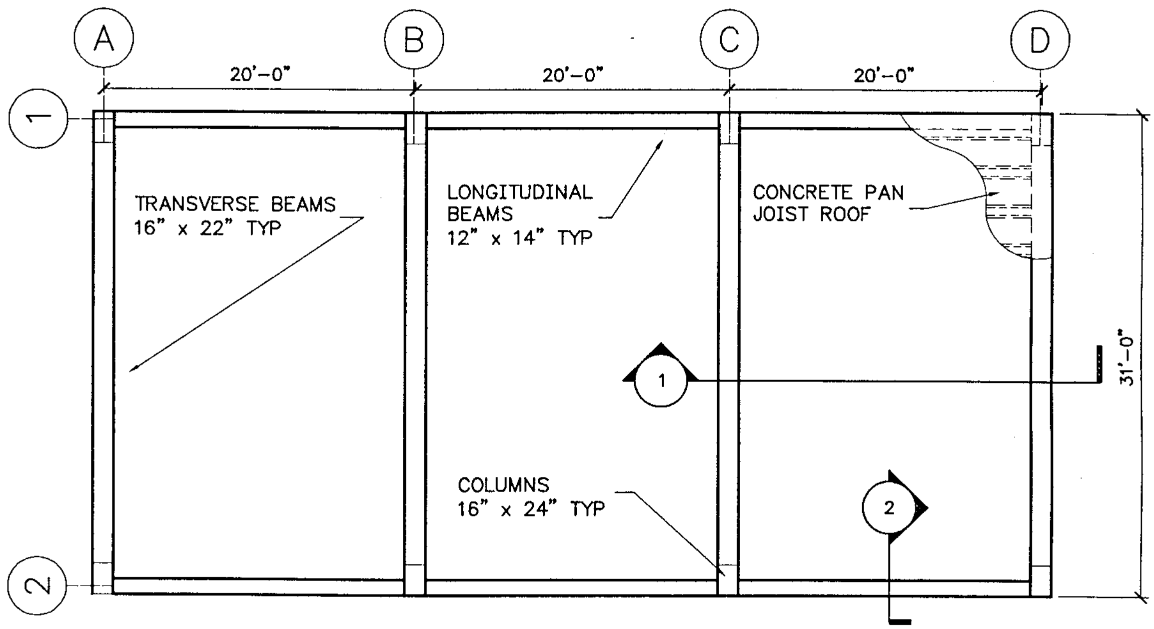
D4. Infilled Concrete Moment Frame Building

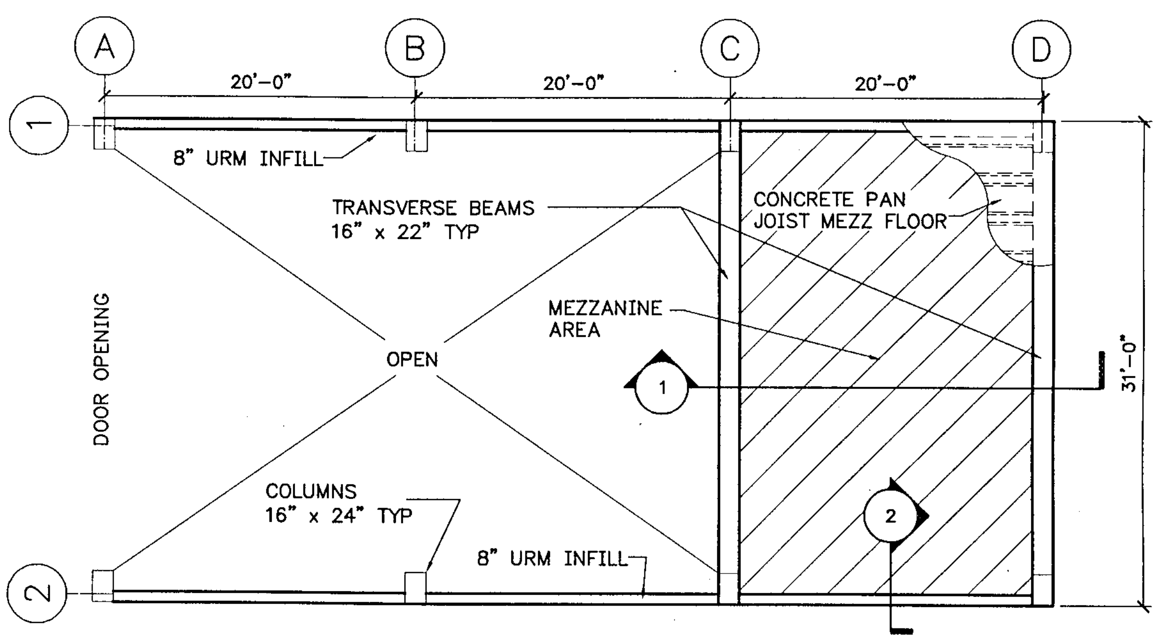
a. Description. This building is a fire station with a high bay 31' x 40' (9.5 m x 12.2 m) vehicle storage area and an attached 20' x 31' (6.1 m x 9.5 m) two-story dormitory over an office and storage area. The vehicle area has ordinary concrete moment frames in the transverse direction. The frames are infilled with URM in the longitudinal direction and in the transverse direction at the juncture with the two-story portion. The front is open to accommodate large nonstructural door framing. The roof consist of a concrete pan joist and slab system supported by the moment frames. The second floor is also a concrete joist system supported on intermediate beams at midheight of the two moment frames at the rear of the building.

b. Performance Objective. The fire station is designated as a Seismic Use Group IIIE structure and is assigned an IO performance level.

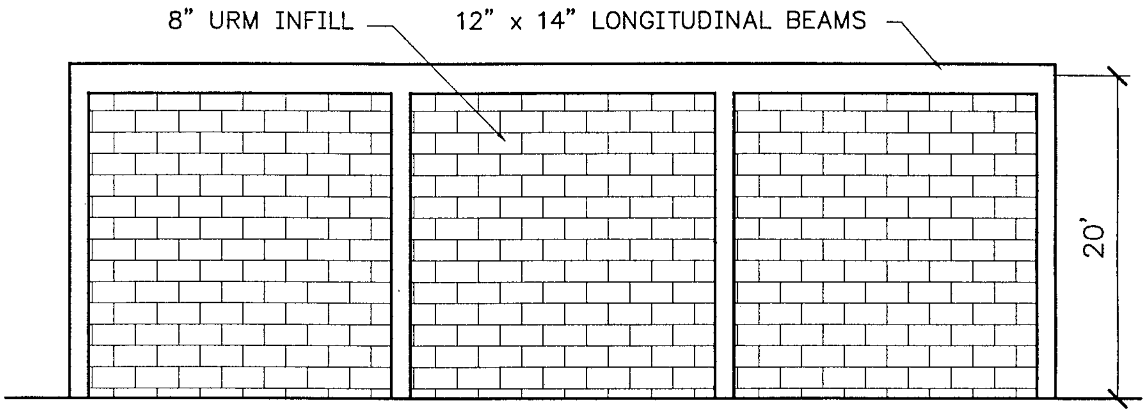
c. Analytical procedures. It will be assumed that the structure was designed for gravity loads only ignoring the infill. The building will be subjected to a full building Tier 2 evaluation including the infill participation and the rehabilitation will be designed based on a Linear Static Procedure analysis.



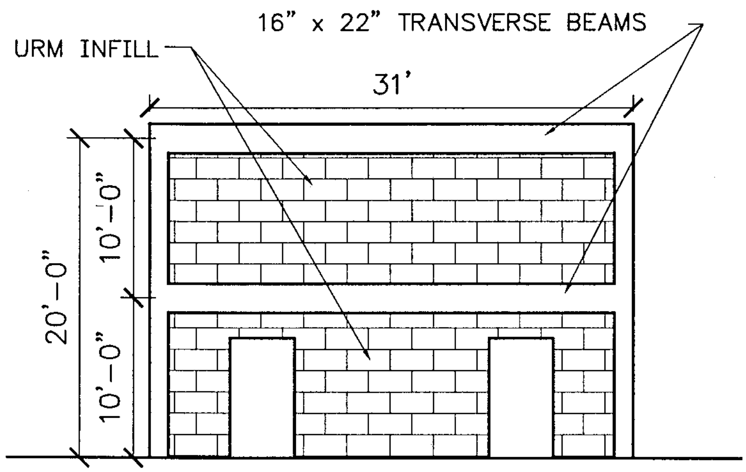
ROOF FRAMING



FRAMING AT MEZZANINE LEVEL

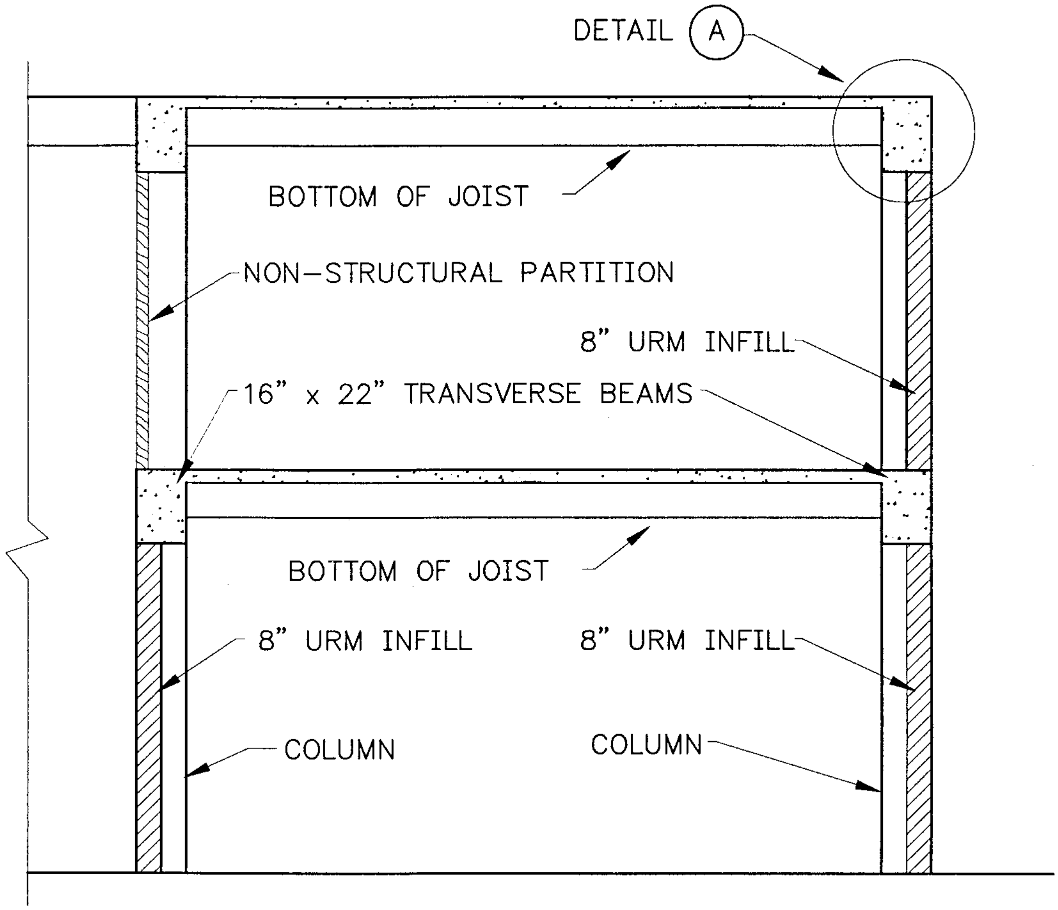


LONGITUDINAL ELEVATION

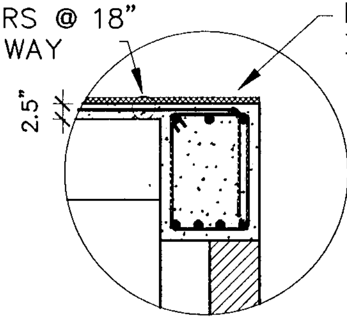


TRANSVERSE ELEVATION WALL LINE D

1 ft = 0.305 m
1 in = 25.4 mm

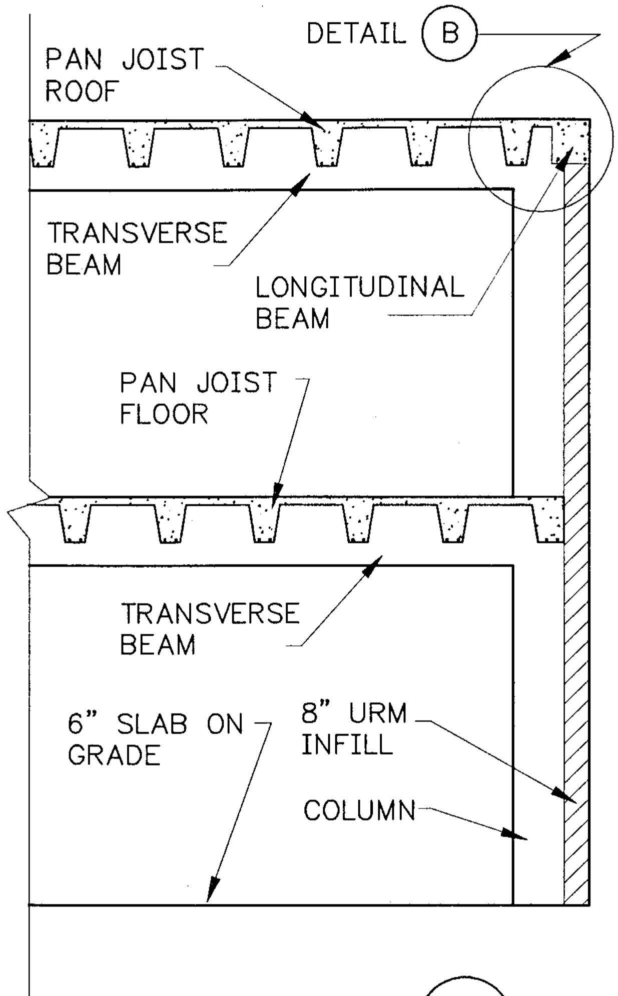


SECTION 1

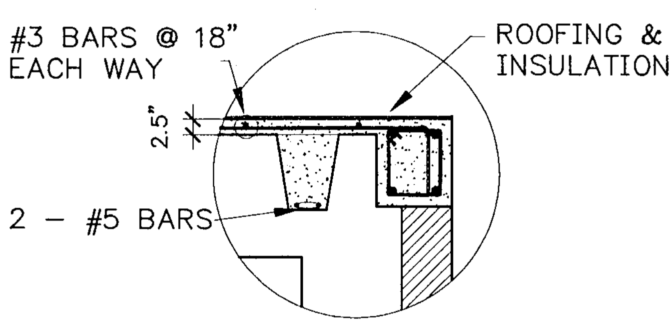


DETAIL A

1 in = 25.4 mm

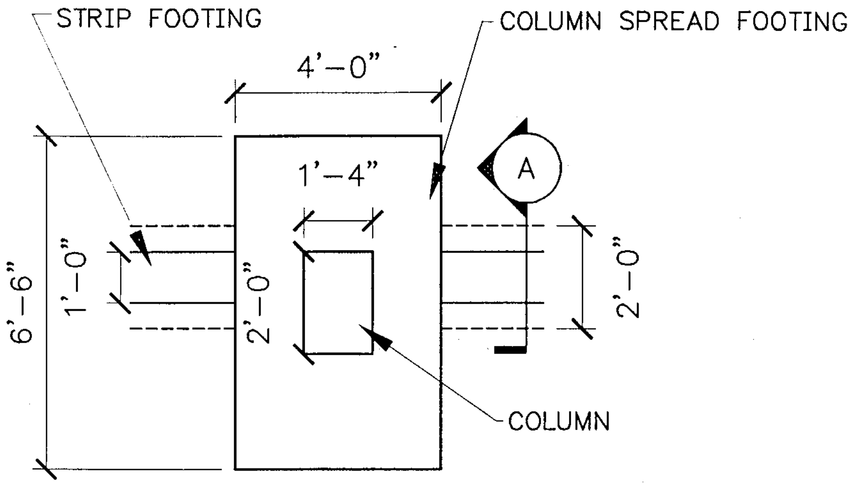


SECTION 2

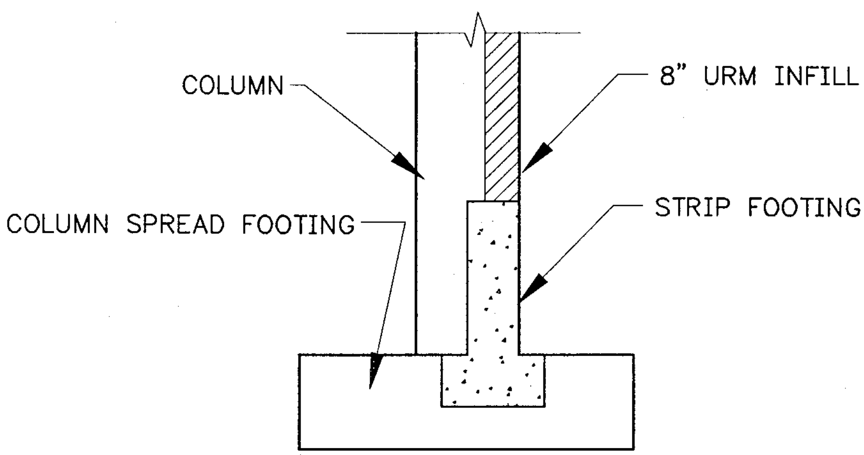


DETAIL B

1 in = 25.4 mm



PLAN VIEW OF COLUMN FOOTING



SECTION (A) THROUGH FOOTING

A. Preliminary Determinations (following steps laid out in Table 2-1)

1. *Obtain building and site data:*

a. *Seismic Use Group.* The fire station is an Essential Facility due to its occupancy. Therefore, from Table 2-2, the building falls into Seismic Use Group IIIE.

b. *Structural Performance Level.* This structure is to be analyzed for the Immediate Occupancy Performance Level as described in Table 2-3.

c. *Applicable Ground Motions (Performance Objective).* A ground motion of 2/3 MCE is prescribed for all Seismic Use Groups in Table 2-4. The spectral accelerations are determined from the MCE maps for the given location.

(1) Determine the short-period and one-second period spectral response accelerations:

$S_S = 1.00$ g	(MCE Maps)
$S_1 = 0.38$ g	(MCE Maps)

(2) Determine the site response coefficients: A geotechnical report of the building site classifies the soil as Class D (See TI 809-04 Table 3-1). The site response coefficients are determined by interpolation of Tables 3-2a and 3-2b of TI 809-04.

$F_a = 1.1$	(TI 809-04 Table 3-2a)
$F_v = 1.64$	(TI 809-04 Table 3-2b)

(3) Determine the adjusted MCE spectral response accelerations:

$S_{MS} = F_aS_S = (1.1)(1.0) = 1.1$	(TI 809-04 Eq. 3-1)
$S_{M1} = F_vS_1 = (1.64)(0.38) = 0.62$	(TI 809-04 Eq. 3-2)
$S_{MS} \leq 1.5\ F_a = 1.5(1.1) = 1.65 > 1.1$, use 1.1	(TI 809-04 Eq. 3-5)
$S_{M1} \leq 0.6\ F_v = 0.6(1.64) = 0.984 > 0.62$, use 0.62	(TI 809-04 Eq. 3-6)

(4) Determine the design spectral response accelerations:

$S_{DS} = 2/3\ S_{MS} = (2/3)(1.1) = 0.73$	(TI 809-04 Eq. 3-3)
$S_{D1} = 2/3\ S_{M1} = (2/3)(0.62) = 0.41$	(TI 809-04 Eq. 3-4)

d. *Determine seismic design category:*

Seismic design category: D	(Table 2-5a)
Seismic design category: D	(Table 2-5b)
Use Seismic Design Category D	

2. *Screen for geologic hazards and foundations.* Screening for hazards was performed in accordance with Paragraph F-3 of Appendix F in TI 809-04. It was determined that no hazards existed. Table 4-2 of this document requires that the geologic site hazard and foundation checklists contained in FEMA 310 be completed for structures assigned to Seismic Design Category D. See step C.2 for the completed checklist.

3. *Evaluate geologic hazards.* Not necessary.

4. *Mitigate geologic hazards.* Not Necessary.

B. Preliminary Structural Assessment (following steps laid out in Table 4-1)

1. *Definitely needs rehabilitation without further evaluation.* The structure has continuous load paths to resist lateral forces and there are no obvious signs of distress. Therefore, it is not obvious whether the building needs rehabilitation or not without an evaluation.

2. *Requires evaluation.* Paragraph 4-2a states that Seismic Use Group IIIE buildings will be evaluated only by a Tier 2 or Tier 3 evaluation. The building is fairly regular, with the exception of the mezzanine, and is therefore evaluated with a Tier 2 analysis.

C. Structural Screening (Tier 1) (following steps laid out in Table 4-2)

The structure is to be evaluated with a Tier 2 analysis. However, the Geologic Sited Hazard & Foundation Checklist is completed for all structures at this step.

1. *Determine applicable checklists.* Table 4-3 requires that the Geologic Site Hazard and Foundation checklist be completed for structures being evaluated with a Tier 2 analysis (The Basic Nonstructural and Supplemental Nonstructural checklists would also be completed at this point. However, this example does not address the nonstructural evaluation and rehabilitation.)

2. *Complete applicable checklists*

Geologic Site Hazards and Foundations Checklist (FEMA 310, Section 3.8)

This Geologic Site Hazards and Foundations Checklist shall be completed when required by Table 4-3 of TI 809-05. 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 Handbook, 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.

Geologic Site Hazards

The following statements shall be completed for buildings in regions of high or moderate seismicity.

C NC N/A LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.7.1.1).

C NC N/A SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure (Tier 2: Sec. 4.7.1.2).

C NC N/A SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated (Tier 2: Sec. 4.7.1.3).

Condition of Foundations

The following statement shall be completed for all Tier 1 building evaluations.

C NC N/A FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure (Tier 2: Sec. 4.7.2.1).

The following statement shall be completed for buildings in regions of high or moderate seismicity being evaluated to the Immediate Occupancy Performance Level.

C NC N/A DETERIORATION: There shall not be evidence that foundation elements have deteriorated due to corrosion, sulfate attack, material breakdown, or other reasons in a manner that would affect the integrity or strength of the structure (Tier 2: Sec. 4.7.2.2).

Capacity of Foundations

The following statement shall be completed for all Tier 1 building evaluations.

C NC N/A POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 ft. for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.7.3.1).

The following statements shall be completed for buildings in regions of high seismicity and for buildings in regions of moderate seismicity being evaluated to the Immediate Occupancy Performance Level.

C NC N/A OVERTURNING: The ratio of the effective horizontal dimension, at the foundation level of the lateral-force-resisting system, to the building height (base/height) shall be greater than $0.6S_a$ (Tier 2: Sec. 4.7.3.2).
Longitudinal: $b/h = 60' / 20' = 3.0 > 0.6S_a = 0.6(0.73) = 0.44$
Transverse: $b/h = 40'/20' = 2.0 > 0.44$
Note: $S_a = S_{DS}$. *See the Tier 2 evaluation in Section F for the determination of* S_a

C NC N/A TIES BETWEEN FOUNDATION ELEMENTS: The foundation shall have ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Class A, B, or C (Tier 2: Sec. 4.7.3.3).

C NC N/A DEEP FOUNDATIONS: Piles and piers shall be capable of transferring the lateral forces between the structure and the soil. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.7.3.4).

C NC N/A SLOPING SITES: The grade difference from one side of the building to another shall not exceed one-half the story height at the location of embedment. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.7.3.5).

3. *Evaluate screening results.* There are no 'Noncompliant' statements from the Geologic Hazards checklist. This structure is designated as a Seismic Use Group IIIE structure and may now be evaluated by a Tier 2 analysis.

D. Preliminary Nonstructural Assessment (from Table 4-4)

Nonstructural assessment is not in the scope of this example.

E. Nonstructural Screening (Tier 1) (from Table 4-5)

Nonstructural assessment is not in the scope of this example.

F. Structural Evaluation (Tier 2) (following steps laid out in Table 5-1)

1. Select appropriate analytical procedure. Based on the guidance of paragraph 5-2 of TI 809-04 the building shall be analyzed by the Linear Static Procedure (LSP).

2. Determine applicable ground motion. The ground motion was determined in Section A to be:

$S_{DS} = 0.73 \qquad S_{D1} = 0.41$

3. Perform structural analysis. A mathematical model of the building is developed in accordance with FEMA 310 Sections. 4.2.2 and 4.2.3: The building is analyzed using the LSP procedure outlined in Section 4.2.2.1 of FEMA 310. A three-dimensional model is used to capture the torsional effects due to the rigid diaphragm action.

a. Develop a mathematical building model in accordance with Section 4.2.3 of FEMA 310

- *Horizontal torsion:* The total torsional moment at a given floor level is equal to the eccentricity between the center of mass and the center of rigidity and an accidental torsion produced by horizontal offset in the center of mass equal to 5% of the horizontal dimension at the given floor level. The actual torsion is captured directly by the three-dimensional model. The accidental torsion is captured by applying a moment equal to the product of the shear at a level times the 5% offset to the diaphragms (Note: the accidental torsion is calculated at both the roof and mezzanine levels by multiplying the tributary weight times 5% of the dimension of the roof or mezzanine.)

- *Primary and secondary components:* All of the columns, infills, beams and concrete slab components are classified as primary components. The concrete moment frames are assumed to resist all of the gravity loads in addition to a portion of the lateral loads. The masonry infill panels are assumed to resist lateral loads only.

- *Diaphragms:* The roof and mezzanine level diaphragms are assumed to be rigid. Therefore, the vertical resisting elements resist lateral forces based on their relative rigidities.

- *Multidirectional excitation effects:* The building is torsionally irregular due to the high stiffness at the east end of the building. Therefore, multidirectional excitation is evaluated by applying 100% of the seismic force in one horizontal direction plus 30% of the seismic forces in the perpendicular horizontal direction.

- *Vertical Acceleration:* The effects of vertical excitation are negligible.

b. Determine the pseudo lateral forces in accordance with FEMA 310 Sec. 4.2.2.1.1:

The pseudo lateral force applied in the LSP is calculated in accordance with FEMA 310 Section 3.5.2.1. The building is assumed to behave as a URM building due to the high stiffness of the infill panels compared to the frames. Although the structure has two stories in the mezzanine level area, the majority of the building has only one story. Therefore, for determination of the pseudo lateral force, assume the structure is one story. This produces a more conservative pseudo lateral force due to the higher 'C' factor for a one story structure.

$V = CS_aW$ (FEMA 310 Eq. 3-1)
$C = 1.4$ (C3, Concrete frames with masonry infill and stiff diaphragms) (FEMA 310 Table 3-4)
$S_a = S_{D1} / T$, but S_a need not exceed S_{DS}; (FEMA 310 Eq. 3-4)
$T = C_t h_n^{3/4} = 0.020(20 \text{ ft.})^{3/4} = 0.19$ sec. (FEMA 310 Eq. 3-7)
$S_{DS} = 0.73$, $S_{D1} = 0.41$ (determined previously)
$S_a = 0.41 / 0.19 = 2.16 > 0.73$, use $S_a = 0.73$

W = Total dead load and 10 psf partition load (assume snow load = 0.0 psf)

(Calculations of seismic weights not shown)

Weight tributary to the roof level diaphragm = 299 kips (1330 kN)
Weight tributary to the mezzanine level diaphragm = 78 kips (346 kN)
Total seismic weight = (299 kips + 78 kips) = 377 kips (1677 kN)

The pseudo lateral forces are the same in both the transverse and longitudinal directions.

$V = (1.4)(0.73)(377 \text{ kips}) = 385$ kips (1712 kN)

c. Distribute the lateral forces vertically in accordance with FEMA 310 Sec. 4.2.2.1.2: The lateral force is distributed to the roof and mezzanine levels assuming that the building acts as a one-story structure. The pseudo lateral force is distributed to the roof and mezzanine based on tributary mass.

Level	w_x (kips)	CS_a	F_x (kips)	F_x (kN)
Roof	299	1.022	305	1357
Mezzanine	78	1.022	80	355
		$\Sigma =$	385	1712

d. Determine the component forces and displacements using linear, elastic analysis methods.

The actions due to earthquake forces, Q_E, are determined first:

The structure is analyzed using RISA 3D software with the following assumptions:

- The roof and mezzanine diaphragms are assumed to be rigid.

- Per FEMA 273 Table 6-4, the effective stiffness of the concrete moment frame members for flexural rigidity = 0.5EI for the beams and 0.7EI for the columns
  $E = 57\sqrt{f_c'} = 57\sqrt{3000\text{psi}} = 3122$ ksi for 3000 psi concrete
  $I_{\text{transverse beams}} = 1/12(14")(22")^3 = 12423 \text{ in.}^4$, $0.5I = 0.5(12423 \text{ in.}^4) = 6212 \text{ in.}^4$
  $I_{\text{longitudinal beams}} = 1/12(12")(14")^3 = 2744 \text{ in.}^4$, $0.5I = 0.5(2744 \text{ in.}^4) = 1372 \text{ in.}^4$
  $I_{\text{columns}} = 1/12(14 \text{ in.})(14 \text{ in.})^3 = 3201 \text{ in.}^4$, $0.7I = 0.7(3201 \text{in.}^4) = 2241 \text{ in.}^4$

- Mechanical Properties of Masonry:
  The mechanical properties of the masonry infill are taken as the default values given in FEMA 273 Section 7.5

  Compressive Strength: $f_{me} = 900$ psi (FEMA 273 Sec. 7.3.2.1)
  $f_{me}' = 1.25 f_{me} = 1.25(900 \text{ psi}) = 1125$ psi (expected strength)

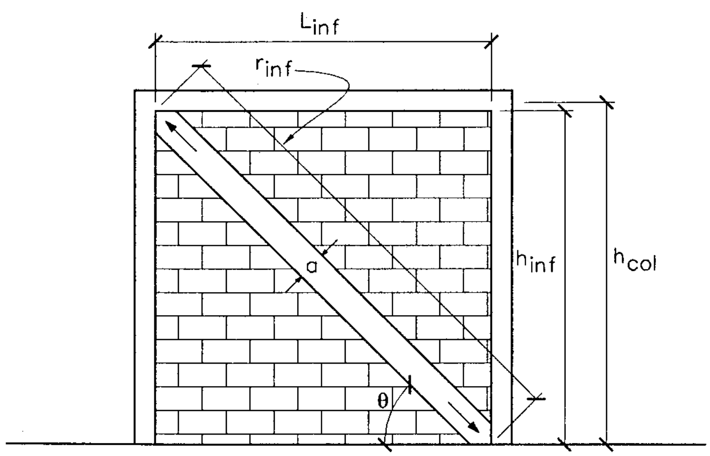
Elastic Modulus:	$E_{me} = 550\, f_{me}' = 550(1125 \text{ psi}) = 619 \text{ ksi}$	(FEMA 273 Sec. 7.3.2.2)
Tensile Strength:	$f_{te} = 20$ psi $f_{te}' = 1.25 f_{te} = 1.25(20 \text{ psi}) = 25$ psi	(FEMA 273 Sec. 7.3.2.3) (expected strength)
Shear Strength:	$v_{me} = 27$ psi $v_{me}' = 1.25 v_{me} = 1.25(27 \text{ psi}) = 33.8$ psi	(FEMA 273 Sec. 7.3.2.4) (expected strength)
URM Panel Thickness:	t_{inf} = equivalent solid thickness = 3.0 in (TM 5-809-3 Table 5-2)	

- Compression Struts: The infill panels along the exterior walls and the mezzanine wall on grid line C are modeled as compression struts following the procedure outlined in FEMA 273 Section 7.5.2

The elastic in-plane stiffness of a solid unreinforced masonry infill panel is represented with an equivalent diagonal compression strut of width 'a'. The equivalent strut has the same thickness and modulus of elasticity as the infill panel it represents.

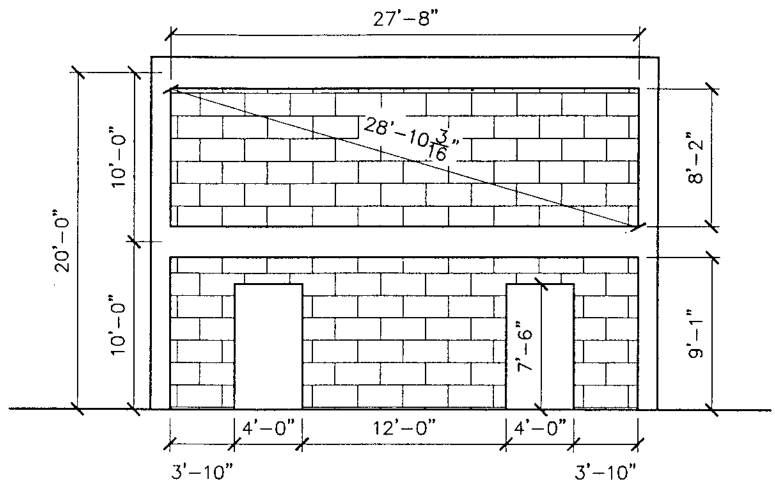
$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}$$ (FEMA 273 Eq. 7-14)

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{1/4}$$



COMPRESSION STRUT TERM DEFINITIONS

Transverse wall line D:



WALL LINE D

The bottom portion of the wall below the mezzanine contains openings. The equations above are applicable only for the case where the infill is solid with no openings. FEMA 273 suggests the use of a finite element program such as FEM/I to determine the equivalent strut properties for the case of a wall with openings. This can be very time consuming and may lead to inconsistent results when compared to the value predicted from the above equations. For this example, a simpler method is used. The compression strut stiffness of the complete infill is taken as the value computed from the above equations. The compression strut stiffness of the bottom infill portion with openings is then taken as a fraction of the solid infill section. The ratio is taken as the ratio of stiffness of the walls when considered as flexural-shear wall panel elements.

The flexural-shear stiffness of the walls is calculated assuming that they act as fixed-fixed pier elements.

The deflection of a fixed-fixed wall pier is calculated as:

$\Delta_f = \dfrac{Ph^3}{12EI} + \dfrac{1.2Ph}{AG}$, and the wall pier rigidity or stiffness is taken as $1/\Delta_f$

where
h = height of wall pier
E = Elastic modulus of masonry
I = Inertia of wall cross-section (Note: 0.5 I is used to model the wall as a cracked-section)
A = Cross-sectional area of wall pier
G = Shear modulus, = 0.4E for concrete and masonry

Determine flexural-shear stiffness of top portion of wall:

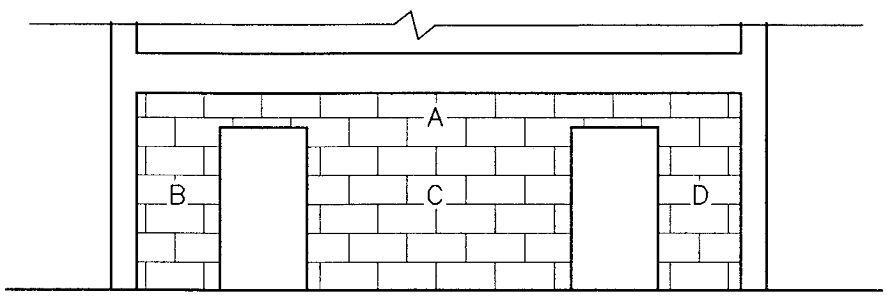
$$\Delta_f(h_w, l_w, t_w) := \frac{\left(P \cdot h_w^{3}\right)}{12 \cdot E_m \cdot I(l_w, t_w)} + \frac{\left(1.2 \cdot P \cdot h_w\right)}{A(l_w, t_w) \cdot E_v}$$

Deflection of Solid Wall $\quad \Delta_{solid} := \Delta_f(98 \cdot in, 27.67 \cdot ft, 2.5 \cdot in) \qquad \Delta_{solid} = 0.00061 \circ in$

$$R_{wall} := \frac{1 \cdot kip}{\Delta_{solid}} \qquad R_{wall} = 1651 \circ \frac{kip}{in}$$

Flexural-shear stiffness of top wall portion = 1651 kips / in (2891 kN / cm)

Determine flexural-shear stiffness of bottom portion of wall:



Deflection of Solid Wall $\quad \Delta_{solid} := \Delta_f(109 \cdot in, 27.67 \cdot ft, 2.5 \cdot in) \qquad \Delta_{solid} = 0.00068 \circ in$

Subract Bottom Strip $\quad \Delta_{strip} := \Delta_f(90 \cdot in, 27.67 \cdot ft, 2.5 \cdot in) \qquad \Delta_{strip} = 0.00055 \circ in$

$$\Delta_A := \Delta_{solid} - \Delta_{strip} \qquad \Delta_A = 0.00013 \circ in$$

Add Back in Piers B, C & D $\quad \Delta_B := \Delta_f(90 \cdot in, 46 \cdot in, 2.5 \cdot in) \qquad \Delta_B = 0.01348 \circ in$

$$\Delta_C := \Delta_f(90 \cdot in, 12 \cdot ft, 2.5 \cdot in) \qquad \Delta_C = 0.00153 \circ in$$

$$\Delta_D := \Delta_f(90 \cdot in, 46 \cdot in, 2.5 \cdot in) \qquad \Delta_D = 0.01348 \circ in$$

$$R_{BCD} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C} + \frac{1}{\Delta_D} \qquad R_{BCD} = 803 \circ \frac{1}{in}$$

$$\Delta_{BCD} := \frac{1}{R_{BCD}} \qquad \Delta_{BCD} = 0.00125 \circ in$$

$$\Delta_{wall} := \Delta_A + \Delta_{BCD} \qquad \Delta_{wall} = 0.00138 \circ in$$

$$R_{wall} := \frac{1 \cdot kip}{\Delta_{wall}} \qquad R_{wall} = 727 \circ \frac{kip}{in}$$

Flexural-shear stiffness of bottom wall portion = 727 kips / in (1273 kN / cm)

α = ratio of flexural-shear stiffness of bottom portion compared to top portion;

$\alpha = 727 / 1651 = 0.44$

Determine compression strut properties of top portion of wall per FEMA 273;

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{1/4}, \ \lambda_1 = \left[\frac{619000\,\text{psi}(2.5")\sin(2*16.45°)}{4(3122000\,\text{psi})(3201\,\text{in.}^4)(98")}\right]^{1/4} = 0.0215$$

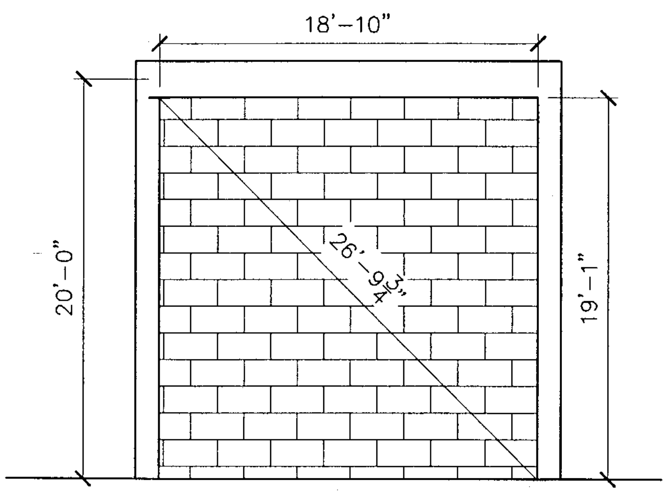
$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}, \ a = (0.175)[(0.0215)(120")]^{-0.4} 346" = 41.4"\ (105\ \text{cm})$$

Therefore, the equivalent compression strut width for the bottom portion of the wall is:

$a_{bottom} = \alpha \times a_{top} = 0.44 \times 41.4" = 18.2"(46.2\ \text{cm})$

Note: This equivalent compression strut is used to model mezzanine wall line C since it is the same in elevation as the bottom portion of wall line D.

Determine compression-strut properties of typical longitudinal infill panel;



TYPICAL LONGITUDINAL INFILL PANEL

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}}\right]^{1/4}, \ \lambda_1 = \left[\frac{619000\,\text{psi}(2.5")\sin(2*45.38°)}{4(3122000\,\text{psi})(3201\,\text{in.}^4)(229")}\right]^{1/4} = 0.020$$

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}, \ a = (0.175)[(0.02)(240")]^{-0.4} 322" = 30.1"\ (765\ \text{cm})$$

- Horizontal Torsion: In torsionally irregular buildings, the effect of accidental torsion must be amplified by the factor, A_x. A building is considered torsionally irregular if the building has rigid diaphragms and the ratio $\delta_{max} / \delta_{avg}$ due to torsional moment exceeds 1.2.

Actual torsion: Captured directly by 3-dimensional model.
Accidental torsion: Taken as the product of the shear and a 5% horizontal offset.

Note: The 5% horizontal offset is based on the entire building dimensions for the roof diaphragm and from the mezzanine dimensions for the mezzanine diaphragm.

Determine accidental torsional forces:

Roof Level;

Transverse Seismic Forces: e_x = 5%(60') = 3', V = 305 k, T_x = (305 k)(3') = 915 kip-ft (1241 kN-m)
Longit. Seismic Forces: e_y = 5%(30') = 1.5', V = 305 k, T_y = (305 k)(1.5') = 458 kip-ft (621 kN-m)

Mezzanine Level;

Transverse Seismic Forces: e_x = 5%(20') = 1', V = 80 k, T_x = (80 k)(1') = 80 kip-ft (108 kN-m)
Longit. Seismic Forces: e_y = 5%(30') = 1.5', V = 80 k, T_y = (80 k)(1.5') = 120 kip-ft (163 kN-m)

Calculate displacements to determine need for torsional amplification: The program SAP 2000 was used to determine nodal displacements. The shear forces and accidental torsional moments are applied to the structure separately for forces in the longitudinal and transverse directions. The nodal displacements at the roof level are then averaged and checked against the maximum nodal displacement for each of the orthogonal loading conditions.

Transverse Seismic Forces:

δ_{ave} = 1.64" (42 mm), δ_{max} = 2.16" (55 mm) $\delta_{max} / \delta_{ave} = 2.16 / 1.64 = 1.32 > 1.2$

Therefore, the accidental torsion for seismic excitation in the transverse direction must be amplified by A_x:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{ave}}\right)^2 = \left(\frac{2.16}{(1.2)(1.64)}\right)^2 = 1.2 \qquad \text{(FEMA 310 Eq. 4-5)}$$

The accidental torsion for transverse seismic forces become:

Roof Level: T = 1.2(915 kip-ft) = 1098 kip-ft (1489 kN-m)
Mezzanine Level: T = 1.2(80 kip-ft) = 96 kip-ft (130 kN-m)

Longitudinal Seismic Forces:

δ_{ave} = 0.725" (18.4 mm), δ_{max} = 0.763" (19.4 mm) $\delta_{max} / \delta_{ave} = 0.763 / 0.725 = 1.05 < 1.2$
Therefore, no amplification is needed for seismic excitation in the longitudinal direction.

– Load combinations:

The gravity load combinations used for analysis are:
$Q_G = 1.2\ Q_D + 0.5\ Q_L + 0.2Q_S$ (Q_S = 0 for this example) (Eq. 7-1)
$Q_G = 0.9\ Q_D$ (FEMA 310 Eq. 4-7)

Q_E = Earthquake forces = Direct shear and torsional forces determined previously

Deformation-controlled actions: The deformation-controlled design actions, Q_{UD}, are calculated according to:

$$Q_{UD} = Q_G \pm Q_E$$ (FEMA 310 Eq. 4-8)

Force-controlled actions: The force-controlled design actions, Q_{UF}, are calculated by one of the three following methods:

(1) Q_{UF} = the sum of the forces due to gravity and the maximum force that can be delivered by deformation-controlled actions,

(2) When the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system,

$$Q_{UF} = Q_G \pm \frac{Q_E}{CJ}$$ (FEMA 310 Eq. 4-9)

where $J = 1.5 + S_{DS} = 1.5 + 0.73 = 2.23 < 2.5$ (FEMA 310 Eq. 4-11)
and $C = 1.4$ (previously determined)

$$Q_{UF} = Q_G \pm \frac{Q_E}{CJ} = Q_G \pm \frac{Q_E}{(1.4)(2.23)} = Q_G \pm 0.32Q_E\,,$$

(3) For all other cases,

$$Q_{UF} = Q_G \pm \frac{Q_E}{C} = Q_G \pm \frac{Q_E}{1.4} = Q_G \pm 0.71Q_E$$ (FEMA 310 Eq. 4-10)

4. *Acceptance Criteria*

(a.) LSP – Linear Static Procedure

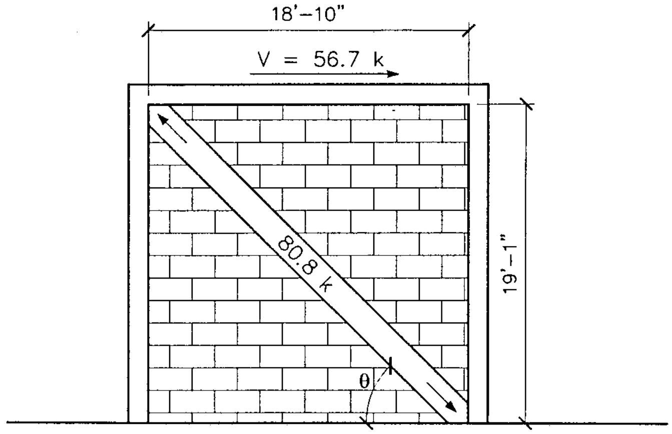
Deformation-controlled Actions: Deformation-controlled actions for the structure include beam and column bending, and shear in the infill panels. Deformation-controlled actions in primary and secondary components and elements shall satisfy:

$$mQ_{CE} \geq Q_{UD}$$ (Eq. 5-1)

Shear stress of infill panels: (Infill panels resist seismic loads only; no gravity loads on panels)

Longitudinal Infill Panels:

Maximum axial force in compression strut = 80.8 kips (359 kN)
Angle of strut elevation = atan(height / length) = 0.79 rad
Horizontal shear component = Axial x cos(θ) = (80.8 kips) x cos(0.79) = 56.7 kips (252 kN)
Q_{UD} = 56.7 kips (252 kN)

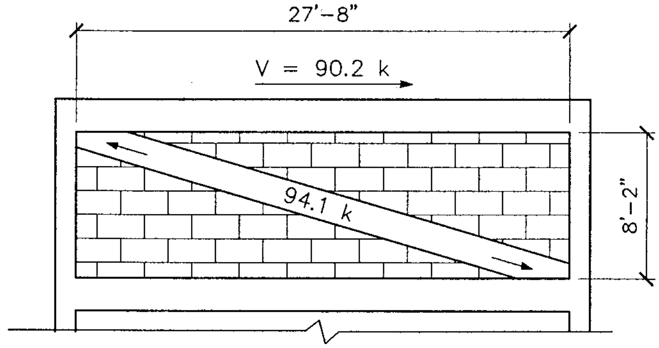


TYPICAL LONGITUDINAL INFILL PANEL FORCES

$Q_{CE} = V_{ine} = A_{ni}f_{vie}$ (FEMA 273 Eq. 7-15)
$f_{vie} = v_{me}$ = 33.8 psi (determined previously)
A_{ni} = Equivalent solid thickness x Panel Length
A_{ni} = (2.5")(18'-10") = 565 in.^{2}
Q_{CE} = (565 in.^{2})(33.8 psi) = 19.1 kips (85 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(19.1 kips) = 19.1 kips (85 kN) < 56.7 kips (252 kN), FAILS

Transverse Infill Panels: Panels in upper portion of wall line D above mezzanine level beam

Maximum axial force in compression strut = 94.1 kips (419 kN)
Angle of strut elevation = atan(height / length) = 0.29 rad
Horizontal shear component = Axial x cos(θ) = (94.1 kips) x cos(0.29) = 90.2 kips (401 kN)
Q_{UD} = 90.2 kips (401 kN)



TRANSVERSE WALL LINE D ABOVE MEZZANINE

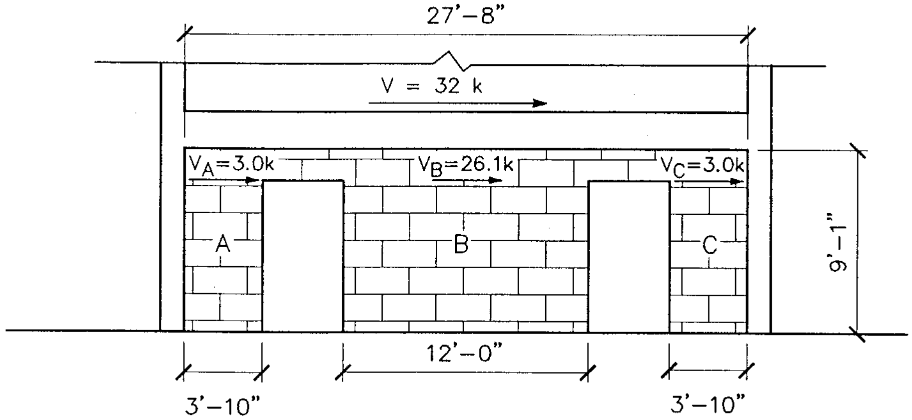
A_{ni} = (2.5")(27'-8") = 830 in.^{2}
Q_{CE} = (830 in.^{2})(33.8 psi) = 28.1 kips (125 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(28.1 kips) = 28.1 kips (125 kN)< 90.2 kips (401 kN), FAILS

Transverse Infill Panels: Panels in lower portion of wall line D below mezzanine level beam: The infill panels below the mezzanine level (along wall lines C and D) have holes for doors. The shear in the wall panels are distributed to each of the piers between the doors based on the pier's relative rigidities.

Angle of strut elevation = atan(height / length) = 0.32 rad

Rigidity A = R_A = 74 k / in (calculations not shown)
R_B = 654 k / in
R_C = 74 k / in
ΣR = 802 k / in

Wall line C
Maximum axial force in compression strut = 33.7 kips (150 kN)
Horizontal shear component = Axial x cos(θ) = (33.7 kips) x cos(0.32) = 32.0 kips (142 kN)
$V_A = V_C = V(R / \Sigma R)$ = 32 k (74 / 802) = 3 k (13 kN)
V_B = 32 k (654 / 802) = 26.1 k (116 kN)

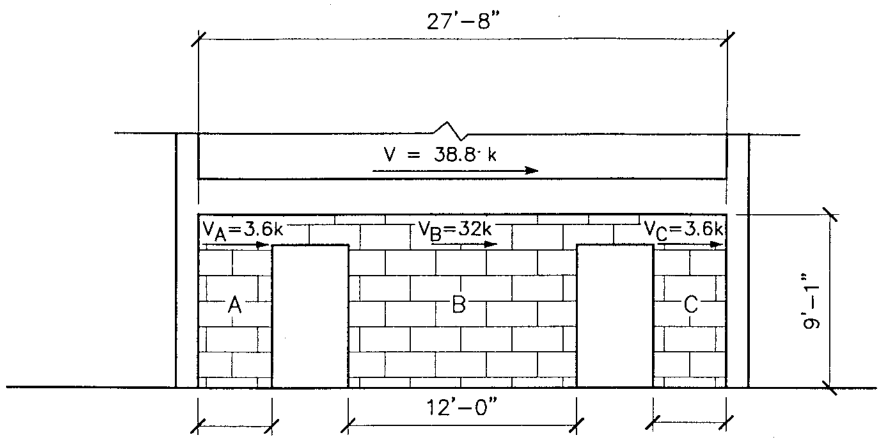


TRANSVERSE WALL LINE C BELOW MEZZANINE

Piers A and C:
Q_{UD} = 3 kips (13 kN)
A_{ni} = (2.5")(3'-10") = 115 in.^{2}
Q_{CE} = (115 in.^{2})(33.8 psi) = 3.9 kips (17 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(3.9 kips) = 3.9 kips (17 kN) > Q_{UD} = 3 kips (13 kN), OK
Pier B:
Q_{UD} = 26.1 k (116 kN)
A_{ni} = (2.5")(12') = 360 in.^{2}
Q_{CE} = (360 in.^{2})(33.8 psi) = 12.2 kips (54.3 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(12.2 kips) = 12.2 kips (54.3 kN)< 26.1 k (116 kN), FAILS

Total shear strength of wall line = 3.9 k + 3.9 k + 12.2 k = 20 k (89 kN)

Wall line D
Maximum axial force in compression strut = 40.8 kips (181 kN)
Horizontal shear component = Axial x cos(θ) = (40.8 kips) x cos(0.32) = 38.8 kips (173 kN)
$V_A = V_C = V(R / \Sigma R)$ = 38.8 k (74 / 802) = 3.6 k (16 kN)
V_B = 38.8 k (654 / 802) = 32 k (142 kN)



TRANSVERSE WALL LINE D BELOW MEZZANINE

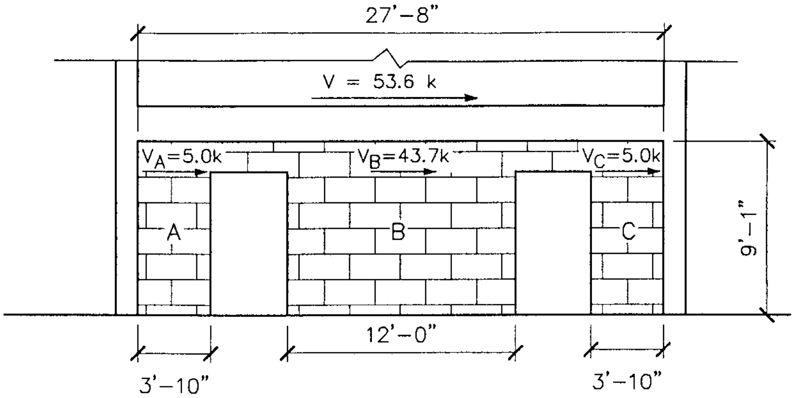
Piers A and C:
Q_{UD} = 3.6 k (16 kN)
A_{ni} = (2.5")(3'-10") = 115 in.^{2}
Q_{CE} = (115 in.^{2})(33.8 psi) = 3.9 kips (17 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(3.9 kips) = 3.9 kips (17 kN) > Q_{UD} = 3.6 k (16 kN), OK
Pier B:
Q_{UD} = 32 k (142 kN)
A_{ni} = (2.5")(12') = 360 in.^{2}
Q_{CE} = (360 in.^{2})(33.8 psi) = 12.2 kips (54.3 kN)
m = 1.0 (FEMA 310 Table 4-5)
mQ_{CE} = (1.0)(12.2 kips) = 12.2 kips (54.3 kN)< 32 k (142 kN), FAILS

Total shear strength of wall line = 3.9 k + 3.9 k + 12.2 k = 20 k (89 kN)

Transverse Infill Panels: Panels in Wall line C below mezzanine level beam
This wall line is analyzed as was done for wall line D below the mezzanine.

Maximum axial force in compression strut = 56.4 kips
Angle of strut elevation = atan(height / length) = 0.32 rad
Horizontal shear component = Axial x cos(θ) = (56.4 kips) x cos(0.32) = 53.6 kips

Rigidity A = R_A = 119 k / in (calculations not shown)
R_B = 1047 k / in
R_C = 119 k / in
ΣR = 1285 k / in = 119 k/in + 119 k/in + 1047 k/in
$V_A = V_C = V(R / \Sigma R)$ = 53.6 k (119 / 1285) = 5.0 k
V_B = 53.3 k (1047 / 1285) = 43.7 k



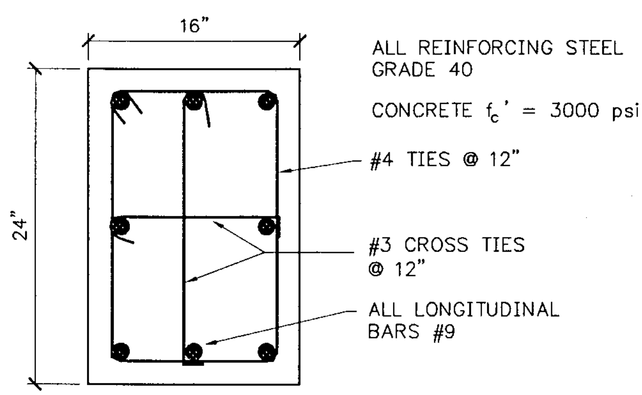
TRANSVERSE WALL LINE C BELOW MEZZANINE

Piers A and C:
$Q_{UD} = 5.0$ kips
$A_{ni} = (4")(3'\text{-}10") = 184$ in.^{2}
$Q_{CE} = (184 \text{ in.}^2)(33.8 \text{ psi}) = 6.2$ kips
$m = 1.0$ (FEMA 310 Table 4-5)
$mQ_{CE} = (1.0)(6.2 \text{ kips}) = 6.2 \text{ kips} > 5.0$ kips, OK

Pier B:
$Q_{UD} = 43.7$ kips
$A_{ni} = (4")(12') = 576$ in.^{2}
$Q_{CE} = (576 \text{ in.}^2)(33.8 \text{ psi}) = 19.5$ kips
$m = 1.0$ (FEMA 310 Table 4-5)
$mQ_{CE} = (1.0)(19.5 \text{ kips}) = 19.5 \text{ kips} < 43.7$ kips, FAILS

Total shear strength of wall line = 6.2 k + 6.2 k + 19.5 k = 31.9 k

Column flexure



TYP. COLUMN

The columns resist moment in both their strong and weak directions. Rectangular columns have a complicated biaxial load-moment interaction surface that requires the use of computer programs to solve exactly. A solution that is suitable for hand calculations is used for this example. The biaxial interaction is checked with:

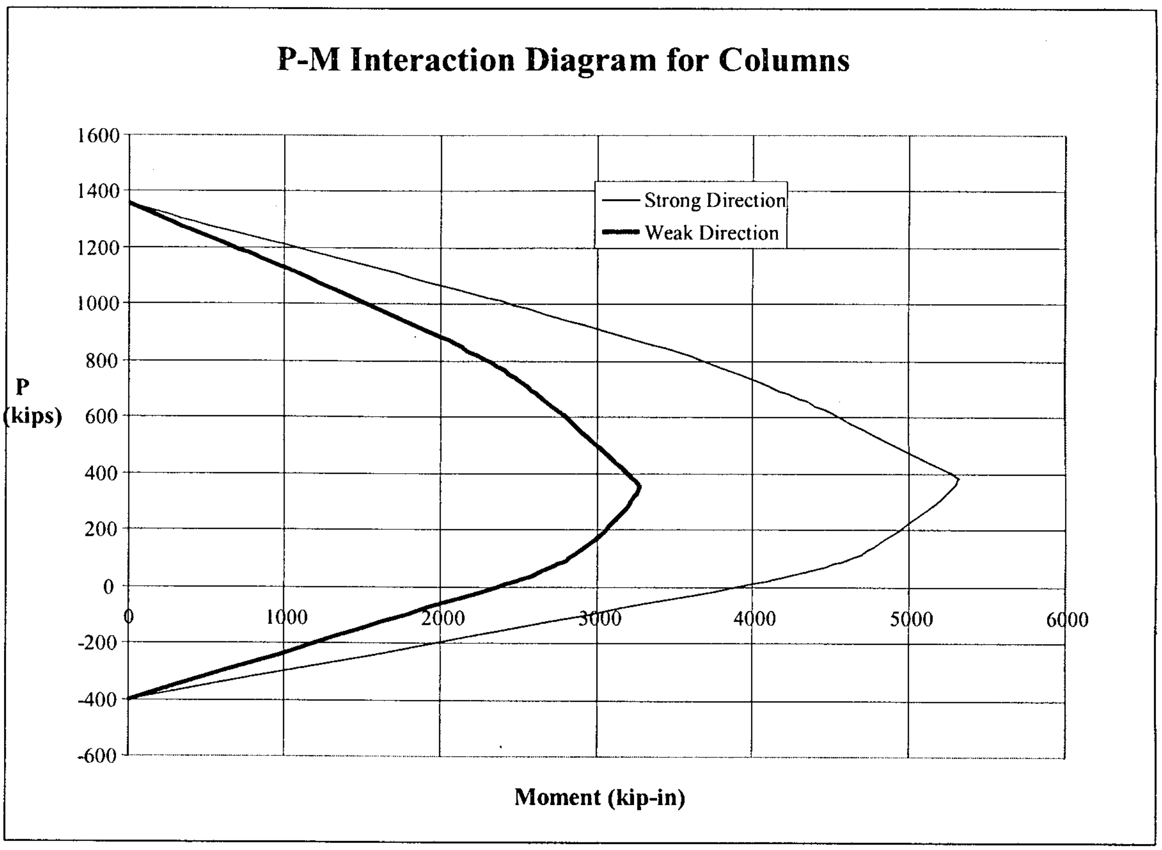
$\frac{M_x}{mM_{CEx}} + \frac{M_y}{mM_{CEy}} \le 1.0$, where M_x and M_y are the design moments occurring simultaneously, and M_{CEx} and M_{CEy} are the uniaxial expected moment strengths for bending only about either the x or y-axes at the design axial load. The m-factor is included in the denominator to account for element ductility. This is a linear interpolation of the column expected strength between the two axes and is a conservative approach for axial load-moment interactions below the balance point

A check of a column at the garage opening is shown to illustrate the column flexural check.

Check of columns along grid line A:

Load combination; $Q_E = 100\%EQ_y + 30\%EQ_x$, $Q_G = 0.9D$

$M_x = Q_{Ex} = 540$ kip-ft (732 kN-m)
$M_y = Q_{Ey} = 46$ kip-ft (62.3 kN-m)
Axial Load ≈ 0
The column expected flexural strength, M_{CE}, is calculated assuming the tensile stress in yielding longitudinal reinforcement is 1.25 times the nominal yield stress = 1.25(40 ksi) = 50 ksi (per FEMA 273 Section 6.4.2.2). The column capacity was calculated using the BIAX computer program:



$M_{CEx} = 324$ kip-ft (439 kN-m) $M_{CEy} = 196$ kip-ft (266 kN-m)

The m-factor from FEMA 310 Table 4-4 for the Immediate Occupancy Performance Level is 1.5

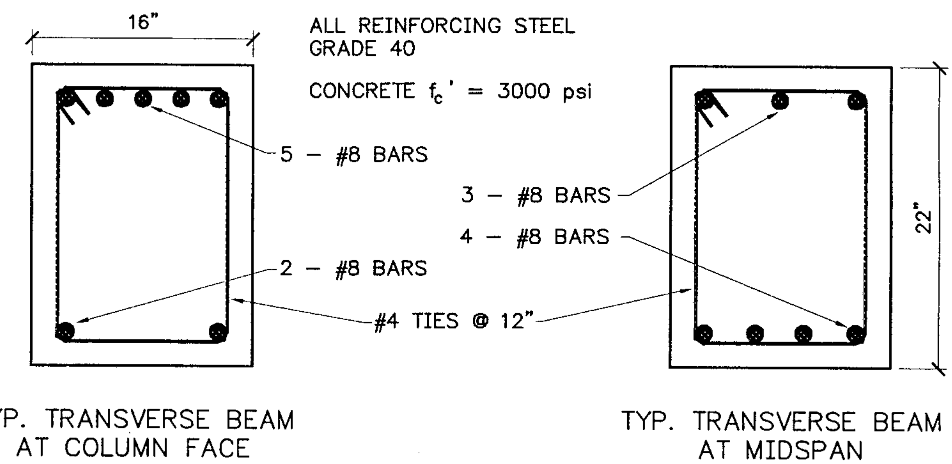
$$\frac{M_x}{mM_{CEx}} + \frac{M_y}{mM_{CEy}} \le 1.0 = \frac{540\text{kip}-\text{ft}}{(1.5)324\text{kip}-\text{ft}} + \frac{46\text{kip}-\text{ft}}{(1.5)196\text{kip}-\text{ft}} = 1.3 > 1.0\text{, FAILS}$$

The columns at the door opening along grid line A and the columns along grid line B were found to fail this condition. All of the rest of the columns were found to be adequate using this evaluation method. A check of the demand / capacity ratios for all of the columns was made using $\frac{M_x}{M_{CEx}} + \frac{M_y}{M_{CEy}}$ without the m-factors in the denominator to determine the ductility demand on the columns for flexure. This is required to evaluate the shear strength of the columns. FEMA 273 Table 6-5 states that elements with DCR's less than 2.0 are classified as having a low ductility demand. All of the columns in the structure were found to have DCR's less than 2.0 and are therefore classifies as having a low ductility demand.

Beam Flexure:

The beam flexure demands must be checked along the entire beam length due to the variations is longitudinal steel layout. The beam expected flexural strength, M_{CE}, is calculated assuming the tensile stress in yielding longitudinal reinforcement is 1.25 times the nominal yield stress = 1.25(40 ksi) = 50 ksi (per FEMA 273 Section 6.4.2.2). The beam flexural capacities were determined using the BIAX computer program.

Transverse Beams:



1 in = 25.4 mm
1 psi = 6.89 kPa

Beams at roof level:

At beam ends;

Largest positive flexural demand = M_{UD}^+ = 162 kip-ft (220 kN-m)
Largest negative flexural demand = M_{UD}^- = 301 kip-ft (408 kN-m)

Expected strengths at beam ends: M_{CE}^+ = 121 kip-ft (164 kN-m), M_{CE}^- = 287 kip-ft (389 kN-m)

D/C ratios: M^+ D/C =162 / 121 = 1.3 < 2.0 (low ductility demand from FEMA 273 Table 6-5)
M^-D/C = 301 / 287 = 1.05 < 2.0 (low ductility demand)

(Note: The D/C ratios are needed to determine the shear strength of the beams in the force-controlled actions section.)

The m-factors for reinforced concrete beams listed in FEMA 310 Table 4-4 for non-ductile beams = 1.5 for the Immediate Occupancy Performance Level.

mM_{CE}^{+} = (1.5)(121 kip-ft) = 182 kip-ft (247 kN-m) > 162 kip-ft (220 kN-m), OK
mM_{CE}^{-} = (1.5)(287 kip-ft) = 431 kip-ft (584 kN-m) > 301 kip-ft (408 kN-m), OK

At beam midpoints;

Largest positive flexural demand = M_{UD}^{+} = 114 kip-ft (155 kN-m)
Largest negative flexural demand = negligible negative moments at midspan

Expected strength at beam midpoint: M_{CE}^{+} = 233 kip-ft (316 kN-m)

D/C ratios: M^{+} D/C =114 / 233 = 0.5 < 2.0 (low ductility demand from FEMA 273 Table 6-5)

mM_{CE}^{+} = (1.5)(233 kip-ft) = 350 kip-ft > Q_{UD} = 114 kip-ft (155 kN-m), OK

<u>Beams at mezzanine level:</u>

At beam ends;

Largest positive flexural demand = M_{UD}^{+} = 88 kip-ft (119 kN-m)
Largest negative flexural demand = M_{UD}^{-} = 256 kip-ft (399 kN-m)

Expected strengths at beam ends: M_{CE}^{+} = 121 kip-ft (164 kN-m), M_{CE}^{-} = 287 kip-ft (389 kN-m)

D/C ratios: M^{+} D/C =88 / 121 = 0.7 < 2.0 (low ductility demand from FEMA 273 Table 6-5)
M^{-}D/C = 256 / 287 = 0.9 < 2.0 (low ductility demand)

mM_{CE}^{+} = (1.5)(121 kip-ft) = 182 kip-ft (247 kN-m) > 88 kip-ft (119 kN-m), OK
mM_{CE}^{-} = (1.5)(287 kip-ft) = 431 kip-ft (584 kN-m) > 256 kip-ft (399 kN-m), OK

At beam midpoints;

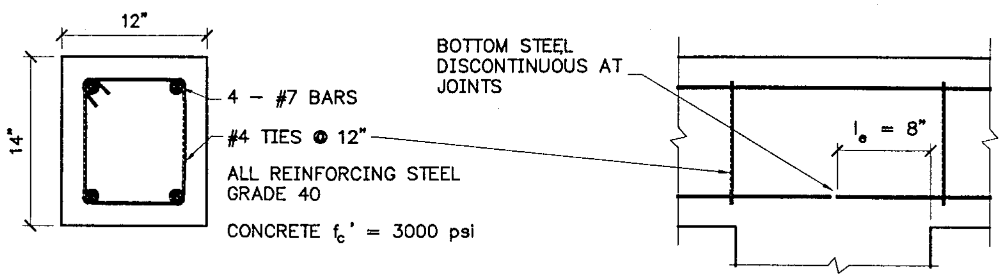
Largest positive flexural demand = M_{UD}^{+} = 136 kip-ft (184 kN-m)
Largest negative flexural demand = negligible negative moments at midspan

Expected strength at beam midpoint: M_{CE}^{+} = 233 kip-ft (316 kN-m)

D/C ratios: M^{+} D/C =136 / 233 = 0.6 < 2.0 (low ductility demand from FEMA 273 Table 6-5)

mM_{CE}^{+} = (1.5)(233 kip-ft) = 350 kip-ft > Q_{UD} = 136 kip-ft (184 kN-m), OK

Longitudinal Beams:



TYP. LONGITUDINAL BEAM CROSS SECTION

TYP. LONGITUDINAL BEAM–COLUMN JOINT ELEVATION

1 in = 25.4 mm
1 psi = 6.89 kPa

The bottom longitudinal steel in the longitudinal beams is not continuous through the beam-column joints. The steel is cut off at mid-depth of the columns (8" embedment length). FEMA 273 Section 6.4.5 states that the strength of straight, discontinuous bars embedded in concrete sections (including beam-column joints) with clear cover over the embedded bar not less than 3db may be calculated according to:

$$f_s = \frac{2500}{d_b} l_e \le f_y \qquad \text{(FEMA 273 Eq. 6-2)}$$

where f_s = maximum stress (in psi) that can be developed in an embedded bar having embedment length = l_e, (in inches), d_b = diameter of embedded bar (in inches), and f_y = bar yield stress (in psi).

$f_s = \frac{2500}{7/8"} 8" = 22857 \le 40000$, use f_s =22.9 ksi (158 MPa) for the bottom longitudinal bar strengths at the beam-column joints.

The top steel of the longitudinal beams is spliced at the beam midpoint with short splices = $20d_b$ = 20(7/8") = 17.5". The strength of the beam at the short splices would need to be evaluated using the methods of FEMA 273 Section 6.4.5 if there were negative flexural demands on the beams at their midpoints. For this example, the longitudinal beams experience only negligible negative moments at their midpoints so this condition does not need to be investigated.

At beam ends;

Largest positive flexural demand = M_{UD}^+ = 18.3 kip-ft (24.8 kN-m)
Largest negative flexural demand = M_{UD}^- = 34.1 kip-ft (46.2 kN-m)

Expected strengths at beam ends: M_{CE}^+ = 27 kip-ft (36.6 kN-m), M_{CE}^- = 52 kip-ft (70.5 kN-m)

D/C ratios: M^+ D/C =18.3 / 27 = 0.7 < 2.0 (low ductility demand from FEMA 273 Table 6-5)
M^-D/C = 34.1 / 52 = 0.7 < 2.0 (low ductility demand)

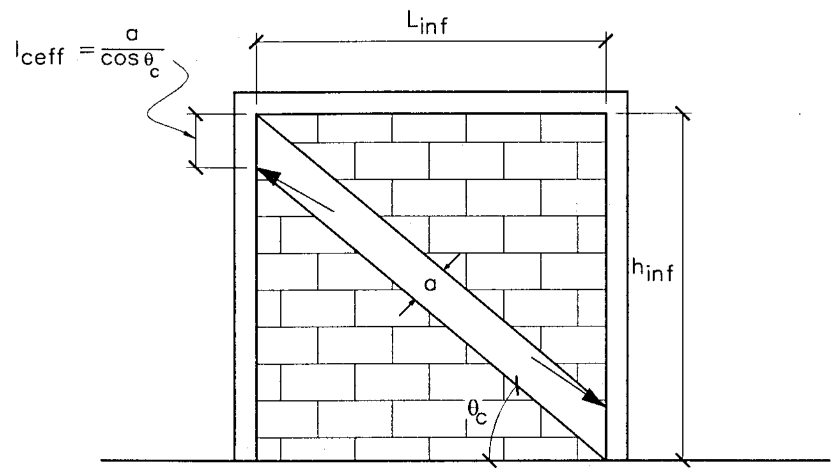
mM_{CE}^+ = (1.5)(27 kip-ft) = 40.5 kip-ft (55 kN-m) > 18.3 kip-ft (24.8 kN-m), OK
mM_{CE}^- = (1.5)(52 kip-ft) = 78 kip-ft (106 kN-m) > 34.1 kip-ft (46.2 kN-m), OK

Force-controlled Actions: Force-controlled actions for the structure include beam, column, and diaphragm shear. Force-controlled actions in primary and secondary components and elements shall satisfy:

$$Q_{CN} \geq Q_{UF} \qquad \text{(Eq. 5-2)}$$

<u>**Column Shear:**</u>

FEMA 273 Section 7.5.2.2.B describes two methods for determining the required strength of column members adjacent to infill panels. Method 2, used here, states that the expected shear strength of column members adjacent to an infill panel shall exceed the forces resulting from the development of expected column flexural strengths at the top and bottom of a column with reduced height equal to l_{ceff}. (Note: FEMA 273 states that this requirement can be waived if the expected masonry shear strength, v_{me}, as measured per the test procedures of FEMA 273 Section 7.3.2.4, is less than 50 psi. The expected masonry shear strength, v_{me}, is taken as the default value of 33.8 psi for this example, which is less than 50 psi. However, since this is a default value the actual shear strength may be greater than 50 psi. Therefore, this condition is checked to be conservative.)



ESTIMATING FORCES APPLIED TO COLUMNS

$$l_{ceff} = \frac{a}{\cos\theta_c} \qquad \text{(FEMA 273 Eq. 7-16)}$$

$$\tan\theta_c = \frac{h_{inf} - \dfrac{a}{\cos\theta_c}}{L_{inf}} \qquad \text{(FEMA 273 Eq. 7-17)}$$

Equation 7-17 is solved by iterating on values of θ_c, then l_{ceff} is determined with the previously determined value of 'a' and θ_c.

Shear in column weak direction:

The longitudinal infill panels produce moments in the columns in the column's weak direction. The flexural capacity of the columns in the weak direction is 196 kip-ft

<u>Determine l_{ceff} for typical longitudinal infill panels:</u>
a = 28.6", h_{inf} = 229", L_{inf} = 226"
Iterate to determine:
$\theta_c = 0.7$
l_{ceff} = 39.3"

Determine shear in column weak direction:

$V_{col} = 2M_{pcol} / l_{ceff} = 2(197 \text{ kip-ft}) / (39.3" / 12'/")$ = 120 kips (534 kN)
$Q_{UF} = V_{col}$ = 120 kips (534 kN)

Shear strength of column per FEMA 273 Section 6.4.4:

The columns were shown to have low ductility demands in the check of their flexural capacities. FEMA 273 states within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using Chapter 11 of ACI 318. The ACI method is used to determine the transverse steel contribution to the shear strength, while FEMA 273 Eq. 6-3 is used to calculate the contribution of concrete to shear strength.

$$V_n = V_c + V_s \qquad \text{(ACI 318 Eq. 11-2)}$$

The columns are reinforced with #4 ties at every 12" and #3 cross-ties at every 12". Therefore, the area of the shear steel = 2(0.20 in^2) + 0.11 in.^{2} = 0.51 in.^{2} in both the strong and weak directions.

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(ACI 318 Eq. 11-15)}$$

$$V_s = \frac{(0.51\,\text{in.}^2)(40\,\text{ksi})(13.5")}{12"} = 23\,\text{kips}\,(103\text{ kN})$$

$$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d \qquad \text{(FEMA 273 Eq. 6-3)}$$

λ = 1.0 for normal weight concrete, k = 1.0 for elements with low ductility demands, and assume $N_u = 0$ to be conservative.

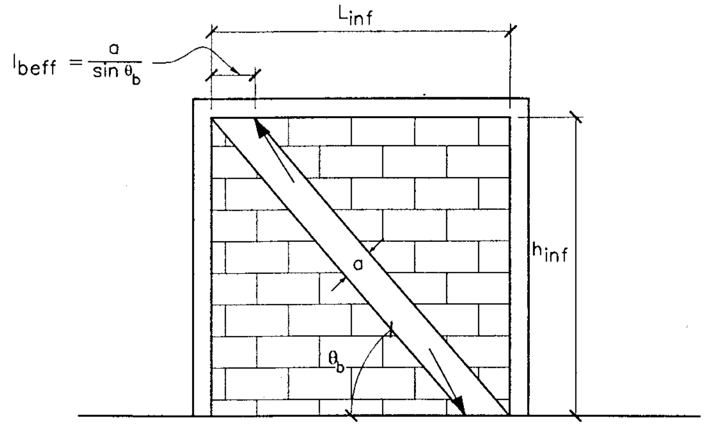
$$V_c = 3.5(1.0)\left((1.0) + \frac{0}{2000A_g}\right)\sqrt{3000}(24")(13.5") = 62\,\text{kips}\ (276\text{ kN})$$

$V_n = Q_{CN}$ = 62 kips + 23 kips = 85 kips (378 kN) < Q_{UF} = 120 kips (534 kN), NO GOOD

All of the columns are inadequate for shear in their weak direction (longitudinal direction) since they are all adjacent to infill panels. The shear in the strong direction (transverse direction) is not checked since the columns have already been shown to possess inadequate shear capacity.

Beam Shear:

FEMA 273 Section 7.5.2.2.C describes two methods for determining the required strength of beam members adjacent to an infill panel. Method 2, used here, states that the expected shear strength of beam members adjacent to an infill panel shall exceed the forces resulting from the development of expected beam flexural strengths at the ends of a beam member with a reduced length equal to l_{beff}. . (Note: FEMA 273 states that this requirement can be waived if the expected masonry shear strength, v_{me}, as measured per the test procedures of FEMA 273 Section 7.3.2.4, is less than 50 psi. The expected masonry shear strength, v_{me}, is taken as the default value of 33.8 psi for this example, which is less than 50 psi. However, since this is a default value the actual shear strength may be greater than 50 psi. Therefore, this condition is checked to be conservative.)



ESTIMATING FORCES APPLIED TO BEAMS

$$l_{beff} = \frac{a}{\sin\theta_b}$$ (FEMA 273 Eq. 7-18)

$$\tan\theta_b = \frac{h_{inf}}{L_{inf} - \frac{a}{\sin\theta_b}}$$ (FEMA 273 Eq. 7-19)

Equation 7-19 is solved by iterating on values of θ_b, then l_{beff} is determined with the previously determined value of 'a' and θ_b.

Longitudinal beams:
The flexural strengths of the longitudinal beams at the end zones are $M_{CE}^+ = 27$ kip-ft, $M_{CE}^- = 52$ kip-ft.

Determine l_{beff} for the longitudinal infill panels:
a = 30.1", $h_{inf} = 229$", $L_{inf} = 226$"
Iterate to determine:
$\theta_b = 0.9$
$l_{beff} = 38.7$"

Determine beam shear:
$V_{beam} = M_{CE}^+ + M_{CE}^- / l_{beff}$ = (27 kip-ft + 52 kip-ft)/ (38.7" / 12'/") = 24.5 kips
$Q_{UF} = V_{beam} = 24.5$ kips (109 kN)

Shear strength of beams per FEMA 273 Section 6.4.4:
The beams were shown to have low ductility demands in the check of their flexural capacities. FEMA 273 states within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using Chapter 11 of ACI 318.

$$V_n = V_c + V_s$$ (ACI 318 Eq. 11-2)

The beams are reinforced with #4 ties at every 12". Therefore, the area of the shear steel = 2(0.20 in^2) = 0.40 in.^{2}.

$$V_s = \frac{A_v f_y d}{s}$$ (ACI 318 Eq. 11-15)

$$V_s = \frac{(0.40\,\text{in.}^2)(40\,\text{ksi})(11.5")}{12"} = 15\,\text{kips}\,(66.7\text{ kN})$$

$$V_c = 2\sqrt{f_c'}\,b_w d \qquad \text{(ACI 318 Eq. 11-3)}$$

$$V_c = 2\sqrt{3000}(12")(11.5") = 15\,\text{kips}\,(66.7\text{ kN})$$

V_n = 15 kips + 15 kips = 30 kips (133 kN) > Q_{UF} = 24.5 kips (109 kN)

Transverse Beams:

The transverse mezzanine beams along grid lines C and D and the upper transverse beam along grid line D are adjacent to infill panels. The transverse beams are all the same size with the same longitudinal flexural reinforcement. The flexural strengths of the transverse beams at the end zones are M_{CE}^+ = 121 kip-ft, M_{CE}^- = 287 kip-ft.

Mezzanine Beams:

Determine l_{beff} for the transverse infill panel along wall lines C and D below mezzanine:
a = 18.2", h_{inf} = 109", L_{inf} = 332"
Iterate to determine:
θ_b = 0.37
l_{beff} = 50.3"

Determine beam shear:
V_{beam}= M_{CE}^+ + M_{CE}^- / l_{beff} = (121 kip-ft + 287 kip-ft)/ (50.3" / 12'/") = 97 kips (431 kN)
Q_{UF} = V_{beam} = 97 kips (431 kN)

Shear strength of beams per FEMA 273 Section 6.4.4:
The beams were shown to have low ductility demands in the check of their flexural capacities. FEMA 273 states within yielding regions of components with low ductility demands, and outside yielding regions, shear strength may be calculated using Chapter 11 of ACI 318.

$$V_n = V_c + V_s \qquad \text{(ACI 318 Eq. 11-2)}$$

The beams are reinforced with #4 ties at every 12" . Therefore, the area of the shear steel = 2(0.20 in^2) = 0.40 in.^{2}.

$$V_s = \frac{A_v f_y d}{s} \qquad \text{(ACI 318 Eq. 11-15)}$$

$$V_s = \frac{(0.40\,\text{in.}^2)(40\,\text{ksi})(19.5")}{12"} = 26\,\text{kips}\,(116\text{ kN})$$

$$V_c = 2\sqrt{f_c'}\,b_w d \qquad \text{(ACI 318 Eq. 11-3)}$$

$$V_c = 2\sqrt{3000}(16")(19.5") = 34\,\text{kips}\,(151\text{ kN})$$

V_n = 26 kips + 34 kips = 60 kips (267 kN) < Q_{UF} = 97 kips (431 kN), NO GOOD

Transverse beam along grid line D at roof level:

Determine l_{beff} for the transverse infill panel along wall line D above mezzanine:
a = 41.4", h_{inf} = 106", L_{inf} = 332"
Iterate to determine:
θ_b = 0.43
l_{beff} = 99"

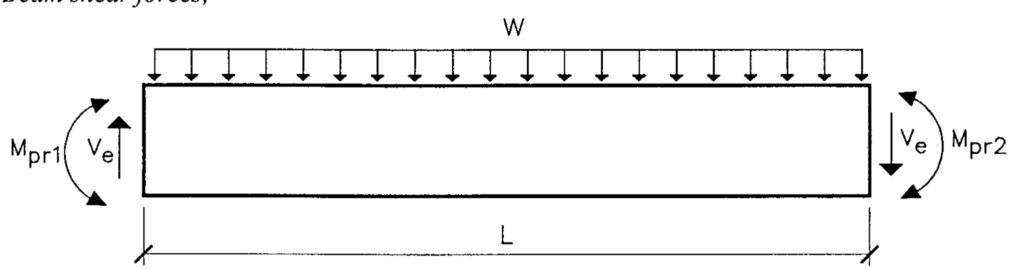
Determine the beam shear:
$V_{beam} = M_{CE}^{+} + M_{CE}^{-} / l_{beff}$ = (121 kip-ft + 287 kip-ft)/ (99" / 12'/") = 50 kips (222 kN)
$Q_{UF} = V_{beam}$ = 50 kips (222 kN)

The shear strength is the same as for the mezzanine beams;
V_n = 60 kips (267 kN) > Q_{UF} = 50 kips (222 kN), OK

Transverse beams along grid lines A, B, and C at roof level:

The transverse beams develop flexural hinges at their ends due to the different seismic load combinations. The beam shear demand is based on the flexural capacity of the beams per ACI 318 Section 21.3.4. The design shear force V_e is determined from consideration of the statical forces on the portion of the member between faces of the joints. It is assumed that moments of opposite signs corresponding to probable strength M_{pr} act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

Beam shear forces;



Beam moment capacities (from BIAX): Side 1 is the left end, Side 2 is the right end
M_{pr1}^{+} = 121 kip-ft M_{pr1}^{-} = 287 kip-ft M_{pr2}^{+} = 121 kip-ft M_{pr2}^{-} = 287 kip-ft

w = Gravity loads = 1.2D + 0.5L = 1.2(2.02 klf) + 0.5(0.4 klf) = 2.6 klf
L = 28 ft. (clear distance between column faces)

$V_e = (M_{pr1}^{+} + M_{pr2}^{-}) / L + wL/2$ = (121 kft + 287 kft) / 28' + (2.6 klf)(28') / 2 = 51 kips (227 kN)

$Q_{UF} = V_e$ = 51 kips (227 kN)
V_n = 60 kips (267 kN) (determined previously)
V_n = 60 kips > 51 kips, OK

Diaphragm shear forces:

The mezzanine diaphragm is not directly connected to the infill panels along the longitudinal walls (lines 1 and 2). Therefore, the diaphragms are not evaluated and rehabilitation is needed.

5. *Evaluation results:*

The building lacks the required strength to resist seismic forces. The components found to be deficient include:

- The infill panels are overstressed in shear by up to 300%. Nearly all of the URM infill panels were found to possess inadequate shear strength.
- The columns and beams were found to lack the required shear strength for forces imposed by the infill panels.

- The columns along grid lines A and B were found to possess inadequate flexural strength. The high flexural demands are due to building torsion. The large door opening along grid line A has very low stiffness compared to the shear walls along grid line D and the partial shear wall along grid line C, causing large torsional demands.
- The mezzanine diaphragm lacks direct connection detailing for transfer of shear forces to the longitudinal infill panels.

G. Structural Evaluation (Tier 3) (from Table 5-2)

A Tier 3 is not completed as it would only show that the building is deficient as was shown in the Tier 2 evaluation.

H. Nonstructural Evaluation (Tier 2) (from Table 5-3)

Nonstructural components are not considered in this example.

I. Final Assessment (from Table 6-1)

1. Structural evaluation assessment:

The structure was found to lack strength to resist the prescribed lateral forces (see step F.5 above for a list of deficiencies). The building is a serious life safety hazard due to the high overstress in the infill panels and the nonductile detailing of the concrete-framing members, but rehabilitation is possible.

2. Structural rehabilitation strategy:

The rehabilitation strategy is to add strength to the structure and reduce the torsional demands on the framing. The infill panels may be strengthened by adding a layer of shotcrete to the masonry. The shotcrete will be detailed such that the rehabilitated panels will act as complete shear walls rather than compression struts. This will reduce the demands on the frames as the walls will be much stiffer and resist more force. The torsion problem may be reduced by moving the center of rigidity away from the rear of the building (along grid line D) towards the garage opening (along grid line A). The addition of exterior buttresses to the columns along grid line A will add rigidity to the front of the building, thus reducing the torsion.

3. Structural rehabilitation concept:

The infill panels will be strengthened by adding a 4" (102 mm) layer of shotcrete to the interior of the walls. The shotcrete is placed on the interior so that it can be connected to the existing framing, allowing the rehabilitated walls to act as shear elements rather than compression struts. The new shotcrete will be doweled to the existing framing so that it will act as a composite section. At the mezzanine area, the new vertical steel in the shotcrete will pass through holes drilled and grouted in the mezzanine slab. This will provide a direct shear transfer mechanism between the mezzanine diaphragm and the new shotcrete walls for seismic forces in the longitudinal direction.

Buttresses will be added to the front of the building (along grid line A) to reduce the torsional response of the structure. The buttresses will be tapered from 8' (2.44 m) long at the base to 2' (0.61 m) long at the top. New foundations with hold-down piles must be constructed for the buttresses to resist the large overturning demands. Hold-down piles are designed to mobilize the weight of a tributary wedge of soil to resist the uplift forces. The piles consist of a high strength steel bar grouted in a drilled hole. The end of the bar is initially grouted a sufficient length to develop the strength of the bar and at the appropriate depth to mobilize the necessary soil wedge. The remainder of the bar is sheathed so as to preclude bonding with the final grouting.

4. Nonstructural evaluation assessment:

Nonstructural assessment is not in the scope of this example.

5. Nonstructural rehabilitation strategy:

Nonstructural assessment is not in the scope of this example.

6. Nonstructural rehabilitation concept:

Nonstructural assessment is not in the scope of this example.

At this point a cost estimating specialist will develop the programming level cost estimate for the project. This estimate will include the structural seismic rehabilitation costs, based on the material quantities developed by the structural evaluator, along with the costs for nonstructural seismic rehabilitation and all other items associated with the building upgrade.

J. Evaluation Report (from Table 6-2)

At this point an evaluation report would be completed per the steps in Table 6-2. This step is not done for this design example.

The Evaluation Process is complete.

Seismic Rehabilitation Design (Chapter 7)

Since rehabilitation of the structural system was the seismic hazard mitigation method selected , the following procedures are completed.

K. Rehabilitation (from Table 7-1)

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

The evaluation report completed earlier was reviewed along with the available drawings.

2. *Site Visit*

The site was visited during the building evaluation. No further meaningful information could be gathered by another visit.

3. *Supplementary analysis of existing building (if necessary)*

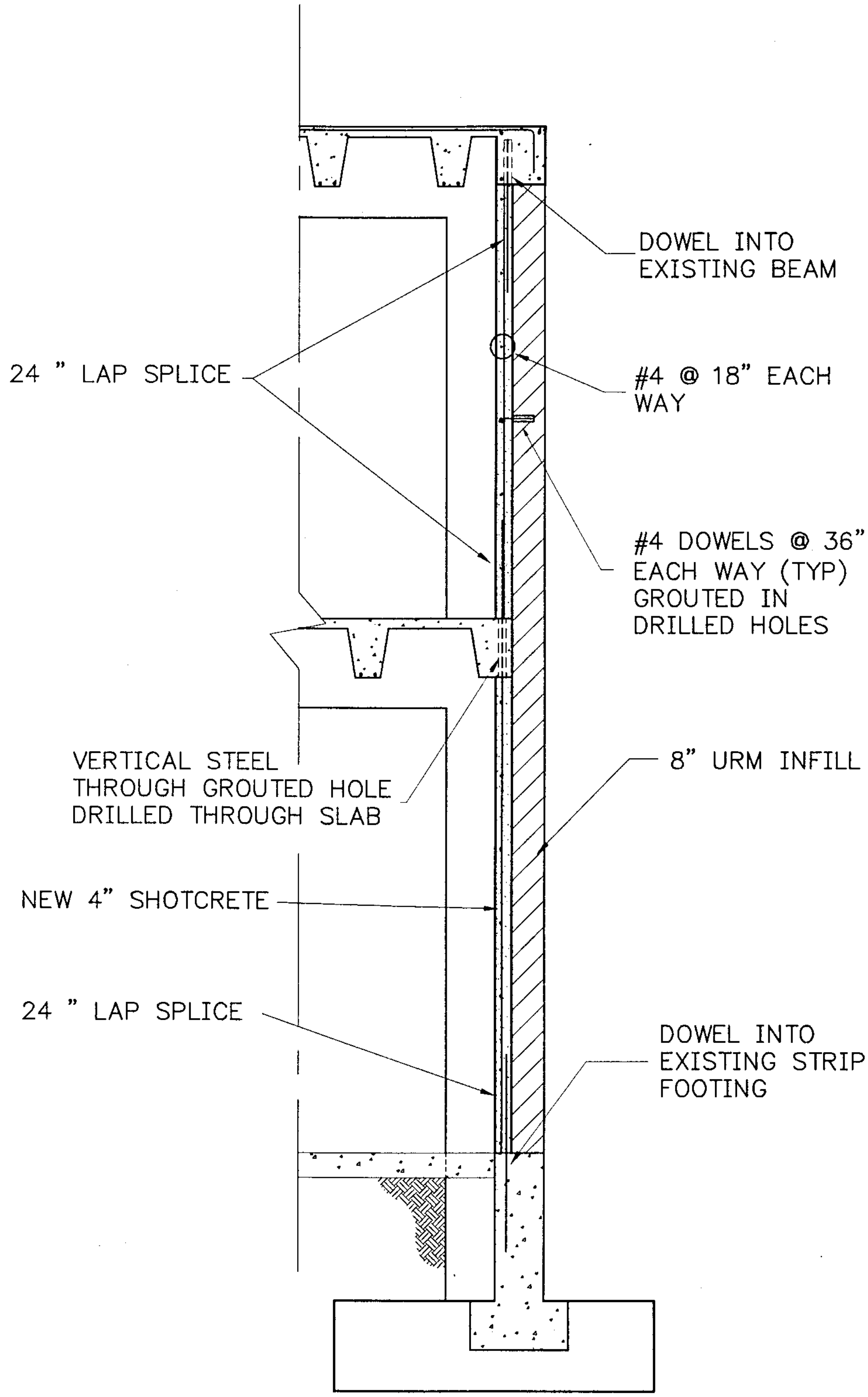
Supplementary analysis of the existing building is not necessary. The evaluation report contains sufficient detail to commence with the rehabilitation design.

4. *Rehabilitation concept selection*

The rehabilitation concept selected is discussed in step I.3 above.

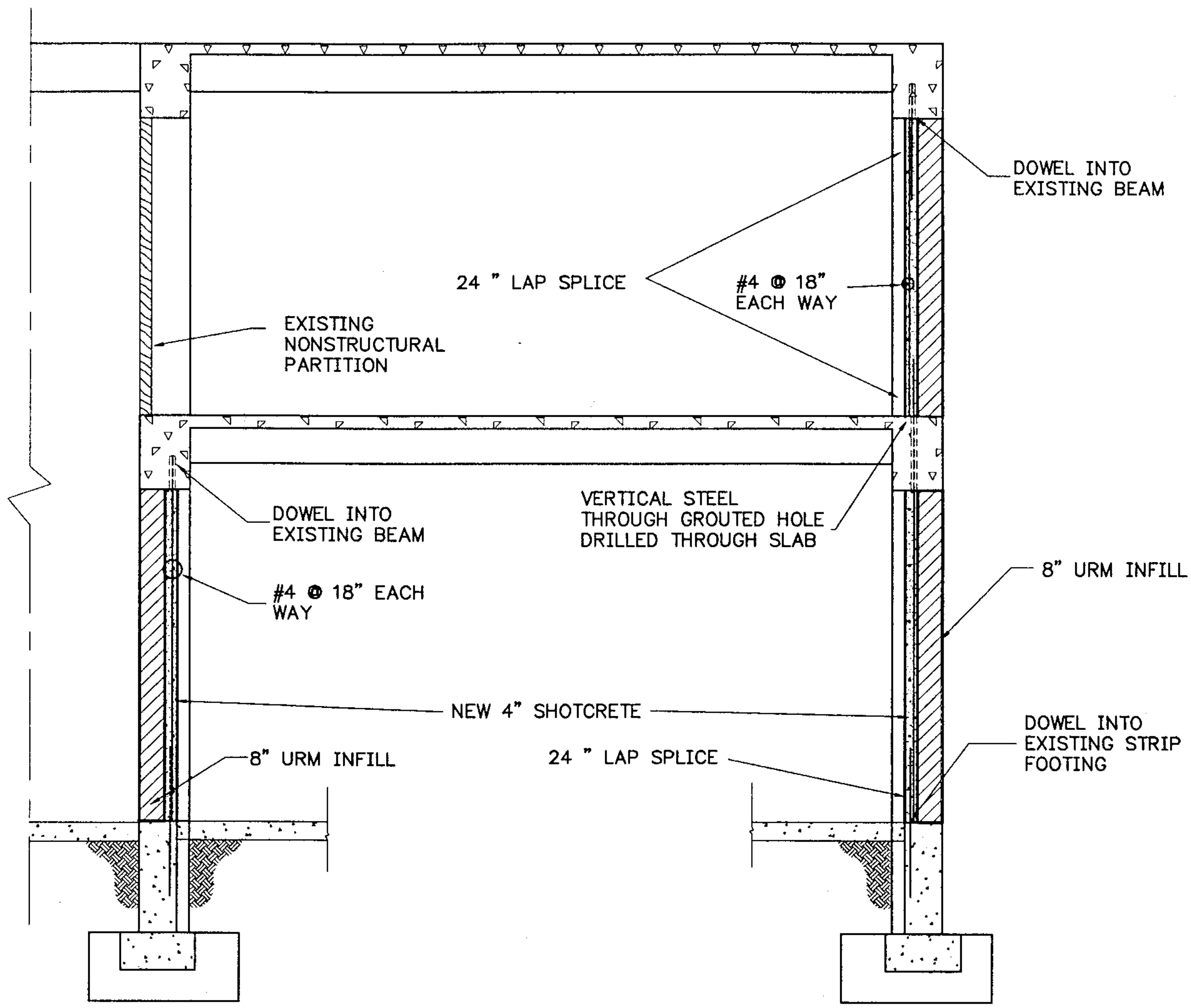
5. *Rehabilitation design*

The following figures show the rehabilitation design selected:



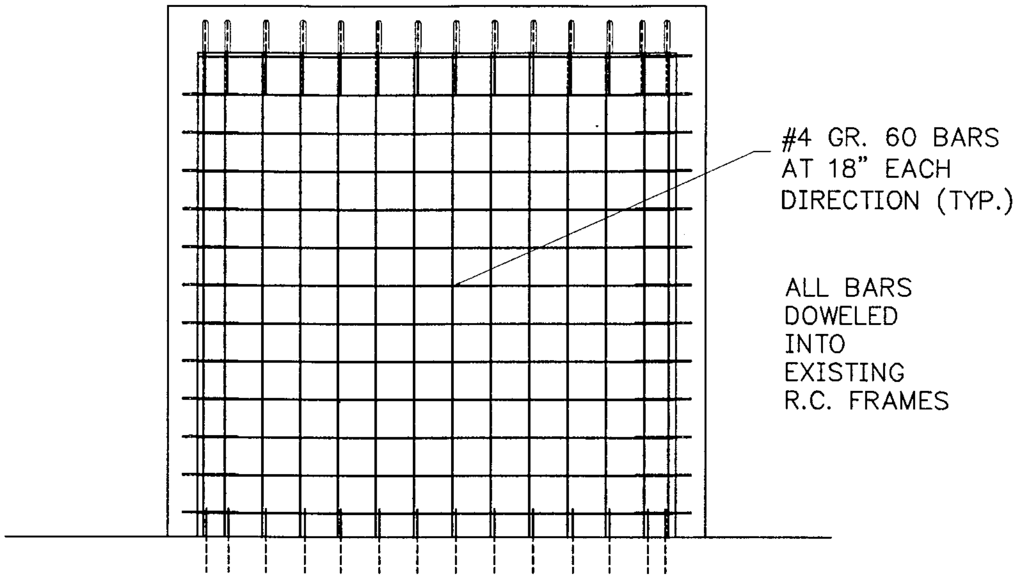
REHABILITATED WALL SECTION THROUGH

WALL LINE 2 AT MEZZANINE

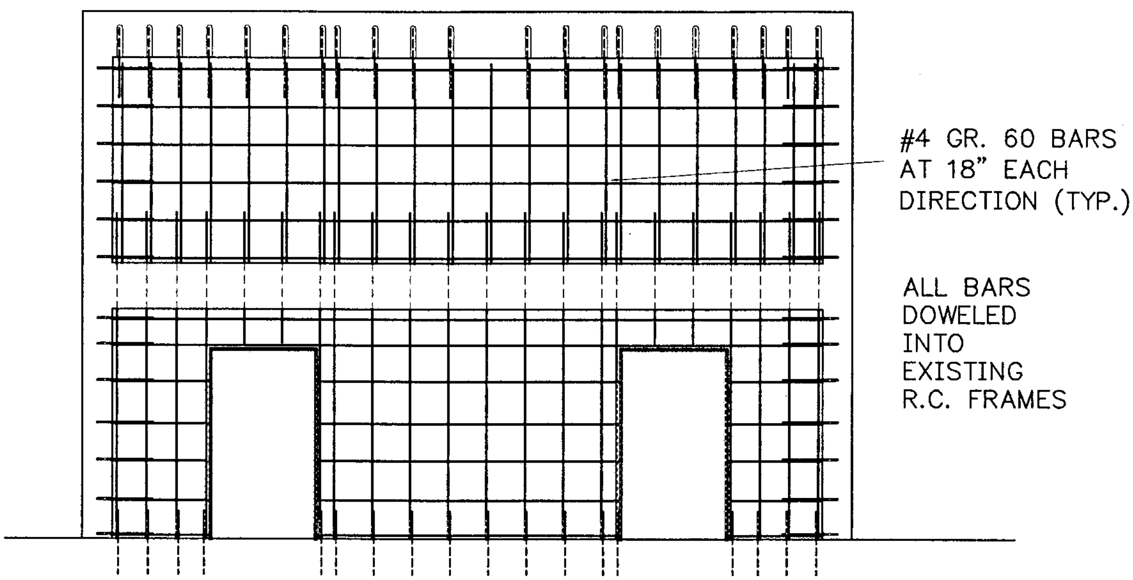


REHABILITATED WALL SECTION THROUGH

WALL LINES (C) AND (D) AT MEZZANINE

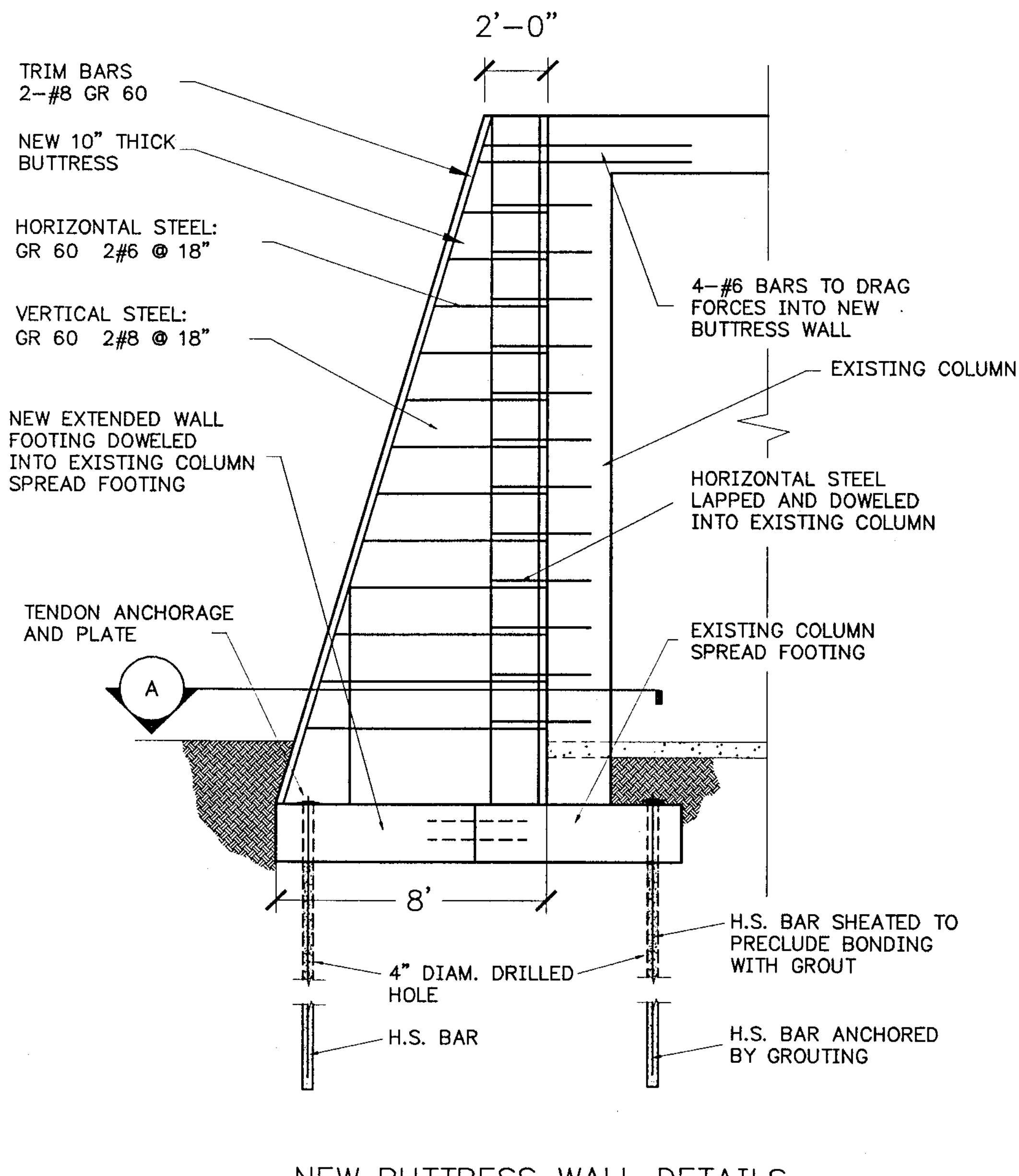


TYPICAL SHOTCRETE REINFORCMENT
FOR LONGITUDINAL INFILL PANEL

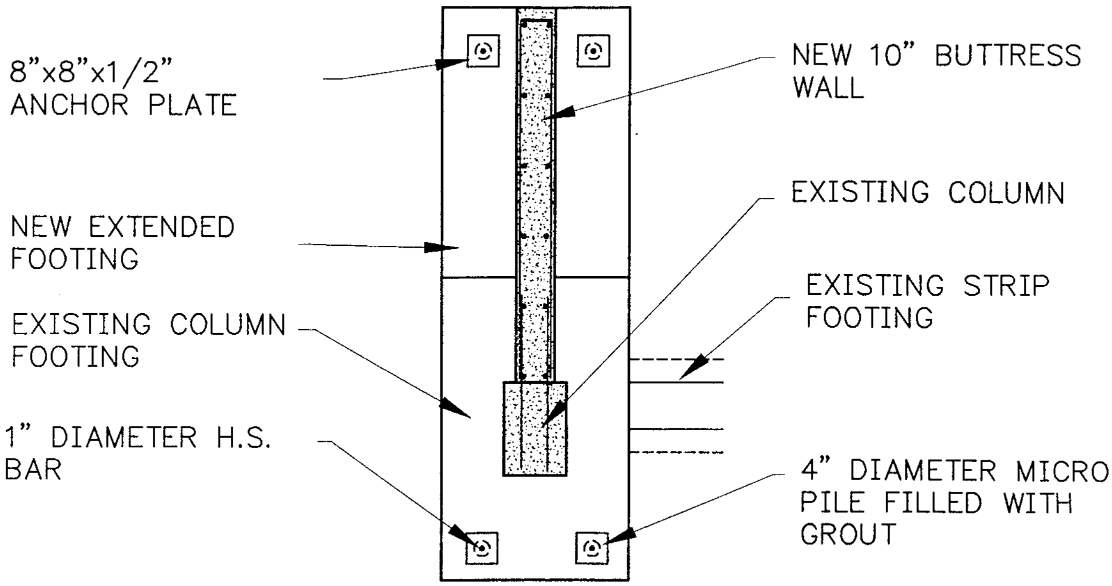


SHOTCRETE REINFORCEMENT FOR

WALL LINE D

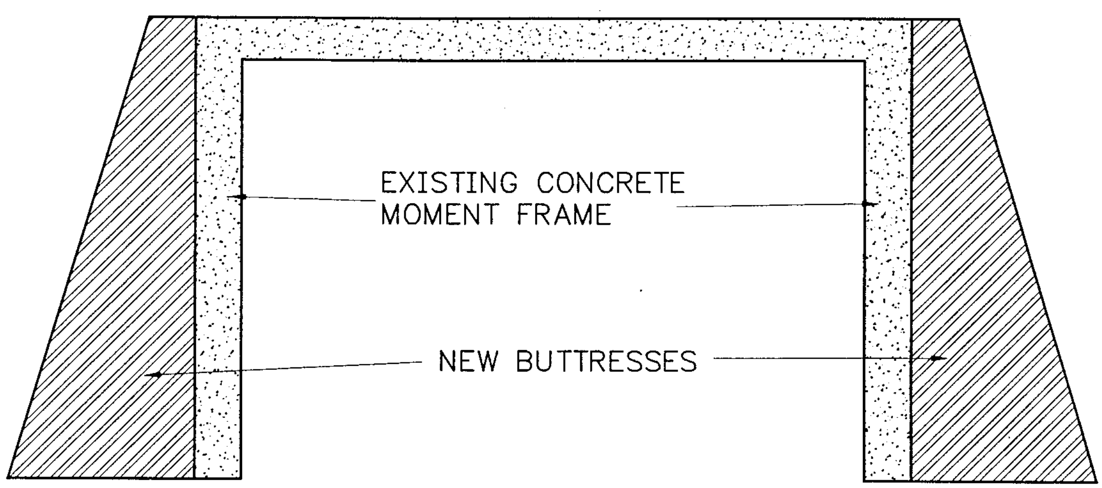


NEW BUTTRESS WALL DETAILS



SECTION (A) THROUGH BUTTRESS

1 in = 25.4 mm



ELEVATION OF WALL LINE (A) SHOWING NEW BUTTRESSES AT DOOR OPENING

6. Confirming evaluation of rehabilitation

a. Analytical procedures

The rehabilitated structure is evaluated using the Linear Static Procedure (LSP) outlined in FEMA 273 Section 3.3.1. The building model is created using the mathematical modeling assumptions of FEMA 273 Section 3.2.

- Basic Assumptions (FEMA 273 Section 3.2.2.1)

The building is modeled, analyzed and evaluated as a three-dimensional assembly of elements and components. The roof and mezzanine diaphragms are assumed to be rigid and capable of transmitting torsional forces. The computer program ETABS was used for the modeling of the structure.

- Horizontal Torsion (FEMA 273 Section 3.2.2.2)

The total torsional moment at a given floor level is set equal to the sum of the following two torsional moments:

1. Actual Torsion: The moment resulting from the eccentricity between the centers of mass at all floor levels above including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor. The effects of actual torsion are captured directly by the ETABS computer model.
2. Accidental Torsion: An accidental torsional moment produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load. The effect of accidental torsion shall be considered if the maximum lateral displacement due to this effect at any point on any floor diaphragm exceeds the average displacement by more than 10%. This effect shall be calculated independent of the effect of actual torsion. For linear analysis of building with rigid diaphragms, when the ratio $\delta_{max} / \delta_{avg}$ due to total torsional moment exceeds 1.2, the effect of accidental torsion shall be amplified by a factor A_x:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \qquad \text{(FEMA 273 Eq. 3-1)}$$

The torsional forces and the need for the amplification factor are determined after the vertical distribution of lateral forces is calculated.

- Primary and Secondary Actions, Components, and Elements (FEMA 273 Section 3.2.2.3)

All of the columns, beams, and walls are considered to be primary elements.

- Stiffness Assumptions (FEMA 273 Section 6.4.1.2 for concrete components)

The effective stiffness values for beams, columns, and walls are taken from FEMA 273 Table 6-4

Beams:	Flexural Rigidity = $0.5E_cI_g$	Shear rigidity = $0.4E_cA_w$
Columns in compression:	Flexural Rigidity = $0.7E_cI_g$	Shear rigidity = $0.4E_cA_w$
Columns in tension:	Flexural Rigidity = $0.5E_cI_g$	Shear rigidity = $0.4E_cA_w$
Walls (cracked):	Flexural Rigidity = $0.5E_cI_g$	Shear rigidity = $0.4E_cA_w$

E_c = modulus of elasticity for concrete and shotcrete = $w_c^{1.5} 33\sqrt{f_c'}$ (ACI Section 8.5.1)

E_c for normal weight concrete may be taken as $57000\sqrt{f_c'}$

E_c for existing concrete moment frames and new buttresses $= 57000\sqrt{3000} = 3122\,\text{ksi}$

E_c for 3000 psi lightweight shotcrete $= (120\text{pcf})^{1.5} 33\sqrt{3000} = 2376\,\text{ksi}$

The rehabilitated walls are assumed to act as composite sections due to the presence of the existing unreinforced masonry and the new shotcrete. The stiffness of the walls is evaluated by assuming that the walls are 4" (102 mm) thick (this is equal to the new shotcrete thickness) but with a modified modulus of elasticity equal to the combination of the new shotcrete and the existing masonry.

E_{me} = 619 ksi (determined previously)

The modulus of elasticity used for the wall elements is = 2376 ksi + 619 ksi = 2995 ksi

- Foundation Modeling (FEMA 273 Section 3.2.2.6)

The foundation is assumed to be rigid and is not included in the mathematical model.

- P-Δ Effects (FEMA 273 Section 3.2.5)

Two type of P-Δ (second-order) effects are addressed:

1. Static P-Δ effects: The stability coefficient θ, is assumed to be less than 0.1. Therefore, static P-Δ effects are ignored.
2. Dynamic P-Δ effects: The coefficient C_3 captures this effect for the linear procedures.

- Multidirectional Excitation Effects (FEMA 273 Section 3.2.7)

The multidirectional (orthogonal) excitation effects are captured by evaluating the forces and deformations associated with 100% of the seismic displacement in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction.

- Component Gravity Loads and Load Combinations

There are two gravity load combinations that must be considered. The first combination is different than the FEMA 273 equation while the second is taken directly from FEMA 273.

$Q_G = 1.2\ Q_D + 0.5\ Q_L + 0.2Q_S$ ($Q_S = 0$ for this example) (Eq. 7-1)
$Q_G = 0.9\ Q_D$ (FEMA 273 Eq. 3-3)

- Period Determination (FEMA 273 Section 3.3.1.2)

The building period is determined using Method 2 of FEMA 273 Section 3.3.1.2.

$T = C_t h_n^{3/4}$ (FEMA 273 Eq. 3-4)
The building is assumed to act as a shear wall structure in both the longitudinal and transverse direction. The C_t factor is 0.020 for shear wall structures.

$T = 0.020(20')^{3/4} = 0.19$ seconds

- Pseudo Lateral Load (FEMA 273 Section 3.3.1.3)

$V = C_1C_2C_3S_aW$ (FEMA 273 Eq. 3-6)

C_1 factor:
$C_1 = 1.5$ for $T < 0.10$, $C_1 = 1.0$ for $T \geq T_0$

where $T_0 = S_{D1} / S_{DS}$ for 5% damping, $T_0 = 0.41 / 0.73 = 0.56$ seconds
$C_1 = 1.40$ by linear interpolation for $0.10 < T = 0.19 < 0.56$ seconds

C_2 factor:

The C_2 coefficient is taken form FEMA 273 Table 3-1 to be equal 1.0 for the Immediate Occupancy Performance Level.

$C_2 = 1.0$

C_3 factor:

It is assumed that the building does not stability problems due to stiffness of the shear walls.

$C_3 = 1.0$

S_a, Spectral Acceleration:

$S_a = 0.73$ (determined previously)

W, Seismic Weight

The weight of the building is updated to account for the additional weight of the new shotcrete and buttress shear walls.

The new building seismic weights are:

Weight tributary to the roof level:	352.3 kips
Weight tributary to the mezzanine level:	125.4 kips
Total weight, W:	478 kips (2126 kN)

$V = C_1C_2C_3S_aW = (1.40)(1.0)(1.0)(0.73)(478 \text{ kips}) = 489 \text{ kips } (2175 \text{ kN})$

- Vertical Distribution of Seismic Forces:

  The lateral force is distributed to the roof and mezzanine levels assuming that the building acts as a one-story structure. The pseudo lateral force is distributed to the roof and mezzanine based on tributary mass.

Level	w_x (kips)	$C_1C_2C_3S_a$	F_x (kips)	F_x (kN)
Roof	352	1.025	361	1606
Mezzanine	125	1.025	128	572

- Determine Torsional Forces and need for Amplification Factor

The total torsion has contributions from both actual and accidental torsion. The actual torsion is automatically captured by the three dimensional ETABS computer model. The accidental torsion must be calculated.

Transverse Seismic Forces:

Roof Level:

Shear:	361 kips
Perpendicular dimension:	60 ft.
5% offset:	3 ft.
Torsion = (361 k)(3') =	1083 kip-ft (1469 kN-m)

Mezzanine Level:

Shear:	128 kips
Perpendicular dimension:	20 ft.
5% offset:	1 ft.
Torsion = (44 k)(1') =	128 kip-ft (174 kN-m)

Longitudinal Seismic Forces:

Roof Level:

Shear:	361 kips
Perpendicular dimension:	30 ft.
5% offset:	1.5 ft.
Torsion = (397 k)(1.5') =	542 kip-ft (735 kN-m)

Mezzanine Level:

Shear:	80 kips
Perpendicular dimension:	30 ft.
5% offset:	1.5 ft.
Torsion = (44 k)(1.5') =	120 kip-ft (163 kN-m)

These accidental torsional forces are placed upon the computer model of the structure to determine the need for torsional amplification.

Seismic forces in the longitudinal direction:

Average displacement of the roof diaphragm, δ_{avg} = 0.0165"
Maximum displacement of point on diaphragm, δ_{max} = 0.0176"
$\delta_{max} / \delta_{ave}$ = (0.0176") / (0.0165") = 1.07 < 1.2
∴ No Torsional amplification needed
It was determined that the mezzanine level torsion required no amplification either (calculations not shown).

Seismic forces in the transverse direction:

Average displacement of the roof diaphragm, δ_{avg} = 0.092"
Maximum displacement of point on diaphragm, δ_{max} = 0.118"
$\delta_{max} / \delta_{ave}$ = (0.118") / (0.092") = 1.28 > 1.2
∴ Torsional amplification needed

$$A_x = \left(\frac{0.118"}{1.2(0.092")}\right)^2 = 1.14 < 3.0$$

Amplified Accidental Roof Level Torsion = A_xT = (1.14)(1083 kip-ft) = 1235 kip-ft (1675 kN-m)

Amplified Accidental Mezzanine Level Torsion = A_xT = (1.14)(128 kip-ft) = 146 kip-ft (198 kN-m)

Component Forces:

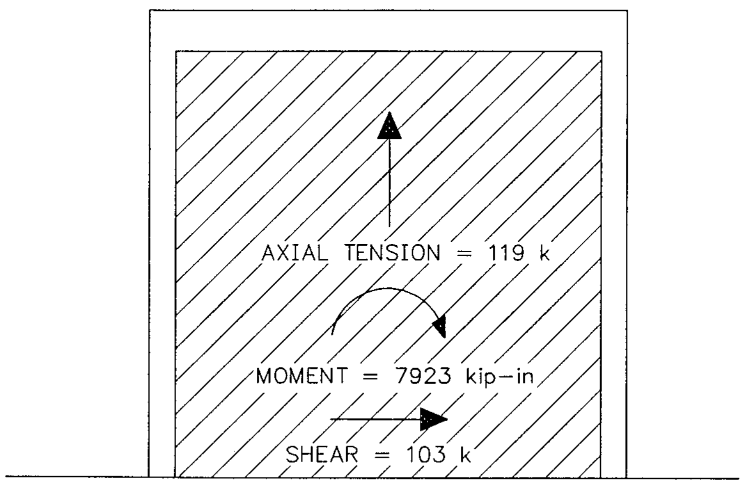
Deformation-Controlled Components

The deformation-controlled actions consist of wall, beam, and column flexure, and shear in the wall elements (Footnote (1) from TI 809-04 Table 7-3 states that for shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15Agfc'$, the longitudinal reinforcement must be symmetrical, and the maximum shear stress must be $\leq 6\sqrt{f_c'}$, otherwise the shear shall be considered to be a force-controlled action). The design actions Q_{UD} are calculated according to:

$Q_{UD} = Q_G \pm Q_E$ (FEMA 273 Eq. 3-14)
Q_E = Design earthquake loads
Q_G = Design gravity loads

Wall Flexural Forces: The maximum moment from all of the load combinations is shown for each typical wall panel element.

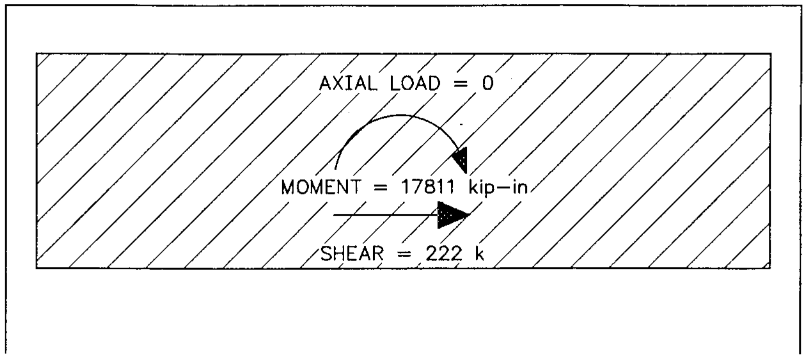
Typical Longitudinal Walls (grid lines 1 and 2):



TYPICAL LONGITUDINAL INFILL PANEL

1 kip = 4.448 kN
1 kip-in = 0.113 kN-m

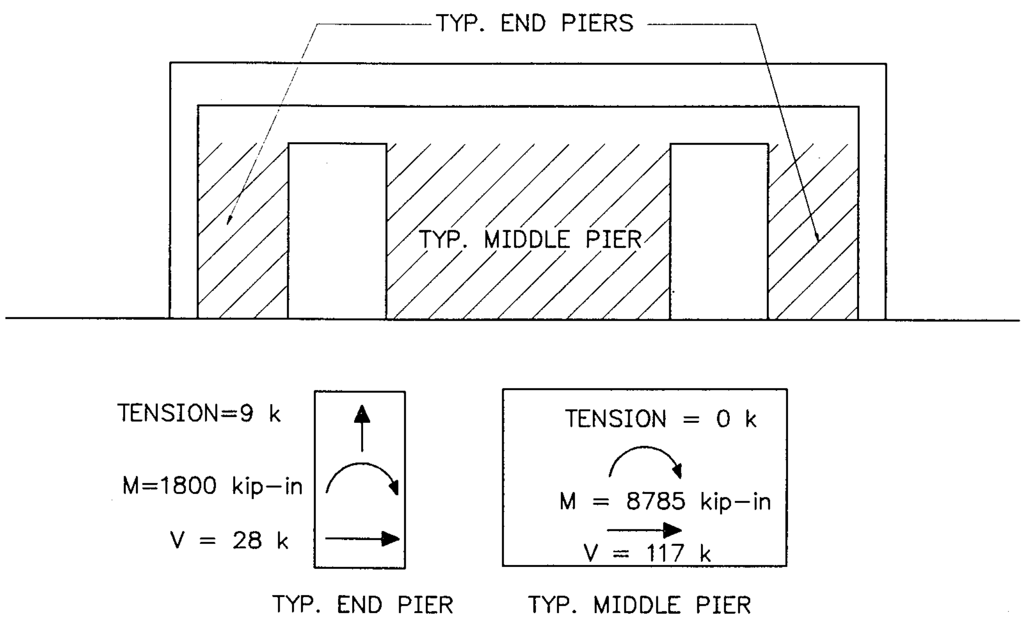
Wall Line D, Panel Above Mezzanine Level:



TRANSVERSE WALL LINE D ABOVE MEZZANINE

1 kip = 4.448 kN
1 kip-in = 0.113 kN-m

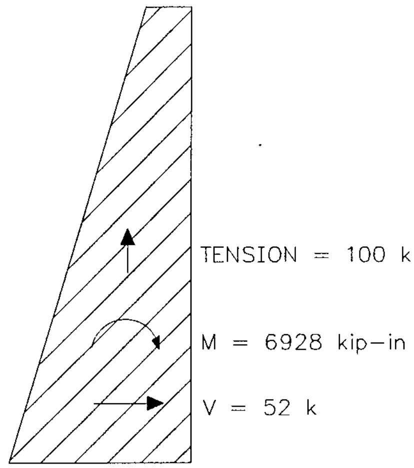
Wall Lines C & D; Typical Panels Below the Mezzanine Level:



TYPICAL PIERS FOR WALL LINES C & D BELOW MEZZANINE LEVEL

1 kip = 4.448 kN
1 kip-in = 0.113 kN-m

Typical Buttress Wall at Grid Line A:



TYPICAL BUTTRESS

1 kip = 4.448 kN
1 kip-in = 0.113 kN-m

Wall Shear Forces:

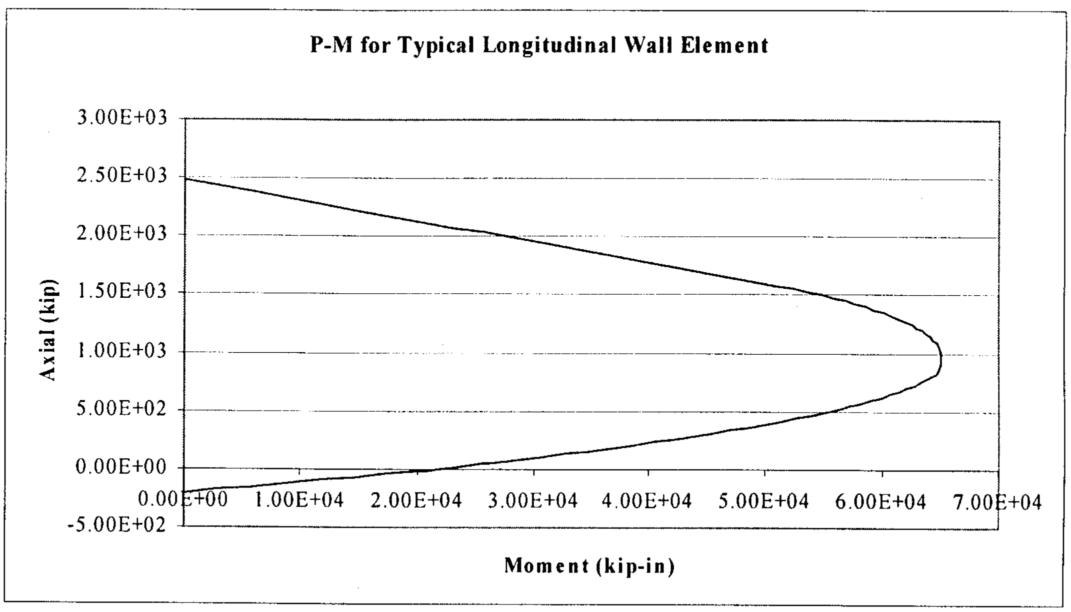
The wall shear force demand is taken as either the force from the ETABS analysis or the maximum force that can be developed by the wall. If the wall remains elastic in flexure, the shear demand is taken as the force from the ETABS output. If the wall is pushed beyond its elastic limit in flexure, the shear demand is taken as the maximum force that can be developed by the wall. FEMA 273 Section 6.8.2.3 states that the nominal flexural strength of a shear wall or wall segment shall be used to determine the maximum force likely to act in shear walls and wall segments. For cantilever shear walls the design shear force is equal to the magnitude of the lateral force required to develop the nominal flexural strength at the base of the wall, assuming the lateral force is distributed uniformly over the height of the wall. For wall segments, the design shear force is equal to the shear corresponding to the development of the positive and negative nominal moment strengths at opposite ends of the wall segment.

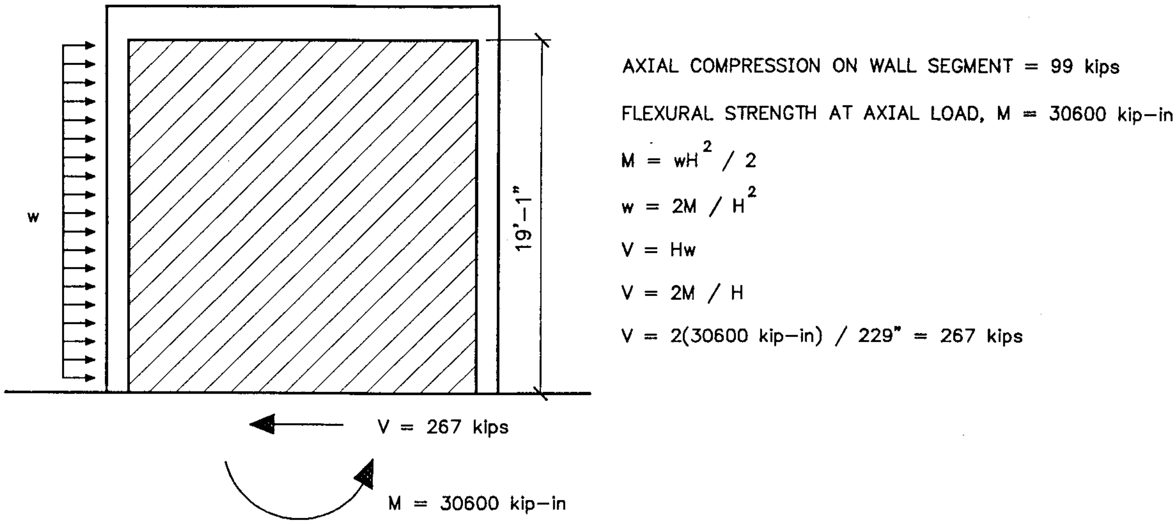
For the determination of the wall flexural strength the yield strength of the longitudinal reinforcement should be taken as 125% of the specified yield strength to account for material overstrength and strain hardening. For all moment strength calculation, the axial load acting on the wall shall be considered.

The P-M interaction diagrams for the wall elements were calculated using the computer program BIAX with f_c' of the shotcrete = 3000 psi and the yield strength of the steel = $1.25f_y$ = 1.25(60ksi) = 75 ksi.

Typical Longitudinal Walls (grid lines 1 and 2):

The longitudinal walls are assumed to act as cantilevers.



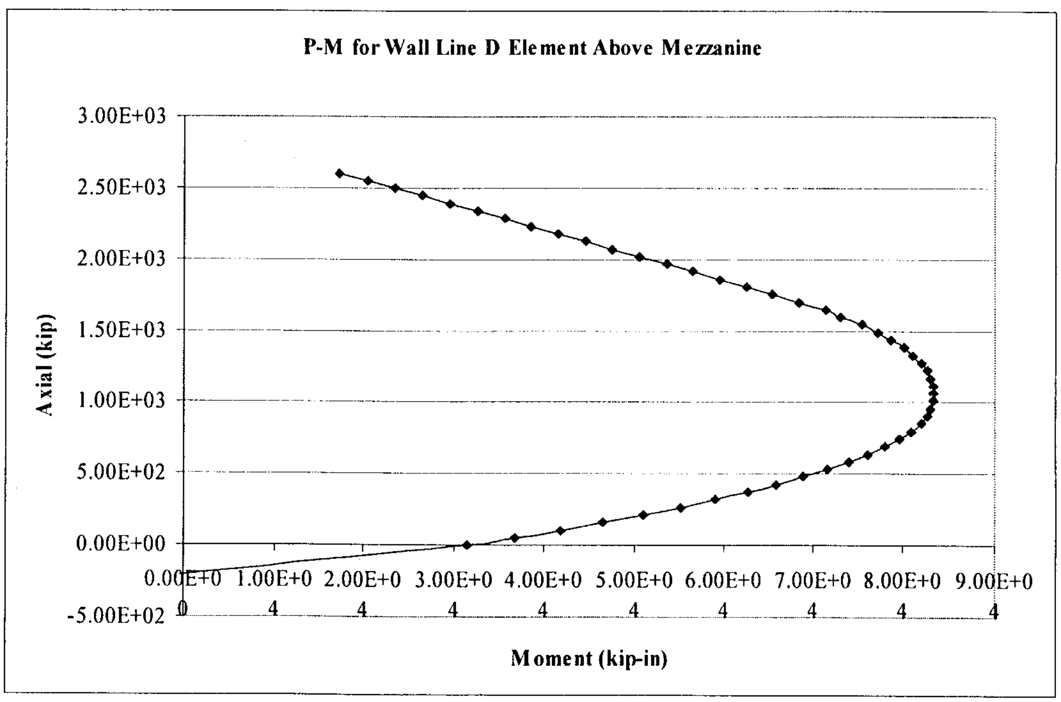


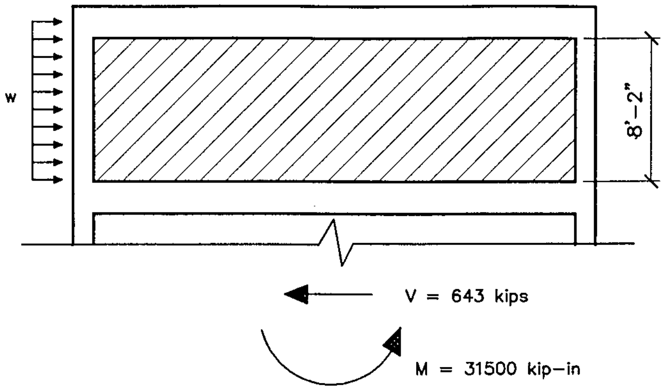
TYPICAL LONGITUDINAL WALL SEGMENT

Flexural shear demand on wall = V = 267 kips (1188 kN)

Wall Line D, Panel Above Mezzanine Level:

This wall segment is assumed to act as a cantilever.





AXIAL LOAD ON WALL = 0 kips

MOMENT STRENGTH, M = 31500 kip-in

$M = wH^2 / 2$

$w = 2M / H^2$

$V = Hw$

$V = 2M / H$

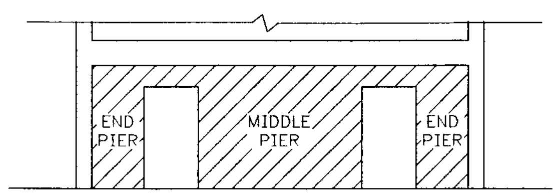
V = 2(31500 kip-in)/98" = 643 kips

WALL LINE D, PANEL ABOVE MEZZANINE LEVEL

Flexural shear demand on wall = V = 643 kips (2860 kN)

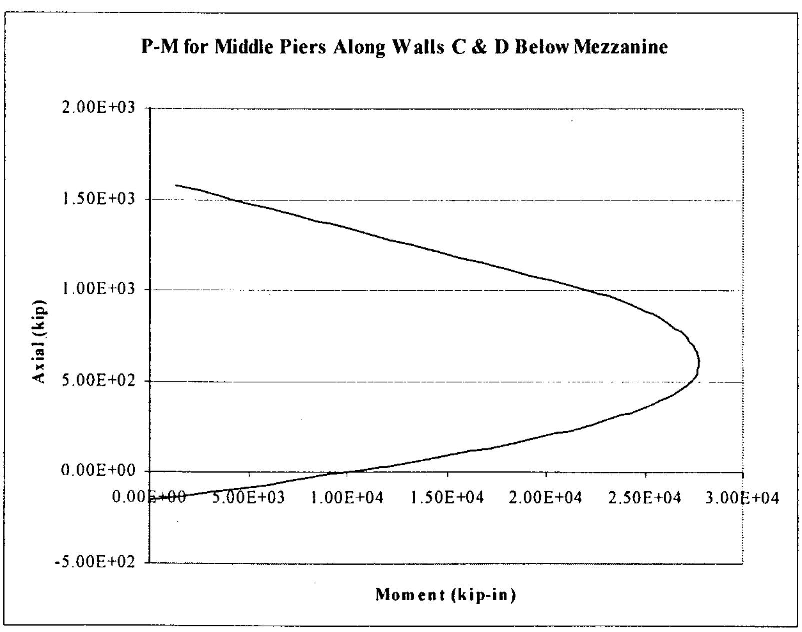
Wall Lines C & D; Typical Panels Below the Mezzanine Level:

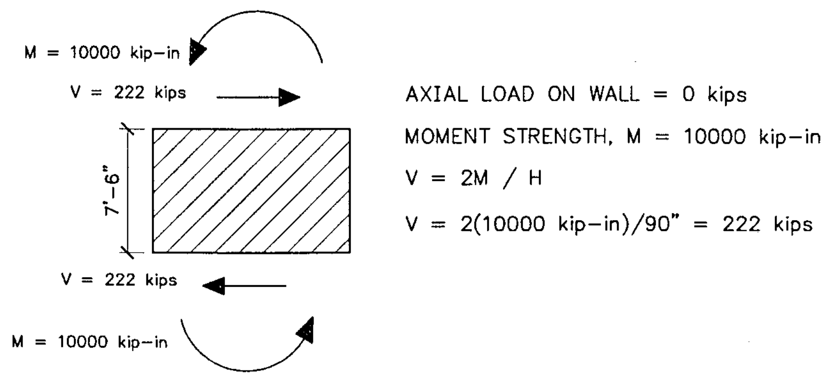
Wall lines C and D below the mezzanine level consist of two narrow end piers and a larger pier in the middle of the wall. Each of the piers is assumed to act as a fixed-fixed wall element.



WALL LINES C & D BELOW MEZZANINE LEVEL

Typical Middle Pier:

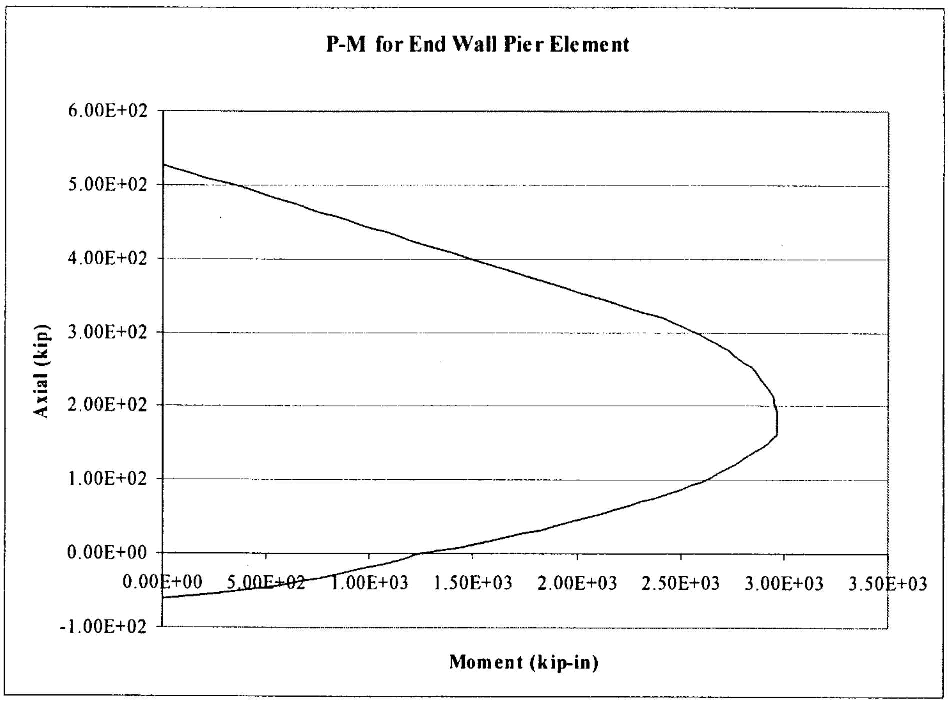


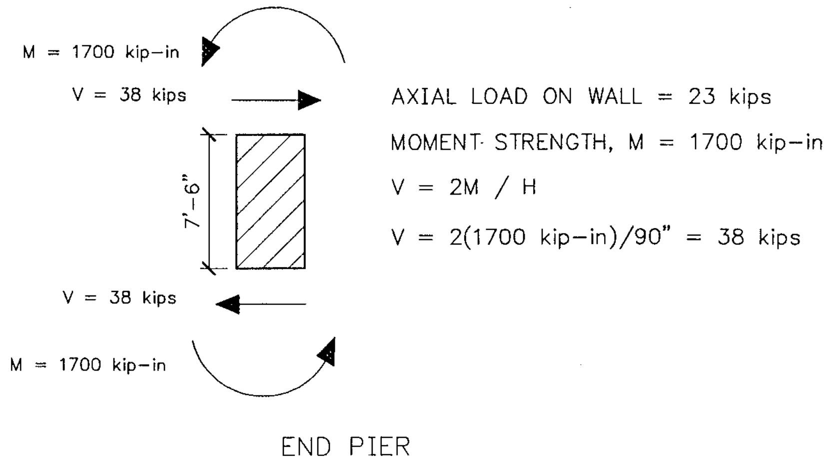


MIDDLE PIER

Q_{UD} = 222 kips (987 kN)

Typical End Pier:

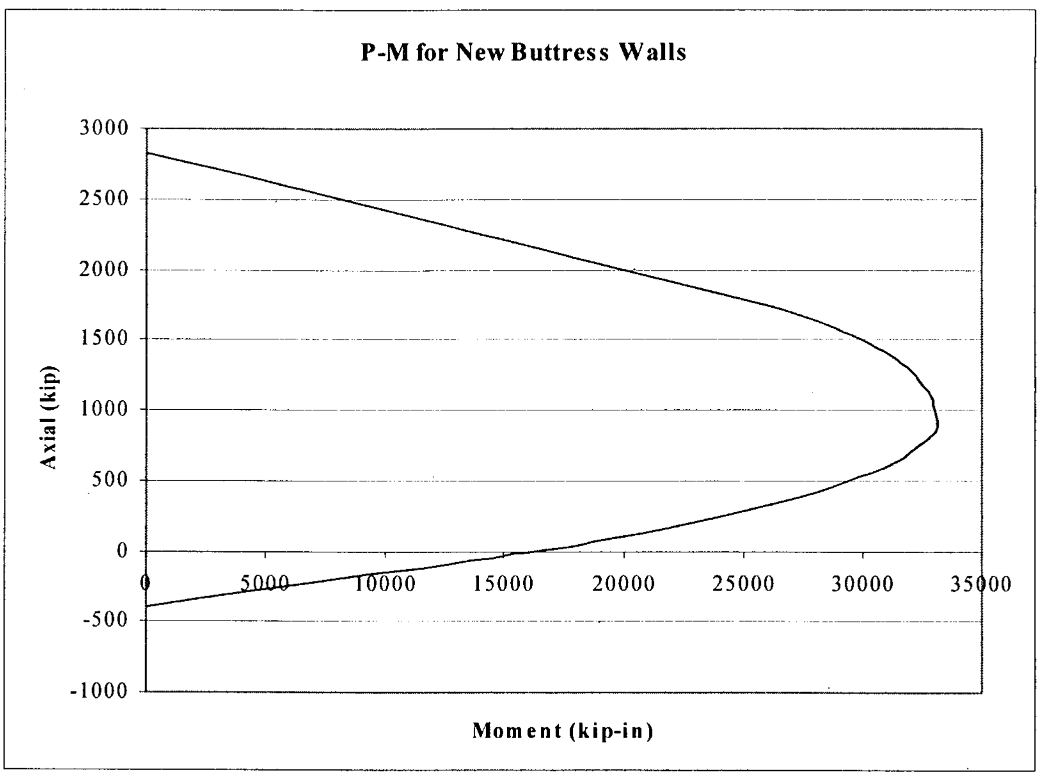


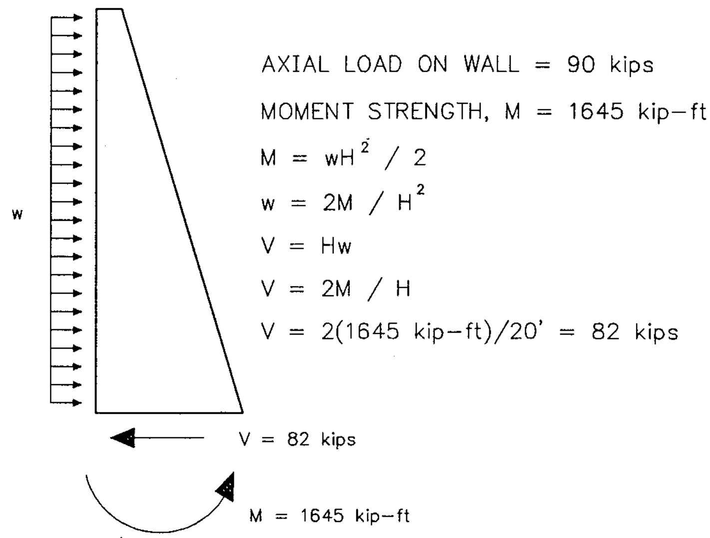


Q_{UF} = V = 38 kips (169 kN)

Typical New Buttress Walls:

The buttress walls are assumed to act as cantilevers.





NEW BUTTRESS WALLS

Flexural shear demand on wall = V = 82 kips (365 kN)

<u>Beam Flexural Forces:</u>

Transverse Beams:
The transverse beams all have the same dimensions and reinforcement. The maximum forces along the transverse beams are:

Ends of beams:
Maximum positive moment demand = 977 kip-in (110 kN-m)
Maximum negative moment demand = 1929 kip-in (218 kN-m)

Midpoint of beams:
Maximum positive moment demand = 1021 kip-in (115 kN-m)
Maximum negative moment demand = 821 kip-in (93 kN-m)

Maximum shear demand = 36 kips (160 kN)

Longitudinal Beams:

The longitudinal beams all have the same dimensions and reinforcement. The maximum forces along the longitudinal beams are:

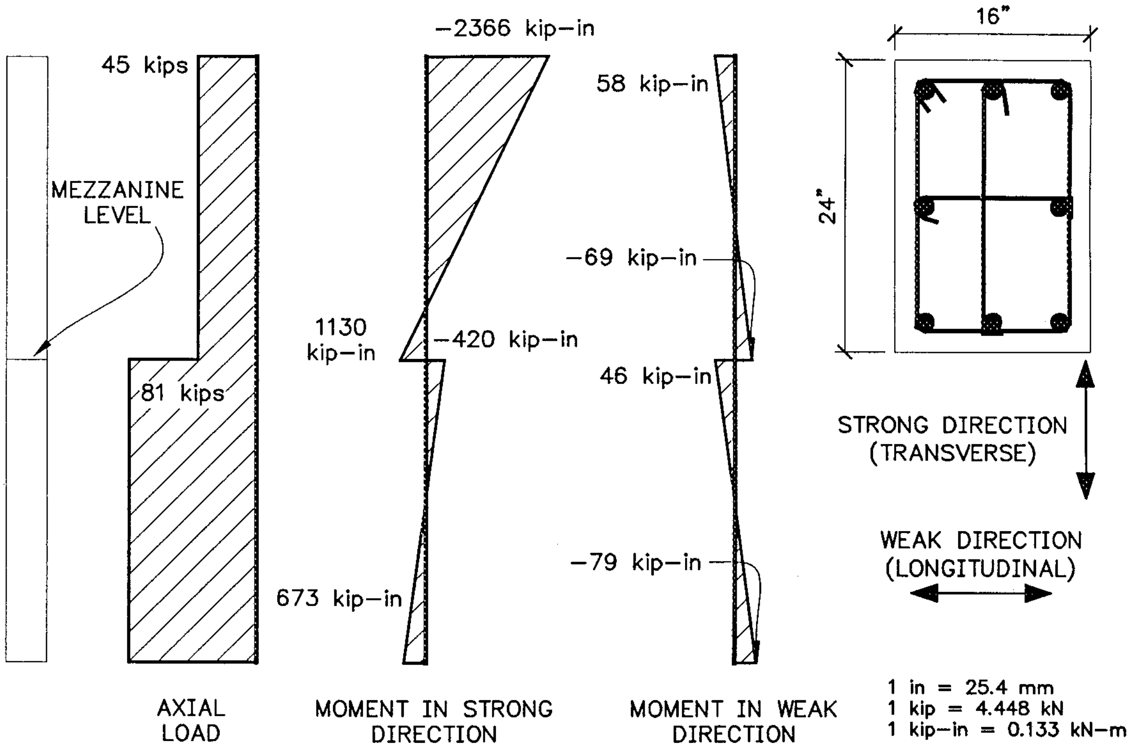
Ends of beams:
Maximum positive moment demand = No positive moments at beam ends
Maximum negative moment demand = 128 kip-in (14 kN-m)

Midpoint of beams:
Maximum positive moment demand = 72 kip-in (8.1 kN-m)
Maximum negative moment demand = No negative moments at beam midspan

Maximum shear demand = 3 kips (13.3 kN)

Column Flexural Forces

The columns resist forces in both the transverse and longitudinal directions. The flexural strength of the columns is a function of the axial load present due to axial-moment interaction. Therefore, the column with the highest flexural demands may not be the most critical due to the axial load present. Only the forces on the most critical column is shown for the check of acceptance for flexure (columns located along grid line C are the most critical).



COLUMN AXIAL AND MOMENT DIAGRAMS

Force-Controlled Components

The force-controlled actions consist of column, beam and diaphragm shear. The design actions Q_{UF} are taken as either the maximum action that can be developed in a component considering the nonlinear behavior of the building or the value calculated according to:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} \qquad \text{(FEMA 273 Eq. 3-16)}$$

Beam Shear Forces:

The beam shear and moment demands were listed earlier in the deformation-controlled components section. The beams in both the longitudinal and transverse directions do not develop flexural hinges when subjected to the design earthquake forces. Therefore, their shear demand is calculated using FEMA 273 Eq. 3-16 for force-controlled components. To be conservative, the Q_E term in equation 3-16 is not divided by the 'C' factors.

Typical Transverse Beam:

Q_{UF} = 36 kips (160 kN)

Typical Longitudinal Beam:

Q_{UF} = 3 kips (13.3 kN)

Column Shear Forces:

The columns resist shear forces in both the longitudinal and transverse direction. In the longitudinal direction (the column weak direction) the shear wall panels resist essentially all of the shear force. Therefore, the columns are checked for shear in their strong direction only (transverse direction).

All of the columns have the same dimensions and reinforcement details so only the one with the highest shear force is checked. The maximum shear force (determined from equation 3-16) occurs in the columns along grid line 3 above the mezzanine level.

Q_{UF} = 36 kips (160 kN)

Diaphragm Shear Forces:

Roof diaphragm:

Transverse Direction: The highest diaphragm shear for seismic forces in the transverse direction occurs along wall line D due to the high stiffness of the wall. The diaphragm must transfer 222 kips into wall line D.

V = 222 kips (987 kN)
L = 31'
Q_{CE} = 222 kips / 31' = 7.2 klf

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3} = Q_{UF} = \frac{7.2\,\text{klf}}{(1.4)(1.0)(1.0)} = 5.1\,\text{klf}\ (74\ \text{kN/m})$$

Longitudinal Direction:

V = 205 kips (from ETABS output)
L = 60'
Q_{CE} = 205 kips / 60' = 3.4 klf

$$Q_{UF} = \frac{3.4\,\text{klf}}{(1.4)(1.0)(1.0)} = 2.4\,\text{klf}\ (35.0\ \text{kN/m})$$

Mezzanine diaphragm:

Transverse Direction:

V = 161 kips (from ETABS output)
L = 31'
Q_{CE} = 161 kips / 31' = 5.2 klf

$$Q_{UF} = \frac{5.2\,\text{klf}}{(1.4)(1.0)(1.0)} = 3.7\,\text{klf}\ (54\ \text{kN/m})$$

Longitudinal Direction:

V = 75 kips (from ETABS output)
L = 20'
Q_{CE} = 75 kips / 20' = 3.75 klf

$$Q_{UF} = \frac{3.75\,\text{klf}}{(1.4)(1.0)(1.0)} = 2.68\,\text{klf}\ (39.1\ \text{kN/m})$$

b. *Acceptance criteria*

Deformation-Controlled Components

Deformation-controlled actions in primary components and elements must satisfy:

$$mQ_{CE} \geq Q_{UD} \qquad \text{(Eq. 7-2)}$$

<u>*Wall Flexural and Shear Forces:*</u>

The expected flexural strength of the walls was determined using the computer program BIAX. Per FEMA 273 Section 6.8.2.3, the yield strength of the longitudinal reinforcement is taken as 125% of the specified yield strength to account for material overstrength and strain hardening. The strength of the wall is based on the new shotcrete and reinforcement only; contribution to the strength by the original masonry is neglected.

Typical Longitudinal Walls (grid lines 1 and 2):

Determine if wall is flexure or shear-controlled:

$$V_n = A_{CV}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) \qquad \text{(ACI 318 Eq. 21-7)}$$

h / l for the wall = 229" / 226" = 1.01 < 1.5, α = 3.0

$$V_n = (226"\text{x}4")\left(3.0\sqrt{3000\,\text{psi}} + 0.0028(60000\,\text{psi})\right) = 300\,\text{kips}$$

V_n = 300 kips > Flexural shear demand = 267 kips (determined previously), therefore the wall is flexure-controlled.

<u>Flexure:</u>
Q_{CE} = 9400 kip-in (at an axial tension load of 119 kips)
Q_{UD} = 7923 kip-in (895 kN-m)
m = 2.0 (TI 809-04 Table 7-2)
mQ_{CE} = (2.0)(9400 kip-in) = 18800 kip-in (2124 kN-m) > 7923 kip-in (895 kN-m), OK

The wall panels will not yield in flexure (9400 kip-in > 7923 kip-in), therefore the shear demand is taken as the force from the ETABS analysis.

Shear:
Shear, V = Q_{UD} =103 kips
Q_{CE} = V_n = 300 kips (1334 kN)
mQ_{CE} = (2.0)(300 kips) = 600 kips (2669 kN) > 103 kips (458 kN), OK

Wall Line D, Panel Above Mezzanine Level:

Determine if wall is flexure or shear-controlled:

$$V_n = A_{CV}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right)$$ (ACI 318 Eq. 21-7)

h / l for the wall = 98" / 336" = 0.29 < 1.5, α = 3.0

$$V_n = (336\text{"} \times 4\text{"})\left(3.0\sqrt{3000\,\text{psi}} + 0.0028(60000\,\text{psi})\right) = 447\,\text{kips}$$

V_n = 447 kips < Flexural shear demand = 643 kips, therefore the wall is shear-controlled

Flexure:
Q_{CE} = 31500 kip-in (3560 kN-m)
Q_{UD} = 17811 kip-in (2013 kN-m)
m = 2.0 (TI 809-04 Table 7-3)
mQ_{CE} = (2.0)(31500 kip-in) = 63000 kip-in (7119 kN-m) > 17811kip-in (2013 kN-m), OK

The wall panels will not yield in flexure (31500 kip-in > 17811 kip-in), therefore the shear demand is taken as the force from the ETABS analysis.

Shear:
Shear, V = Q_{UD} = 222 kips
Q_{CE} = V_n = 447 kips
mQ_{CE} = (2.0)(447 kips) = 894 kips(3977 kN) > 222 kips (987 kN), OK

Wall Lines C & D; Typical Panels Below the Mezzanine Level:

Typical Middle Pier:

Determine if wall is flexure or shear-controlled:

$$V_n = A_{CV}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right)$$ (ACI 318 Eq. 21-7)

h / l for the wall = 90" / 144" = 0.63 < 1.5, α = 3.0

$$V_n = (144\text{"} \times 4\text{"})\left(3.0\sqrt{3000\,\text{psi}} + 0.0028(60000\,\text{psi})\right) = 191\,\text{kips}$$

V_n = 191 kips < Flexural shear demand = 222 kips (determined previously), therefore the wall is shear-controlled.

Flexure:
Q_{CE} = 10000 kip-in (at an axial load of 0 kips)
Q_{UD} = 8785 kip-in (determined previously)
m = 2.0 (TI 809-04 Table 7-3)
mQ_{CE} = (2.0)(10000 kip-in) = 20000 kip-in (2260 kN-m) > 8785 kip-in (993 kN-m), OK

The wall panels will not yield in flexure (10000 kip-in > 8785 kip-in), therefore the shear demand is taken as the force from the ETABS analysis.

Shear:
Shear, V = Q_{UD} =117 kips
$Q_{CE} = V_n$ = 191 kips
mQ_{CE} = (2.0)(191 kips) = 382 kips (1699 kN) > 117 kips (520 kN), OK

Typical End Pier:

Determine if wall is flexure or shear-controlled:

$$V_n = A_{CV}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) \qquad \text{(ACI 318 Eq. 21-7)}$$

h / l for the wall = 90" / 46" = 1.95 > 1.5 but less than 2.0, α = 2.0

$$V_n = (46\text{"}\,\text{x}4\text{"})\left(2.0\sqrt{3000\,\text{psi}} + 0.0028(60000\,\text{psi})\right) = 51\,\text{kips}$$

V_n = 51 kips > Flexural shear demand = 38 kips (determined previously), therefore the wall is flexure-controlled.

Flexure:
Q_{CE} = 1075 kip-in (at an axial tension load of 9 kips)
Q_{UD} = 1800 kip-in (determined previously)
m = 2.0 (TI 809-04 Table 7-2)
mQ_{CE} = (2.0)(1075 kip-in) = 2150 kip-in (243 kN-m) > 1800 kip-in (203 kN-m), OK

The wall panels will yield in flexure (1075 kip-in < 1800 kip-in), therefore the shear demand is taken as the shear force corresponding to the development of the wall-pier flexural capacity.

Shear:
Shear, V = Flexural shear capacity = 38 kips
$Q_{CE} = V_n$ = 51 kips
mQ_{CE} = (2.0)(51 kips) = 102 kips (454 kN) > 38 kips (169 kN), OK

New Buttress Walls:
A check of the new buttress walls is shown for forces at the base of the wall:

Determine if wall is flexure or shear-controlled: (use shear strength at top of wall to be conservative)

$$V_n = A_{CV}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) \qquad \text{(ACI 318 Eq. 21-7)}$$

h / l for the wall = 240" / 96" = 2.5 > 2.0, α = 2.0

$$V_n = (24\text{"}\,\text{x}10\text{"})\left(2.0\sqrt{3000\,\text{psi}} + 0.0049(60000\,\text{psi})\right) = 97\,\text{kips}\ \ (431\text{ kN})\text{ at top of wall}$$

V_n = 97 kips > Flexural shear demand = 82 kips (determined previously), therefore the wall is flexure-controlled.

Flexure:
Q_{UD} = 6928 kip-in (783 kN-m)
Q_{CE} = 12500 kip-in (1413 kN-m) (at an axial tension load of 100 kips)
m = 2.0 (TI 809-04 Table 7-2)
mQ_{CE} = (2.0)(12500 kip-in) = 25000 kip-in (2825 kN-m) > 6928 kip-in (783 kN-m), OK

Shear:
Shear, V = Flexural shear capacity = 82 kips (365 kN)
$Q_{CE} = V_n$ = 97 kips (431 kN)
mQ_{CE} = (2.0)(97 kips) = 194 kips (863 kN) > 82 kips (365 kN), OK

Beam Flexure:

Transverse Beams:

At beam ends; Q_{UD}^+ = 977 kip-in (110 kN-m) (largest positive moment demand)
Q_{UD}^- = 1929 kip-in (218 kN-m) (largest negative moment demand)
M_{CE}^+ = 1452 kip-in (164 kN-m), M_{CE}^- = 3444 kip-in (389 kN-m)

m = 2.0 (from TI 809-04 Table 7-14)
mQ_{CE}^+ = (2.0)(1452 kip-in) = 2904 kip-in (328 kN-m) > 977 kip-in (110 kN-m), OK
mQ_{CE}^-= (2.0)(3444 kip-in) = 6888 kip-in (9340 kN-m) > 1929 kip-in (218 kN-m), OK

At midspan; Q_{UD}^+ = 1021 kip-in (115 kN-m) (largest positive moment demand)
Q_{UD}^- = 821 kip-in (93 kN-m) (largest negative moment demand)
M_{CE}^+ = 2797 kip-in (316 kN-m), M_{CE}^- = 2124 kip-in (240 kN-m)

m = 2.0 (from TI 809-04 Table 7-14)
mQ_{CE}^+ = (2.0)(2797 kip-in) = 5594 kip-in (632 kN-m) > 1021 kip-in (115 kN-m), OK
mQ_{CE}^-= (2.0)(2124 kip-in) = 4248 kip-in (480 kN-m) > 821 kip-in (93 kN-m), OK

Longitudinal Beams:

At beam ends; Q_{UD}^+ = No positive moment demands at beam ends
Q_{UD}^- = 128 kip-in (14 kN-m) (largest negative moment demand)
M_{CE}^+ = 324 kip-in (36.6 kN-m), M_{CE}^- = 624 kip-in (70.5 kN-m)
m = 2.0 (from TI 809-04 Table 7-14)
mQ_{CE}^-= (2.0)(624 kip-in) = 1248 kip-in (141 kN-m) > 128 kip-in (14 kN-m), OK

At midspan; *Flexural demands at midspan are negligible; OK by inspection.*

Column Flexure:

A check of a column along grid line C is shown to illustrate the check of component acceptance. The expected flexural strengths of the column in the strong (x) and weak (y) directions are evaluated at the given axial load. See the evaluation section for the column P-M interaction diagram.

Q_{UDx} = 2366 kip-in (267 kN-m)
Q_{UDy} = 58 kip-in (6.6 kN-m)
Axial load = 45 kips (200 kN) compression
Q_{CEx} = 4250 kip-in (480 kN-m) (at given axial load)
Q_{CEy} = 2500 kip-in (283 kN-m) (at given axial load)
m = 2.0 (from TI 809-04 Table 7-15)

$$\left(\frac{Q_{UDx}}{mQ_{CEx}} + \frac{Q_{UDy}}{mQ_{CEy}}\right) = \left(\frac{2366\,\text{kip}-\text{in}}{(2.0)(4250\,\text{kip}-\text{in})} + \frac{58\,\text{kip}-\text{in}}{(2.0)(2500\,\text{kip}-\text{in})}\right) = 0.3 < 1.0\text{, OK}$$

Force-Controlled Components

Force-controlled actions in primary components and elements must satisfy:

$$Q_{CN} \geq Q_{UF} \qquad \text{(Eq. 7-3)}$$

Column Shear:

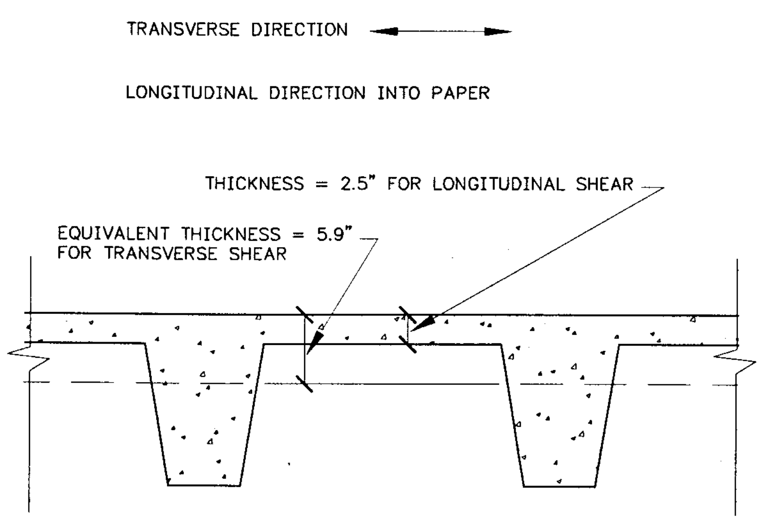
Q_{CN} = 85 kips (378 kN) (Q_{CN} determined in the evaluation section)
Q_{UF} = 36 kips (160 kN), OK
$Q_{CN} > Q_{UF}$, OK

Beam Shear:

(Q_{CN} for the beams determined in the evaluation section)
Transverse beams: Q_{CN} = 60 kips (267 kN) > 36 kips (160 kN),
Longitudinal beams: Q_{CN} = 30 kips (133 kN) > Q_{UF} = 3 kips (13.3 kN), OK

Diaphragm Shear:

The thickness of the diaphragm is different in the longitudinal and transverse directions. In the longitudinal direction, the joist ribs run parallel to the shear forces. The weak link is in between the ribs and is taken equal to the thickness of the slab. The ribs run perpendicular to the transverse direction. The equivalent thickness for transverse shear is taken as a weighted average of the concrete area (see diagram below).



EQUIVALENT THICKNESSES FOR DIAPHRAGM SHEAR

The shear strength of the diaphragm is taken as:

$$V_n = A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right) \qquad \text{(ACI 318 Eq. 21-6)}$$

The slabs are reinforced with #3 bars at 18" in both directions. (The contribution to the shear strength by the longitudinal steel at the bottom of the pan joists is neglected.)

Longitudinal Direction: (Check shown for roof level only)

$\rho_n = 0.11 \text{ in.}^2 / (2.5" \times 18") = 0.0024$

$V_n = (2.5" \text{ x length})\left(2\sqrt{3000\,\text{psi}} + (0.0024)(60\,\text{ksi})\right) = 633\,\text{pli} = 7.6\,\text{klf}\ (111\ \text{kN / m})$

$Q_{CN} = 7.6\ \text{klf}\ (111\ \text{kN / m}) > Q_{UF} = 2.4\ \text{klf}\ (35.0\ \text{kN / m})$, OK

Transverse Direction: (Check shown for roof level only)

$\rho_n = 0.11 \text{ in.}^2 / (5.9" \times 18") = 0.001$

$V_n = (5.9" \text{ x length})\left(2\sqrt{3000\,\text{psi}} + (0.001)(60\,\text{ksi})\right) = 1000\,\text{pli} = 12.0\,\text{klf}\ (175\ \text{kN / m})$

$Q_{CN} = 12.0\ \text{klf}\ (175\ \text{kN / m}) > Q_{UF} = 5.1\ \text{klf}\ (74\ \text{kN / m})$, OK

7. *Prepare construction documents:*

Construction documents are not included for this design example.

8. *Quality assurance / quality control:*

QA / QC is not included for this design example.