APPENDIX E
ARCHITECTURAL COMPONENT EXAMPLES

This appendix illustrates the implementation of the provisions of this document for the seismic evaluation and rehabilitation of architectural nonstructural components in military buildings. The examples in the following sections of this appendix were selected to demonstrate the application of various rehabilitation techniques to mitigate seismic deficiencies identified in typical architectural components in existing military buildings.

E1. Unreinforced Masonry Parapet

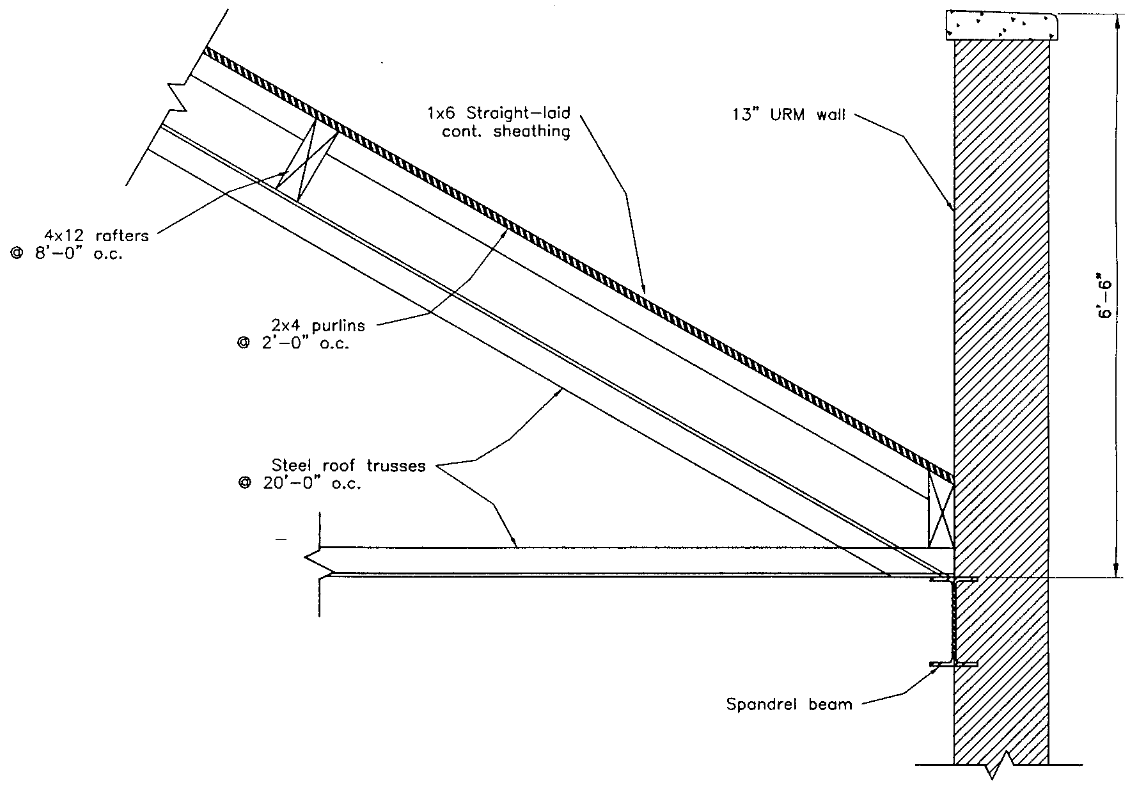
E2. Canopy at Building Entrance

E3. Bracing of Library Shelving

DESIGN EXAMPLE PROBLEM E1: Retrofit of Unreinforced Masonry Parapet

Description

The parapet is part of the exterior wall of a 3-story structure built in the 1930's with structural steel frames and infilled unreinforced masonry walls. A wood roof is supported on steel trusses that are spaced at 20' o.c. (6.1 m) and bear on perimeter steel columns. The top chords of the trusses are sloped at 30 degrees from the horizontal. The roof framing consists of 4 by 12 inch (102 mm x 305 mm)wood rafters at 8 foot (2.44 m) supported by the steel trusses. The rafters support 2 x 4 inch purlins at 2 foot (610 mm) on center and the purlins support 1 x 6 inch straight-laid sheathing with tar and gravel roof. The exterior walls are 13-inch (330 mm) thick unreinforced brick and the parapet rises 6.5 feet (1.98 mm) above the spandrel beam line.



1 in = 25.4 mm
1 ft = 0.305 m

Figure E1-1: Section at Parapet

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is a Standard Occupancy Structure, and from Table 3-1, falls into Seismic Use Group I.

b. Structural Performance Level. The parapet is to be analyzed for the Life Safety Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective for the parapet is determined to be 1A, defined as the combination of Life Safety Performance Level with a ground motion of 2/3 MCE as prescribed for Seismic Use Group I. For this example, the design spectral response acceleration is assumed to be as follows:

$S_{DS} = 2/3\ S_{MS} = 0.65$ (TI 809-04 Eq. 3-3)

d. Seismic design category:
Based on Short Period Response Acceleration:
Seismic design category: D (Table 3-4a)
Based on 1 second period Response Acceleration:
Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

1) Exempt Components

Not applicable. The parapet is not considered an exempt component.

2) Classification of Componenet

The parapet is assigned an importance factor, I_p of 1.0.

3) Disposition

The parapet has been screened by the Tier 1 evaluation of FEMA 310 in Example Problem H3 of TI 809-51. It was determined that the building definitely needs rehabilitation.

G. Nonstructural Screening (Tier 1)

This step has already been completed as part of Example Problem H3 of TI 809-51.

H. Nonstructural Evaluation (Tier 2)
This step is skipped here since the building has already been designated as definitely requiring rehabilitation.

I. Evaluation Report

The evaluation report of the Example Problem H3 of TI 809-51 would include the following:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations

A judgmental assessment of the results of the evaluation determined that the building definitely needs rehabilitation.

6. Rehabilitation strategy

The potential rehabilitation options included:

a. Remove parapet
b. Strengthen masonry parapet with concrete overlay.
c. Strengthen masonry parapet with steel bracing.

The last alternative to strengthen the parapet with bracing was selected as the rehabilitation alternative.

7. Rehabilitation concept

The rehabilitation concept is shown in Figure E1-2. It consists of the addition of steel channel bracing attached to the roof and to the parapet at 1'-0" (305 mm) below the top of parapet. Horizontal steel channel walers are provided along the parapet for horizontal brace reactions, and vertical flat bars mobilize the weight of the wall to provide vertical reactions.

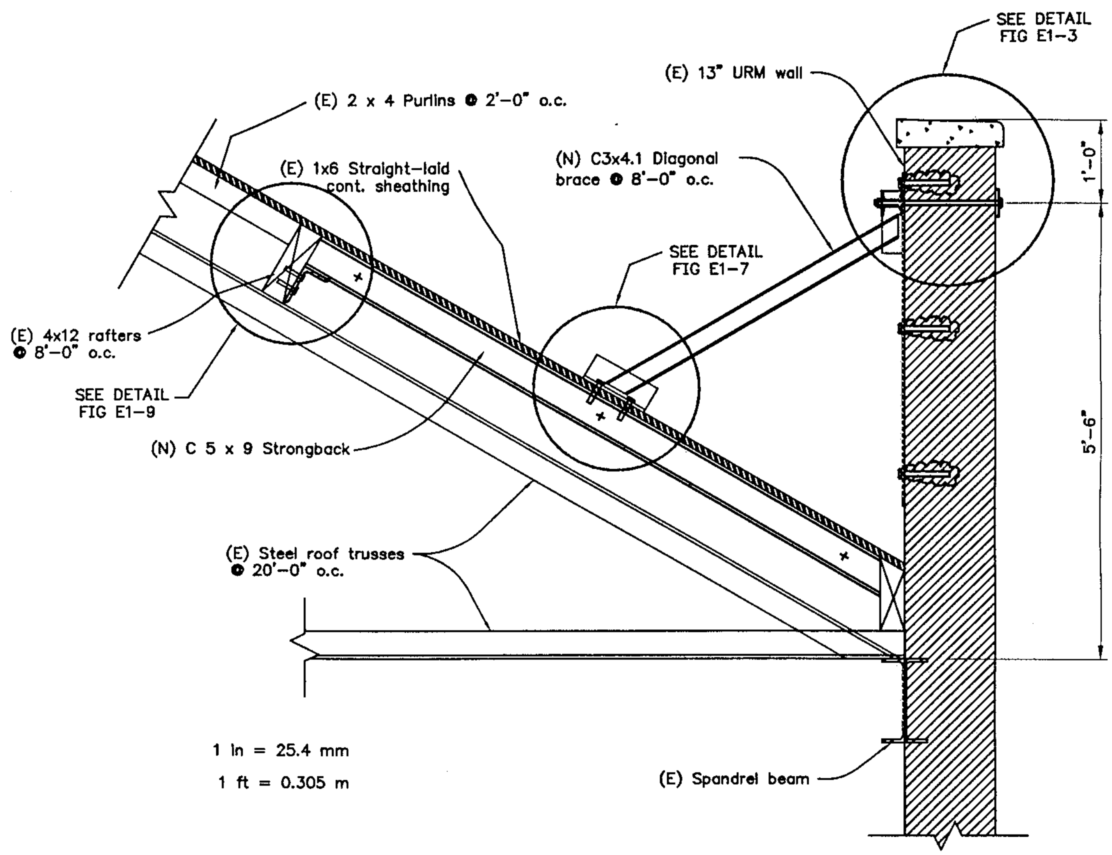


Figure E1-2: Parapet Bracing

J. Rehabilitation

The procedures for rehabilitation are outlined below:

1. *Review Evaluation Report and other available data.*

2. *Site Visit.*

3. *Confirming evaluation of existing building (if necessary).*

4. *Prepare alternative structural rehabilitation concepts.*

5. *Rehabilitation design.*

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 9 of this document. A detailed analysis follows.

Determine Seismic Forces

Select R_p and a_p, factors:

$a_p = 2.5$ (TI 809-04, Table 10-1)
$R_p = 1.25$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4a_p I_p S_{DS} W_p}{R_p}\left(1+2\frac{x}{h}\right) \qquad \text{(TI 809-04, EQ. 10-1)}$$

where; $x/h = 41/36$ (3rd story of a 3-story building)
$W_p =$ Dead load = 130psf (13-in. Brick)
$\therefore W_p = (1'+5.5'/2)(130\text{psf}) = 488\text{plf}\ (7.12\ \text{kN / m})$

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-3)}$$

$$F_p = \frac{0.4(2.5)(1.0)0.65(488\text{plf})}{1.25}\left(1+2\frac{41}{36}\right) = 1.73(488\text{plf}) = 846\text{plf}\ (12.3\ \text{kN / m})$$

$(F_p)_{max} = 1.6(0.65)1.0(488\text{plf}) = 508\text{plf} < 846\text{plf} = F_p\ (12.3\ \text{kN / m})$ **Governs**

$(F_p)_{min} = 0.3(0.65)1.0(488\text{plf}) = 95\text{plf} < 846\text{plf} = F_p\ (12.3\ \text{kN / m})$ **O.K.**

$$\therefore F_p = 508\,plf\ (7.4\ \text{kN / m})$$

Brace to wall connection

Try 5/8-in. ϕ bolts (A-307) extending through wall with steel bearing plates.

The design axial strength, B_a for headed anchor bolts embedded in masonry shall be the least of:

$B_a = 4\phi A_p \sqrt{f'_m}$ (strength governed by masonry breakout) (FEMA 302 Eq. 11.3.12.2-1)

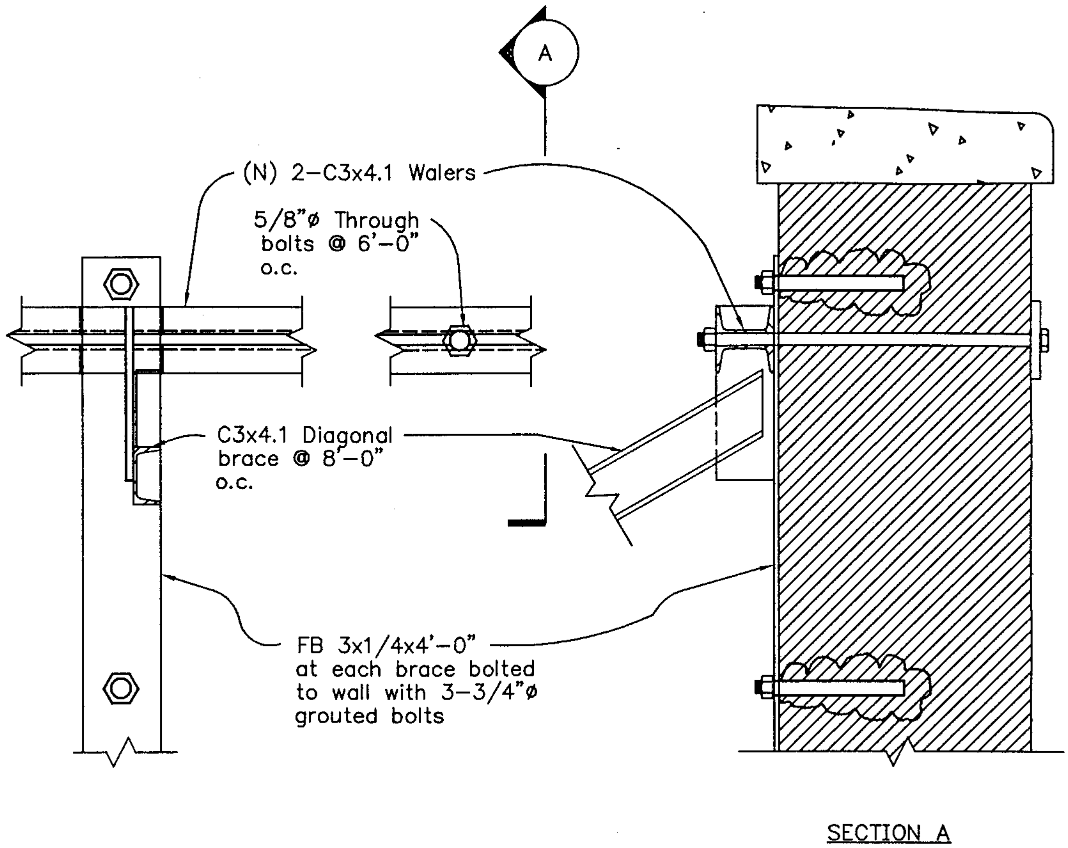
where $A_p = \pi l_b^2 = \pi(13")^2 = 530\ \text{in.}^2$

$B_a = 4(0.5)(530\,\text{in.}^2)\sqrt{900\,\text{psi}} = 31.8\,\text{kips/anchor}\ (141\ \text{kN})$

$B_a = \phi A_b f_y$ (strength governed by steel) (FEMA 302 Eq. 11.3.12.2-2)

$B_a = (0.9)(0.31\,\text{in.}^2)(60\,\text{ksi}) = 16.7\,\text{kips/anchor}\ (74.3\ \text{kN})$

For anchors at 6' (1.83 m) on center Q_N = 16.7 kips / 6ft² = 2780 psf
Q_{CN} =2780 plf (40.6 kN / m) > Q_{UF} = 508 plf (7.4 kN /m), OK



1 in = 25.4 mm
1 ft = 0.305 m

Figure E1-3: Detail at Top of Parapet Brace

Design of Walers:

Check flexure for bracing at 8'-0" (2.44 m) on center:

Assume simple beam moment for channel spanning between bolts;

$$fM_n > M_u$$

$$M_u = \frac{W_p L^2}{8} = \frac{508\,plf(8')^2(12''/1')}{8} = 48{,}700^{in-lb} \text{ or } 4.06 \text{ kip-ft } (5.5 \text{ kN} - \text{m})$$

For C3x4.1, L_b = 8'-0"; Z = 1.04-in^3, L_p=1.7', L_r=12.1'

For $L_p < L_b < L_r$:

$$\phi M_n = \phi M_p - \phi(M_p - M_r)\frac{L_b - L_p}{L_r - L_p} < \phi M_p$$

$$\phi M_n = 3.51 - 0.90(3.9 - 2.38)\frac{8'-1.7'}{12.1'-1.7'} \text{ x 2 walers} > 4.06 \text{ kip-ft } (5.5 \text{ kN} - \text{m})$$

$$\phi M_n = 5.36 \text{kip} - \text{ft } (7.23 \text{kN} - \text{m}) > 4.06 \text{kip} - \text{ft } (5.5 \text{kN} - \text{m})$$ **O.K.**

Check deflection at service level loads ($W_p/1.4$):

$$\Delta_{flex\ trans} = \frac{5wL^4}{384EI} = \frac{5(508\ plf)/1.4(1'/12")(8ft * 12"/')^4}{384(29000000psi)(2)(1.66)in^4} = 0.35in = \frac{1}{275} \quad \textbf{O.K.}$$

Design of channel brace

(Brace is sloped at 60 degrees)

Check axial compression in brace:

$$P_{BR} = \frac{2(l)(w_p)}{\sqrt{3}} = \frac{2(8')(0.508)}{\sqrt{3}} = 4.70 \text{ kips (20.9 kN) per brace}$$

Try 2-C3x4.1 r_{min}=0.40 in. A=1.21 in.^{2}

L_{max} = 48"

L/r = 48/0.40 = 120

$\phi_c F_{cr} = 14.34$ ksi

$\phi P_n = \phi_c F_{cr}(A) = 14.34(1.21) = 17.30$ kips (77.5 kN) per brace > 4.70 kips (20.9 kN) **O.K.**

Check Upward Reactions on wall

$$P_V = \frac{P_u}{\sqrt{3}} = \frac{(8')(0.508)}{\sqrt{3}} = 2.4 \text{ kips (10.7 kN) per brace}$$

Weight of brick above waler = (1)(8)(0.13)=1.04 kips (4.6 kN)
Provide vertical member to mobilize additional weight of wall (See Figure E1-2)
Use flat bar 3x1/4 with 3-3/4"ϕ shear bolts to wall

Bolt Capacity:

$$B_v = 1750\phi\left(\sqrt{f'_m A_b}\right)^4$$ (strength governed by masonry) (FEMA 302 Eq. 11.3.12.3-1)

$= 1750(0.5)(900\text{psi x } 0.31)^4 = 3.6$ kips/anchor (16.0 kN)

$$B_v = 0.6\phi A_b f_y$$ (strength governed by steel) (FEMA 302 Eq. 11.3.12.3-2)

$= 0.6(0.9)(0.31)(60 \text{ ksi}) = 10.0$ kips/anchor (44.5 kN)

V_{bolts} = 3 x 3.6 x 0.85 = 8.6 kips (38.3 kN > 2.4 kips (10.7 kN) **O.K.**

Connection of brace to walers

P_{BR} = 4.70kips (20.9 kN) per brace

For a 3/16" fillet weld (L60 electrodes), the strength of weld metal is:

$$0.707x\frac{3}{16}x\phi F_w = 0.133x0.75[0.60(60)] = 3.58k/in$$

For a 4" weld, $\phi R_n = 4x3.58 = 14.32\,\text{kips}\,(63.7\,\text{kN}) > 4.70\,\text{kips}\,(20.9\,\text{kN}$

Connection of walers to 3/8" plate

$$P = ¼ \text{ x } 4.04k = 1.01\text{kips } (4.5 \text{ kN})$$

1/8" fillet weld

$$0.707x\frac{1}{8}x\phi F_w = 0.088x0.75[0.60(60)] = 2.39k/in$$

For a 2.5" weld, $\phi R_n = 2.5x2.39 = 5.98k\,(26.6\,\text{kN}) > 1.01k\,(4.5\,\text{kN})$

3/8" base plate to ¼" flat bar

P_v = 2.4kips (10.7 kN) per brace P_h = 4.04 kips (18.0 kN) per brace

For a 3/16" fillet weld,

$$\phi R_n = 2x6x3.58 = 43\text{kips}\,(191\text{kN})$$

$$\frac{2.4k}{43k} + \frac{4.04k}{43k} \leq 1.0$$

$$0.056 + 0.094 = 0.15 \leq 1.0 \quad \textbf{O.K.}$$

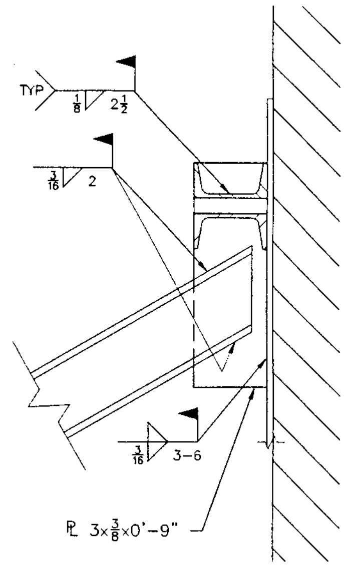


Figure E1-4: Parapet Brace Connection

Note:

Existing 4x12 rafters are on 8'-0" (2.44 m) on center, so locate parapet braces over rafters. In the orthogonal direction, 2x4 purlins are on 2'-0"(610 mm) on center, so locate parapet braces over every fourth purlin.

Design channel strongback for 2x4 purlin to carry the brace load

2x4 Purlins are at 8'-0" o.c.

$$M_{MAX} = \frac{Pxaxb}{L} = \frac{4.04x4.5x3.5}{8} = 7.95\text{kip}-\text{ft} = 95.44\text{kip}-\text{in}\,(10.8\text{kN}-\text{m})$$

Try C5x6.7   Z=3.51 in^3

$$Z_{req'd} = \frac{M_{MAX}}{\phi_b F_y} = \frac{95.44\text{in}-k}{0.9(60)} = 1.76\text{in}^3 < 3.51 \text{ in}^3 \text{ O.K.}$$

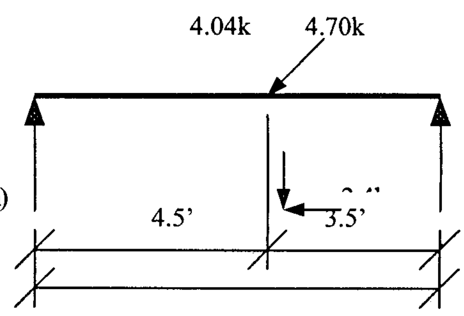


Figure E1.5: Parapet Brace Reactions at Existing 2x4 Purlin

Use C5x9 (Minimum size for 5/8"-diameter bolt to flange)

Connection of Diagonal Brace to Strongback

Use L4x3½x3/8

Weld channel to angle as at top connection (See Figure e1-3)

Bolt L to strongback through existing sheathing. Since there are no published values for this type of connection, an allowable value will be derived based on calculated maximum combined stress.

Calculate bolt capacity assuming 1" sheathing is only a spacer between two steel plates.

Try 5/8"-diameter bolt at roof of thread: D = 0.514"
A = 0.208 in^2

For A307 bolt, f_y = 36 ksi

For 2.40k load (10.7 kN)(horiz.) and 4.04k (18.0 kN) (vert):

Allowable stresses:

bending = ϕF_{bn} = 0.9 x 0.75 x 36 = 24.3 ksi x 1.7 = 41.3 ksi
shear = ϕF_{vn} = 0.85 x 0.4 x 36 = 12.2 ksi x 1.7 = 20.7 ksi
bearing = ϕF_{bn} = 0.90 x 0.6 x 36 = 19.4 ksi x 1.7 = 33.0 ksi
tension = ϕF_{tn} = 0.90 x 0.6 x 36 = 19.4 ksi x 1.7 = 33.0 ksi

M = 2.40k x 1.22" x 0.5 = 1.47 kip-in (166 N - m)

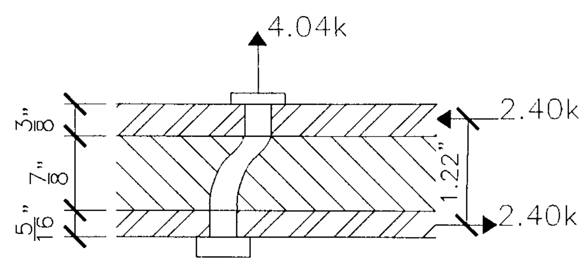
$$f_b = \frac{M}{S} = \frac{1.47}{0.13} = 11.30\text{ksi}$$

$$S = \frac{\Pi D^3}{32} = \frac{\Pi x 0.514^3}{32} = 0.13\text{in}^3$$

$$f_v = \frac{F}{A} = \frac{2.40}{0.208} = 11.54\text{ksi}$$

$$f_{bearing} = \frac{2.4}{0.514 x 0.320} = 14.6\text{ksi}$$

$$f_{tn} = \frac{4.04}{0.208} = 19.42\text{ksi}$$



1 kip = 4.448 kN

Figure E1-6: Forces on Bolt at Roof Sheathing

Since connection will not develop the strength of the brace,

Assume

ϕF_{tn} = 0.67 x 20.0 x 1.7 = 22.78 ksi (157 MPa)

$$= \sqrt{11.30^2 + 11.54^2 + 19.42^2}$$

= 25.3 ksi/bolt (174 MPa)

Use 2-5/8 ϕ bolts f_{tmax} = 25.3ksi/2=12.65ksi (87 MPa) < 22.78 ksi (157 MPa) **O.K.**

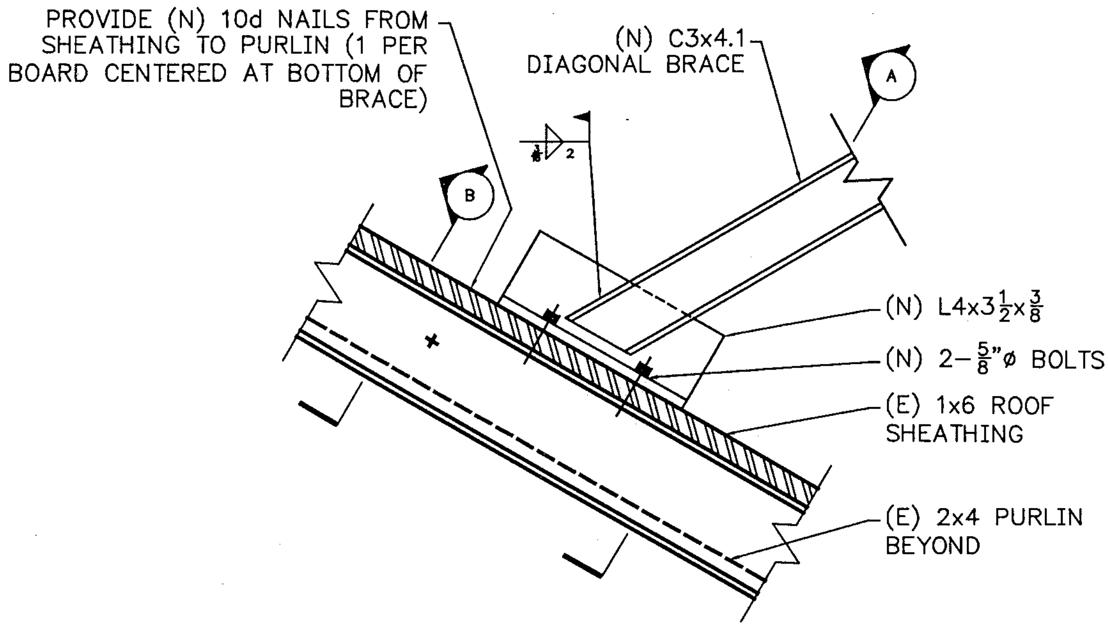
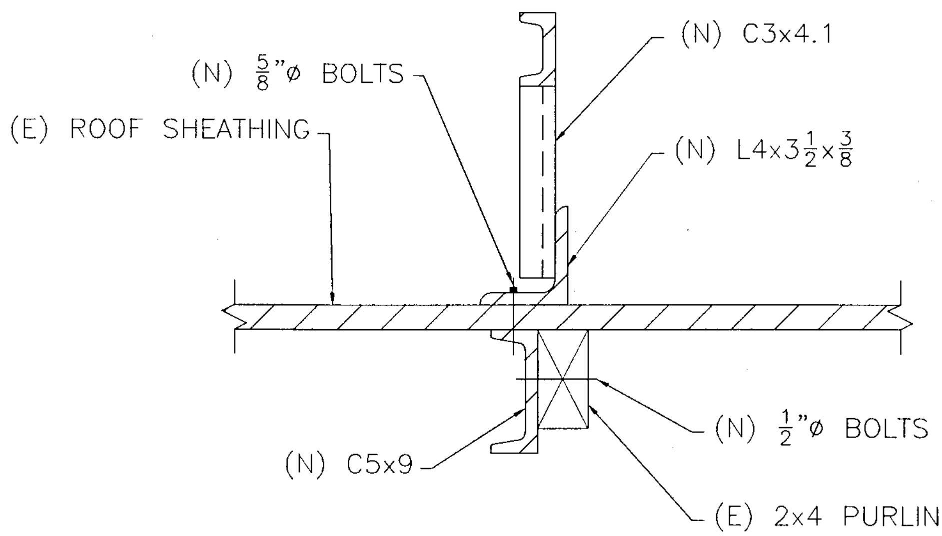


Figure E1-7: Detail at Bottom of Parapet Bracing



1 in = 25.4 mm

Figure E1-8: Section A-A

Connection of C5x9 to 4x12 rafter

$V_{max} = \frac{4.5}{8.0} x4.04k = 2.27k$ ips (10.1 kN) (moments about left end of C5x9)

5/8" ϕ lag screw (parallel to grain) 3" penetration

$F_{all} = \phi F_n = 0.9 \text{ x } 790 = 711$ lbs/bolt (3.2 kN) (UBC 97, Table 23-III U)

Use 4-5/8" ϕ lag screws $\sum F_{all} = 4x711 = 2.84k\ (12.6\ kN) > 2.27k\ (10.1\ kN)$ O.K.

Connection of C5x9 to 2x4 purlins

$V_{max} = 2.27k$ (10.1 kN)

For ½"ϕ bolts (parallel to grain)

$F_{all} = \phi \text{ x } F_n$ $\phi=1.0$

$F_n = 2 \text{ x } 1.75 \text{ x } 745 = 2.61k$

$F_{all} = 1.0 \text{ x } 2.61$

$=2.61$k/bolt (11.6 kN)

2 = NEHRP multiplier 1.75 = metal side plate
745lbs (3.3 kN)= National Design Specification value

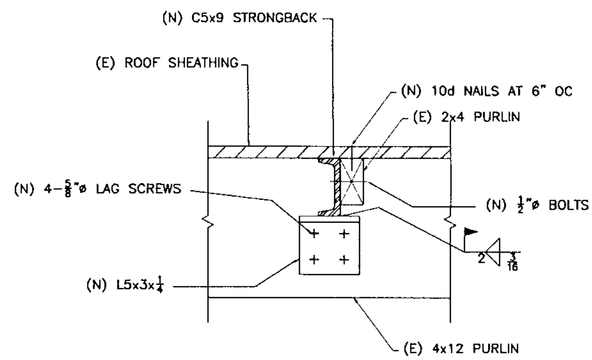


Figure E1-9: Section B-B

Use 3-½" ϕ bolts $F_{act} = 3 \text{ x } 2.61 = 7.83k\ (34.8\ kN) > 2.27k\ (10.1\ kN)$ O.K.

Nailing of sheathing to 2x4 purlin.

$V_{max} = 2.27k$ (10.1 kN) $v = \frac{2.27k}{8} = 0.284$ k/ft (4.14 kN / m)

For 10d nails- $F_{all} = \phi F_n = 1.0 \text{ x } 2 \text{ x } 76 = 152$ lb/nail (676 N)

Use 10d nails- 6" on center to 2x4, $F_{act} = \frac{12}{6} x152 = 304 lb/ft\ (4.4\ kN/m) > 284\ lb/ft\ (4.14\ kN/m)$ O.K.

Design Condition where Parapet Brace is Parallel to 4x12 rafter

Dead Load = 20.0 psf (958 Pa) (Assumed)

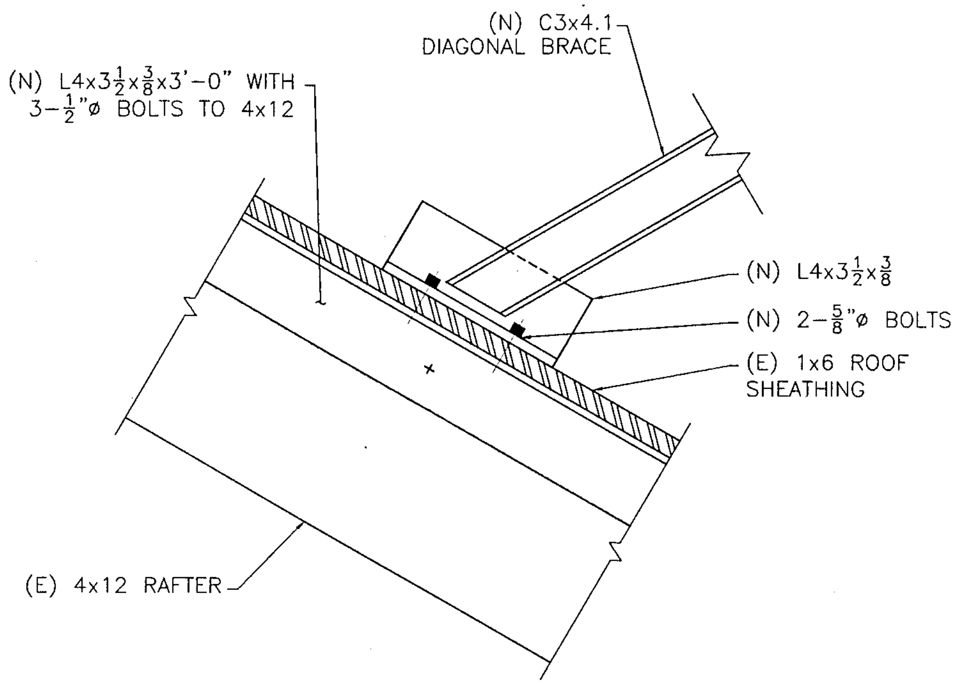
L = 20' w = 20.0 x 8 = 160 lb/ft (876 N / m)

$$M = \frac{160 lb/ft(20ft)^2}{8} = 8k-ft = 96k-in\ (10.8\ kN\text{-}m) \quad S = \frac{3.625(11.625)^2}{6} = 81.6 in^3$$

$$f = \frac{M}{S} = \frac{96}{81.6} = 1176psi \text{ O.K., Assume } F_{all} = \sim 1400 \text{ psi}$$

Rafters are sized for stiffness and have a large excess capacity for brace loads.

Use L4x3½x3/8x3'-0" to receive parapet load brace load and to transfer it to 4x12 rafters.



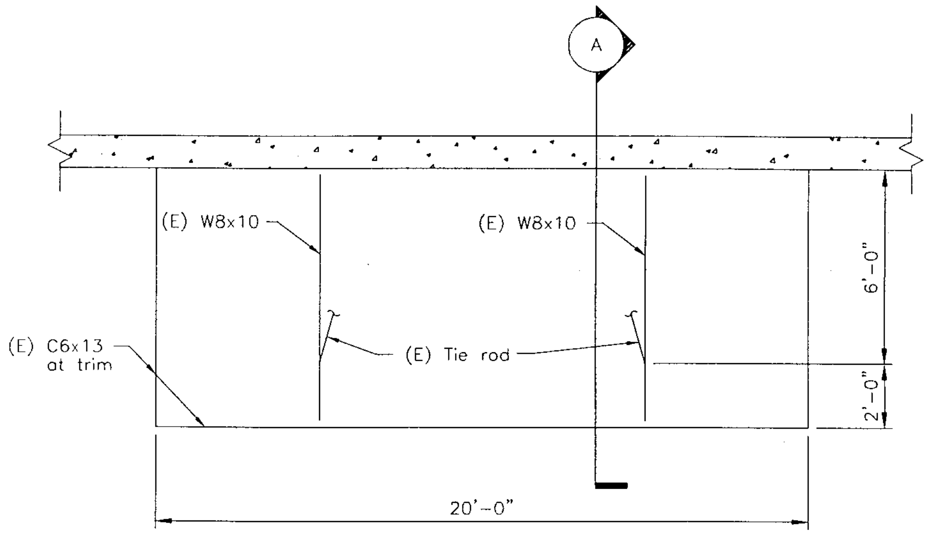
1 in = 25.4 mm

Figure E1-10: Detail for Parapet Brace at 4x12 Rafter

DESIGN EXAMPLE PROBLEM E2: Evaluation and Retrofit of Canopy at Building Entrance

Description

This example consists of the evaluation and rehabilitation of an existing steel frame canopy over the main entrance of a two-story military building. The canopy measures 20' long in plan and extends out 8' from the exterior of the building. It was designed for gravity loads only, with steel decking over steel wide flange beams and channels supported by two tie rods connected to existing concrete walls.



1 ft = 0.305 m

Figure E2-1: Canopy Plan

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is a Standard Occupancy Structure, and from Table 3-1, falls into Seismic Use Group I.

b. Structural Performance Level. The canopy is to be analyzed for the Life Safety Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective for the canopy is determined to be 1A, defined as the combination of Life Safety Performance Level with a ground motion of 2/3 MCE as prescribed for Seismic Use Group I. For this example, the spectral response acceleration is assumed to be as follows:

$S_{DS} = 2/3\ S_{MS} = 0.80$ (TI 809-04 Eq. 3-3)

The canopy will be evaluated for vertical acceleration equal to 2/3 S_{DS}.

d. Seismic design category:

Based on Short Period Response Acceleration:

Seismic design category: D (Table 3-4a)

Based on 1 second period Response Acceleration:
Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

Preliminary assessment of the canopy is based upon available drawings and visual inspection of the accessible components. A plan view and section of the canopy is given in Figure E2-1.

1) Exempt Components

Not applicable. The canopy is not considered an exempt component.

2) Classification of Component

The canopy is assigned an importance factor, I_p of 1.5 and classified as important because it could impede safe egress from a principle building exit.

3) Disposition

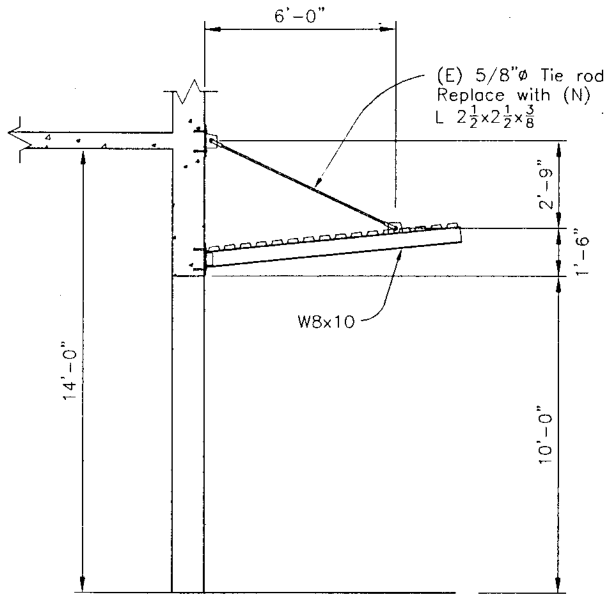
The canopy shall be screened by the Tier 1 evaluation of FEMA 310.

G. Nonstructural Screening (Tier 1)

The canopy is anchored at a spacing greater than 10 feet for Life Safety, and so a Tier 2 evaluation is required.

H. Nonstructural Evaluation (Tier 2)

The canopy is subjected to a Tier 2 analysis according to the provisions of Section 4.8 of FEMA 310 except as modified by Section 6.3 of this document. Analysis is performed as follows.



1 ft = 0.305 m
1 in = 25.4 mm

Figure E2-2: Section A of Canopy

Determine Gravity Forces on Supports

Dead Load:

Metal Deck	3 psf
Steel support members	2 psf
Roofing	3 psf
Total:	8 psf (383 Pa)

Live Load: 5 psf (239 Pa)

Tributary Area = 20' x 5' = 100 sqft

Total Dead Load: 8 psf x 100 sqft = 800 lbs. (3.56 kN)
Total Live Load: 5 psf x 100 sqft = 500 lbs. (2.22 kN)

Dead Load on each 5/8" tie rod: (For Brace at 60 degrees from vertical)
2(0.8 k) / 2 tie rods = 0.8 kips (3.56 kN) dead load per tie rod

Live Load on each 5/8" tie rod:
2(0.5 k) / 2 tie rods = 0.5 kips (2.22 kN) live load per tie rod

Determine Seismic Forces

Select R_p and a_p, factors:

$a_p = 2.5$ (TI 809-04, Table 10-1)
$R_p = 1.5$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4 a_p I_p S_{DS} W_p}{R_p}\left(1+2\frac{x}{h}\right);\ W_p = 0.8\text{ k }(3.56\text{ kN});\ x/h = 0.5 \qquad \text{(TI 809-04, EQ. 10-1)}$$

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-3)}$$

$$F_p = \frac{0.4(2.5)(1.5)(2/3)0.65(0.8\text{ k})}{1.5}(1+2(0.5)) = 0.87(0.8\text{ k}) = 0.7\text{ k }(3.11\text{ kN})$$

$(F_p)_{max} = 1.6(0.65)1.5(0.8\text{k}) = 1.3\text{ k} > 0.7\text{ k} = F_p$ **O.K.**

$(F_p)_{min} = 0.3(0.65)1.5(0.8\text{ k}) = 0.3\text{ k} < 0.7\text{ k} = F_p$ **O.K.**

$\therefore\ F_p = 0.7\text{ k }(3.11\text{ kN})$

Seismic Force on each 5/8" tie rod at 60 degrees from vertical:
2 x 0.7 k /2 tie rods = 0.7 kips (3.11 kN) seismic load per tie rod

Check 5/8" supporting tie rods

Since the rods are tension-only members, the ability of the dead load to resist the seismic forces imposed by vertical acceleration of the canopy must be checked:

$Q_u = 0.9D - 1.0Q_E$
$= 0.9(0.6\text{ k}) - 1.0(0.7\text{ k}) = -0.2\text{ k}$ (-.89 kN) (Net compression in tie rod) **NG**

I. Evaluation Report

The Evaluation Report shall summarize the following as required for this example problem:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations

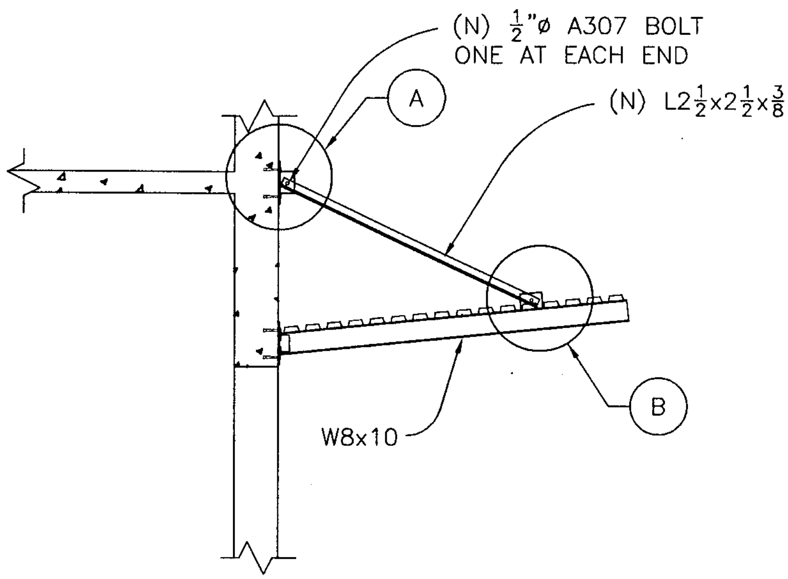
A judgmental assessment of the results of the evaluation and a statement of the evaluator's assessment of the level of confidence are to be included as part of the report. The dead load for the canopy is not capable of resisting the seismic force associated with vertical acceleration of the canopy, and supporting tie rods are capable of resisting tension loads only. It is determined that the canopy definitely needs rehabilitation.

6. Rehabilitation strategy

If the canopy is to remain, the most cost-effective rehabilitation option for the canopy is to replace the tie rods with a steel member capable of resisting the compressive forces associated with vertical seismic forces.

7. Rehabilitation concept

The rehabilitation concept is shown in Figure E2-2. It consists of the addition of steel angle bracing attached to the roof and to the concrete wall.



1 in = 25.4 mm

Figure E2-3: Canopy Bracing

J. Rehabilitation Design

The procedures for rehabilitation are outlined below:

1. Review Evaluation Report and other available data.

2. Site Visit.

3. Confirming evaluation of existing building (if necessary).

4. Prepare alternative structural rehabilitation concepts.

5. Rehabilitation design.

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 7 and 9 of this document and FEMA 302 for the design and detailing of new structural components. A detailed analysis follows.

Determine gravity load effects

For the canopy, gravity loads are determined thus:

P_u=1.4D
P_u=1.2D+1.6L_r

(ANSI/ASCE 7-95)

Note: Wind loads are not included in this analysis. For a complete design, any nonstructural component must also be checked for the effects of any applicable wind loads in accordance with the load combinations prescribed by ANSI/ASCE 7-95.

P_u=1.4(0.8 k) = 1.1 kips (4.89 kN) axial tension per brace
P_u= [1.2(0.8 k) + 1.6(0.4k)] = 1.6 kips (7.12 kN) axial tension per brace (Governs)

Try L2 ½ x 2 ½ x 3/8 A=1.73 in.^{2}

1.6 k (7.12 kN) per brace < 0.9(36ksi)(1.73) = 56.0 k (249 kN) per brace OK

Design for Combination of seismic and gravity load effects

$Q_u = 1.0Q_G + 1.0Q_E$

where Q_G:
=1.2 Q_D + 0.5Q_L + 0.2 Q_S (Eq. 7-1)
=0.9 Q_D
and Q_E= F_p

For Tension on brace (gravity and seismic forces are additive):

Q =1.2(0.8k)+0.5(0.4k)+1.0(0.7k) = 1.9 k (8.45 kN) per brace

1.9 k (8.45 kN) per brace < 0.9(36ksi)(1.73) = 56.0 k (249 kN) per brace **OK**

For compression in brace:

Q =0.9(0.8k)-1.0(0.7k) = -0.02 k (89 N) per brace net compression

For L2 ½ x 2 ½ x 3/8 r_{min}=0.753 in. A=1.73 in.^{2}

L_{max} = 79"

L/r = 79/0.753 = 105

$\phi_c F_{cr}$ = 13 ksi

$\phi P_n = \phi_c F_{cr}(A) = 13(1.73) = 23\,k$ (102 kN) per brace **O.K.**

0.02 k per brace (89 N) $< = 23$ k (102 kN) per brace compression **OK**

USE L2 ½ x 2 ½ x 3/8 Brace to wall

<u>Design connection to concrete wall:</u>

Anchor steel angle to wall using a bolted connection to a 6x4x¼ gusset plate that is welded to a ¼" plate. The plate is bolted to the wall using 4-3/8" ϕ adhesive anchors.

Total Demand per bolt = 1.9 k per brace/4 bolts = 0.48k (2.15 kN)

Demand shear per bolt = $0.48/2 = 0.24$k (1.08 kN)

Demand tension per bolt = $0.48\mathrm{x}\sqrt{3}/2 = 0.42$k (1.87 kN)

Bolt shear capacity:

Test data and design values for various proprietary post-installed systems are available from various sources, including International Conference of Building Officials (ICBO) reports and in manufacturer's literature.

Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, FEMA 273 Sec. C6.4.6.2 recommends using a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropriate to use a design strength equal to half the tabulated average ultimate value.

For a 3/8" ϕ (9.5 mm) adhesive anchor (ASTM A36) with 3 ½" (89 mm) embedment depth and minimum spacing requirements satisfied, a working load value of 1110 lbs (4.89 kN) in shear and 1550 lbs (6.89 kN) in tension is obtained from ICBO reports. The design values used are 2 x 1110 lbs = 2220 lbs (9.87 kN) in shear and 2 x 1550 lbs = 3100 lbs (13.8 kN) in tension.

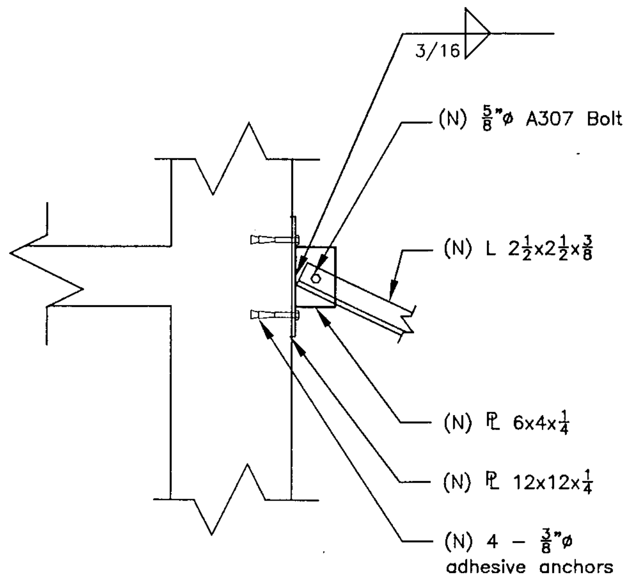
$$\frac{V_u}{V_c}+\frac{P_u}{P_c}=\frac{240}{2220}+\frac{420}{3100}=0.11+0.14=0.25\leq 1.0 \qquad \textbf{O.K.}$$

Use 4-3/8" ϕ adhesive anchor at each brace.

The 6x4x¼ gusset plate is shop welded to a 12x12x¼ plate with a 3/16" fillet weld on both sides.

The angle brace is connected to the gusset plate a with 5/8" ϕ (15.9 mm) A307 bolt. From the LRFD Manual Volume II, the design shear strength of a bolt is 5520 lbs (24.6 kN).

Bolt capacity = 5.52 k (24.6 kN) > Demand force = 1.9 k (8.4 kN). O.K.



1 in = 25.4 mm

Figure E2-4: Detail A - Connection to wall

Design connection to canopy:

The connection of the angle bracing to the canopy framing is similar to the connection to the wall, except that the gusset plate is welded directly to the top flange of the W8x10 framing with 3/16" fillet weld on both sides. The capacity of the weld is o.k. by inspection.

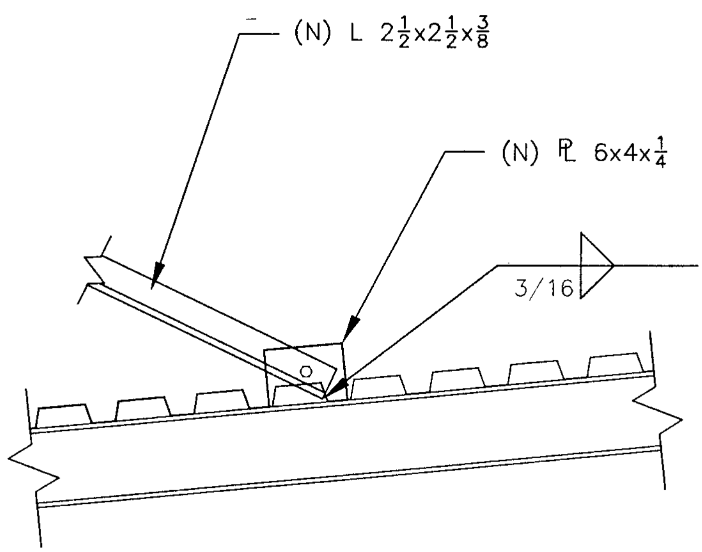
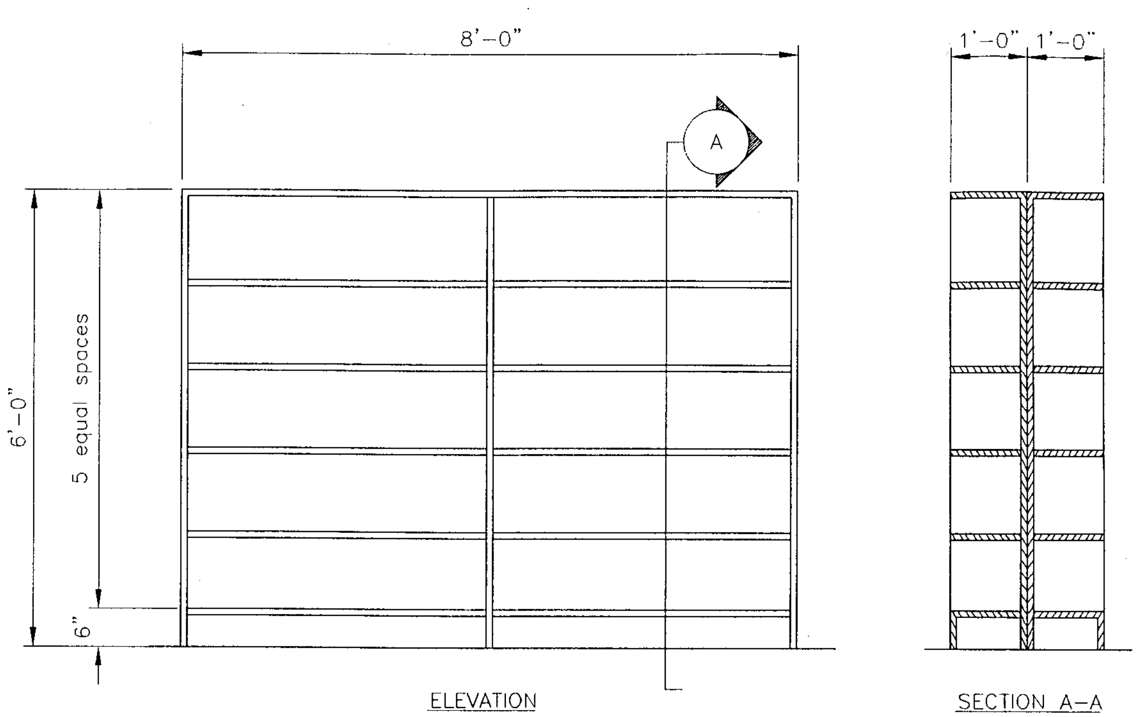


Figure E2-5: Detail B - Connection to Canopy Framing

DESIGN EXAMPLE PROBLEM E3: Bracing of Library Shelving

Description

This example consists of the evaluation and bracing of two free-standing library bookshelves located on the second floor of a two-story library building. The shelves are entirely constructed of 1-inch plywood and are to be evaluated and rehabilitated as required only for stability under ground motion.



1 ft = 0.305 m
1 in = 25.4 mm

Figure E3-1. Two Free-standing Library Shelves

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is a Standard Occupancy Structure, and from Table 3-1, falls into Seismic Use Group I.

b. Structural Performance Level. The shelves are to be analyzed for the Life Safety Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective for the shelves is determined to be 1A, defined as the combination of Life Safety Performance Level with a ground motion of 2/3 MCE as prescribed for Seismic Use Group I. For this example, the spectral response acceleration is assumed to be as follows:

$$S_{DS} = 2/3\ S_{MS} = 0.90\ g \qquad \text{(TI 809-04 Eq. 3-3)}$$

d. Seismic design category:
Based on Short Period Response Acceleration:
Seismic design category: D (Table 3-4a)
Based on 1 second period Response Acceleration:
Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

Preliminary assessment of the shelves is based upon available drawings and visual inspection of the accessible components. .

1) Exempt Components

Not applicable. The shelves are not considered an exempt component.

2) Classification of Component

The shelves do not constitute a significant life safety hazard and are assigned an importance factor, I_p of 1.0.

3) Disposition

The shelves shall be screened by the Tier 1 evaluation of FEMA 310.

G. Nonstructural Screening (Tier 1)

The shelves have a height-to-depth ratio greater than 4 and are not anchored to the floor or adjacent walls. A Tier 2 evaluation is required.

H. Nonstructural Evaluation (Tier 2)

The library shelves are subjected to a Tier 2 analysis according to the provisions of Section 4.8 of FEMA 310 except as modified by Section 6.3 of this document. Analysis is performed as follows.

Determine Seismic Forces

Select R_p and a_p, factors:

$a_p = 1.0$ (TI 809-04, Table 10-1)
$R_p = 3.0$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4 a_p I_p S_{DS} W_p}{R_p}\left(1 + 2\frac{x}{h}\right); \; W_p = 1.92 \text{ k } (8.54 \text{ kN}); \; x/h = 0.5 \qquad \text{(TI 809-04, EQ. 10-1)}$$

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-3)}$$

$$F_p = \frac{0.4(1.0)(1.0)0.90(1.92 \text{ k})}{3.0}(1 + 2(0.5)) = 0.24(1.92k) = 0.46 \text{ kips } (2.05 \text{ kN})$$

$(F_p)_{max} = 1.6(0.90)1.0(1.92\text{k}) = 2.77 \text{ k} > 0.46 \text{ k} = F_p$ **O.K.**

$(F_p)_{min} = 0.3(0.90)1.0(1.92 \text{ k}) = 0.52 \text{ k} > 0.46 \text{ k} = F_p$ **Governs**

$F_p = 0.52$ k (2.3 kN)

Check Overturning Stability of Bookshelves

$Q_u = 0.9D - 1.0Q_E$

$M_{OT} = F_p(h/2) - 0.9W_p(L/2)$

=0.52k(6'/2) – 0.9(1.92k)(1'/2) = 0.70 kip-ft (0.95 kN-m) NET OVERTURNING **- Retrofit required**

I. Evaluation Report

The Evaluation Report shall summarize the following as required for this example problem:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations
A judgmental assessment of the results of the evaluation and a statement of the evaluator's assessment of the level of confidence are to be included as part of the report. The dead load for the bookshelves is not adequate to resist overturning forces from seismic loads. It is determined that the bookshelves require rehabilitation.

6. Rehabilitation strategy/ concept
Rehabilitation concept will involve the addition of seismic bracing elements strapped across the bookshelves and bolted to the roof slab.

J. Rehabilitation Design

The procedures for rehabilitation are outlined below:

1. Review Evaluation Report and other available data.

2. Site Visit.

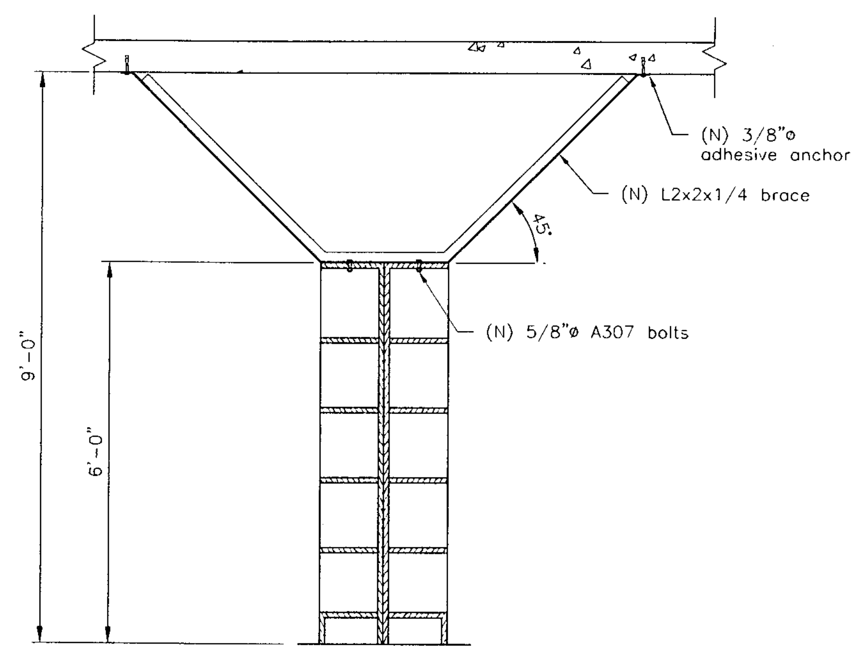
3. Confirming evaluation of existing building (if necessary).

4. Prepare alternative structural rehabilitation concepts.

5. Rehabilitation design.

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 7 and 9 of this document and FEMA 302 for the design and detailing of new structural components.

<u>Design new braces:</u>



1 ft = 0.305 m

Figure E3-2. Bracing at shelves

Provide 4 braces strapped to top of shelves and bolted to underside of roof slab above.

P_{brace}=1.41(520 lbs)/4 = 183 lbs (814 N)

Try L 2x2x1/4": r_{min}=0.609 in. A=0.938 in.^{2} $L_{max} = 36"x\sqrt{2}$ =51 in.

L/r = 51/0.609 = 84

$\phi_c F_{cr} = 9$ ksi

$\phi P_n = \phi_c F_{cr}(A) = 9(0.938) = 8.42$ k per brace (37.5 kN) > 0.183 k (814 N) per brace **O.K.**

Use L2x2x1/4" braces in each direction.

Design connection to slab:

Anchor steel angles to underside of slab with 1-3/8" ϕ adhesive anchor at each side.

Total Demand per bolt = 183 lbs (814 N)

Demand shear per bolt = $183/\sqrt{2} = 129$ lbs (574 N)

Demand tension per bolt = $183/\sqrt{2} = 129$ lbs (574 N)

Bolt shear capacity:

Test data and design values for various proprietary post-installed systems are available from various sources, including International Conference of Building Officials (ICBO) reports and in manufacturer's literature.

Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, FEMA 273 Sec. C6.4.6.2 recommends using a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropriate to use a design strength equal to half the tabulated average ultimate value.

For a 3/8" ϕ (9.5 mm) adhesive anchor (ASTM A36) with 3½" (90 mm) embedment depth and minimum spacing requirements satisfied, a working load value of 1110 lbs (4.94 kN) in shear and 1550 lbs (6.89 kN) in tension is obtained from ICBO reports. The design values used are 2 x 1110 lbs = 2220 lbs (9.87 kN) in shear and 2 x 1550 lbs = 3100 lbs (13.8 kN) in tension.

$$\frac{V_u}{V_c} + \frac{P_u}{P_c} = \frac{129}{2220} + \frac{129}{3100} = 0.06 + 0.04 = 0.10 < 1.0 \qquad \text{O.K.}$$

Use a 3/8" ϕ adhesive anchor at each brace.

Design connection between brace and shelves

To connect the angles to the braces use 5/8" ϕ (15.9 mm) A307 bolts. From the LRFD Manual Volume II, the design shear strength of a bolt is 5520 lbs (24.6 kN) and the design tensile strength is 10400 lbs (46.3 kN). O.K. by inspection for 129 lbs (574 N) in shear and tension.

APPENDIX F
MECHANICAL AND ELECTRICAL COMPONENT EXAMPLES

This appendix illustrates the implementation of the provisions of this document for the seismic evaluation and rehabilitation of nonstructural mechanical and electrical components in military buildings. The examples in the following sections of this appendix were selected to demonstrate the application of various rehabilitation techniques to mitigate seismic deficiencies identified in typical mechanical and electrical components in existing military buildings.

F1. Electrical Control Panel

F2. Emergency Motor Generator

F3. Suspended Chiller Unit

DESIGN EXAMPLE PROBLEM F1: Electrical Control Panel

Description

This example consists of the evaluation and bracing of a free-standing electrical control panel on the ground floor of a two-story Seismic Use Group IIIE building.

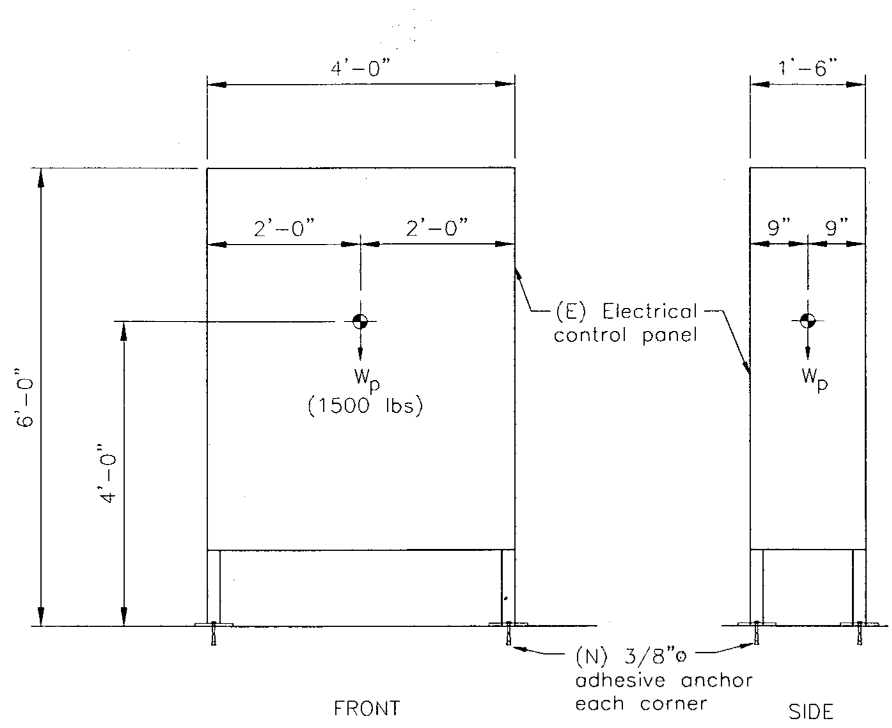


Figure F1-1. Electrical Control Panel

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is assumed from the problem statement to be Seismic Use Group IIIE.

b. Structural Performance Level. The electric panels are to be analyzed for the Immediate Occupancy Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective for the electric panels is determined to be 3B, defined as the combination of Immediate Occupancy Performance Level with a ground motion of 3/4 MCE as prescribed for Seismic Use Group IIIE. For this example, the spectral response acceleration is assumed to be as follows:

$$S_{DS} = 2/3\ S_{MS} = 0.90\ g \qquad \text{(TI 809-04 Eq. 3-3)}$$

d. Seismic design category:
Based on Short Period Response Acceleration:
Seismic design category: D (Table 3-4a)
Based on 1 second period Response Acceleration:
Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

Preliminary assessment of the electrical panel is based upon available drawings and visual inspection of the accessible components.

1) Exempt Components

Not applicable. The panels are not considered an exempt component.

2) Classification of Component

The panel controls electrical circuits that must be functional during and following a severe earthquake. The panel is therefore assigned an importance factor, I_p of 1.5.

3) Disposition

The panels shall be screened by the Tier 1 evaluation of FEMA 310.

G. Nonstructural Screening (Tier 1)

The electrical panel is free-standing. A Tier 2 evaluation is required to check if it needs anchorage to the floor.

H. Nonstructural Evaluation (Tier 2)

The electrical panel is subjected to a Tier 2 analysis according to the provisions of Section 4.8 of FEMA 310 except as modified by Section 6.3 of this document. Analysis is performed as follows.

Determine Seismic Forces

Select R_p and a_p, factors:

$a_p = 2.5$ (TI 809-04, Table 10-1)
$R_p = 3.0$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4a_pI_pS_{DS}W_p}{R_p}\left(1+2\frac{x}{h}\right);\ W_p = 1500 \text{ lbs } (6.67 \text{ kN}),\ x/h = 0 \quad \text{(TI 809-04, EQ. 10-1)}$$

F_p is not required to be greater than:

$$F_p = 1.6S_{DS}I_pW_p \quad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

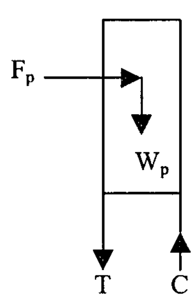
$$F_p = 0.3S_{DS}I_pW_p \quad \text{(TI 809-04, EQ. 10-3)}$$

$$F_p = \frac{0.4(2.5)(1.5)0.90(1500\text{lbs})}{3.0}(1+2(0)) = 0.45(1500\text{lbs}) = 675 \text{ lbs } (3.0 \text{ kN})$$

$$(F_p)_{max} = 1.6(0.90)1.5(1500\text{lbs}) = 3240 \text{ lbs} > 675 \text{ lbs} = F_p \quad \textbf{O.K.}$$
$$(F_p)_{min} = 0.3(0.90)1.5(1500 \text{ lbs}) = 610 \text{ lbs} < 675 \text{ lbs} = F_p \quad \textbf{O.K.}$$

$$\therefore\ F_p = 675 \text{ lbs } (3.0 \text{ kN})$$

Check Overturning of Electrical Panel



$Q_u = 0.9D - 1.0Q_E$

$M_{OT} = F_p(ht) - 0.9W_p(L/2)$

=675lbs(4') – 0.9(1500)(0.75) = 1690 lbs-ft (2.29 kN-m) NET OVERTURNING
Anchorage required to resist overturning.

I. Evaluation Report

The Evaluation Report shall summarize the following as required for this example problem:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations

A judgmental assessment of the results of the evaluation and a statement of the evaluator's assessment of the level of confidence are to be included as part of the report. The dead load for the electrical panel is not adequate to resist sliding and overturning forces from seismic loads. It is determined that the electrical panel requires rehabilitation.

6. Rehabilitation strategy/ Concept

The rehabilitation will require the addition of anchor bolts drilled into the concrete to resist the seismic forces.

J. Rehabilitation Design

The procedures for rehabilitation are outlined below:

1. Review Evaluation Report and other available data.

2. Site Visit.

3. Confirming evaluation of existing building (if necessary).

4. Prepare alternative structural rehabilitation concepts.

5. Rehabilitation design.

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 7 and 9 of this document and FEMA 302 for the design and detailing of new structural components. A detailed analysis follows.

Design Anchor bolts:

T=C=M/L = 1690 lbs / 1.5' = 1130 lbs (5.03 kN)

For 2 bolts, tension load per bolt = 1130 lbs/2 = 565 lbs/bolt (2.51 kN)

Shear per bolt = 675 lbs / 4 = 170 lbs (756 N)

Check bolt capacity:

Test data and design values for various proprietary post-installed systems are available from various sources, including International Conference of Building Officials (ICBO) reports and in manufacturer's literature.

Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, FEMA 273 Sec. C6.4.6.2 recommends using a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropriate to use a design strength equal to half the tabulated average ultimate value.

For a 3/8" (9.5 mm) ϕ adhesive anchor (ASTM A36) with 3½" (89 mm) embedment depth and minimum spacing requirements satisfied, an ICBO report provides the following allowable working loads:

Shear: 1110 lbs (4.94 kN) > 170 lbs (756 N) **OK**
Tension: 1550 lbs (6.89 kN) > 565 lbs (2.51 kN) **OK**

According to the ICBO report, allowable loads for anchor subjected to combined shear and tension forces are determined by the ratio of the actual shear to the allowable shear, plus the ratio of the actual tension to the allowable tension, not exceeding 1.0.

$$\left[\left(\frac{P_u}{P_c}\right)+\left(\frac{V_u}{V_c}\right)\right] \le 1.0$$

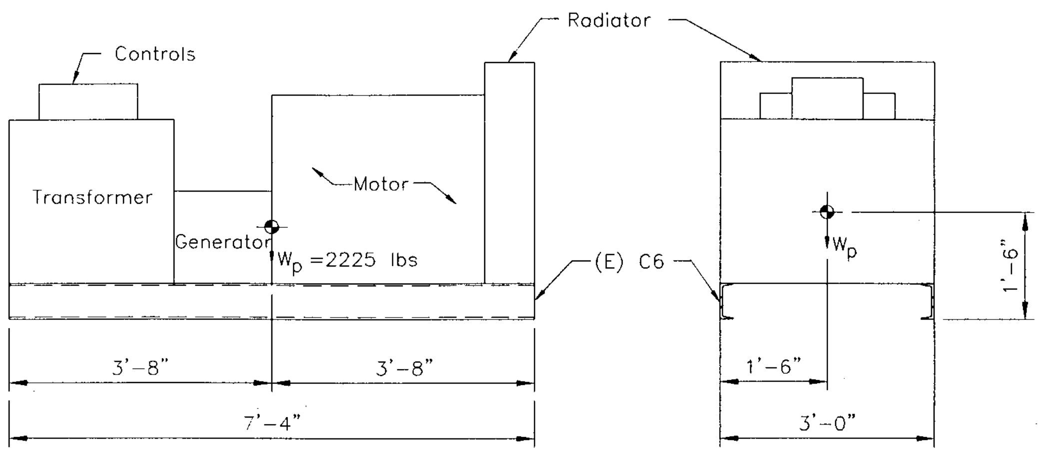
$$\left[\left(\frac{565}{1712}\right)+\left(\frac{170}{1110}\right)\right] = 0.48 \le 1.0 \quad \textbf{OK}$$

USE 3/8" ϕ chemical anchor at each corner.

DESIGN EXAMPLE PROBLEM F2: Emergency Motor Generator

Description

This example consists of the evaluation of an emergency motor generator set shown in Figure F2-1 on the ground floor of a 3-story military hospital. The unit has been mounted on four springs for vibration control. The stiffness factor for each spring is 300 lbs/in.



1 in = 25.4 mm
1 ft = 0.305 m

Figure F2-1. Emergency Motor Generator

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is in Seismic Use Group IIIE, Essential Facilities.

b. Structural Performance Level. The performance level prescribed for buildings in Seismic Use Group IIIE is the Immediate Occupancy Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective is determined to be 3B, defined as the combination of Immediate Occupancy Performance Level with a ground motion of 3/4 MCE as prescribed for Seismic Use Group IIIE. For this example, the spectral response acceleration is assumed to be as follows:

$S_{DS} = 3/4\ S_{MS} = 0.90$ g (TI 809-04 Eq. 3-3)

The design vertical acceleration is assumed to be 2/3 $S_{DS} = 0.60$ g.

d. Seismic design category:

Based on Short Period Response Acceleration:
Seismic design category: D (Table 3-4a)
Based on 1 second period Response Acceleration:
Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

Preliminary assessment is based upon available drawings and visual inspection of the accessible components.

1) Exempt Components

Not applicable. The generator is not considered an exempt component.

2) Classification of Component

The generator must be operable during and after the design earthquake, and is therefore assigned an importance factor, I_p of 1.5.

3) Disposition

The emergency motor generator shall be screened by the Tier 1 evaluation of FEMA 310.

G. Nonstructural Screening (Tier 1)

The generator is mounted on vibration isolators without restraints or snubbers. Restraints are required to prevent movement in all directions. A Tier 2 evaluation is not available for non-compliant equipment mounted on vibration isolators, and rehabilitation is necessary to achieve the selected performance level.

H. Nonstructural Evaluation (Tier 2)

Not required. Equipment was found to be non-compliant as part of the Tier 1 evaluation, and rehabilitation was recommended.

I. Evaluation Report

The Evaluation Report shall summarize the following as required for this example problem:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations

With the lack of restraint against lateral and vertical movement, earthquake forces can cause the equipment to fall off its isolaters. Without restraints or snubbers, mitigation is required to achieve the selected performance level.

6. Rehabilitation strategy/ Concept

The rehabilitation will require the design of a horizontal and vertical stop assembly to maintain stability of the isolated unit.

J. Rehabilitation Design

The procedures for rehabilitation are outlined below:

1. Review Evaluation Report and other available data.

2. Site Visit.

3. Confirming evaluation of existing building (if necessary).

4. Prepare alternative structural rehabilitation concepts.

5. Rehabilitation design.

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 7 and 9 of this document and FEMA 302 for the design and detailing of new structural components. A detailed analysis follows.

<u>Determine Seismic Forces</u>

Select R_p and a_p, factors:

$a_p = 2.5$ (TI 809-04, Table 10-1)
$R_p = 3.0$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4 a_p I_p S_{DS} W_p}{R_p}\left(1 + 2\frac{x}{h}\right);\ W_p = 2225 \text{ lbs } (9.90 \text{ kN})\ ;\ x/h = 0 \qquad \text{(TI 809-04, EQ. 10-1)}$$

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-3)}$$

$$F_{p\text{-}H} = \frac{0.4(2.5)(1.5)0.90(2225\text{lbs})}{3.0}(1+2(0)) = 0.45(2225\text{lbs}) = \text{ 1000 lbs (4.45 kN)}$$

$$(F_{p\text{-}H})_{max} = 1.6(0.90)1.5(2225\text{lbs}) = 4800 \text{ lbs} > 1000 \text{ lbs} = F_{p\text{-}H} \qquad \textbf{O.K.}$$

$$(F_{p\text{-}H})_{min} = 0.3(0.90)1.5(2225 \text{ lbs}) = 900 \text{ lbs} < 1000 \text{ lbs} = F_{p\text{-}H} \qquad \textbf{O.K.}$$

$$\therefore\ F_{p\text{-}H} = 1000 \text{ lbs (4.45 kN)}$$

$$F_{p\text{-}V} = (2/3)F_{p\text{-}H} = 667 \text{ lbs (2.97 kN)}$$

Forces at Support

Shear (ΣV_H=0):

V_H=$F_{p\text{-}H}$

V_H=1000 lbs. (4.45 kN)

Overturning (ΣM_0=0):

When gravity and seismic loads are additive:

Load Combination: $Q_u = 1.2D + 1.0Q_E$ (EQ. 7-1)

$C(L) = F_{p\text{-}H}\,(h_{CG}) + 1.2W_c(L/2) + F_{p\text{-}V}\,(h_{CG})$

$C(36) = [1000(18)+1.2(2225)(15)+667(18)]$

C=1950 lbs (8.67 kN)

When gravity loads counteract seismic loads (Uplift):

Load Combination : $Q_u = 0.9D - 1.0Q_E$

$T(L) = F_{p\text{-}H}\,(h_{CG}) - 0.9W_c(L/2) + F_{p\text{-}V}\,(L/2)$

$T(36) = [1000(18) - 0.9(2225)(15) + 667(15)]$

T=-56 lbs (-249 N) **No Net Uplift**

Forces at top of each support (spring):

$V_{H\text{-}Support}$(at top of support) = 1000/4 = 250 lbs. (1112 N)

C_{down}=1950 lbs (8674 N)

Mitigation

The maximum displacement of the isolater springs subjected to the vertical acceleration shall be calculated and a horizontal and vertical stop assembly designed to maintain stability of the isolated unit. Restraint must be able to resist the vertical and horizontal reactions.

The maximum acceleration experienced by isolator spring with natural period, T, is equal to the vertical spectral acceleration $S_{a\text{-vert}}$, corresponding to the period, T, from a vertical response spectrum provided by a geotechnical engineer.

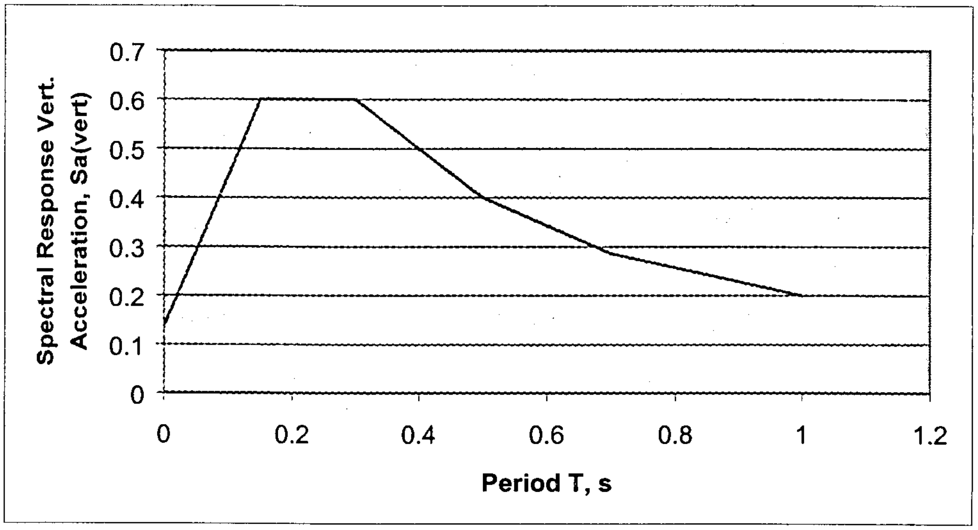


Figure F2-3. Vertical Response Spectrum (provided by Geotechnical Engineer)

Find natural period for vertical translation:

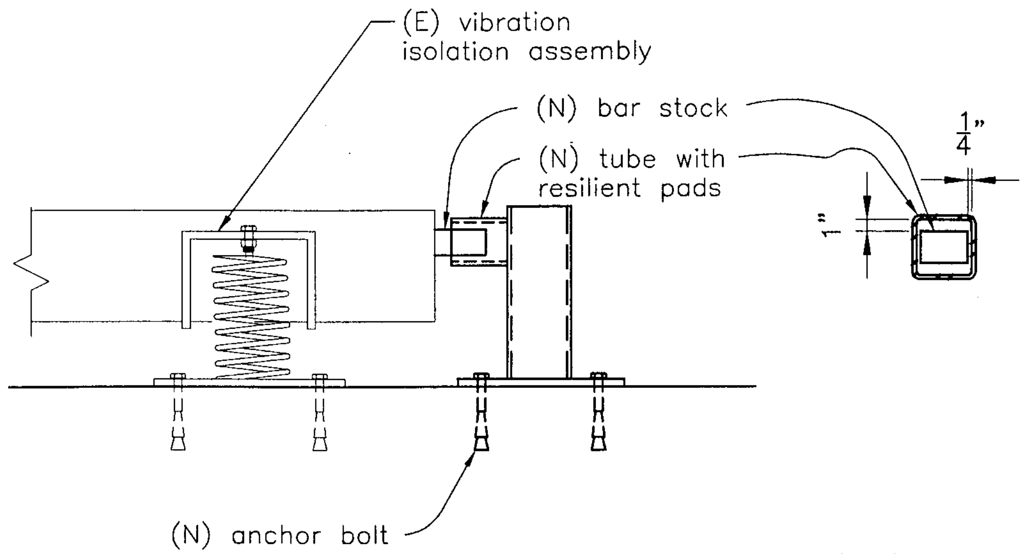
$$T = 2\pi\sqrt{\frac{W}{kg}} = 2\pi\sqrt{\frac{2225\text{lbf} / 4 \text{ springs}}{300\text{lbs} / \text{in.}(386 \text{ in.}/\text{s}^2)}} = 0.44s$$

From vertical response spectrum, $S_{a\,(vert)} = 0.45$ s

For undamped, single-degree-of-freedom system in simple harmonic motion, the spectral displacement, $S_d = S_a / \omega^2 = S_a(T/2\pi)^2$
$=0.45(386.4 \text{ in/s}^2)\,(0.44/(2\pi))^2 = 0.85$ in (22 mm)

Set gap at 1" (25 mm) to allow for vertical displacement.

A seismic restraint may be designed as shown in Figure F2-1.



1 in = 25.4 mm

Figure F2-2. Seismic Restraint for Vibration isolated Equipment

Use a total of eight anchor assemblies as shown above, two on each side with TS4x4x1/4 sections welded all around with 3/16" fillet weld to a 1/2" thick 10"x10" base plate with 4 Hilti Kwik bolts.

Check anchor capacity:

It is assumed that only two anchor assemblies would resist the lateral force demand at a time.
Therefore, the shear demand on one anchor assembly = 1000/2 = 500 lbs (2.22 kN)
Shear demand on one anchor bolt = 500/4 = 125 lbs (556 N)

The height of the anchor assembly will be based on the height of the equipment base. It is assumed that the height for this example is 12" (305 mm).
The lateral force on the top of the anchor assembly will force two of the anchor bolts to act in tension.

Tension force on one anchor bolt = {(500lbs/2) x 12"} /7" = 429 lbs (1908 N)

A 3/8" (9.5 mm) ϕ Hilti Kwik Bolt with 4 1/4" (108 mm) embedment depth has the following allowable working loads:

Shear: 1470 lbs (6.53 kN) > 125 lbs (556 N) **OK**
Tension: 1390 lbs (6.18 kN) > 429 lbs (1908 N) **OK**

According to the ICBO report, allowable loads for anchor subjected to combined shear and tension forces are determined by the ratio of the actual shear to the allowable shear, plus the ratio of the actual tension to the allowable tension, not exceeding 1.0.

$$\left[\left(\frac{P_u}{P_c}\right)+\left(\frac{V_u}{V_c}\right)\right] \leq 1.0$$

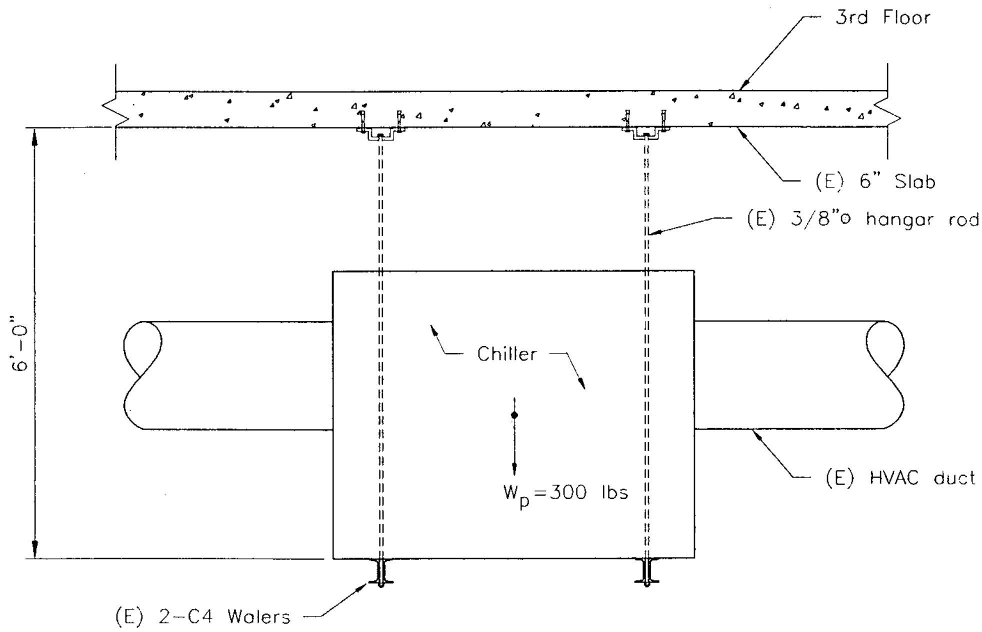
$$\left[\left(\frac{429}{1390}\right)+\left(\frac{125}{1470}\right)\right] = 0.40 \leq 1.0 \quad \textbf{OK}$$

USE 4-3/8" ϕ Hilti Kwik bolts at each anchor assembly.

DESIGN EXAMPLE PROBLEM F3: Suspended Chiller Unit

Description

This example consists of the evaluation and retrofit of a chiller that is part of the HVAC system in the 2^{nd} story of a 3-story building. The chiller is suspended by hangar rods from the 3^{rd} floor slab as shown in Figure F3-1.



1 ft = 0.305 m
1 in = 25.4 mm
1 lb = 4.448 N

Figure F3-1. Suspended Chiller Unit

A. Preliminary Determinations

1. Obtain building and site data:

a. Seismic Use Group. The building is a Standard Occupancy Building, and from Table 3-1, falls into Seismic Use Group I.

b. Structural Performance Level. The chiller constitutes a life-safety hazard but its failure would not impact any essential function. It is to be analyzed for the Life Safety Performance Level as described in Table 3-2.

c. Applicable Ground Motions (Performance Objective). The Performance Objective is determined to be 1A, defined as the combination of Life Safety Performance Level with a ground motion of 2/3 MCE as prescribed for Seismic Use Group I. For this example, the spectral response acceleration is assumed to be as follows:

$S_{DS} = 3/4\ S_{MS} = 0.80$ g (TI 809-04 Eq. 3-3)

d. Seismic design category:

Based on Short Period Response Acceleration:

Seismic design category: D (Table 3-4a)

Based on 1 second period Response Acceleration:

Seismic design category: D (Table 3-4b)

B. Preliminary Structural Assessment

Not in scope of this example problem.

C. Structural Screening (Tier 1)

Not in scope of this example problem.

D. Structural Screening (Tier 2)

Not in scope of this example problem.

E. Structural Screening (Tier 3)

Not in scope of this example problem.

F. Preliminary Nonstructural Assessment

Preliminary assessment is based upon available drawings and visual inspection of the accessible components.

1) Exempt Components

Not applicable. The chiller is not considered an exempt component.

2) Classification of Component

The chiller constitutes a life-safety hazard but its failure would not impact any essential function. The chiller is therefore assigned an importance factor, I_p of 1.0.

3) Disposition

The chiller shall be screened by the Tier 1 evaluation of FEMA 310.

G. Nonstructural Screening (Tier 1)

The chiller weighs over 20 pounds and is suspended from the ceiling more than 4 feet above the floor. Without bracing, such equipment is non-compliant and mitigation is required. A Tier 2 evaluation is not available for non-compliant suspended equipment; rehabilitation is recommended.

H. Nonstructural Evaluation (Tier 2)

Not required. Equipment was found to be non-compliant as part of the Tier 1 evaluation, and rehabilitation was recommended.

I. Evaluation Report

The Evaluation Report shall summarize the following as required for this example problem:

1. Building and Site Data

2. Preliminary nonstructural assessment

3. Nonstructural screening

4. Nonstructural evaluation

5. Judgmental Evaluations

6. Rehabilitation strategy/ Concept

The rehabilitation will require the design of bracing to laterally support chiller unit from seismic loads.

J. Rehabilitation Design

The procedures for rehabilitation are outlined below:

1. Review Evaluation Report and other available data.

2. Site Visit.

3. Confirming evaluation of existing building (if necessary).

4. Prepare alternative structural rehabilitation concepts.

5. Rehabilitation design.

The rehabilitation design follows the procedures laid out in accordance with the provisions of Chapter 7 and 9 of this document and FEMA 302 for the design and detailing of new structural components. A detailed analysis follows.

<u>*Determine Seismic Forces*</u>

Select R_p and a_p, factors:

$a_p = 1.0$ (TI 809-04, Table 10-1)
$R_p = 3.0$ (TI 809-04, Table 10-1)

Seismic forces (F_p) shall be determined in accordance with Chapter 6 as follows:

$$F_p = \frac{0.4 a_p I_p S_{DS} W_p}{R_p}\left(1 + 2\frac{x}{h}\right) \qquad \text{(TI 809-04, EQ. 10-1)}$$

W_p = 300 lbs (1.33 kN); x/h = 20.5'/36'=0.57 (Assume 12' floor-to-floor height

F_p is not required to be greater than:

$$F_p = 1.6 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-2)}$$

nor less than:

$$F_p = 0.3 S_{DS} I_p W_p \qquad \text{(TI 809-04, EQ. 10-3)}$$

$$F_p = \frac{0.4(1.0)(1.0)0.80(300\text{lbs})}{3.0}(1+2(0.57)) = 0.11(300\text{lbs}) = 33 \text{ lbs}$$

$(F_p)_{max} = 1.6(0.80)1.0(300\text{lbs}) = 384 \text{ lbs}(1.71 \text{ kN}) > 33 \text{ lbs}(147\text{N}) = F_p$ **O.K.**

$(F_p)_{min} = 0.3(0.80)1.0(300 \text{ lbs}) = 72 \text{ lbs}(320 \text{ N}) > 33 \text{ lbs} (147 \text{ N}) = F_p$ **Governs**

$\therefore\ F_p = 72 \text{ lbs } (320 \text{ N})$

Design new braces:

Provide brace in each direction:

P_{brace}=1.41(72 lbs) = 102 lbs (454 N)

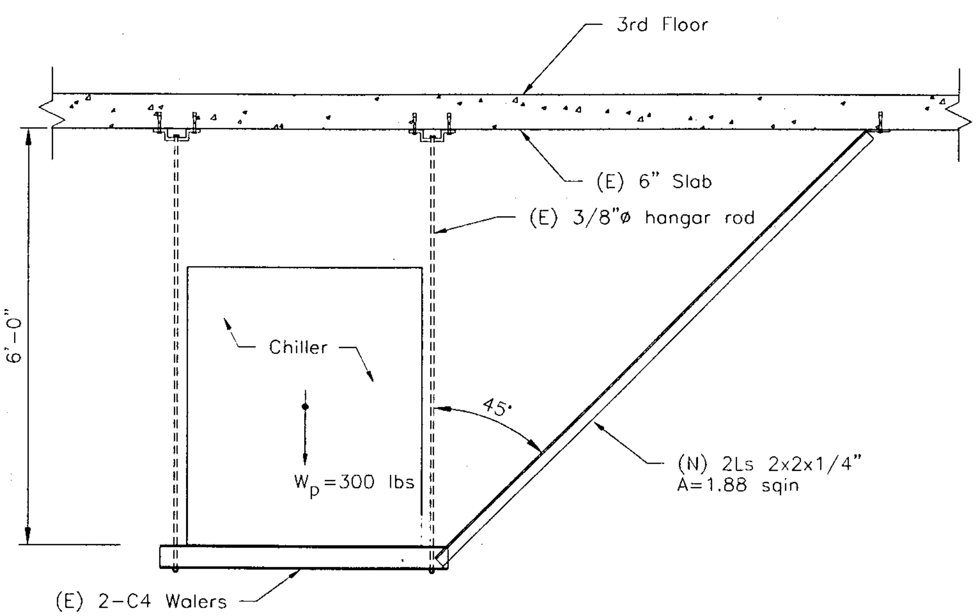
Try 2-L's 2x2x1/4": r_{min}=0.609 in. A=1.88 in.^{2} $L_{max} = 72\text{"}x\sqrt{2} = 102$ in.

L/r = 102/0.609 = 167

$\phi_c F_{cr} = 7.65$ ksi

$\phi P_n = \phi_c F_{cr}(A) = 7.65(1.88) = 14.4$ k (64.1 kN) per brace > 0.072 k (320 N) per brace **O.K.**

Use 2-L's 2x2x1/4" braces in each direction.



1 ft = 0.305 m
1 in = 25.4 mm
1 lb = 4.448 N

Figure F3-2. Bracing of Suspended Chiller Unit

Design connection to slab:

Anchor steel angles to underside of slab with 1-3/8" ϕ chemical anchor.

Demand shear per bolt = 72 lbs (320 N)

Bolt shear capacity:

Test data and design values for various proprietary post-installed systems are available from various sources, including International Conference of Building Officials (ICBO) reports and in manufacturer's literature.

Because there commonly is a relatively wide scatter in ultimate strengths, common practice is to define working loads as one-quarter of the average of the ultimate test values. Where working load data are defined in this manner, FEMA 273 Sec. C6.4.6.2 recommends using a design strength equal to twice the tabulated working load. Alternatively, where ultimate values are tabulated, it may be appropiate to use a design strength equal to half the tabulated average ultimate value.

For a 3/8" (9.5 mm) ϕ chemical anchor (ASTM A36) with 1 ¾" (44.5 mm) embedment depth and minimum spacing requirements satisfied, a working load value of 935 lbs is obtained from ICBO reports, and a design value of 2 x 935 lbs = 1870 lbs (8.3 kN) is used.

1870 lbs (8.3 kN) > 72 lbs (320 N) **OK**

USE 3/8" ϕ chemical anchor at each brace.

APPENDIX G
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APPENDIX H
CHECKLISTS FOR UNREINFORCED MASONRY BEARING-WALL BUILDINGS

This appendix contains checklists for Tier 1 structural screening of unreinforced masonry bearing-wall buildings. The checklists are adapted from the Third Ballot version of the proposed ASCE draft standard to replace FEMA 310. References in the checklists pertain to sections and tables in FEMA 310.

H1. Basic Structural Checklist for Building Type URM: Unreinforced Masonry Bearing-Wall Buildings with Flexible Diaphragms.

H2. Supplemental Structural Checklist for Building Type URM: Unreinforced Masonry Bearing-Wall Buildings with Flexible Diaphragms.

H3. Basic Structural Checklist for Building Type URMA: Unreinforced Masonry Bearing-Wall Buildings with Stiff Diaphragms.

H4. Supplemental Structural Checklist for Building Type URMA: Unreinforced Masonry Bearing-Wall Buildings with Stiff Diaphragms.

H1. Basic Structural Checklist for Building Type URM: Unreinforced Masonry Bearing-Wall Buildings with Flexible Diaphragms

This Basic Structural Checklist shall be completed when required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 evaluation procedure; the section numbers in parentheses following each evaluation statement correspond to Tier 2 evaluation procedures.

Basic Structural Checklist for Building Type URM

These buildings have bearing walls that consist of unreinforced (or lightly reinforced) brick or concrete block masonry. Wood floor and roof framing consists of wood joists, glulam beams, and wood posts or small steel columns. Steel floor and roof framing consists of steel beams or open-web joists, steel girders, and steel columns. Lateral forces are resisted by the reinforced brick or concrete block masonry shear walls. Diaphragms consist of straight or diagonal wood sheathing, plywood, or untopped metal deck, and are flexible relative to the walls. Foundations consist of brick or concrete spread footings.

Building System

C	NC		LOAD PATH: The structure shall contain one complete load path for Life Safety for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (Tier 2: Sec. 4.3.1.1).
C	NC	N/A	ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4% of the height of the shorter building for Life Safety (Tier 2: Sec. 4.3.1.2).
C	NC	N/A	MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure (Tier 2: Sec. 4.3.1.3).
C	NC	N/A	WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety (Tier 2: Sec. 4.3.2.1).
C	NC	N/A	SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below, or less than 80% of the average stiffness of the three stories above or below for Life Safety (Tier 2: Sec. 4.3.2.2).
C	NC	N/A	GEOMETRY: There shall be no changes in horizontal dimensions of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety, excluding one-story penthouses (Tier 2: Sec. 4.3.2.3).

C	NC	N/A	VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation (Tier 2: Sec. 4.3.2.4).
C	NC	N/A	MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety (Tier 2: Sec. 4.3.2.5).
C	NC	N/A	DETERIORATION OF WOOD: There shall be no signs of decay, shrinkage, splitting, fire damage, or sagging in any of the wood members, and none of the metal accessories shall be deteriorated, broken, or loose (Tier 2: Sec. 4.3.3.1).
C	NC	N/A	MASONRY UNITS: There shall be no visible deterioration of masonry units (Tier 2: Sec. 4.3.3.7).
C	NC	N/A	MASONRY JOINTS: The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar (Tier 2: Sec. 4.3.3.8).
C	NC	N/A	REINFORCED MASONRY WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for Life Safety; shall not be concentrated in one location; and shall not form an X pattern (Tier 2: Sec. 4.3.3.10).

Lateral-Force-Resisting System

C	NC	N/A	REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety (Tier 2: Sec. 4.4.2.1.1).
C	NC	N/A	SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 30 psi for clay units, and 70 psi for concrete units for Life Safety (Tier 2: Sec. 4.4.2.5.1).

Connections

C	NC	N/A	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check Procedure of Section 3.5.3.7 (Tier 2: Sec. 4.6.1.1).
C	NC	N/A	WOOD LEDGERS: The connection between the wall panels and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers (Tier 2: Sec. 4.6.1.2).
C	NC	N/A	TRANSFER TO SHEAR WALLS: Diaphragms shall be reinforced and connected for transfer of loads to the shear walls for Life Safety (Tier 2: Sec. 4.6.2.1).
C	NC	N/A	GIRDER/COLUMN CONNECTION: There shall be a positive connection utilizing steel plates or straps between the girder and the column support (Tier 2: Sec. 4.6.4.1).

H2. Supplemental Structural Checklist for Building Type URM: Unreinforced Masonry Bearing-Wall Buildings with Flexible Diaphragms

This Supplemental Structural Checklist shall be completed when required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

Lateral-Force-Resisting System

C NC N/A PROPORTIONS: The thickness of masonry walls, supported by flexible diaphragms, shall exceed twice the expected displacement of the diaphragm as calculated by the Quick Check procedure of Section 3.5.3.8 for Life Safety (Tier 2: Sec. 4.4.2.5.2.2).

C NC N/A MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls shall have negligible voids (Tier 2: Sec. 4.4.2.5.3).

Diaphragms

C NC N/A CROSS TIES: There shall be continuous cross ties between diaphragm chords (Tier 2: Sec. 4.5.1.2).

C NC N/A OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length for Life Safety (Tier 2: Sec. 4.5.1.4).

C NC N/A OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls shall not be greater than 8 feet long for Life Safety (Tier 2: Sec. 4.5.1.6).

C NC N/A STRAIGHT SHEATHING: All straight-sheathed diaphragms shall have aspect ratios less than 2 to 1 for Life Safety (Tier 2: Sec. 4.5.2.1).

C NC N/A SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety shall consist of wood structural panels or diagonal sheathing (Tier 2: Sec. 4.5.2.2).

C NC N/A UNBLOCKED DIAPHRAGMS: All unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety, and shall have aspect ratios less than or equal to 4 to 1 for Life Safety (Tier 2: Sec. 4.5.2.3).

C NC N/A OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than those described in Section 4.5 (Tier 2: Sec. 4.5.7.1).

Connections

C NC N/A ANCHOR SPACING: Exterior masonry walls shall be anchored to the floor and roof systems at a spacing of 4 feet or less for Life Safety (Tier 2: Sec. 4.6.1.3).

C NC N/A STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements shall be installed taut, and shall be stiff enough to prevent movement between the wall and the diaphragm (Tier 2: Sec. 4.6.1.5).

H3. Basic Structural Checklist for Building Type URMA: Unreinforced Masonry Bearing-Wall Buildings with Stiff Diaphragms

This Basic Structural Checklist shall be completed when required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 evaluation procedure; the section numbers in parentheses following each evaluation statement correspond to Tier 2 evaluation procedures.

Basic Structural Checklist for Building Type URMA

These buildings have perimeter bearing walls that consist of unreinforced clay brick masonry. Interior bearing walls, when present, also consist of unreinforced clay brick masonry. Diaphragms are stiff relative to the unreinforced masonry walls and interior framing. In older construction or large, multistory buildings, diaphragms consist of cast-in-place concrete. In regions of low seismicity, more recent construction consists of metal deck and concrete fill supported on steel framing.

Building System

C NC LOAD PATH: The structure shall contain one complete load path for Life Safety for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (Tier 2: Sec. 4.3.1.1).

C NC N/A MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure (Tier 2: Sec. 4.3.1.3).

C NC N/A WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety (Tier 2: Sec. 4.3.2.1).

C NC N/A SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below, or less than 80% of the average stiffness of the three stories above or below for Life Safety (Tier 2: Sec. 4.3.2.2).

C NC N/A GEOMETRY: There shall be no changes in horizontal dimensions of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety, excluding one-story penthouses (Tier 2: Sec. 4.3.2.3).

C NC N/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation (Tier 2: Sec. 4.3.2.4).

C NC N/A MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety (Tier 2: Sec. 4.3.2.5).

C NC N/A TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension for Life Safety (Tier 2: Sec. 4.3.2.6).

C NC N/A DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements (Tier 2: Sec. 4.3.3.4).

C NC N/A MASONRY UNITS: There shall be no visible deterioration of masonry units (Tier 2: Sec. 4.3.3.7).

C NC N/A MASONRY JOINTS: The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar (Tier 2: Sec. 4.3.3.8).

Lateral-Force-Resisting System

C NC N/A REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety (Tier 2: Sec. 4.4.2.1.1).

C NC N/A SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 30 psi for clay units, and 70 psi for concrete units for Life Safety (Tier 2: Sec. 4.4.2.5.1).

Connections

C NC N/A WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check Procedure of Section 3.5.3.7 (Tier 2: Sec. 4.6.1.1).

C NC N/A TRANSFER TO SHEAR WALLS: Diaphragms shall be reinforced and connected for transfer of loads to the shear walls for Life Safety (Tier 2: Sec. 4.6.2.1).

C NC N/A GIRDER/COLUMN CONNECTION: There shall be a positive connection utilizing steel plates or straps between the girder and the column support (Tier 2: Sec. 4.6.4.1).

H4. Supplemental Structural Checklist for Building Type URMA: Unreinforced Masonry Bearing-Wall Buildings with Stiff Diaphragms

This Supplemental Structural Checklist shall be completed when required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

Lateral-Force-Resisting System

C NC N/A PROPORTIONS: The height-to-thickness ratio of the shear walls at each story shall be less than the following for Life Safety (Tier 2: Sec. 4.4.2.5.2.1).

Top story of multi-story building	9
First story of multi-story building:	15
All other conditions:	13

C NC N/A MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls shall have negligible voids (Tier 2: Sec. 4.4.2.5.3).

Diaphragms

General

C NC N/A OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length for Life Safety (Tier 2: Sec. 4.5.1.4).

C NC N/A OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls shall not be greater than 8 feet long for Life Safety (Tier 2: Sec. 4.5.1.6).

Connections

C NC N/A ANCHOR SPACING: Exterior masonry walls shall be anchored to the floor and roof systems at a spacing of 4 feet or less for Life Safety (Tier 2: Sec. 4.6.1.3).