13

DESIGN FOR NONSTRUCTURAL COMPONENTS

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Chapter 6 of the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereinafter, the *Provisions* and *Commentary*) addresses architectural, mechanical, and electrical components of buildings. Two examples are presented here to illustrate many of the requirements and procedures. Design and anchorage are illustrated for exterior precast concrete cladding, and for a roof-mounted HVAC unit. The rooftop unit is examined in two common installations: directly attached, and isolated with snubbers. This chapter also contains an explanation of the fundamental aspects of the *Provisions,* and an explanation of how piping, designed according to the ASME Power Piping code, is checked for the force and displacement requirements of the *Provisions*.

A large variety of materials and industries are involved with nonstructural components is large, and numerous documents define and describe methods of design, construction, manufacture, installation, attachment, etc. Some of the documents address seismic issues but many do not. *Provisions* Sec. 6.1.1 [6.1.2] contains a listing of approved standards for various nonstructural components.

Although the *Guide* is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

A few noteworthy changes were made to the nonstructural components requirements of the 2003 *Provisions*. These include explicit definition of load effects (including vertical seismic forces) within the chapter and revised classification of nonductile anchors (based on demonstrated ductility or prequalification rather than embedment-length-to-diameter ratio).

In addition to changes *Provisions* Chapter 6, the basic earthquake hazard maps were updated and the concrete design reference was updated to ACI 318-02 (with a significant resulting changes to the calculations for anchors in concrete).

Where they affect the design examples in this chapter of the *Guide*, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the *Provisions*, the following are referenced in this chapter:

ACI 318	American Concrete Institute. 1999 [2002]. *Building Code Requirements and Commentary for Reinforced Concrete*.
ASCE 7	American Society of Civil Engineers. 1998 [2002]. *Minimum Design Loads for Buildings and Other Structures*.
ASHRAE APP IP	American Society of Heating, Refrigeration, and Air-Conditioning Engineers (ASHRAE). 1999. *Seismic and Wind Restraint Design*, Chapter 53.
ASME B31.1	American Society of Mechanical Engineers. *Power Piping Code*.
IBC	International Code Council. 2000. *International Building Code*.

The symbols used in this chapter are drawn from Chapter 2 of the *Provisions* or reflect common engineering usage. The examples are presented in U.S. customary units.

[In the 2003 *Provisions*, definitions and symbols specific to nonstructural components appear in Sec. 6.1.3 and 6.1.4, respectively.]

13.1 DEVELOPMENT AND BACKGROUND OF THE PROVISIONS FOR NONSTRUCTURAL COMPONENTS

13.1.1 Approach to Nonstructural Components

The *Provisions* requires that nonstructural components be checked for two fundamentally different demands placed upon them by the response of the structure to earthquake ground motion: resistance to inertial forces and accommodation of imposed displacements. Building codes have long had requirements for resistance to inertial forces. Most such requirements apply to the component mass an acceleration that vary with the basic ground motion parameter and a few broad categories of components. The broad categories are intended distinguish between components whose dynamic response couples with that of the supporting structure in such a fashion as to cause the component response accelerations to be amplified above the accelerations of the structure and those components that are rigid enough with respect to the structure so that the component response is not amplified over the structural response. In recent years, a coefficient based on the function of the building or of the component have been introduced as another multiplier for components important to life safety or essential facilities.

The *Provisions* includes an equation to compute the inertial force that involves two additional concepts: variation of the acceleration with relative height within the structure, and reduction in design force based upon available ductility in the component, or its attachment. The *Provisions* also includes a quantitative measure for the deformation imposed upon nonstructural components. The inertial force demands tend to control the seismic design for isolated or heavy components, whereas, the imposed deformations are important for the seismic design for elements that are continuous through multiple levels of a structure, or across expansion joints between adjacent structures, such as cladding or piping.

The remaining portions of this section describe the sequence of steps and decisions prescribed by the *Provisions* to check these two seismic demands on nonstructural components.

13.1.2 Force Equations

The following seismic force equations are prescribed for nonstructural components:(*Provisions* Eq. 6.1.3-1 [6.2-1], 6.1.3-2 [6.2-3], and 6.1.3-3 [6.2-4]):

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right)$$

$$F_{p_{max}} = 1.6 S_{DS} I_p W_p$$

$$F_{p_{min}} = 0.3 S_{DS} I_p W_p$$

where:

F_p = horizontal equivalent static seismic design force centered at the component's center of gravity and distributed relative to the component's mass distribution.

a_p = component amplification factor (either 1.0 or 2.5) as tabulated in *Provisions* Table 6.2.2 [6.3-1] for architectural components and *Provisions* Table 6.3.2 [6.4-1] for mechanical and electrical components (Alternatively, may be computed by dynamic analysis)

S_{DS} = five percent damped spectral response acceleration parameter at short period as defined in *Provisions* Sec. 4.1.2 [3.3.3]

W_p = component operating weight

R_p = component response modification factor (varies from 1.0 to 5.0) as tabulated in *Provisions* Table 6.2.2 [6.3-1]for architectural components and *Provisions* Table 6.3.2 [6.4-1] for mechanical and electrical components

I_p = component importance factor (either 1.0 or 1.5) as indicated in *Provisions* Sec. 6.1.5 [6.2-2]

z = elevation in structure of component point of attachment relative to the base

h = roof elevation of the structure or elevation of highest point of the seismic-force-resisting system of the structure relative to the base

The seismic design force, F_p, is to be applied independently in the longitudinal, and transverse directions. The effects of these loads on the component are combined with the effects of static loads. *Provisions* Eq. 6.1.3-2 [6.2-3 and 6.2-4] and 6.1.3-3, provide maximum and minimum limits for the seismic design force.

For each point of attachment, a force, F_p, should be determined based on *Provisions* Eq. 6.1.3-1 [6.2-1]. The minima and maxima determined from *Provisions* Eq. 6.1.3-2 and 6.1.3-3 [6.2-1] must be considered in determining each F_p. The weight, W_p, used to determine each F_p should be based on the tributary weight of the component associated with the point of attachment. For designing the component, the attachment force, F_p, should be distributed relative to the component's mass distribution over the area used to establish the tributary weight. With the exception of the bearing walls, which are covered by *Provisions* Sec. 5.2.6.2.7 [4.6.1.3], and out-of-plane wall anchorage to flexible diaphragms, which is covered by *Provisions* Sec. 5.2.6.3.2 [4.6.2.1], each anchorage force should be based on simple statics determined by using all the distributed loads applied to the complete component. Cantilever parapets that are part of a continuous element, should be separately checked for parapet forces.

13.1.3 Load Combinations and Acceptance Criteria

13.1.3.1 Seismic Load Effects

When the effects of vertical gravity loads and horizontal earthquake loads are additive, *Provisions* Eq. 5.2.7.1-1 [4.2-1] is used:

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

When the effects of vertical gravity load counteract those of horizontal earthquake loads, *Provisions* Eq. 5.2.7.1-2 [4.2-2] is used:

$$E = \rho Q_E - 0.2S_{DS}D$$

where:

E = effect of horizontal and vertical earthquake-induced forces

ρ = redundancy factor (= 1.0 for nonstructural components)

Q_E = effect of horizontal seismic forces (due to application of F_p for nonstructural components)

D = effect of dead load

$0.2S_{DS}D$ = effect of vertical seismic forces

13.1.3.2 Strength Load Combinations

Provisions Sec. 5.2.7 [4.2.2] requires the use of ASCE 7 factored load combinations. The combinations from ASCE 7 Sec. 2.3.2 that include earthquake effects are:

$$U = 1.2D + 1.0E + 0.5L + 0.2S$$

$$U = 0.9D + 1.0E + 1.6H$$

13.1.4 Component Amplification Factor

The component amplification factor, a_p, found in *Provisions* Eq. 6.1.3-1 [6.2-1] represents the dynamic amplification of the component relative to the maximum acceleration of the component support point(s). Typically, this amplification is a function of the fundamental period of the component, T_p, and the fundamental period of the support structure, T. It is recognized that at the time the components are designed or selected, the effective fundamental period of the structure, T, is not always available. It is also recognized that for a majority of nonstructural components, the component fundamental period, T_p, can be accurately obtained only by expensive shake-table or pullback tests. As a result, the determination of a component's fundamental period by dynamic analysis, considering T/T_p ratios, is not always practicable. For this reason, acceptable values of a_p have been provided in the *Provisions* tables. Therefore, component amplification factors from either these tables or a dynamic analysis may be used. Values for a_p are tabulated for each component based on the expectation that the component will behave in either a rigid or a flexible manner. For simplicity, a step function increase based on input motion amplifications is provided to help distinguish between rigid and flexible behavior. If the fundamental period of the component is less than 0.06 seconds, no dynamic amplification is expected and a_p may be taken to equal 1.00. If the fundamental period of the component is greater than 0.06 seconds, dynamic amplification is expected, and a_p is taken to equal 2.50. In addition, a rational analysis determination of

a_p is permitted if reasonable values of both T and T_p are available. Acceptable procedures for determining a_p are provided in *Commentary* Chapter 6.

13.1.5 Seismic Coefficient at Grade

The short period design spectral acceleration, S_{DS}, considers the site seismicity and local soil conditions. The site seismicity is obtained from the design value maps (or CD-ROM), and S_{DS} is determined in accordance with *Provisions* Sec. 4.1.2.5 [3.3.3]. The coefficient S_{DS} is the used to design the structure. The *Provisions* approximates the effective peak ground acceleration as $0.4S_{DS}$, which is why 0.4 appears in *Provisions* Eq. 6.1.3-1 [6.2-1].

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

13.1.6 Relative Location Factor

The relative location factor, $\left(1+2\frac{z}{h}\right)$, scales the seismic coefficient at grade, resulting in values linearly varying from 1.0 at grade to 3.0 at roof level. This factor approximates the dynamic amplification of ground acceleration by the supporting structure.

13.1.7 Component Response Modification Factor

The component response modification factor, R_p, represents the energy absorption capability of the component's construction and attachments. In the absence of applicable research, these factors are based on judgment with respect to the following benchmark values:

1. R_p = 1.0 or 1.5, brittle or buckling failure mode is expected
2. R_p = 2.5, some minimal level of energy dissipation capacity
3. R_p = 3.5 or 5.0, highly ductile materials and detailing

13.1.8 Component Importance Factor

The component importance factor, I_p, represents the greater of the life safety importance and/or the hazard exposure importance of the component. The factor indirectly accounts for the functionality of the component or structure by requiring design for a lesser amount of inelastic behavior (or higher force level). It is assumed that a lesser amount of inelastic behavior will result in a component that will have a higher likelihood of functioning after a major earthquake.

13.1.9 Accommodation of Seismic Relative Displacements

The *Provisions* requires that seismic relative displacements, D_p, be determined in accordance with several equations. For two connection points on Structure A (or on the same structural system), one at Level x and the other at Level y, D_p is determined from *Provisions* Eq. 6.1.4-1 [6.2-5] as:

$$D_p = \delta_{xA} - \delta_{yA}$$

Because the computed displacements are frequently not available to the designer of nonstructural components, one may use the maximum permissible structural displacements per *Provisions* Eq. 6.1.4-2 [6.2-6]:

$$D_{p_{max}} = (X - Y)\frac{\Delta_{aA}}{h_{sx}}$$

For two connection points on Structures A and B (or on two separate structural systems), one at Level *x*, and the other at Level *y*, D_P and D_{Pmax} are determined from *Provisions* Eq. 6.1.4-3 [6.2-7] and 6.1.4-4 [6.2-8], respectively, as:

$$D_p = |\delta_{xA}| + |\delta_{yB}|$$

$$D_{p_{max}} = \frac{X\Delta_{aA}}{h_{sx}} + \frac{Y\Delta_{aB}}{h_{sx}}$$

where:

D_p = seismic relative displacement that the component must be designed to accommodate

δ_{xA} = deflection of building Level *x* of Structure A, determined by an elastic analysis as defined in *Provisions* Sec. 5.4.6.1 or 5.5.5 [5.2.6, 5.3.5, or 5.4.3] and multiplied by the C_d factor

δ_{yA} = deflection of building Level *y* of Structure A, determined in the same fashion as δ_{xA}

X = height of upper support attachment at Level *x* as measured from the base

Y = height of lower support attachment at Level *y* as measured from the base

Δ_{aA} = allowable story drift for Structure A as defined in *Provisions* Table 5.2.8 [4.5-1]

h_{sx} = story height used in the definition of the allowable drift, Δ_a, in *Provisions* Table 5.2.8 [4.5-1]

δ_{yB} = deflection of building Level *y* of Structure B, determined in the same fashion as δ_{xA}

Δ_{aB} = allowable story drift for Structure B as defined in *Provisions* Table 5.2.8 [4.5-1]

The effects of seismic relative displacements must be considered in combination with displacements caused by other loads as appropriate. Specific methods for evaluating seismic relative displacement effects of components and associated acceptance criteria are not specified in the *Provisions*. However, the intention is to satisfy the purpose of the *Provisions*. Therefore, for nonessential facilities, nonstructural components can experience serious damage during the design level earthquake provided they do not constitute a serious life safety hazard. For essential facilities, nonstructural components can experience some damage or inelastic deformation during the design level earthquake provided they do not significantly impair the function of the facility.

13.1.10 Component Anchorage Factors and Acceptance Criteria

Design seismic forces in the connected parts, F_p, are prescribed in *Provisions* Sec. 6.1.3 [6.2.6].

When component anchorage is provided by expansion anchors or shallow anchors, a value of R_p = 1.5 is used. Shallow anchors are defined as those with embedment length-to-diameter ratios of less than 8. Anchors embedded in concrete or masonry are proportioned to carry the least of the following:

1. The design strength of the connected part

2. 1.3 times the prescribed seismic design force or
3. The maximum force that can be transferred to the connected part by the component structural system

Determination of design seismic forces in anchors must consider installation eccentricities, prying effects, multiple anchor effects, and the stiffness of the connected system.

Use of powder-driven fasteners is not permitted for seismic design tension forces in Seismic Design Categories D, E, and F unless approved for such loading.

[In the 2003 *Provisions* reference is made to “power-actuated” fasteners so as to cover a broader range of fastener types than is implied by “powder-driven.”]

The design strength of anchors in concrete is determined in accordance with, *Provisions* Chapter 9, which is basically the same as IBC Sec. 1913, and has been updated somewhat in Appendix D of ACI 318-2002. (These rules for anchors in concrete will probably be deleted from the next edition of the *Provisions* in favor of a reference to ACI 318.)

[The 2003 *Provisions* refer to Appendix D of ACI 318-02 rather than providing specific, detailed requirements.]

13.1.11 Construction Documents

Construction documents must be prepared by a registered design professional and must include sufficient detail for use by the owner, building officials, contractors, and special inspectors; *Provisions* Table 6.1.7 [6.2-1] includes specific requirements.

13.2 ARCHITECTURAL CONCRETE WALL PANEL

13.2.1 Example Description

In this example, the architectural components are a 4.5-in.-thick precast normal weight concrete spandrel panel and a column cover supported by the structural steel frame of a five-story building as shown in Figures 13.2-1 and 13.2-2.

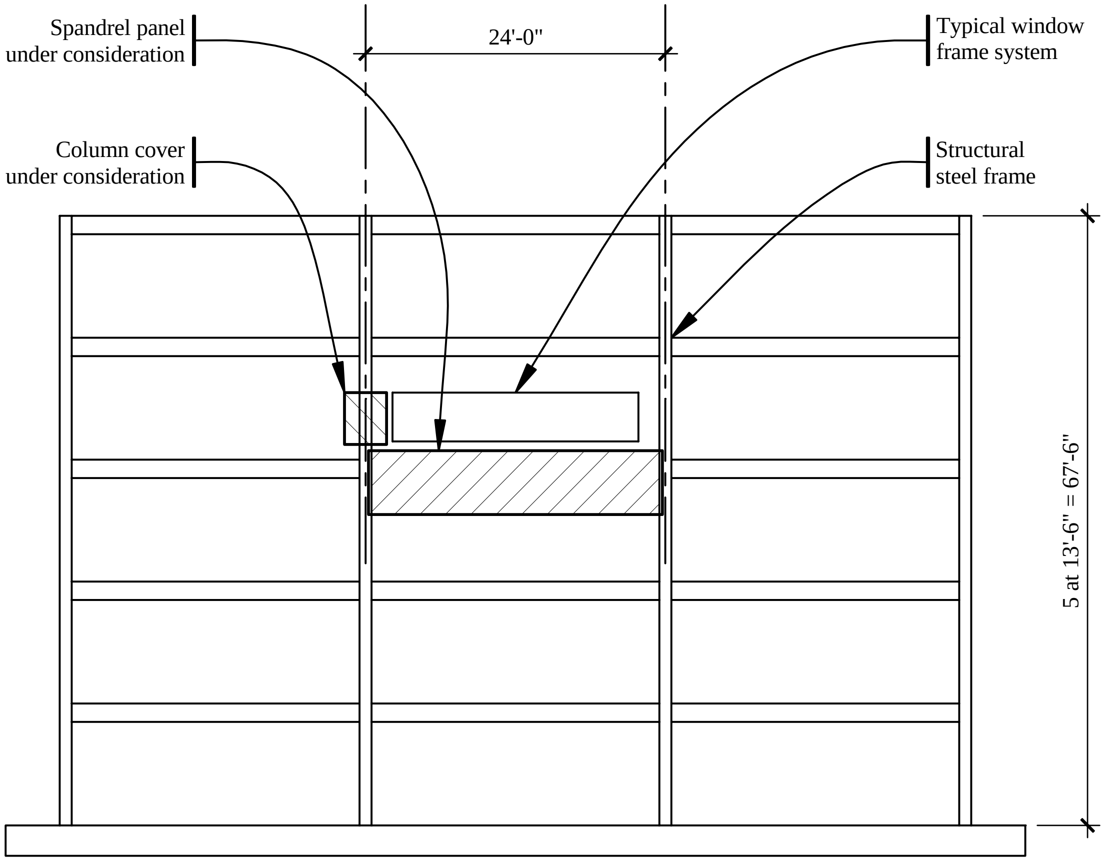

Figure 13.2-1 Five-story building elevation showing panel location (1.0 ft = 0.3048 m)

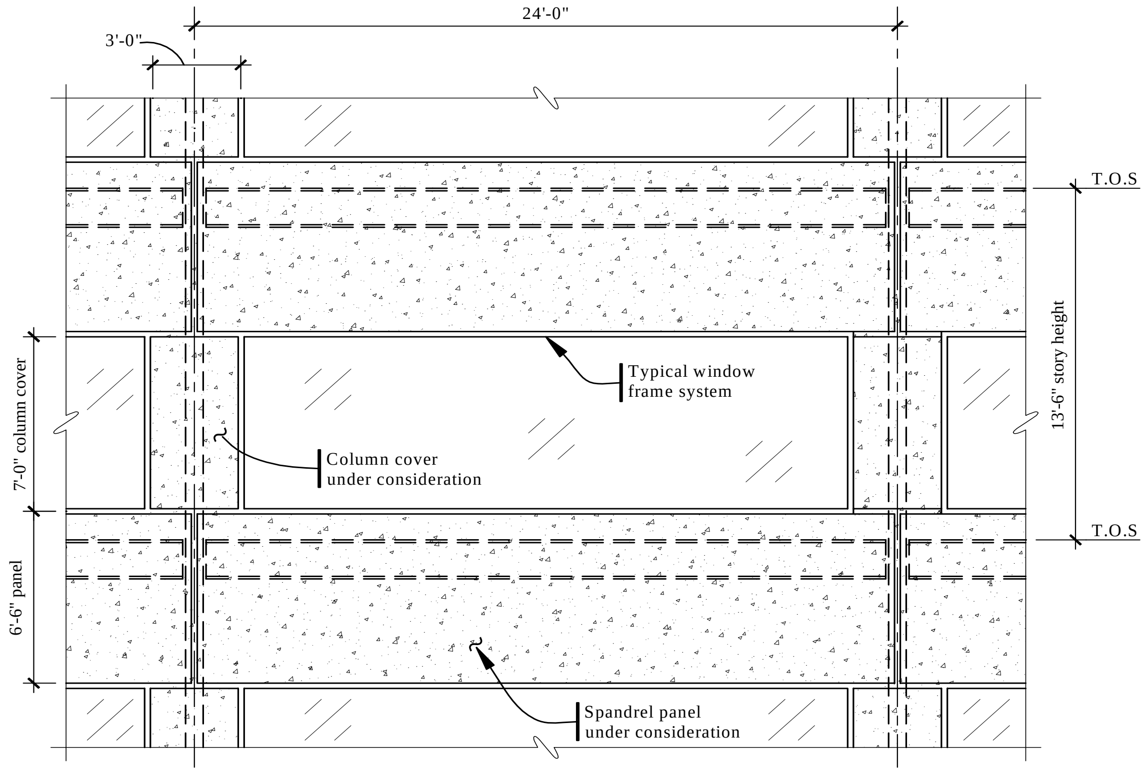

Figure 13.2-2 Detailed building elevation (1.0 ft = 0.3048 m).

The columns, at the third level of the five-story office building, support the spandrel panel under consideration. The columns between the third and fourth levels of the building support the column cover under consideration. The building, located near a significant active fault in Los Angeles, California, is assigned to Seismic Use Group I. Wind pressures normal to the building are 17 psf determined in accordance with ASCE 7. The spandrel panel supports glass windows weighing 10 psf.

This example develops prescribed seismic forces for the selected spandrel panel and prescribed seismic displacements for the selected column cover.

It should be noted that details of precast connections vary according to the preferences and local practices of the precast panel supplier. In addition, some connections may involve patented designs. As a result, this example will concentrate on quantifying the prescribed seismic forces and displacements. After the prescribed seismic forces and displacements are determined, the connections can be detailed and designed according to the appropriate AISC and ACI codes and PCI (Precast/Prestressed Concrete Institute) recommendations.

13.2.2 Design Requirements

13.2.2.1 *Provisions* Parameters and Coefficients

a_p = 1.0 for wall panels (*Provisions* Table 6.2.2 [6.3-1])

a_p = 1.25 for connection fasteners (*Provisions* Table 6.2.2 [6.3-1])

S_{DS} = 1.487 (Design Values CD-ROM for the selected location and site class)

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

Seismic Design Category = D (*Provisions* Table 4.2.1a [1.4-1] for S_1 < 0.75)

[In the footnote to 2003 *Provisions* Table 1.4-1, the value of S_1 used to trigger assignment to Seismic Design Category E or F was changed from 0.75 to 0.6.]

Spandrel Panel W_p = (150 lb/ft^3)(24 ft)(6.5 ft)(0.375 ft) = 8775 lb

Glass W_p = (10 lb/ft^2)(21 ft)(7 ft) = 1470 lb (supported by spandrel panel)

Column Cover W_p = (150 lb/ft^3)(3 ft)(7 ft)(0.375 ft) = 1181 lb

R_p = 2.5 for wall panels (*Provisions* Table 6.3.2 [6.3-1])

R_p = 1.0 for connection fasteners ((*Provisions* Sec. 6.1.6.1 [6.2.8.1])

[In the 2003 *Provisions* component anchorage is designed using R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

I_p = 1.0 (*Provisions* Sec. 6.1.5 [6.2.2])

$$\frac{z}{h} = \frac{40.5\text{ ft}}{67.5\text{ ft}} = 0.6$$ (at third floor)

ρ = 1.0 (*Provisions* Sec. 6.1.3)

[The 2003 *Provisions* indicate that the redundancy factor does not apply to the design of nonstructural components. Although the effect is similar to stating that ρ = 1, there is a real difference since load effects for such components and their supports and attachments are now defined in Chapter 6 rather than by reference to Chapter 4.]

13.2.2.2 Performance Criteria

Component failure should not cause failure of an essential architectural, mechanical, or electrical component (*Provisions* Sec. 6.1 [6.2.3]).

Component seismic attachments must be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (*Provisions* Sec. 6.1.2 [6.2.5]).

The effects of seismic relative displacements must be considered in combination with displacements caused by other loads as appropriate (*Provisions* Sec. 6.1.4 [6.2.7]).

Exterior nonstructural wall panels that are attached to or enclose the structure shall be designed to resist the forces in accordance with *Provisions* Eq. 6.1.3-1 or 6.1.3-2 [6.2.6] and must be able to accommodate movements of the structure resulting from response to the design basis ground motion, D_p, or temperature changes (*Provisions* Sec. 6.2.4 [6.3.2]).

13.2.3 Spandrel Panel

13.2.3.1 Connection Details

Figure 13.2-3 shows the types and locations of connections that support one spandrel panel.

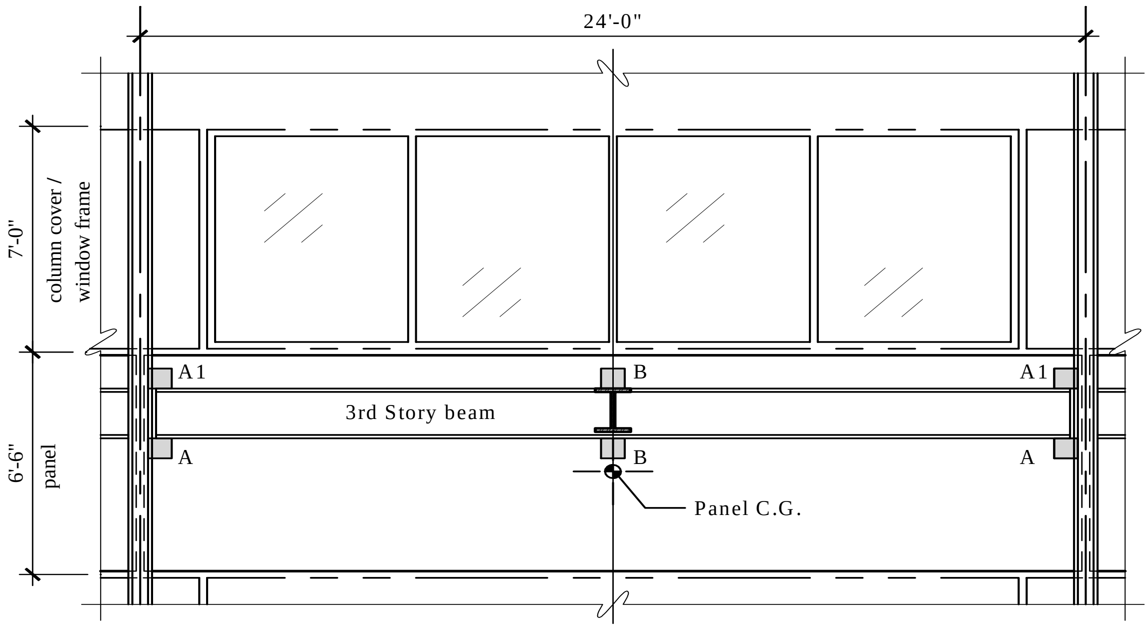

Figure 13.2-3 Spandrel panel connection layout from interior (1.0 ft = 0.3048 m).

The connection system must resist the weight of the panel and supported construction including the eccentricity between that load and the supports as well as inertial forces generated by response to the seismic motions in all three dimensions. Furthermore, the connection system must not create undue interaction between the structural frame and the panel, such as restraint of the natural shrinkage of the panel or the transfer of floor live load from the beam to the panel. The panels are usually very stiff compared to the frame, and this requires careful release of potential constraints at connections. PCI's *Architectural Precast Concrete* (2nd Ed. 1989), provides an extended discussion of important design concepts for such panels.

For this example, the basic gravity load, and vertical accelerations are resisted at Points A, which provides the recommended simple and statically determinant system for the main gravity weight. The eccentricity of vertical loads is resisted by a force couple at the two pairs of A1 and A connections. Horizontal loads parallel to the panel are resisted by the A connections. Horizontal loads perpendicular to the panel are resisted by all six connections. The A connections, therefore, restrain movement in three dimensions while the A1 and B connections restrain movement in only one dimension, perpendicular to the panel. Connection components can be designed to resolve some eccentricities by bending of the element; for example, the eccentricity of the horizontal in-plane force with the structural frame can be resisted by bending the A connection.

The practice of resisting the horizontal in-plane force at two points varies with seismic demand and local industry practice. The option is to resist all of the in-plane horizontal force at one connection in order to avoid restraint of panel shrinkage. The choice made here depends on local experience indicating that precast panels of this length have been restrained at the two ends without undue shrinkage restraint problems.

The A and A1 connections are often designed to take the loads directly to the columns, particularly on steel moment frames where attachments to the flexural hinging regions of beams are difficult to accomplish. The lower B connection often require an intersecting beam to provide sufficient stiffness and strength to resist the loads.

The column cover is supported both vertically and horizontally by the column, transfers no loads to the spandrel panel, and provides no support for the window frame.

The window frame is supported both vertically and horizontally along the length of the spandrel panel and transfers no loads to the column covers.

13.2.3.2 Prescribed Seismic Forces

Lateral forces on the wall panels and connection fasteners include seismic loads in accordance with the *Provisions* and wind loads in accordance with ASCE 7 as indicated in the problem statement. Wind forces are not illustrated here.

13.2.3.2.1 Panels

$D = W_p$ = 8775 lb + 1470 lb = 10245 lb (vertical gravity effect)

$$F_p = \frac{0.4(1.0)(1.487)(10245\text{ lb})}{\left(2.5/1.0\right)}\left(1+2(0.6)\right) = 5362\text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$F_{p_{max}} = 1.6(1.487)(1.0)(10245\text{ lb}) = 24375\text{ lb}$ (*Provisions* Eq. 6.1.3-2 [6.2-3])

$F_{p_{min}} = 0.3(1.487)(1.0)(10245\text{ lb}) = 4570\text{ lb}$ (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following five steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 5362 lb (*Provisions* Sec. 6.1.3)

ρQ_E = (1.0)(5362 lb) = 5362 lb (horizontal earthquake effect)

$0.2S_{DS}D$ = (0.2)(1.487)(10245 lb) = 3047 lb (vertical earthquake effect)

$E = \rho Q_E + 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

13.2.3.2.2 Connection Fasteners

The *Provisions* specifies a reduced R_P, and an increased a_P for "Fasteners," which is intended to prevent premature failure in those elements of connections that are inherently brittle, such as embedments that depend on concrete breakout strength, or are simply too small to adequately dissipate energy inelastically, such as welds or bolts. The net effect more than triples the design seismic force.

$$F_p = \frac{0.4(1.25)(1.487)(10245\text{ lb})}{\left(1.0/1.0\right)}\left(1+2(0.6)\right) = 16757\text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$F_{p_{max}} = 1.6(1.487)(1.0)(10245 \text{ lb}) = 24375 \text{ lb}$ (*Provisions* Eq. 6.1.3-2 [6.2-3])

$F_{p_{min}} = 0.3(1.487)(1.0)(10245 \text{ lb}) = 4570 \text{ lb}$ (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following five steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 16757 lb (*Provisions* Sec. 6.1.3)

$\rho Q_E = (1.0)(16757 \text{ lb}) = 16757 \text{ lb}$ (horizontal earthquake effect)

$0.2S_{DS}D = (0.2)(1.487)(10245 \text{ lb}) = 3047 \text{ lb}$ (vertical earthquake effect)

$E = \rho Q_E + 0.2\text{S}_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2\text{S}_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

13.2.3.3 Proportioning and Design

13.2.3.3.1 Panels

The wall panels should be designed for the following loads in accordance with ACI 318. The design of the reinforced concrete panel is standard and is not illustrated in this example. Spandrel panel moments are shown in Figure 13.2-4. Reaction shears (V_u), forces (H_u), and moments (M_u) are calculated for applicable strength load combinations.

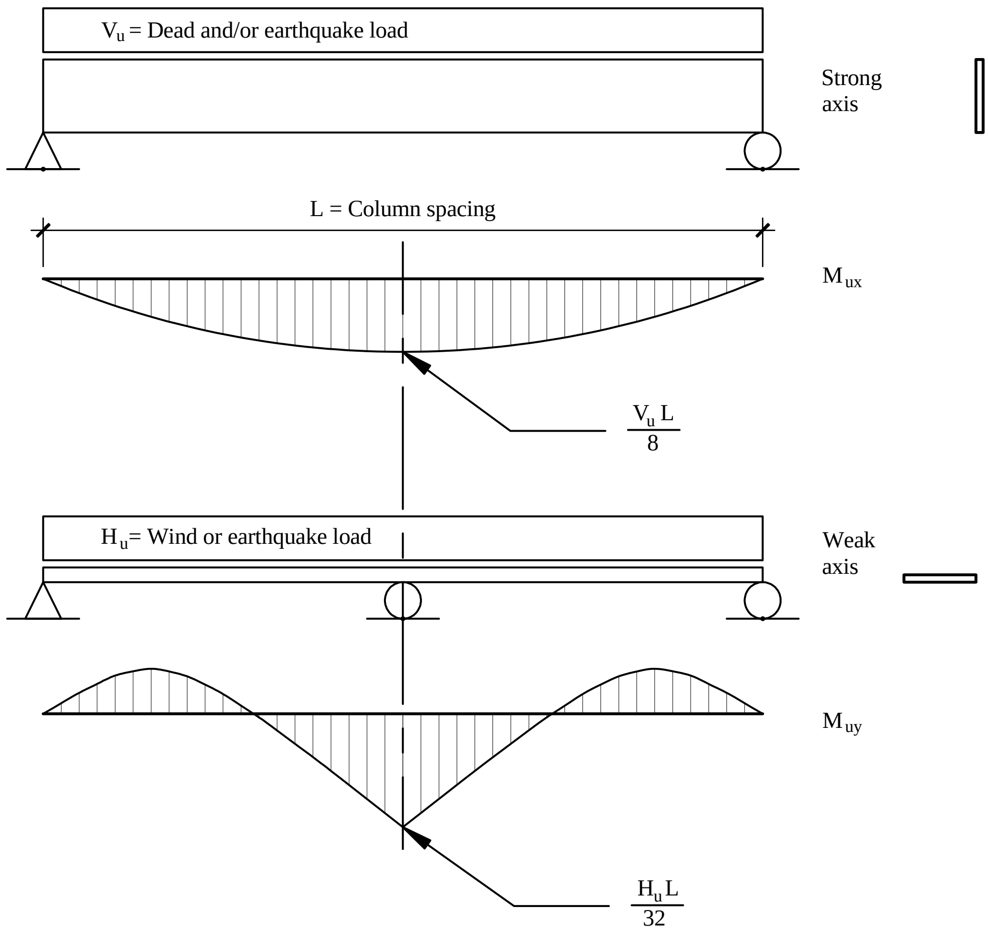

Figure 13.2-4 Spandrel panel moments.

<u>U = 1.4D</u>

V_u = 1.4(10245 lb) = 14343 lb (vertical load downward)

$$M_{ux} = \frac{(14343 \text{ lb})(24 \text{ ft})}{8} = 43029 \text{ ft-lb}$$ (strong axis moment)

<u>U = 1.2D + 1.0E</u>

$V_{u_{max}} = 1.2(10245 \text{ lb}) + 1.0(3047 \text{ lb}) = 15341 \text{ lb}$ (vertical load downward)

$\Leftrightarrow H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load parallel to panel)

$\perp H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load perpendicular to panel)

$$M_{ux_{max}} = \frac{(15341 \text{ lb})(24 \text{ ft})}{8} = 46023 \text{ ft-lb}$$ (strong axis moment)

$$M_{uy} = \frac{(5362 \text{ lb})(24 \text{ ft})}{32} = 4022 \text{ ft-lb}$$ (weak axis moment)

<u>U = 0.9D + 1.0E</u>

$V_{u_{min}} = 0.9(10245 \text{ lb}) - 1.0(3047 \text{ lb}) = 6174 \text{ lb}$ (vertical load downward)

$\Leftrightarrow H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load parallel to panel)

$\perp H_u = 1.0(5362 \text{ lb}) = 5362 \text{ lb}$ (horizontal load perpendicular to panel)

$$M_{ux_{min}} = \frac{(6174 \text{ lb})(24 \text{ ft})}{8} = 18522 \text{ ft-lb}$$ (strong axis moment)

$$M_{uy} = \frac{(5362 \text{ lb})(24 \text{ ft})}{32} = 4022 \text{ ft-lb}$$ (weak axis moment)

13.2.3.3.2 Connection Fasteners

The connection fasteners should be designed for the following loads in accordance with ACI 318-2002 (Appendix D) and the AISC specification. There are special reduction factors for anchorage in high seismic demand locations, and the parameters for this project would invoke those reduction factors. The design of the connection fasteners is not illustrated in this example. Spandrel panel connection forces are shown in Figure 13.2-5. Reaction shears (V_u), forces (H_u), and moments (M_u) are calculated for applicable strength load combinations.

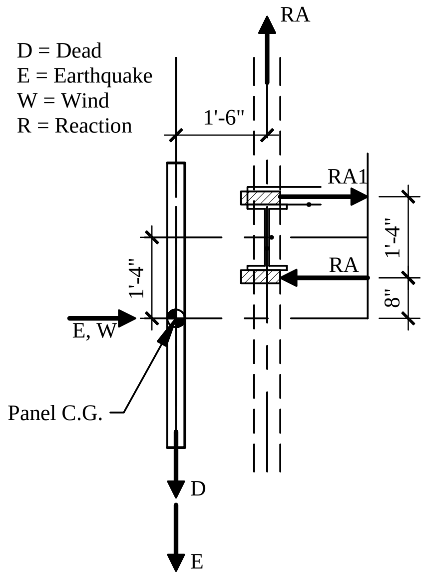

Figure 13.2-5 Spandrel panel connection forces.

<u>U = 1.4D</u>

$$V_{uA} = \frac{1.4(10245\ \text{lb})}{2} = 7172\ \text{lb}$$ (vertical load downward at Point A and A1)

M_{uA} = (7172 lb)(1.5 ft) = 10758 ft-lb (moment resisted by paired Points A and A1)

Horizontal couple from moment at A and A1 = 10758 / 1.33 = 8071 lb

<u>U = 1.2D + 1.0E</u>

$$V_{uA_{max}} = \frac{1.2(10245\ \text{lb}) + 1.0(3047\ \text{lb})}{2} = 7671\ \text{lb}$$ (vertical load downward at Point A)

$$\perp H_{uA} = 1.0(16757\ \text{lb})\frac{3}{16} = 3142\ \text{lb}$$ (horizontal load perpendicular to panel at Points A and A1)

H_{Ain} = (7671 lb)(1.5 ft) / (1.33 ft) + (3142 lb)(2.0 ft) / (1.33 ft) = 13366 lb (inward force at Point A)

H_{A1out} = (7671 lb)(1.5 ft) / (1.33 ft) + (3142 lb)(0.67) / (1.33 ft) = 10222 lb(outward force at Point A1)

$$\Leftrightarrow H_{uA} = \frac{1.0(16757\ \text{lb})}{2} = 8378\ \text{lb}$$ (horizontal load parallel to panel at Point A)

M_{u2A} = (8378 lb)(1.5 ft) = 12568 ft-lb (flexural moment at Point A)

$$\perp H_{uB} = 1.0(16757\ \text{lb})\frac{5}{8} = 10473\ \text{lb}$$ (horizontal load perpendicular to panel at Points B and B1)

H_B = (10743 lb)(2.0 ft) / (1.33 ft) = 15714 lb (inward or outward force at B)

H_{B1} = (10473 lb)(0.67 ft) / (1.33 ft) = 5237 lb (inward or outward force at B1)

U = 0.9D + 1.0E

$$V_{uA_{min}} = \frac{0.9(10245 \text{ lb}) - 1.0(3047 \text{ lb})}{2} = 3086 \text{ lb}$$ (vertical load downward at Point A)

Horizontal forces are the same as combination 1.2 D + 1.0 E. No uplift occurs; the net reaction at A is downward. Maximum forces are controlled by prior combination. It is important to realize that inward and outward acting horizontal forces generate different demands when the connections are eccentric to the center of mass as it is in this example. Only the maximum reactions are computed above.

13.2.3.4 Prescribed Seismic Displacements

Prescribed seismic displacements are not applicable to the building panel because all connections are essentially at the same elevation.

13.2.4 Column Cover

13.2.4.1 Connection Details

Figure 13.2-6 shows the key to the types of forces resisted at each column cover connection.

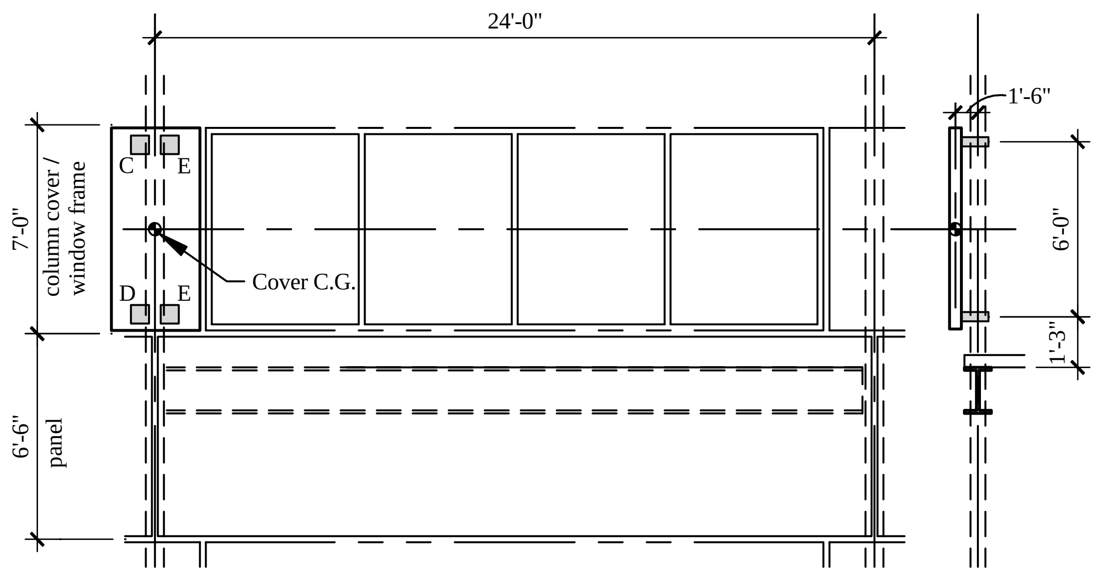

Figure 13.2-6 Column cover connection layout (1.0 ft = 0.3048 m).

Vertical loads, horizontal loads parallel to the panel, and horizontal loads perpendicular to the panel are resisted at Point C. The eccentricity of vertical loads is resisted by a force couple at Points C and D. The horizontal load parallel to the panel eccentricity between the panel and the support is resisted in flexure of the connection. The connection is designed to take the loads directly to the columns.

Horizontal loads parallel to the panel and horizontal loads perpendicular to the panel are resisted at Point D. The vertical load eccentricity between the panel and the support is resisted by a force couple of Points

C and D. The eccentricity of horizontal loads parallel to the panel is resisted by flexure at the connection. The connection must not restrict vertical movement of the panel due to thermal effects or seismic input. The connection is designed to take the loads directly to the columns.

Horizontal loads perpendicular to the panel are resisted at Points E. The connection is designed to take the loads directly to the columns.

There is no load eccentricity associated with the horizontal loads perpendicular to the panel.

In this example, all connections are made to the sides of the column because there usually is not enough room between the outside face of the column and the inside face of the cover to allow a feasible load-carrying connection.

13.2.4.2 Prescribed Seismic Forces

Calculation of prescribed seismic forces for the column cover are not shown in this example. They should be determined in the same manner as illustrated for the spandrel panels.

13.2.4.3 Prescribed Seismic Displacements

The results of an elastic analysis of the building structure are not usually available in time for use in the design of the precast cladding system. As a result, prescribed seismic displacements are usually calculated based on allowable story drift requirements:

h_{sx} = story height = 13 ft 6 in

X = height of upper support attachment = 47 ft 9 in

Y = height of lower support attachment = 41 ft 9 in

$\Delta_a = 0.020 h_{sx}$ (*Provisions* Table 5.2.8 [4.5-1])

$$D_{p_{max}} = (X - Y)\frac{\Delta_a}{h_{sx}} = (72 \text{ in.})\frac{0.020 h_{sx}}{h_{sx}} = 1.44 \text{ in.}$$ (*Provisions* Eq. 6.1.4-2 [6.2-6])

The joints at the top and bottom of the column cover must be designed to accommodate an in-plane relative displacement of 1.44 inches. The column cover will rotate somewhat as these displacements occur, depending on the nature of the connections to the column. If the supports at one level are “fixed” to the columns while the other level is designed to “float,” then the rotation will be that of the column at the point of attachment.

13.2.5 Additional Design Considerations

13.2.5.1 Window Frame System

The window frame system is supported by the spandrel panels above and below. Assuming that the spandrel panels move rigidly in-plane with each floor level, the window frame system must accommodate a prescribed seismic displacement based on the full story height.

$$D_{p_{max}} = (X - Y)\frac{\Delta_a}{h_{sx}} = (162 \text{ in.})\frac{0.020 h_{sx}}{h_{sx}} = 3.24 \text{ in.}$$ (*Provisions* Eq. 6.1.4-2 [6.2-6])

The window frame system must be designed to accommodate an in-plane relative displacement of 3.24 in. between the supports. This is normally accommodated by a clearance between the glass and the frame. *Provisions* Sec. 6.2.10.1 [6.3.7], prescribes a method of checking such a clearance. It requires that the clearance be large enough so that the glass panel will not fall out of the frame unless the relative seismic displacement at the top and bottom of the panel exceeds 125 percent of the value predicted amplified by the building importance factor. If h_p and b_p are the respective height and width of individual panes and if the horizontal and vertical clearances are designated c_1 and c_2, respectively, then the following expression applies:

$$D_{clear} = 2c_1\left(1+\frac{h_p c_2}{b_p c_1}\right) \geq 1.25 D_p$$

For h_p = 7 ft, b_p = 5 ft, and D_p = 3.24 in., and setting $c_1 = c_2$, the required clearance is 0.84 in.

13.2.5.2 Building Corners

Some thought needs to be given to seismic behavior at external building corners. The preferred approach is to detail the corners with two separate panel pieces, mitered at a 45 degree angle, with high grade sealant between the sections. An alternative choice of detailing L-shaped corner pieces, would introduce more seismic mass and load eccentricity into connections on both sides of the corner column.

13.2.5.3 Dimensional Coordination

It is important to coordinate dimensions with the architect and structural engineer. Precast concrete panels must be located a sufficient distance from the building structural frame to allow room for the design of efficient load transfer connection pieces. However, distances must not be so large as to unnecessarily increase the load eccentricities between the panels and the frame.

13.3 HVAC FAN UNIT SUPPORT

13.3.1 Example Description

In this example, the mechanical component is a 4-ft-high, 5-ft-wide, 8-ft-long, 3000-lb HVAC fan unit that is supported on the two long sides near each corner (Figure 13.3-1). The component is located at the roof level of a five-story office building, near a significant active fault in Los Angeles, California. The building is assigned to Seismic Use Group I. Two methods of attaching the component to the 4,000 psi, normal-weight roof slab, are considered as follows:

1. Direct attachment to the structure with 36 ksi, carbon steel, cast-in-place anchors and
2. Support on vibration isolation springs, that are attached to the slab with 36 ksi carbon steel post-installed expansion anchors.

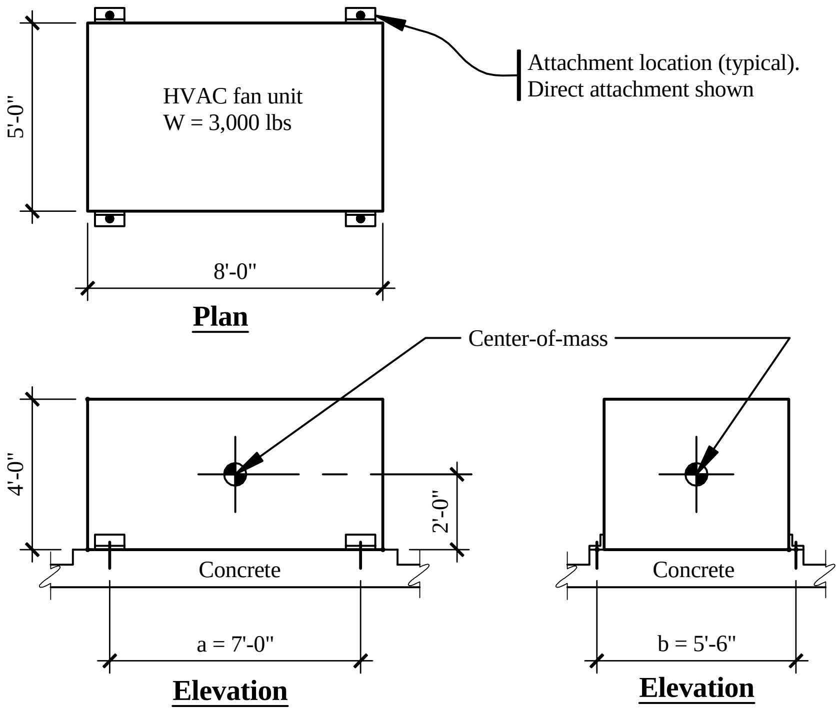

Figure 13.3-1 Air handling fan unit (1.0 ft = 0.3048 m, 1.0 lb = 4.45 N).

13.3.2 Design Requirements

13.3.2.1 *Provisions* Parameters and Coefficients

a_p = 1.0 for direct attachment (*Provisions* Table 6.3.2 [6.4-1])

a_p = 2.5 for vibration isolated (*Provisions* Table 6.3.2 [6.4-1])

S_{DS} = 1.487 (Design Values CD-ROM)

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

Seismic Design Category = D (*Provisions* Table 4.2.1a [14.4-1])

W_p = 3000 lb (given)

R_p = 2.5 for HVAC system equipment (*Provisions* Table 6.3.2 [6.4-1])

R_p = 1.5 for expansion anchors and shallow, cast-in-place anchors* ((*Provisions* Sec. 6.1.6.1 [6.2.8.1])

Shallow anchors are defined by *Provisions* Sec. 2.1 as anchors having embedment-to-diameter ratios of less than 8.

[In the 2003 *Provisions* component anchorage is treated differently. Rather than making distinctions based on an anchor being "shallow," component anchorage is designed using R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

$I_p = 1.0$ (*Provisions* Sec. 6.1.5 [6.2.2])

$z/h = 1.0$ (for roof mounted equipment)

$\rho = 1.0$ (*Provisions* Sec. 6.1.3 [6.2.6])

[The 2003 *Provisions* indicate that the redundancy factor does not apply to the design of nonstructural components. Although the effect is similar to stating that $\rho = 1$, there is a real difference since load effects for such components and their supports and attachments are now defined in Chapter 6 rather than by reference to Chapter 4.]

13.3.2.2 Performance Criteria

Component failure should not cause failure of an essential architectural, mechanical, or electrical component (*Provisions* Sec. 6.1 [6.2.3]).

Component seismic attachments must be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity (*Provisions* Sec. 6.1.2 [6.2.5]).

Anchors embedded in concrete or masonry must be proportioned to carry the least of: (a) the design strength of the connected part, (b) 1.3 times the force in the connected part due to the prescribed forces, or (c) the maximum force that can be transferred to the connected part by the component structural system (*Provisions* Sec. 6.1.6.2 [6.2.8.2]).

Attachments and supports transferring seismic loads must be constructed of materials suitable for the application and must be designed and constructed in accordance with a nationally recognized structural standard (*Provisions* Sec. 6.3.13.2.a [6.4.4, item 6]).

Components mounted on vibration isolation systems must have a bumper restraint or snubber in each horizontal direction. Vertical restraints must be provided where required to resist overturning. Isolator housings and restraints must also be constructed of ductile materials. A viscoelastic pad, or similar material of appropriate thickness, must be used between the bumper and equipment item to limit the impact load (*Provisions* Sec. 6.3.13.2.e). Such components must also resist an amplified design force.

13.3.3 Direct Attachment to Structure

This section illustrates design for cast-in-place concrete anchors with embedment-length-to-diameters ratios of 8 or greater; thus, the use of R_p = 2.5 is permitted. [In 2003 *Provisions* Sec. 6.2.8.1, the value of R_p used in designing component anchorage is no longer based on the embedment depth-to-diameter ratio. Instead R_p = 1.5 unless specific ductility or prequalification requirements are satisfied.]

13.3.3.1 Prescribed Seismic Forces

See Figure 13.3-2 for freebody diagram for seismic force analysis.

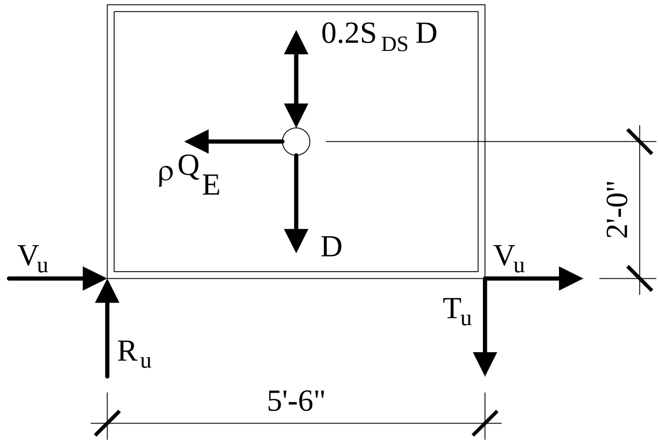

Figure 13.3-2 Free-body diagram for seismic force analysis (1.0 ft = 0.348 m).

$$F_p = \frac{0.4(1.0)(1.487)(3000\text{ lb})}{\left(2.5/1.0\right)}\left(1+2(1)\right) = 2141\text{ lb}$$ (*Provisions* Eq. 6.1.3-1 [6.2-1])

$$F_{p_{max}} = 1.6(1.487)(1.0)(3000\text{ lb}) = 7138\text{ lb}$$ (*Provisions* Eq. 6.1.3-2 [6.2-3])

$$F_{p_{min}} = 0.3(1.487)(1.0)(3000\text{ lb}) = 1338\text{ lb}$$ (*Provisions* Eq. 6.1.3-3 [6.2-4])

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following six steps would be cast differently; the result is the same.]

Q_E (due to application of F_p) = 2141 lb (*Provisions* Sec. 6.1.3)

ρQ_E = (1.0)(2141 lb) = 2141 lb (horizontal earthquake effect)

$0.2S_{DS}D$ = (0.2)(1.487)(3000 lb) = 892 lb (vertical earthquake effect)

$D = W_p$ = 3000 lb (vertical gravity effect)

$E = \rho Q_E + 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (*Provisions* Eq. 5.2.7.1-2 [4.2-2])

U = 1.2D + 1.0E + 0.5L + 0.2S

$$V_u = \frac{1.0(2141\text{ lb})}{4\text{ bolts}} = 535\text{ lb/bolt}$$

$$T_u = \frac{-1.2(3000\text{ lb})(2.75\text{ ft}) + 1.0(2141\text{ lb})(2\text{ ft}) + 1.0(892\text{ lb})(2.75\text{ ft})}{(5.5\text{ ft})(2\text{ bolts})} = -288\text{ lb/bolt}$$ no tension

U = 0.9D - (1.3W or 1.0E)

$$V_u = \frac{1.0(2141\text{ lb})}{4\text{ bolts}} = 535\text{ lb/bolt}$$

$$T_u = \frac{-0.9(3000\text{ lb})(2.75\text{ ft}) + 1.0(2141\text{ lb})(2\text{ ft}) + 1.0(892\text{ lb})(2.75\text{ ft})}{(5.5\text{ ft})(2\text{ bolts})} = -63\text{ lb/bolt}$$ <u>no tension</u>

13.3.3.2 Proportioning and Design

See Figure 13.3-3 for anchor for direct attachment to structure.

Check one ¼-in.-diameter cast-in-place anchor embedded 2 in. into the concrete slab with no transverse reinforcing engaging the anchor and extending through the failure surface. Although there is no required tension strength on these anchors, design strengths and tension/shear interaction acceptance relationships are calculated to demonstrate the use of the *Provisions* equations.

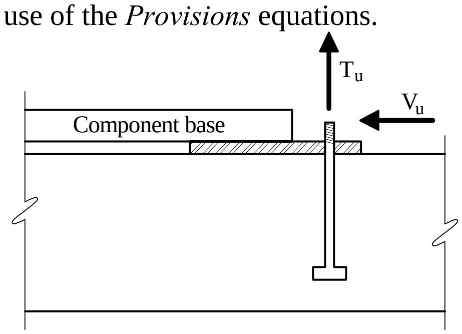

Figure 13.3-3 Anchor for direct attachment to structure.

5.3.3.2.1 Design Tension Strength on Isolated Anchor in Slab, Away from Edge, Loaded Concentrically

[The 2003 *Provisions* refer to Appendix D of ACI 318-02 rather than providing specific, detailed requirements. Note also that some of the resistance factors, ϕ, are different.]

Following *Provisions* Sec. 9.2, for a headed bolt or a rod with a nut at the bottom:

$$\frac{L}{d} = \frac{2\text{ in.}}{0.25\text{ in.}} = 8.0$$ (not shallow, R_p = 2.5 is permitted)

[In 2003 *Provisions* selection of R_p is no longer based on the L/d ratio; see Sec. 6.2.8.1.]

Tension capacity of steel, ϕ = 0.80:

$N_s = A_{se}F_y$ = (0.049 in.^{2})(36000 psi) = 1764 lb (*Provisions* Sec. 9.2.5.1.2 [ACI 318-02 Eq. D-3])

[In Appendix D of ACI 318-02 this capacity calculation is based on f_{ut} rather than F_y, since "the large majority of anchor materials do not exhibit a well-defined yield point."]

Tension capacity of concrete, ϕ = 0.70, with no eccentricity, no pullthrough, and no edge or group effect:

$N_c = k\sqrt{f'_c}h_{ef}^{1.5} = 24\sqrt{4000\text{ psi}}(8^{1.5}) = 4293$ lb (*Provisions* Eq. 9.2.5.2.2-1 [ACI 318-02 Eq. D-7])

[In Appendix D of ACI 318-02 this item is defined as N_b rather than N_c.]

The steel controls but, with no tension demand, the point is moot.

5.3.3.2.2 Design Shear Strength on Isolated Anchor, Away from Edge

Shear capacity of steel, $\phi = 0.80$:

$$V_s = A_{se}F_y = (0.049 \text{ in.}^2)(36000 \text{ psi}) = 1764 \text{ lb} \quad \text{(\textit{Provisions} Eq. 9.2.6.1.2-1 [ACI 318-02 Eq. D-17)}$$

[In Appendix D of ACI 318-02 this capacity calculation is based on f_{ut} rather than F_y, since "the large majority of anchor materials do not exhibit a well-defined yield point."]

Shear capacity of concrete, far from edge, limited to pryout, $\phi = 0.70$:

$$V_{cp} = 2N_c = 2(4293) = 8493 \text{ lb} \qquad \text{(\textit{Provisions} Eq. 9.2.6.3.1 [ACI 318-02 Eq. D-28)}$$

The steel controls, with $\phi V_N = 0.8(1764) = 1411$ lb

Per *Provisions* Sec. 6.1.6.2 [6.2.8.2], anchors embedded in concrete or masonry are to be proportioned to carry at least 1.3 times the force in the connected part due to the prescribed forces. Thus, $V_u = 1.3(535) = 696$ lb and the anchor is clearly adequate.

5.3.3.2.3 Combined Tension and Shear

The *Provisions* gives a new equation (Eq. 9.2.7.3 [ACI 318-02 Eq. D-29]) for the interaction of tension and shear on an anchor or a group of anchors:

$$\frac{N_u}{\phi N_N} + \frac{V_u}{\phi V_N} \leq 1.2 \text{, which applies when either term exceeds 0.2}$$

5.3.3.2.3 Summary

At each corner of the component, provide one ¼-in.-diameter cast-in-place anchor embedded 2 in. into the concrete slab. Transverse reinforcement engaging the anchor and extending through the failure surface is not necessary.

13.3.4 Support on Vibration Isolation Springs

13.3.4.1 Prescribed Seismic Forces

Design forces for vibration isolation springs are determined by an analysis of earthquake forces applied in a diagonal horizontal direction as shown in Figure 13.3-4. Terminology and concept are taken from ASHRAE APP IP.

Angle of diagonal loading:

$$\theta = \tan^{-1}\left(\frac{b}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 17)}$$

Tension per isolator:

$$T_u = \frac{W_p - F_{pv}}{4} - \frac{F_p h}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 18)}$$

Compression per isolator:

$$C_u = \frac{W_p + F_{pv}}{4} + \frac{F_p h}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) \qquad \text{(ASHRAE APP IP. Eq. 19)}$$

Shear per isolator:

$$V_u = \frac{F_p}{4} \qquad \text{(ASHRAE APP IP. Eq. 20)}$$

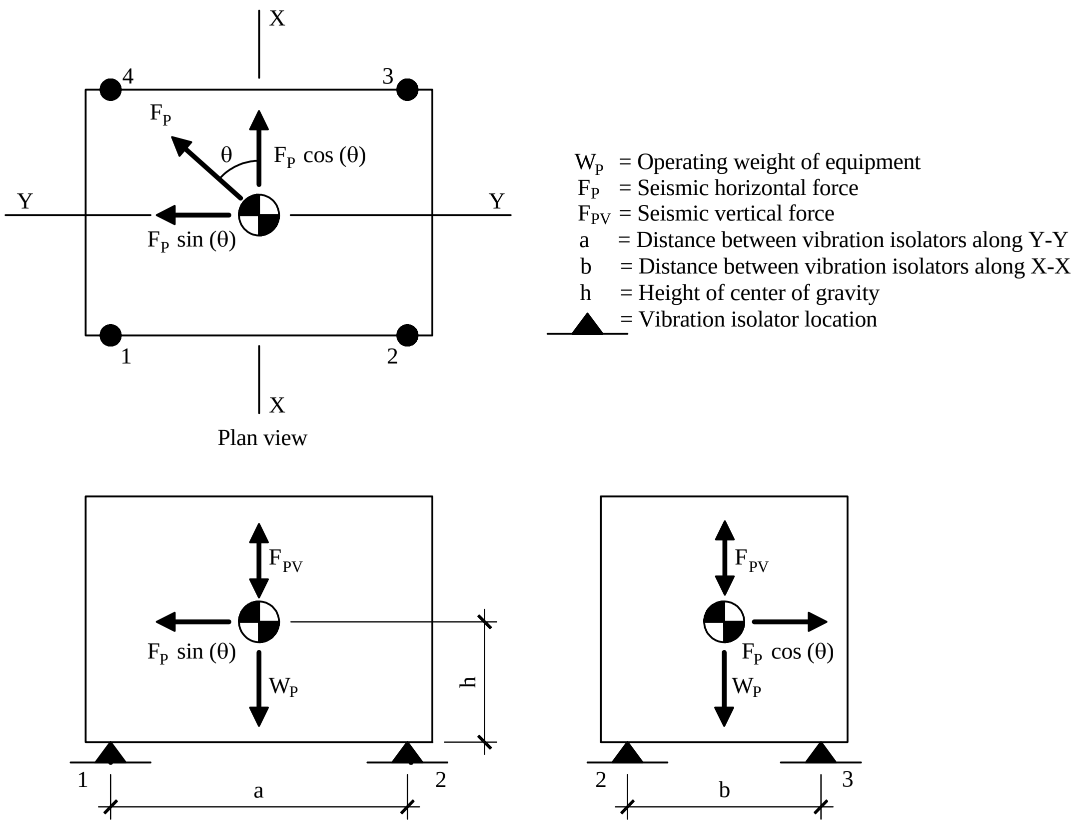

Figure 13.3-4 ASHRAE diagonal seismic force analysis for vibration isolation springs.

Select worst case assumption: Design for post-installed expansion anchors, requiring the use of R_p = 1.5.

$$F_p = \frac{0.4(2.5)(1.487)(3000\text{ lb})}{\left(1.5/1.0\right)}\left(1+2(1)\right) = 8922\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-1 [6.2-1]})$$

$$F_{p_{max}} = 1.6(1.487)(1.0)(3000\text{ lb}) = 7138\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-2 [6.2-3]})$$

$$F_{p_{min}} = 0.3(1.487)(1.0)(3000\text{ lb}) = 1338\text{ lb} \qquad (\textit{Provisions}\text{ Eq. 6.1.3-3 [6.2-4]})$$

Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. Per Provisions Table 6.3.2 [6.4-1], Footnote B, the design force shall be taken as $2F_p$.

[2003 *Provisions* Sec. 6.2.6 now treats load effects differently. The vertical forces that must be considered in design are indicated directly and the redundancy factor does not apply, so the following steps would be cast differently; the result is the same.]

$Q_E = F_p = 2(7138 \text{ lb}) = 14276 \text{ lb}$ (*Provisions* Sec. 6.1.3)

$\rho Q_E = (1.0)(14276 \text{ lb}) = 14276 \text{ lb}$ (horizontal earthquake effect)

$F_{pv(\text{ASHRAE})} = 0.2S_{DS}D = (0.2)(1.487)(3000 \text{ lb}) = 892 \text{ lb}$ (vertical earthquake effect)

$D = W_p = 3000 \text{ lb}$ (vertical gravity effect)

$E = \rho Q_E + 0.2S_{DS}D$ (Eq. 5.2.7.1-1 [4.2-1])

$E = \rho Q_E - 0.2S_{DS}D$ (Eq. 5.2.7.1-2 [4.2-2])

<u>U = 1.2D + 1.0E + 0.5L + 0.2S</u>

$$\theta = \tan^{-1}\left(\frac{7 \text{ ft}}{5.5 \text{ ft}}\right) = 51.8^\circ$$

$$T_u = \frac{1.2(3000 \text{ lb}) - (892 \text{ lb})}{4} - \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8^\circ)}{7 \text{ ft}} + \frac{\sin(51.8^\circ)}{5.5 \text{ ft}}\right) = -2624 \text{ lb}$$

$$C_u = \frac{1.2(3000 \text{ lb}) + (892 \text{ lb})}{4} + \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8^\circ)}{7 \text{ ft}} + \frac{\sin(51.8^\circ)}{5.5 \text{ ft}}\right) = 4424 \text{ lb}$$

$$V_u = \frac{14276 \text{ lb}}{4} = 3569 \text{ lb}$$

<u>U = 0.9D + 1.0E</u>

$$\theta = \tan^{-1}\left(\frac{7 \text{ ft}}{5.5 \text{ ft}}\right) = 51.8^\circ$$

$$T_u = \frac{0.9(3000 \text{ lb}) - (892 \text{ lb})}{4} - \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8^\circ)}{7 \text{ ft}} + \frac{\sin(51.8^\circ)}{5.5 \text{ ft}}\right) = -2849 \text{ lb}$$

$$C_u = \frac{0.9(3000 \text{ lb}) + (892 \text{ lb})}{4} + \frac{(14276 \text{ lb})(2 \text{ ft})}{2}\left(\frac{\cos(51.8^\circ)}{7 \text{ ft}} + \frac{\sin(51.8^\circ)}{5.5 \text{ ft}}\right) = 4199 \text{ lb}$$

$$V_u = \frac{14276 \text{ lb}}{4} = 3569 \text{ lb}$$

13.3.4.2 Proportioning and Details

Anchor and snubber loads for support on vibration isolation springs are shown in Figure 13.3-5.

Check vibration isolation system within housing anchored with two 1-in.-diameter post-installed expansion anchors embedded 9 inches into the concrete.

The *Provisions* does not provide a basis for determining the design strength of post-installed expansion anchors. Although manufacturers provide ultimate strength shear and tension loads for their products, the *Provisions* does not provide resistance or quality values, ϕ, to allow determination of design shear and tension strengths. The 2002 edition of ACI 318 contains provisions for post-installed anchors which depend on anchor testing per the ACI 355.2-01 testing standard. Prior to the adoption of this method, the best course available was to use allowable stress loads published in evaluation reports prepared by model building code agencies. These allowable stress loads were then multiplied by 1.4 to convert to design strengths. This technique will be illustrated in the following.

[The 2003 *Provisions* refer to Appendix D of ACI 318-02.]

Allowable stress combinations are included in the 2000 *IBC*. The 1.4 strength conversion factor is unnecessary when using allowable stress loads published in evaluation reports prepared by model building code agencies.

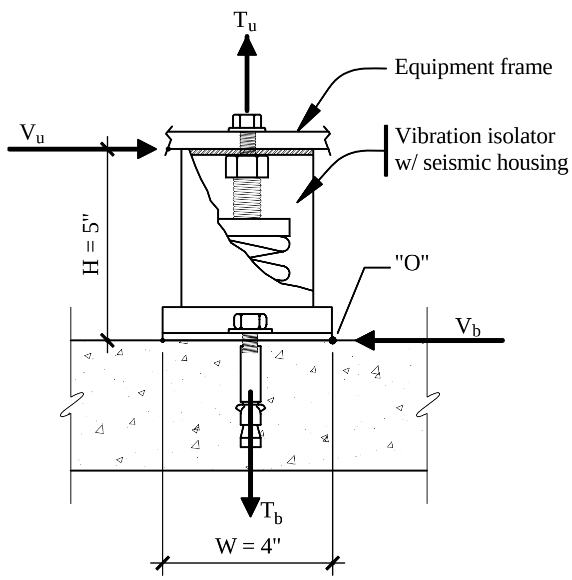

Figure 13.3-5 Anchor and snubber loads for support on vibration isolation springs (1.0 in. = 25.4 mm).

13.3.4.2.1 Design Tension Strength on Isolated Anchor in Slab, Away from Edge

Allowable stress tension values are obtained from ICBO Evaluation Services, Inc., ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar certified allowable values are expected with anchors from other manufacturers.

anchor diameter = 1 in.

anchor depth = 9 in.

f_c' = 4000 psi

with special inspection

T_{allow} = 8800 lb

$P_s = \phi P_c$ = 1.4(8800 lb) = 12320 lb

13.3.4.2.2 Design Shear Strength on Isolated Anchor in Slab, Away from Edge

Allowable stress shear values are obtained from ICBO Evaluation Services, Inc., ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar certified allowable values are expected with anchors from other manufacturers.

anchor diameter = 1 in.

anchor depth = 9 in.

f_c' = 4000 psi

V_{allow} = 8055 lb

$V_s = \phi V_c$ = 1.4(8055 lb) = 11277 lb

13.3.4.2.3 Combined Tension and Shear

Per *Provisions* Sec. 6.1.6.2 [6.2.8.2], anchors embedded in concrete or masonry shall be proportioned to carry at least 1.3 times the force in the connected part due to the prescribed forces.

In the *Provisions* and in the 2000 *IBC*, the factor of 2.0 is reduced to 1.3. This will greatly reduce the prescribed seismic forces.

Interaction relationships for combined shear and tension loads are obtained from ICBO Evaluation Services, Inc. ER-4627 for Hilti Kwik Bolt II concrete anchors. Similar results are expected using other anchors.

As stated in ICBO ES evaluation report:

$$\left(\frac{P_s}{P_t}\right)^{5/3} + \left(\frac{V_s}{V_t}\right)^{5/3} \leq 1$$

Using *Provisions* terminology:

$$\left(\frac{T_b}{\phi P_c}\right)^{5/3} + \left(\frac{V_b}{\phi V_c}\right)^{5/3} \leq 1$$

$$\left(\frac{2 \times 4462 \text{ lb}}{11277 \text{ lb}}\right)^{5/3} + \left(\frac{2 \times 1785 \text{ lb}}{12075 \text{ lb}}\right)^{5/3} = 0.68 + 0.13 = 0.81 < 1 \qquad \text{OK}$$

13.3.4.2.4 Summary

At each corner of the HVAC fan unit, provide a vibration isolation system within a housing anchored with two 1-in.-diameter post-installed expansion anchors embedded 9 in. into the concrete slab. Special inspection is required. A raised concrete pad is probably required to allow proper embedment of the post-installed expansion anchors.

Other post-installed anchors, such as chemical (adhesive) or undercut post-installed anchors also could be investigated. These anchors may require more involved installation procedures, but they may allow the use of R_p = 2.5 if they have an embedment depth-to-diameter ratio of at least 8 (i.e., are not shallow anchors). The higher R_p value will result in much smaller prescribed seismic forces and, therefore, a much reduced embedment depth.

[Again note that in 2003 *Provisions* selection of R_p is no longer based on the *L*/*d* ratio; see Sec. 6.2.8.1.]

13.3.5 Additional Considerations for Support on Vibration Isolators

Vibration isolation springs are provided for equipment to prevent vibration from being transmitted to the building structure. However, they provide virtually no resistance to horizontal seismic forces. In such cases, some type of restraint is required to resist the seismic forces. Figure 13.3-6 illustrates one concept where a bolt attached to the equipment base is allowed to slide a controlled distance (gap) in either direction along its longitudinal axis before it contacts resilient impact material.

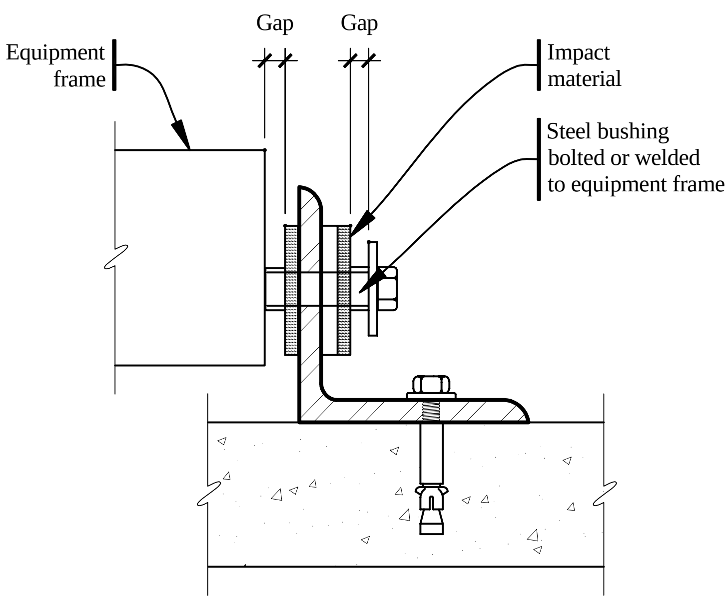

Figure 13.3-6 Lateral restraint required to resist seismic forces.

Design of restraints for vibration-isolated equipment varies for different applications and for different manufacturers. In most cases, restraint design incorporates all directional capability with an air gap, a soft impact material, and a ductile restraint or housing.

Restraints should have all-directional restraint capability to resist both horizontal and vertical motion. Vibration isolators have little or no resistance to overturning forces. Therefore, if there is a difference in height between the equipment center of gravity and the support points of the springs, rocking is inevitable and vertical restraint is required.

An air gap between the restraint device and the equipment prevents vibration from transmitting to the structure during normal operation of the equipment. Air gaps are generally no greater than ¼ in. Dynamic tests indicate a significant increase in acceleration for air gaps larger than ¼ in.

A soft impact material often an elastomer such as bridge bearing neoprene reduces accelerations and impact loads by preventing steel-to-steel contact. The thickness of the elastomer can significantly reduce accelerations to both the equipment and the restraint device and should be specifically addressed for life safety applications.

A ductile restraint or housing is critical to prevent catastrophic failure. Unfortunately, housed isolators made of brittle materials such as cast iron often are assumed to be capable of resisting seismic loads and continue to be installed in seismic zones.

Overturning calculations for vibration- isolated equipment must consider a worst case scenario as illustrated in *Guide* Sec. 13.3.4.1. However, important variations in calculation procedures merit further discussion. For equipment that is usually directly attached to the structure, or mounted on housed vibration isolators, the weight can be used as a restoring force since the equipment will not transfer a tension load to the anchors until the entire equipment weight is overcome at any corner. For equipment installed on any other vibration isolated system (such as the separate spring and snubber arrangement shown in Figure 13.3-5), the weight cannot be used as a restoring force in the overturning calculations.

As the foregoing illustrates, design of restraints for resiliently mounted equipment is a specialized topic. The *Provisions* sets out only a few of the governing criteria. Some suppliers of vibration isolators in the highest seismic zones are familiar with the appropriate criteria and procedures. Consultation with these suppliers may be beneficial.

13.4 ANALYSIS OF PIPING SYSTEMS

13.4.1 ASME Code Allowable Stress Approach

Piping systems are typically designed to satisfy national standards such as ASME B31.1. Piping required to be designed to other ASME piping codes use similar approaches with similar definition of terms.

13.4.1.1 Earthquake Design Requirements

ASME B31.1 Sec. 101.5.3 requires that the effects of earthquakes, where applicable, be considered in the design of piping, piping supports, and restraints using data for the site as a guide in assessing the forces involved. However, earthquakes need not be considered as acting concurrently with wind.

13.4.1.2 Stresses Due to Sustained Loads

The effects of pressure, weight, and other sustained loads must meet the requirements of ASME B31.1 Eq. 11A:

$$S_L = \frac{PD_o}{4t_n} + \frac{0.75iM_A}{Z} \leq 1.0S_h$$

where:

S_L = sum of the longitudinal stresses due to pressure, weight, and other sustained loads

P = internal design pressure, psig

D_o = outside diameter of pipe, in.

t_n = nominal pipe wall thickness, in.

i = stress intensification factor from ASME Piping Code Appendix D, unitless
= 1.0 for straight pipe
$\geq$ 1.0 for fittings and connections

M_A = resultant moment loading on cross section due to weight and other sustained loads, in.-lb

Z = section modulus, in.^{3}

S_h = basic material allowable stress at maximum (hot) temperature from ASME Piping Code Appendix A

For example, ASTM A53 seamless pipe and tube, Grade B: S_h = 15.0 ksi for -20 to 650 degrees F.

13.4.1.3 Stresses Due to Occasional Loads

The effects of pressure, weight, and other sustained loads, and occasional loads including earthquake must meet the requirements of ASME B31.1 Eq. 12A:

$$\frac{PD_o}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_B}{Z} \leq kS_h$$

where:

M_B = resultant moment loading on cross-section due to occasional loads, such as from thrust loads, pressure and flow transients, and earthquake. Use one-half the earthquake moment range. Effects of earthquake anchor displacements may be excluded if they are considered in Eq. 13A, in.-lb

k = duration factor, unitless

= 1.15 for occasional loads acting less than 10% of any 24 hour operating period

= 1.20 for occasional loads acting less than 1% of any 24 hour operating period

= 2.00 for rarely occurring earthquake loads resulting from both inertial forces and anchor movements (per ASME interpretation)

13.4.1.4 Thermal Expansion Stress Range

The effects of thermal expansion must meet the requirements of ASME B31.1 Eq. 13A:

$$S_E = \frac{iM_C}{Z} \leq S_A + f\left(S_h - S_L\right)$$

where:

S_E = sum of the longitudinal stresses due to thermal expansion, ksi

M_C = range of resultant moments due to thermal expansion. Also includes the effects of earthquake anchor displacements if not considered in Eq. 12A, in.-lb

S_A = allowable stress range, ksi (per ASME B31.1 Eq. 1, $S_A = f\left(1.25S_c + 0.25S_h\right)$

f = stress range reduction factor for cyclic conditions from the ASME Piping Code Table 102.3.2.

S_c = basic material allowable stress at minimum (cold) temperature from the ASME Piping Code Appendix A

13.4.1.5 Summary

In the ASME B31.1 allowable stress approach, the earthquake's effects only appear in the M_B and M_C terms.

Earthquake inertial effects ⇒ M_B term

Earthquake displacement effects ⇒ M_C term

13.4.2 Allowable Stress Load Combinations

ASME B31.1 utilizes an allowable stress approach; therefore, allowable stress force levels and allowable stress load combinations should be used. While the *Provisions* are based on strength design, the IBC provides the following two sets of allowable stress loads and load combinations. The IBC load combinations are appropriate for use for piping systems when considering earthquake effects. When earthquake effects are not considered, load combinations should be taken from the appropriate piping system design code.

13.4.2.1 IBC Basic Allowable Stress Load Combinations

No increases in allowable stress are permitted for the following set of load combinations:

D (IBC Eq. 16-7)

$D + L + (L_r \text{ or } S \text{ or } R)$ (IBC Eq. 16-9)

$D + (W \text{ or } 0.7\,E)$ (IBC Eq. 16-10)

$0.6D - 0.7\,E$ (IBC Eq. 16-12)

13.4.2.2 IBC Alternate Basic Allowable Stress Load Combinations

Increases in allowable stress (typically 1/3) are permitted for the following alternate set of load combinations that include W or E:

$D + L + (L_r \text{ or } S \text{ or } R)$ (IBC Eq. 16-13)

$D + L + S + E/1.4$ (IBC Eq. 16-17)

$0.9D + E/1.4$ (IBC Eq. 16-18)

13.4.2.3 Modified IBC Allowable Stress Load Combinations

It is convenient to define separate earthquake load terms to represent the separate inertial and displacement effects.

E_I = Earthquake inertial effects ⇒ M_B term

E_Δ = Earthquake displacement effects ⇒ M_C term

It is also convenient to use the IBC Alternate Basic Allowable Stress Load Combinations modified to use ASME Piping Code terminology, deleting roof load effects (L_r or S or R) and multiplying by 0.75 to account for the 1.33 allowable stress increase when W or E is included. Only modified IBC Eq. 16-17 and 16-18 will be considered in the discussion that follows.

$$0.75[D + L + S + (E_I + E_\Delta)/1.4] \qquad \text{(modified IBC Eq. 16-17)}$$

$$0.75[0.9D + (E_I + E_\Delta)/1.4] \qquad \text{(modified IBC Eq. 16-18)}$$

13.4.3 Application of the *Provisions*

13.4.3.1 Overview

Provisions Sec. 6.3.11 [6.4.2, item 4] requires that, in addition to their attachments and supports, piping systems assigned an I_p greater than 1.0 must themselves be designed to meet the force and displacement requirements of *Provisions* Sec. 6.1.3 and 6.1.4 [6.2.6 and 6.2.7] and the additional requirements of this section.

13.4.3.1 Forces

Provisions Sec. 6.1.3 [6.2.6] provides specific guidance regarding the equivalent static forces that must be considered. In computing the earthquake forces for piping systems, the inertial portion of the forces (noted as E_I in this example) are computed using *Provisions* Eq. 6.1.3-1, 6.1.3-2, and 6.1.3-3 [6.2-1, 6.2-3, and 6.2-4] for F_p with a_p = 1 and R_p = 3.5. For anchor points with different elevations, the average value of the F_p may be used with minimum and maximums observed. In addition, when computing the inertial forces, the vertical seismic effects (± $0.2S_{DS}W_p$) should be considered.

The term E_I can be expressed in terms of the forces defined in the *Provisions* converted to an allowable stress basis by the 1.4 factor:

$$E_I = \left(\frac{F_p}{1.4}\right)_{horizontal} \pm \left(\frac{0.2S_{DS}W_p}{1.4}\right)_{vertical}$$

It is convenient to designate the term $\left(1 \pm \dfrac{0.2S_{DS}}{1.4}\right)$ by the variable β.

The vertical component of E_I can now be defined as βM_a and applied to all load combinations that include E_I.

M_B can now be defined as the resultant moment induced by the design force $F_p/1.4$ where F_p is as defined by *Provisions* Eq. 6.1.3-1, 6.1.3-2, or 6.1.3-3.

13.4.3.3 Displacements

Provisions Sec. 6.1.4 [6.2.7] provides specific guidance regarding the relative displacements that must be considered. Typically piping systems are designed considering forces and displacements using elastic analysis and allowable stresses for code prescribed wind and seismic equivalent static forces in combination with operational loads.

However, no specific guidance is provided in the *Provisions* except to say that the relative displacements should be accommodated. The intent of the word "accommodate" was not to require that a piping system remain elastic. Indeed, many types of piping systems typically are very ductile and can accommodate large amounts of inelastic strain while still functioning quite satisfactorily. What was intended was that the relative displacements between anchor and constraining points that displace significantly relative to one another be demonstrated to be accommodated by some rational means. This accommodation can be made by demonstrating that the pipe has enough flexibility and/or inelastic strain capacity to accommodate the displacement by providing loops in the pipeline to permit the displacement or by adding flex lines or articulating couplings which provide free movement to accommodate the displacement. Sufficient flexibility may not exist where branch lines, may be forced to move with a ceiling or other structural system are connected to main lines. Often this “accommodation” is done by using engineering judgment, without calculations. However, if relative displacement calculations were required for a piping system, a flexibility analysis would be required. A flexibility analysis is one in which a pipe is modeled as a finite element system with commercial pipe stress analysis programs (such as Autopipe or CAESAR II) and the points of attachment are displaced by the prescribed relative displacements. The allowable stress for such a condition may be significantly greater than the normal allowable stress for the pipe.

The internal moments resulting from support displacement may be computed by means of elastic analysis programs using the maximum computed relative displacements as described earlier and then adjusted. As with elastic inertial forces, the internal moments caused by relative displacement can be divided by R_p to account for reserve capacity. A reduction factor of 1.4 may be used to convert them for use with allowable stress equations. Therefore, M_C can now be defined as the resultant moment induced by the design relative seismic displacement $D_p/1.4R_p$ where D_p is defined by *Provisions* Eq. 6.1.4-1, 6.1.4-2, 6.1.4-3, or 6.1.4-4 [6.2-5, 6.2-6, 6.2-7, or 6.2-8].

13.4.3.4 Load Combinations

Combining ASME B31.1 Eq. 12A and 13A with modified IBC Eq. 1605.3.2.5 and 1605.3.2.6 yields the following:

For modified *IBC* Eq. 16-17

$$\frac{PD_o}{4t_n} + \beta\left(\frac{0.75iM_A}{Z}\right) \pm \frac{0.75iM_B}{Z} \le kS_h$$

$$\frac{iM_C}{Z} \le S_A + f\left(S_h - S_L\right)$$

and for modified *IBC* Eq. 16-18

$$\frac{PD_o}{4t_n} + 0.9\beta\left(\frac{0.75iM_A}{Z}\right) - \frac{0.75iM_B}{Z} \le kS_h$$

$$\frac{iM_C}{Z} \le S_A + f\left(S_h - S_L\right)$$

where:

$$\beta = \left(1 \pm \frac{0.2S_{DS}}{1.4}\right)$$

M_A = the resultant moment due to weight

M_B = the resultant moment induced by the design force $F_p/1.4$ where F_p is as defined by *Provisions* Eq. 6.1.3-1, 6.1.3-2, or 6.1.3-3 [6.2-1, 6.2-3, or 6.2-4].

M_C = the resultant moment induced by the design relative seismic displacement $D_p/1.4R_p$ where D_p is as defined by *Provisions* Eq. 6.1.4-1, 6.1.4-2, 6.1.4-3, or 6.1.4-4 [6.2-5, 6.2-6, 6.2-7, or 6.2-8].

S_{DS}, W_p, and R_p are as defined in the *Provisions*.

P, D_o, t_n, I, Z, k, S_h, S_A, f, and S_L are as defined in ASME B31.1.