12.1.4, the minimum support connection force is 5 percent of the dead and live load reaction.

$$P_r\,(\text{min}) = 0.05(14\text{ psf} + 16\text{ psf})(36.67\text{ ft})(32\text{ ft}/2) = 880\text{ lb}$$

Thus, the wall anchorage continuous tie force is governed by the subdiaphragm design

$$P_r = 46{,}872\text{ lbs}$$

Note: The continuous ties along lines C and D are not collector elements and thus are not subject to the special overstrength load combinations of §12.10.2.1. The girder line along line B functions both as a continuous tie and as a collector; therefore, both basic and overstrength load combinations must be considered.

10b. Design of joist-girders as continuity ties along lines C and D

Whether using a panelized wood sheathed roof or a metal deck roof, open web steel joist-girders are common roof girders in tilt-up buildings. Specialty engineers in association with the joist manufacturer typically design the steel joist-girder members. As required by IBC §2206.2, the building's design engineer is responsible for providing axial continuity tie forces to the manufacturer along with information stating which load factors, if any, have already been applied.

In this example, it should be reported to the joist manufacture that the unfactored wall anchorage axial force (tension and compression) acting on the joist-girder top chord is $P_r = 46{,}872$ lb. It is necessary to indicate to the joist manufacturer that this tie force is from seismic effects so that the joist-girder's specialty engineer is able to apply the proper load combinations of ASCE/SEI 7-05 §12.4.2.3.

Though not shown in this example, the top chord axial effects of wind W must also be considered if it could lead to a governing design of the joist-girder. Because the load combinations of §2.3.1 (strength design) and 2.4.1 (allowable stress design) contain very different formulas when considering seismic E and wind W, the design engineer cannot simply compare E and W to determine which governs. Currently, the joist industry is largely based on allowable stress design, but it is expected to transition to strength design in the future.

With line B acting as a collector (Figure 5-2), any joist-girders occurring there require an additional check of the overstrength load combinations of ASCE/SEI 7-05 §12.10.2.1. In this situation, the building's engineer must specify an E_m collector load as well as an E continuous tie load. The joist manufacturer's specialty engineer will have to check both the basic load combinations of §12.4.2.3 for E as well as the basic load combinations with overstrength factor of §12.4.3.2 for E_m.

The following is an example of the information to be placed on the drawings for the steel joist manufacturer to properly design his joist-girders for lateral loadings at lines C and D. Note that the wall anchorage force E shown should already include the 1.4 multiplier for steel elements.

Joist-girder
Axial Forces E = 46.9 kips (unfactored)
E_m = 0.0 kips (unfactored) *Applicable only at collectors.*
W = 13.8 kips (unfactored)
Forces shall be checked in both tension and compression.

10c. Design of joist-girders splices along lines C and D

Splicing large axial loads between joist-girder top chords is best done with a knife plate that sets down between the top chords at the joist seat (Figure 5-24). Top chords have a 1-inch gap between them, and the joist-girder manufacturer will keep this space clear if it is known in advance that a knife plate will be installed here. To facilitate installation, the knife plate should be ⅞-inch thick. The height of the knife plate is that necessary to obtain the splice welding, and often the strength of the knife plate is excessive just to accommodate installation.

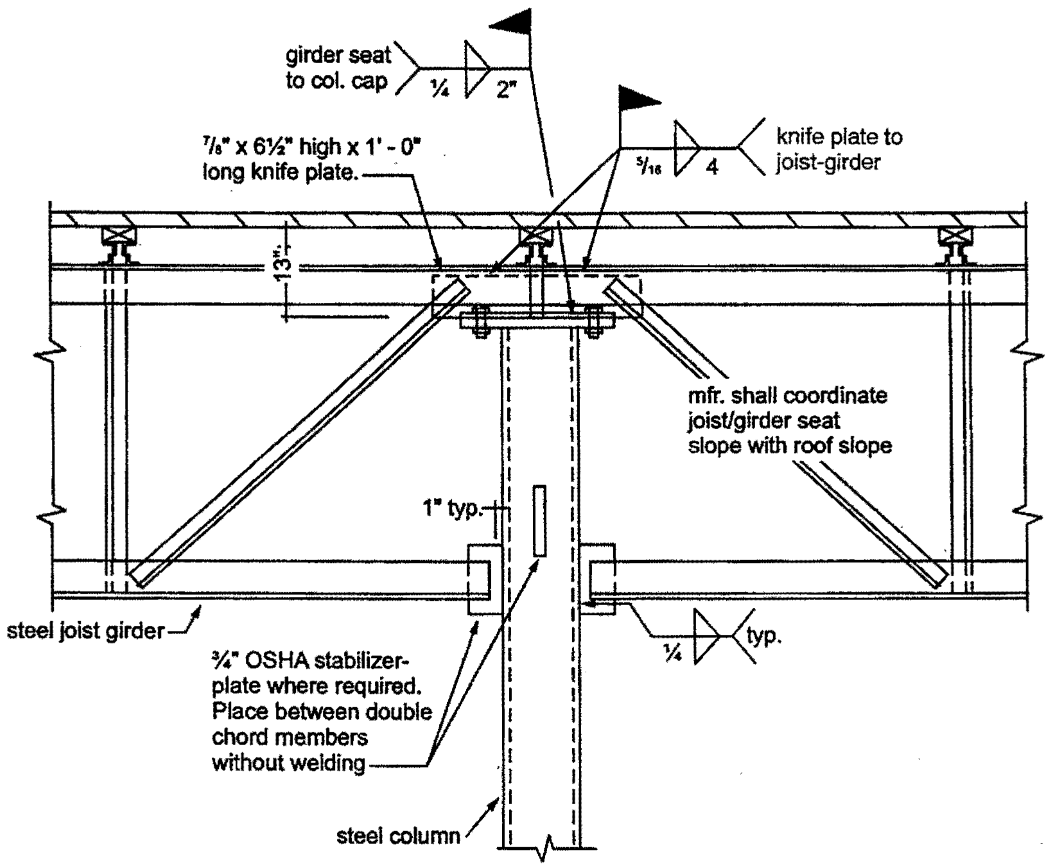

Figure 5-24. Joist-girder splice

Check the ⅞ × 6½-inch splice plate's design tensile strength per AISC §D2

$$\phi_t P_n = \phi_t F_y A_g = 0.90(36{,}000)(0.875)(6.5) = 184{,}000 \text{ lb} \qquad \text{AISC Eq D2-1}$$

Required tie force $P_r = 46{,}872$ lb < 184,000 lb . . . *o.k.*

Using two lines of 5/16 × 4-inch-long fillet welds, check the design weld strength per AISC §J2.4:

$$\phi R_n = \phi F_W A_W = 0.75(0.6 \times 70 \text{ ksi})\left(\frac{0.3125 \text{ in}}{\sqrt{2}} \times 4 \text{ in}\right)2 = 55.7 \text{ kips}$$

Required tie force $P_r = 46{,}872$ lb < 55,700 lb . . . *o.k.*

Therefore, the joist-girder splice across the columns is adequate.

10d. Comments on Metal Deck Diaphragms

Although less common in the southwest than panelized wood sheathing, flexible metal deck diaphragms (without concrete fill) are becoming more common in tilt-up construction in seismically active areas. When designed properly, metal decking can assist in providing wall anchorage and eliminate the need for subdiaphragms by acting itself as the continuous crossties. However, important detailing issues must be carefully considered.

Metal deck can only provide continuous crossties parallel to the deck span direction. ASCE/SEI 7-05 §12.11.2.2.4 specifically prohibits use of metal deck perpendicular to the direction of span for continuity, because the deck flutes will stretch out and flatten. Where the decking is spliced at the ends, a common structural member is needed to receive the attachment from both deck panels. In common steel joist systems with double top chords, it is necessary that both deck panels be attached to the same individual top chord half, otherwise crosstie loads will be inadvertently transferred through the steel joist top chord separation plate or web welding, depending on configuration. Another concern at the metal deck panel splice and direct ledger attachment is the weld tear-out through the metal deck edge. Proper deck gauge and puddle weld edge distance must be maintained for adequate wall anchorage strength.

If the metal decking is expected to carry wall anchorage forces, it must be investigated for tension *and compression* axial loads in conjunction with acting gravity loads. The axial compression loads are associated with inward wall forces and require a special axial/bending analysis of the decking. The *Standard for the Design of Cold-Formed Steel Framing* [AISI, 2004] provides design criteria for the decking, and the Structural Steel Education Council [Mayo, 2001] illustrates one approach for this wall anchorage. A more robust approach to metal deck wall anchorage is to use small steel angles or tubes that provide tension and compression wall support and distribute the load into the metal deck diaphragm.

Another challenge with metal deck diaphragms is the need for thermal expansion joints. Metal deck roof diaphragms are much more vulnerable to temperature swings than wood diaphragm systems; and with the trend toward larger roof dimensions, thermal expansion joints become very important. However, these expansion joints interrupt the continuity of the wall anchorage system and thus create several independent buildings to be analyzed separately. The wall anchorage forces must be fully developed into the main diaphragm and transferred to the applicable shear walls before reaching the expansion joint. This results in larger diaphragm shears.

10e. Design girder (continuity tie) connection to wall panel

In this example, walls are bearing walls and pilasters are not used to support the joist-girder vertically. Consequently, the kind of detail shown in Figure 5-25 must be used. This detail provides both vertical support for the girder and the necessary

wall anchorage capacity. The tie force is the same as that for the wall-roof tie of Part 7a (P_{10} = 7304 lb), but not less than 5 percent of the dead plus live load reaction per ASCE/SEI 7-05 §12.1.4. The detail has the capacity to take both tension and shear forces. Details of the design are not given. The embed design is similar to that shown in Part 7.

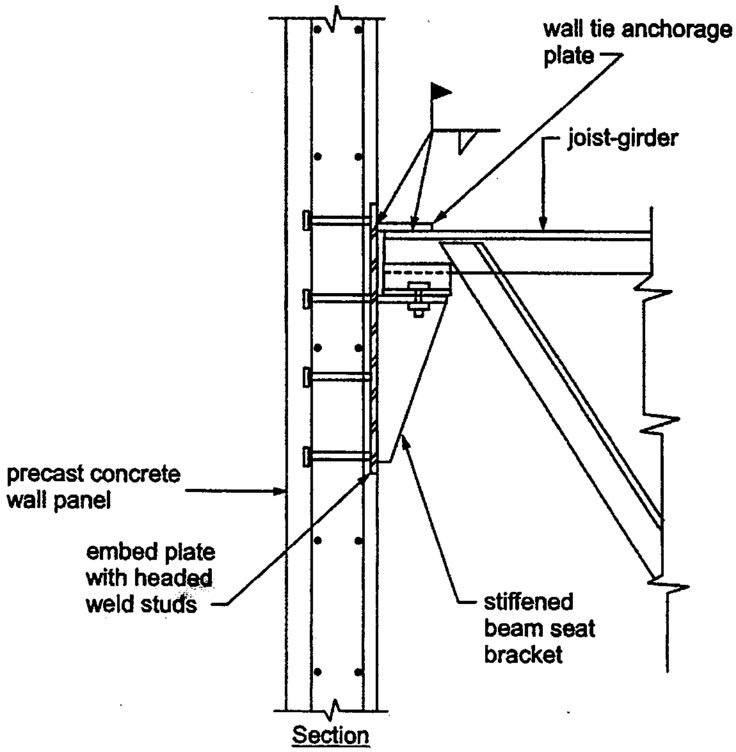

Figure 5-25. Bracket for wall-roof anchorage at joist-girder

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Design Example 6
Tilt-up Wall Panel with Openings

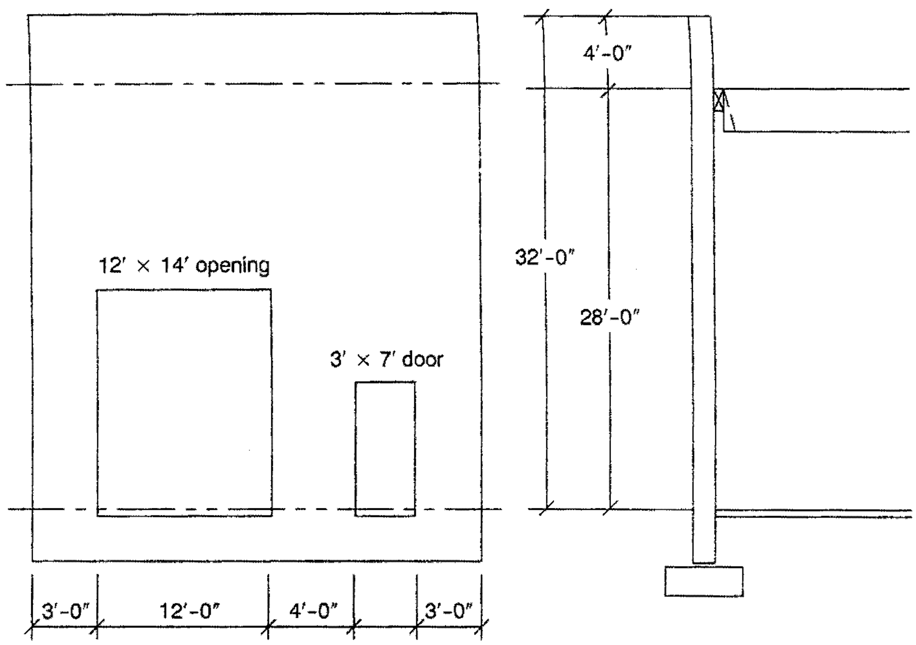

Figure 6-1. Wall elevation and section

Overview

Walls designed under the alternative slender wall method of ACI 318-05 §14.8, 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 (ACI 318 §14.5) and walls designed as compression members (ACI 318 §14.4) in that slender walls have greater restrictions on axial loads and must be a tension-controlled design. 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 of ACI 318 §14.8 as adopted by reference through ASCE/SEI 7-05 §14.2.1. 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. **Primary moment from the out-of-plane forces**
3. **Primary moment from vertical load eccentricity**
4. **Total factored moment including *P*-delta effects**
5. **Nominal moment strength ϕM_n**
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 = seven #5 bars each face at wall section between openings

Loading data

> Roof loading to wall = uniform loading; 40-foot span of 12 psf dead load and 20 psf roof live load; no snow load
>
> Roof loading eccentricity = 4 inches from interior face of panel

Short period spectral response acceleration for design $S_{DS} = 1.0g$

Site class = D

Occupancy importance factor $I = 1.0$

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 1-foot-wide design strips for out-of-plane design. However, for simplicity, where wall openings are involved, the entire pier width between openings is generally used as the design strip. The distributed loading accounts for the strip's self-weight, as well as the tributary loading from above each opening.

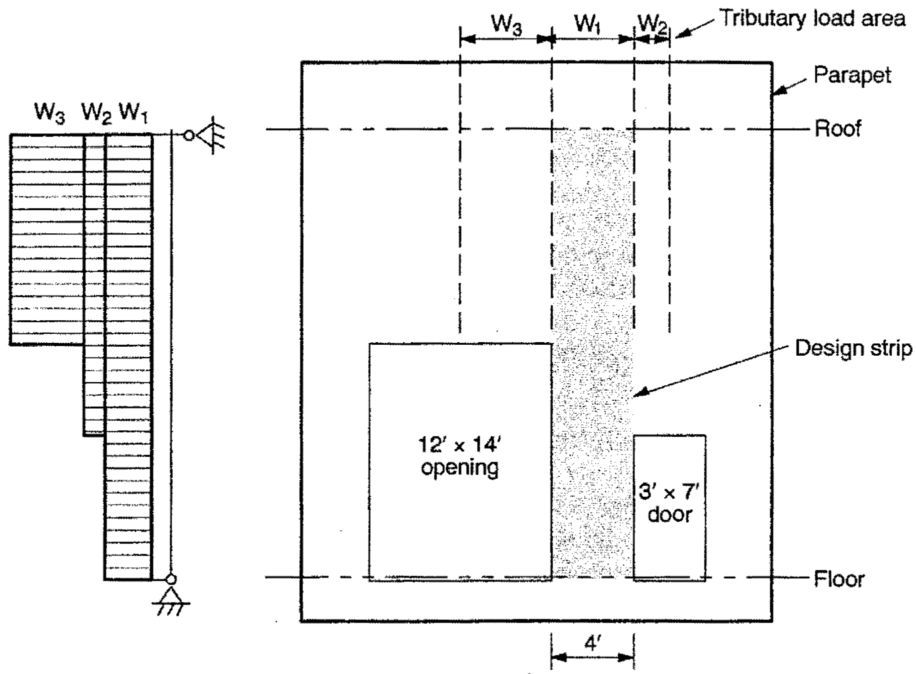

Figure 6-2. Design strip and distributed out-of-plane loading profile

1a. Seismic coefficient of wall element

The wall panel is considered a bearing wall and shear wall, thus §12.11.1 applies in determining the lateral seismic force.

$F_p = 0.40 S_{DS} I W_w$ §12.11.1

but not less than $0.10 w_w$

$I = 1.0$

$S_{DS} = 1.00g$

$F_p = 0.40(1.0)(1.00)w_w = 0.40w_w$

1b. Load combinations for strength design

For this example, the use of IBC load combination (Eq. 16-5) of §1605.2.1 is applicable, and governs for concrete strength design under seismic loading.

$$1.2D + 1.0E + L + 0.2\,S \qquad \text{(Eq 16-5)}$$

where

D = self weight of wall and dead load of roof

$L = 0$ (floor live load)

$S = 0$ (snow load)

$E = E_h + E_v = \rho Q_E + 0.20 S_{DS} D$ where $\rho = 1.0$ for wall elements §12.4.2

IBC load combination (Eq. 16-5) reduces to

$$(1.2 + 0.2S_{DS})D + 1.0Q_E \quad or \quad (1.2 + 0.20)D + 1.0Q_E \qquad \text{§12.4.2.3}$$

or $1.4D + 1.0Q_E$

1c. Lateral out-of-plane wall forces

The lateral wall forces Q_E are determined by multiplying the wall's tributary weight by the lateral force coefficient. Three different distributed loads are determined because of the presence of two door openings of differing heights. See Figure 6-2.

$$\text{Wall weight} = \frac{9.25}{12}\,150\text{ pcf} = 116\text{ lb/ft}^2$$

$F_{p\,wall} = 0.40(116\text{ lb/ft}^2) = 46\text{ lb/ft}^2$

$W_1 = 46\text{ lb/ft}^2 \times 4\text{ ft} = 184\text{ plf}$

$W_2 = 46\text{ lb/ft}^2 \times 3/2\text{ ft} = 69\text{ plf}$

$W_3 = 46\text{ lb/ft}^2 \times 12/2\text{ ft} = 276\text{ plf}$

2. Primary. moment from out-of-plane forces

Our objective is to check $\phi M_n \geq M_u$ where $M_u = M_{ua} + P_u \Delta_u$ (ACI 318 Equations 14-3 and 14-4). M_{ua} is the midheight moment due to applied factored loads and consists of two components: an out-of-plane loading moment ($M_{u\,oop}$) and a vertical eccentricity loading moment ($M_{u\,ecc}$). $P_u \Delta_u$ is a secondary moment created by P-delta effects and is investigated in Part 4.

To determine $M_{u\,oop}$, use the loading diagram in Figure 6-3.

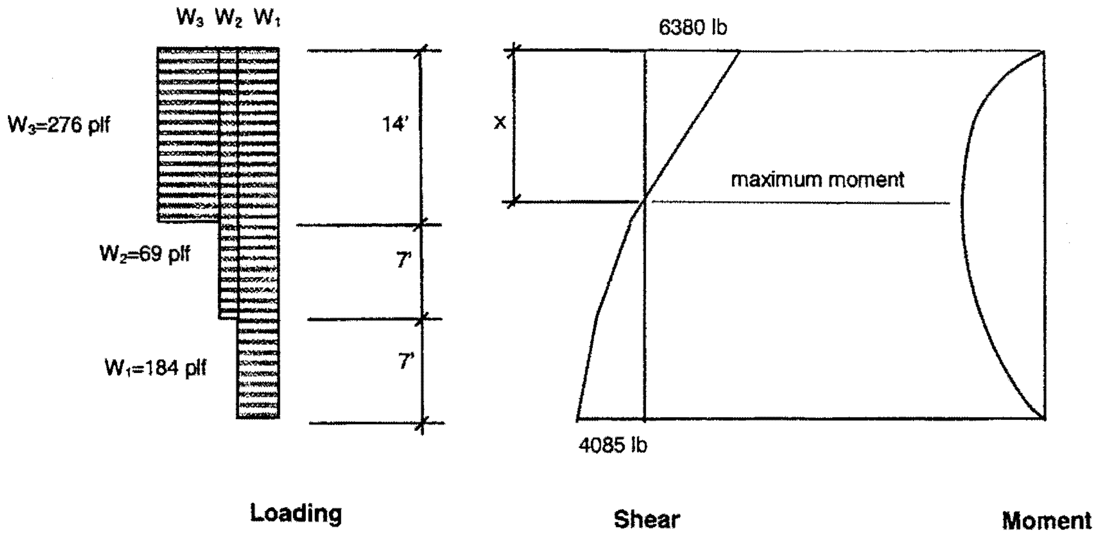

Figure 6-3. Corresponding loading, shear, and moment diagrams

ACI 318 §14.8.2.1 states, "The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan." As evident from Figure 6-3, a pier between openings has neither a uniform lateral load nor a maximum moment occurring at midspan. In this situation, it is acceptable to compute an equivalent uniform load and the more accurate maximum moment $M_{u\,oop}$ located slightly away from midspan. This is then combined with $M_{u\,ecc}$ and $P_u\Delta_u$ as computed at midspan.

Locate the point of zero shear for maximum moment $M_{u\,oop}$. 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 using forces based on §12.11.1. The parapet should be checked separately under §13.3.1, but is not a part of this example.

This example conservatively assumes the maximum moment occurs at a critical section width of 4 feet. 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

$$R_{grade} = \left[184\frac{(28)^2}{2} + 69\frac{(21)^2}{2} + 276\frac{(14)^2}{2}\right]\frac{1}{28} = 4085 \text{ lb}$$

$$R_{roof} = [184(28) + 69(21) + 276(14)] - 4085 = 6380 \text{ lb}$$

Determine the distance of the maximum moment from the roof elevation downward (Figure 6-3)

$$X = \frac{6380}{184+69+276} = 12.1 \text{ feet to point of zero shear (maximum moment)}$$

2b. Determine $M_{u\ out\text{-}of\text{-}plane\ (oop)}$

This is the primary moment due to factored out-of-plane forces, which excludes P-delta effects and vertical load eccentricity effects

$$M_{u\ oop} = 6380(12.1) - (184+69+276)\frac{(12.1)^2}{2} = 38{,}473 \text{ lb-ft}$$

$$M_{u\ oop} = 38.5 \text{ k-ft}$$

3. Primary moment from vertical load eccentricity

Any vertical loads that act at an eccentric distance from the wall's center also apply a moment to the design wall section. In this example only the roof loads are applied to the wall with an eccentricity.

P_{roof} = gravity loads from the roof acting on the design strip

P_{roof} = (roof dead load) × (tributary width of pier) × (tributary width of roof)

$$P_{roof} = (12 \text{ psf})\left(4+\frac{3}{2}+\frac{12}{2}\right)\frac{40}{2} = 2760 \text{ lb}$$

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 units vertical to 1 unit horizontal down to the flexural design section height (ACI 318, §14.8.2.5).

The applicable load combination determined in Part 1 is $1.40D + 1.0Q_E$ for seismic considerations. Roof live load is not combined with seismic loads in the IBC strength design-load combinations. However, when investigating load combinations including wind design, a portion of the roof live load is included.

$$P_{u\ roof} = 1.40\ (2760) = 3864 \text{ lb}$$

The eccentric load places an applied moment at the roof level. With the base of the wall considered pinned, the resulting moment at midheight is half of the applied moment.

$$M_{u\ ecc} = P_{u\ roof}\frac{e}{2}$$

where,

$$e = 4 \text{ in} + \frac{9.25-0.75}{2} = 8.25 \text{ in}$$

$$M_{u\ ecc} = 3864\frac{8.25}{2} = 15{,}939 \text{ lb-in}$$

$$= 1.3 \text{ k-ft}$$

4. Total factored moment including *P*-delta effects

The total factored moment M_u is the applied moment M_{ua} with increase for P-delta effects. From Parts 2 and 3

$$M_{ua} = M_{u\,oop} + M_{u\,ecc}$$

$$= 38.5 + 1.3 = 39.8 \text{ k-ft}$$

M_{ua} is magnified using ACI 318 Equation 14-6

$$M_u = \frac{M_{ua}}{1 - \dfrac{5P_u \ell_c^2}{(0.75)48E_c I_{cr}}} \qquad \text{ACI Eq (14-16)}$$

This provides a direct solution without the need of an iterative calculation process. To use this equation, the wall's vertical loading and section properties must be calculated.

4a. Determine the total vertical load

$P_{total} = P_{roof} + P_{wall\ top}$

$P_{roof} = 2760$ lb (from Part 3)

$P_{wall\ top}$ = 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.

$$P_{wall\ top} = (116\text{ psf})\left(4 + \frac{3}{2} + \frac{12}{2}\right)\left(\frac{28}{2} + 4\right) = 24{,}012 \text{ lb}$$

$P_{total} = P_{roof} + P_{wall\ top} = 2760 + 24{,}012 = 26{,}772$ lb

$P_u = 1.40(26{,}772) = 37{,}481$ lb

$= 37.5$ kips

4b. Determine necessary section properties

Reinforcing depth d can be based on ACI 7.7.3(a). Tilt-up panel reinforcement cover dimensions may comply with those for precast concrete, provided that the construction is similar to that normally expected under plant controlled conditions (ACI R7.7.3). With the panels normally cast on the building's concrete floor slab, reinforcement placement on chairs and form-work dimensions are able to keep to tight tolerances. For wall panels with #11 bars and smaller, the minimum cover dimension is ¾ inch.

d = thickness – reveal – cover – tie diameter – ½ bar diameter

$d = 9\tfrac{1}{4} - \tfrac{3}{4} - \tfrac{3}{4} - \tfrac{3}{8} - (\tfrac{1}{2})(\tfrac{5}{8}) = 7.06$ in

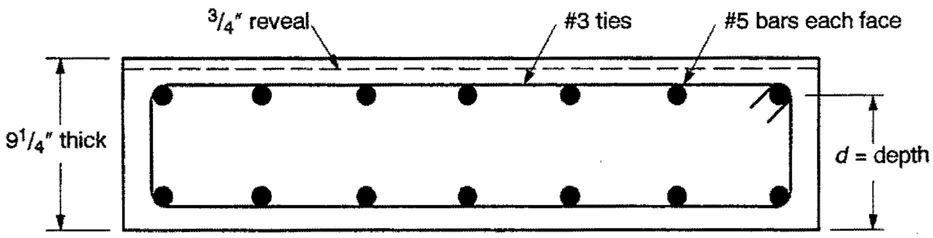

Figure 6-4. Design section

The cracked moment of inertia I_{cr} is necessary to determine the P-delta effects:

$$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_u}{f_y}\right)(d-c)^2 + \frac{\ell_w c^3}{3} \qquad \text{ACI 318 (Eq 14-7)}$$

where

$$E_s = 29{,}000 \text{ ksi}$$

$$E_c = 57\sqrt{f'_c} = 3122 \text{ ksi} \qquad \text{ACI 318 §8.5.1}$$

$$A_{se} = \left(A_s + \frac{P_u}{f_y}\right) = 7(0.31) + \frac{37{,}500}{60{,}000} = 2.80 \text{ in}^2$$

$$a = \frac{P_u + A_s f_y}{0.85 f'_c b} = \frac{37{,}500 + 7(0.31)(60{,}000)}{0.85(3000)(48)} = 1.37 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{1.37}{0.85} = 1.61 \text{ in} \qquad \text{ACI 318 §10.2.7}$$

$$I_{cr} = \frac{29{,}000}{3122} 2.80\,(7.06 - 1.61)^2 + \frac{48(1.61)^3}{3} = 839 \text{ in}^4$$

4c. Determine the total factored moment magnified for *P*-delta effects

$$M_u = \frac{M_{ua}}{1 - \dfrac{5 P_u \ell_c^2}{(0.75) 48 E_c I_{cr}}} = \frac{39.8}{1 - \dfrac{5(37.5)(28 \times 12)^2}{0.75(48)3122(839)}} = 51.3 \text{ k-ft} \qquad \text{ACI 318 (Eq 14-6)}$$

5. Check the design section's adequacy

5a. Nominal strength ϕM_n

The nominal moment strength ϕM_n is given by the following equation

$$\phi M_n = \phi A_{se} f_y \left(d - \frac{a}{2}\right); \text{ where } \phi = 0.90 \text{ per ACI 318 §9.3.2}$$

$$= 0.90(2.80)(60{,}000)\left(7.06 - \frac{1.37}{2}\right)$$

$$= 964 \text{ k-in} = 80.3 \text{ k-ft}$$

$$M_u = 51.3 \text{ k-ft} < 80.3 \text{ k-ft}$$

$$M_u \le \phi M_n \ldots o.k.$$

5b. Check flexural cracking moment

Verify that $M_{cr} \leq \phi M_n$ to determine the acceptability of the slender wall design method (ACI 318 §14.8.2.4). M_{cr} is defined in ACI 318 §9.5.2.3.

$$M_{cr} = f_r \frac{I_g}{y_t} = \frac{7.5\sqrt{3000}(48)\dfrac{(9.25)^3}{12}}{\dfrac{9.25}{2}} = 281{,}187 \text{ lb-in} = 23.4 \text{ k-ft} \quad \text{ACI 318 (Eq 9-9)}$$

$$M_{cr} = 23.4 \text{ k-ft} \leq \phi M_n = 80.3 \text{ k-ft} \ldots \textit{o.k.}$$

Reinforcing is sufficient for the use of the alternative slender wall method.

Note: For the purposes of ACI 318 §14.8.2.4, 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 and relatively shallow.

5c. Check section is tension-controlled

ACI 318 §10.3.4 defines tension-controlled sections as those whose net tensile strain $\varepsilon_t > 0.005$ when the concrete in compression reaches its assumed strain limit of 0.003. The net tensile strain limits can also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. A net tensile strain limit of $\varepsilon_t > 0.005$ is equivalent to $c/d_t < 0.375$ for grade 60 reinforcement (ACI 318 §R9.3.2.2).

$$c/d_t = 1.61/7.06 = 0.228 < 0.375 \ldots \textit{o.k.}$$

Therefore, the slender wall method is acceptable.

5d. Check the maximum vertical stress at midheight

Check the vertical stress at the midheight section to determine whether the alternative slender wall design method is acceptable (ACI 318 §14.8.2.6). ACI requires this check using strength design load levels. With only dead load D and roof live load L_r contributing to P_u, the IBC load combinations of §1605.2.1 with ASCE §12.4.2.3 reduce to the following:

IBC (Eq. 16-1) $1.4(D + F)$ $= 1.4D$

IBC (Eq. 16-2) $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ $= 1.2D + 0.5L_r$

IBC (Eq. 16-3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$ $= 1.2D + 1.6L_r + 0.8W$

IBC (Eq. 16-4) $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$ $= 1.2D + 1.6W + 0.5L_r$

IBC (Eq. 16-5) $(1.2 + 0.2S_{DS})D + 1.0Q_E + L + 0.2S$ $= 1.4D + 1.0Q_E$

IBC (Eq. 16-6) $0.9D + 1.6W + 1.6H$ $= 0.9D + 1.6W$

IBC (Eq. 16-7) $(0.9\text{-}0.2S_{DS})D + 1.0Q_E + 1.6H$ $= 0.7D + 1.0Q_E$

From inspection of the load combinations above, only combinations (16-1), (16-3) and (16-5) can govern vertical load. As determined in Part 4a, the total vertical dead load D is 26,772 lbs. The roof live load L_r is determined as follows

$$L_r = 20 \text{ psf} \times 40/2 \times (4+3/2+12/2) = 4600 \text{ lbs.}$$

Load combinations (16-1), (16-3) and (16-5) result in the following P_u vertical loads

IBC (Eq. 16-1) $1.4D = 1.4(26{,}772)$ = 37,481 lb

IBC (Eq. 16-3) $1.2D + 1.6L_r + 0.8W = 1.2(26{,}772) + 1.6(4600)$ = 39,486 lb (governs)

IBC (Eq. 16-5) $1.4D + Q_E = 1.4(26{,}772)$ = 37,481 lb

Vertical stress $P_u/A_g = 39{,}486/(48 \times (9.25 - 0.75)) = 96.8 \text{ psi} < 0.06(3000) = 180 \text{ psi}$ . . . *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 (ACI 318, §14.5) or the compression member method (ACI 318, §14.4), would be required along with their restrictions on wall slenderness.

6. Service load out-of-plane deflection

In the process of incorporating provisions for slender wall design, ACI 318 included UBC limits for service load deflection Δ_s (including P-delta effects) to a maximum of $\ell_c/150$ (ACI 318 §14.8.4).

$$\Delta_s = \frac{5M\ell_c^2}{48E_cI_e} \qquad \text{ACI 318 (Eq 14-8)}$$

where

$$M = \frac{M_{sa}}{1 - \dfrac{5P_s\ell_c^2}{48E_cI_e}} \qquad \text{(Eq 14-9)}$$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \le I_g; \text{ and } M_a = M \qquad \text{ACI 318 (Eq 9-8)}$$

Unfortunately, during this incorporation no clear direction was given for the service-level load combinations expected to be used in evaluating Δ_s. ACI 318 §8.2.2 simply refers to the "general building code," but with a general transition to strength-based design and the wide variety of load combinations currently in the IBC and ASCE/SEI 7-05 codes, there is no clear direction as to the proper load combination for evaluating service-level seismic deflection Δ_s.

ASCE/SEI 7-05 Appendix C provides a brief discussion on serviceability considerations, and the Appendix C Commentary (added later as errata) provides some guidance for a service-level *wind* load combination. However, no specific discussion on service-level *seismic* load is found in ASCE/SEI 7-05.

As mentioned earlier, many of the slender wall provisions are from the Uniform Building Code. Under the UBC, service-level deflection checks were intended to be determined using the UBC allowable stress load combinations. Those original UBC load combinations are very similar to those currently found in IBC §1605.3.2 *"Alternative basic load combinations."* Until service-level seismic load combinations are more clearly defined in the IBC, ASCE/SEI 7-05, or ACI 318, it is appropriate to use the load combinations in IBC §1605.3.2. For evaluating service-level deflections, IBC Eq. 16-20 will govern.

$$D + L + S + E/1.4 \qquad \text{IBC (Eq. 16-20)}$$

where

$$E = \rho Q_E + 0.2S_{DS}D \qquad \S 12.4.2$$

Thus

$$D + L + S + (\rho Q_E + 0.2S_{DS}D)/1.4$$

or

$$(1 + 0.14S_{DS})D + L + S + \rho Q_E/1.4$$

With $L = 0$, $S = 0$, $\rho = 1.0$, and $S_{DS} = 1.0$, the applicable load combination for service-level seismic loads reduces to the following:

$$1.14D + Q_E/1.4$$

6a. Determine service level moment

M_{sa} is the applied service-level moment, and comprises $M_{s\,oop}$ (out-of-plane) and $M_{s\,ecc}$

$$M_{sa} = M_{s\,oop} + M_{s\,ecc}$$

Because $M_{s\,oop}$is solely caused by seismic loads Q_E,

$$M_{s\,oop} = \frac{M_{u\,oop}}{1.4} = \frac{(38.5)}{1.4} = 27.5 \text{ k-ft}$$

Additionally

$M_{s\,ecc} = P_{roof}\,(e/2) = 1.14(2760)8.25/2 = 12{,}979$ lb-in (See Part 3) = 1.1 k-ft

$M_{sa} = 27.5 + 1.1 = 28.6$ k-ft

$P_s = 1.14\,D = 1.14(26{,}772$ lbs$) = 30{,}520$ lb = 30.5 k (from Part 4a)

$M_{cr} = 23.4$ k-ft (from Part 5b)

$I_{cr} = 839$ in^4 (from Part 4b)

$$I_g = 48\frac{(9.25)^3}{12} = 3166 \text{ in}^4$$

I_g is based on gross thickness, without substracting for the architectural reveal depth, because this produces more accurate results when the reveals are narrow and relatively shallow.

First iteration

Because M and I_e are dependent on each other, some iterations between ACI 318 Equations 9-8 and 14-9 are necessary to obtain an accurate deflection Δ_s. Begin with $M = M_{sa}$

$$I_e = \left(\frac{23.4}{28.6}\right)^3 3166 + \left[1 - \left(\frac{23.4}{28.6}\right)^3\right] 839$$

$$I_e = 2114 \text{ in}^4 \leq I_g \ldots o.k.$$

$$M = \frac{28.6}{1 - \dfrac{5(30.5)(28 \times 12)^2}{48(3122)(2114)}} = 30.2 \text{ k-ft}$$

Second iteration

$$I_e = \left(\frac{23.4}{30.2}\right)^3 3166 + \left[1 - \left(\frac{23.4}{30.2}\right)^3\right] 839$$

$$I_e = 1921 \text{ in}^4 \leq I_g \ldots o.k.$$

$$M = \frac{28.6}{1 - \dfrac{5(30.5)(28 \times 12)^2}{48(3122)(1921)}} = 30.4 \text{ k-ft}$$

Third iteration

$$I_e = \left(\frac{23.4}{30.4}\right)^3 3166 + \left[1 - \left(\frac{23.4}{30.4}\right)^3\right] 839$$

$$I_e = 1900 \text{ in}^4 \leq I_g \ldots o.k.$$

$$M = \frac{28.6}{1 - \dfrac{5(30.5)(28 \times 12)^2}{48(3122)(1900)}} = 30.4 \text{ k-ft} \ldots \text{converged}$$

6b. Check service load deflection — ACI 318 §14.8.4

$$\Delta_s = \frac{5M\ell_c^2}{48E_cI_e}$$

$$= \frac{5(30.4)(28 \times 12)^2 12}{48(3122)(1900)}$$

$$= 0.72 \text{ in} < \frac{\ell_c}{150} = 2.24 \text{ in} \;\; \ldots o.k.$$

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 12-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 1-foot unit horizontal design strip and subject to the out-of-plane loads computed earlier in this example.

$$F_{p\,wall} = 0.40(116 \text{ lb/ft}^2) = 46 \text{ lb/ft}^2$$

The moment is based on a simply supported horizontal beam

$$M_u = F_p \frac{(\text{opening width})^2}{8} = 46\frac{12^2}{8}$$

$$= 828 \text{ lb-ft} = 0.83 \text{ k-ft}$$

Try using #5 bars at 18-inch spacing to match the bar size 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

$$A_s = 0.31\left(\frac{12}{18}\right) = 0.21 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.21)60{,}000}{0.85(3000)(12)} = 0.41 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{0.41}{0.85} = 0.48 \text{ in}$$

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.

$$d = \tfrac{1}{2}(\text{thickness} - \text{reveal}) - \text{bar diameter}$$

$$d = \tfrac{1}{2}(9\tfrac{1}{4} - \tfrac{3}{4}) - \tfrac{5}{8} = 3.63 \text{ in}$$

Determine ϕ per ACI 318 §R9.3.2.2.

$$\frac{c}{d_t} = \frac{0.48}{3.63} = 0.132 \leq 0.375$$

Therefore, it is a tension-controlled section and $\phi = 0.9$

$$\phi M_n = 0.9(0.21)(60)\left(3.63 - \frac{0.41}{2}\right) = 38.8 \text{ k-in}$$

$$= 3.24 \text{ k-ft} \geq 0.83 \text{ k-ft} \ldots \textit{o.k.}$$

$$= \phi M_n \geq M_u \ldots \textit{o.k.}$$

Therefore, the horizontal reinforcing is acceptable.

7b. Typical reinforcing around openings

Two #5 bars are required around all window and door openings per ACI §14.3.7. 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. In addition, it is common to add diagonal bars at the opening corners to assist in limiting the cracking that often occurs because of shrinkage stresses (Figure 6-5).

7c. Horizontal (transverse) reinforcing between the wall openings

The style and quantity of horizontal (transverse) reinforcing between the wall openings is typically dependent on the in-plane shear wall design. For intermediate precast structural walls, ACI 318 §21.13.3 and ASCE/SEI 7-05 §14.2.2.14 provide special reinforcements.

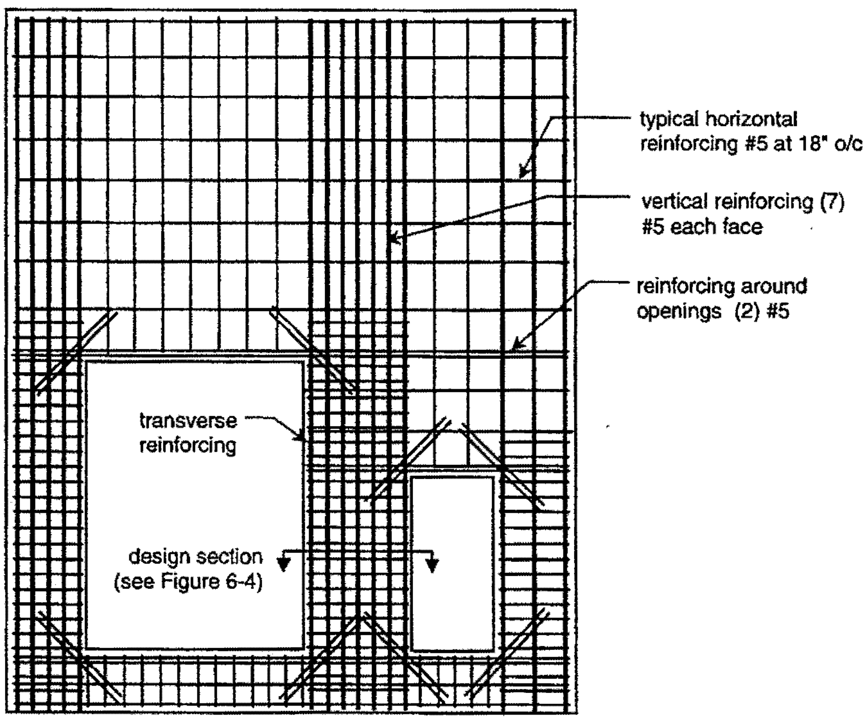

Figure 6-5. Typical wall reinforcing

In this example, two curtains of vertical reinforcing are provided for out-of-plane loads. In this situation, the horizontal reinforcing is often provided in the form of hoops or ties to assist in supporting both layers of the vertical reinforcing during construction even if two curtains of horizontal reinforcing are not required by analysis. See Figure 6-5.

Commentary

The ACI 318 section on the alternative design of slender walls made its debut in the 1999 edition. It is generally based on the 1997 Uniform Building Code, which incorporated 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.

In the process of converting the 1997 UBC slender wall equations to ACI format, the equation for Δ_s was significantly revised. It has been reported that the current ACI procedure overestimates the panel's service-level stiffness compared with the test results of the 1980s.

The calculation for M_u using ACI Eq. 14-6 provides a direct solution for second-order effects including P-delta moments, instead of the interative process of ACI Eq. 14-5. Various software programs on the market today still use an iterative second-order approach or, in some cases, have no second-order analysis. Software program results can have significant errors when improper input assumptions are made. The designer is cautioned to ensure a proper second-order analysis is utlized with proper panel stiffness assumptions.

Tilt-up wall construction has become very popular because of its versatility and its erection speed. Failures of the concrete wall section out-of-plane are extremely rare; however, wall anchorage failures at the roofline have occurred during earthquakes. In response to these failures, the current anchorage design forces and detailing requirements are significantly more stringent than they were under older codes.

References

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