D3. One-story Building with Steel Roof Trusses

Building and Site Data.

Building Description.

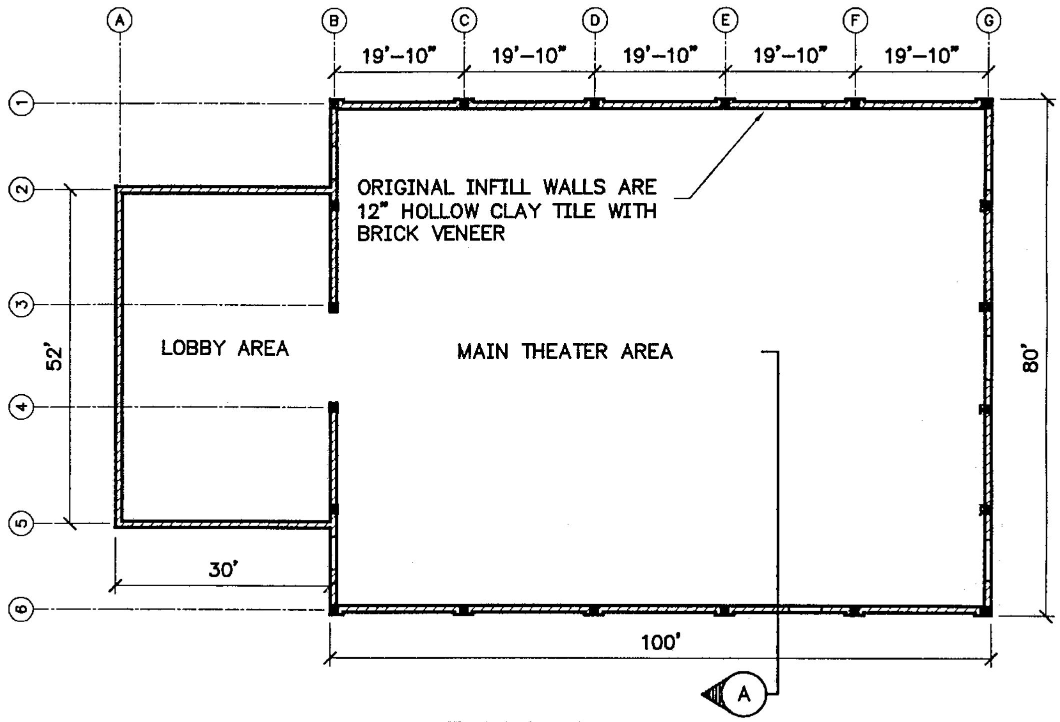
The French Theater is a one-story infilled steel frame building located at Fort Lewis, Washington. The steel frames are infilled with unreinforced hollow clay tile walls. According to the available drawings and information, the building was originally built in 1932. It was apparently enlarged in 1940. The drawings reviewed were prepared for the 1940 modification and generally represent the existing condition of the original building, but do not provide the detailing and reinforcing information of the original building.

The original building had overall plan dimensions of approximately 48' x 120' (14.6 m x 36.6 m) with the main theater section of 48' x 100' (14.6 m x 30.5 m). The modified building has overall plan dimensions of approximately 80' x 130' (24.4 m x 39.7 m) (consisting of an 80' x 100' (24.4 m x 30.5 m) main theater section and a 30' x 52' (9.2 m x 15.9 m) entrance and lobby area). Only the auditorium section of the building is analyzed for this design example.

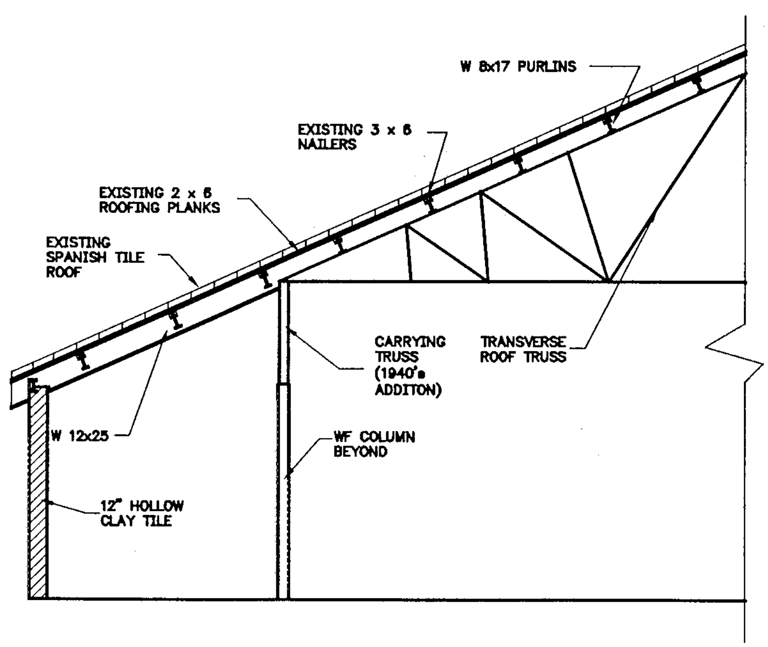
Vertical Load Resisting System. The ground floor is a concrete slab poured on excavated ground to form a sloped surface (reinforcing is unknown). The roof consists of Spanish tile on 2" x 6" roofing planks nailed to 3" x 6" nailers. The 3" x 6" nailers are bolted to a steel purlins which span between transverse trusses. The trusses are supported by steel columns. The steel columns along the exterior are infilled with the hollow clay tile walls. The footings consist of individual spread footings for the columns and strip footings along the perimeter of the structures.

Lateral Load Resisting System. The primary lateral-force resisting system consists of horizontal wood sheathing connected to the top flange of the upper chord of the trusses through 3" x 6" wood nailers and steel joists. The lateral load is resisted by the unreinforced masonry shear walls along the exterior. The lateral load is transferred to the walls through the roof diaphragm with contribution from intermediate collectors and steel framing and X-bracing consisting of angles and rods.

The 1940 Modifications. During the 1940 modification, the auditorium portion was widened and the entrance area was enlarged. The auditorium portion was widened from 49'-4" to 80' and the entrance area was enlarged from 39' x 20'-2" to 52' x 30' (Only the auditorium section of the building is analyzed for this design example.) The transverse framing consists of end shear walls and four interior truss-column frame systems. The columns of the two interior trusses were removed and a carrying truss was installed to transfer the vertical load to the adjacent columns.

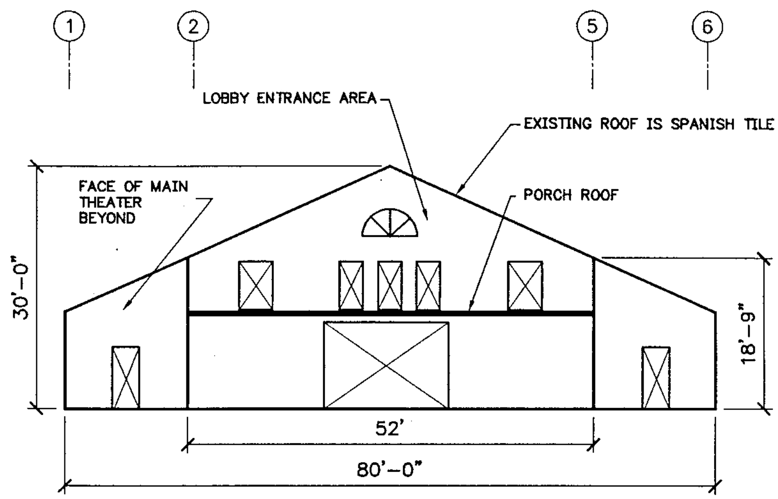


FLOOR PLAN

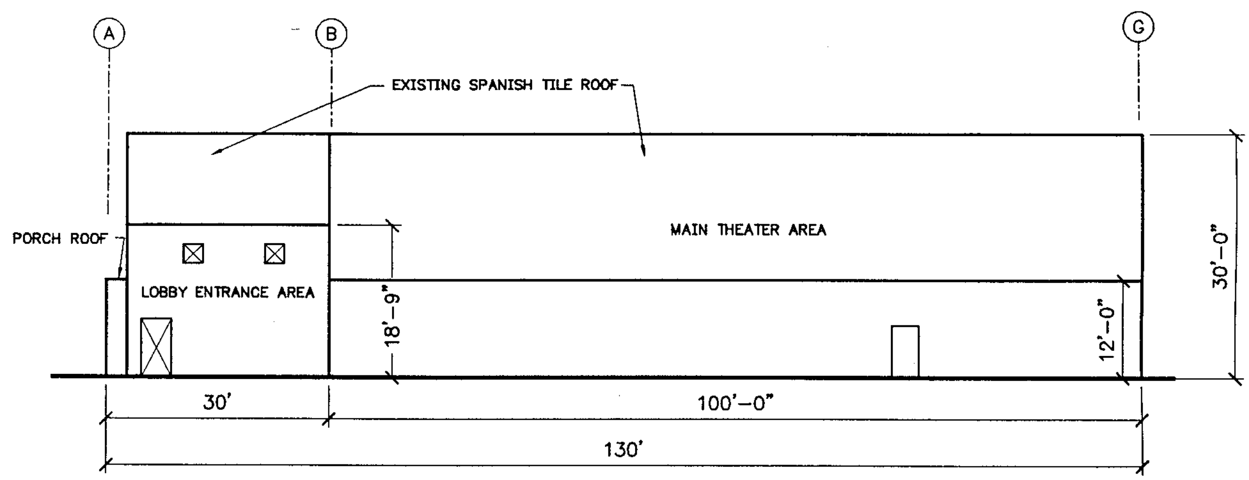


SECTION (A)

1 ft = 0.305 m
1 in = 25.4 mm



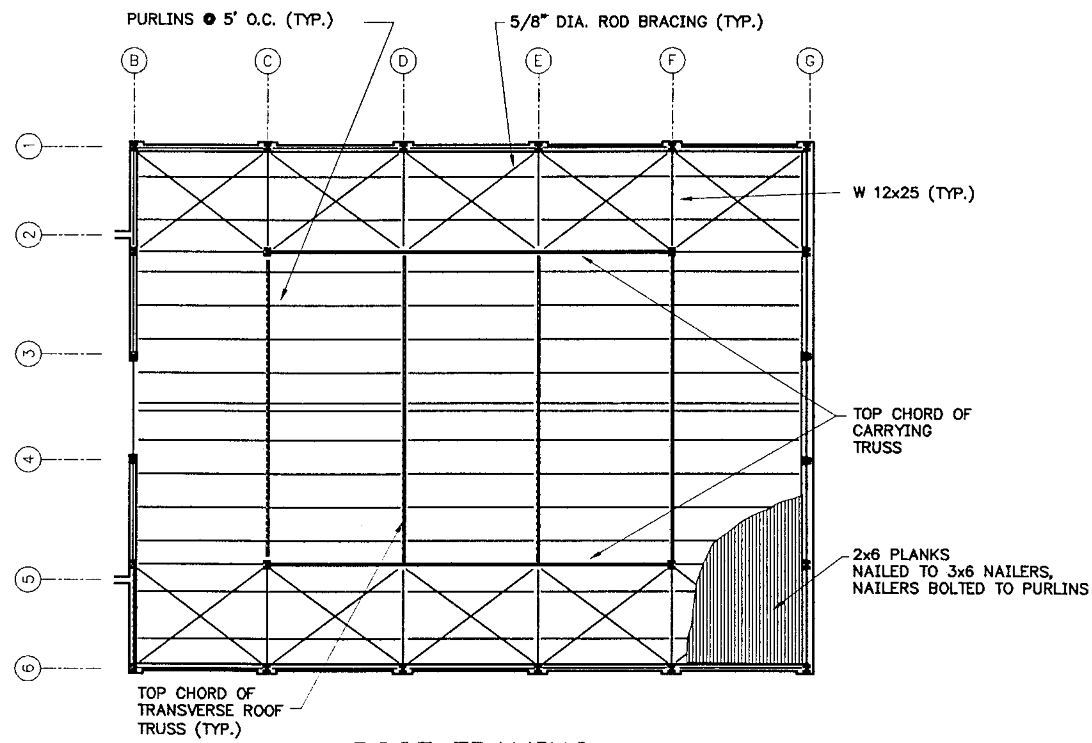
FRONT ELEVATION VIEW OF LOBBY ENTRANCE AREA



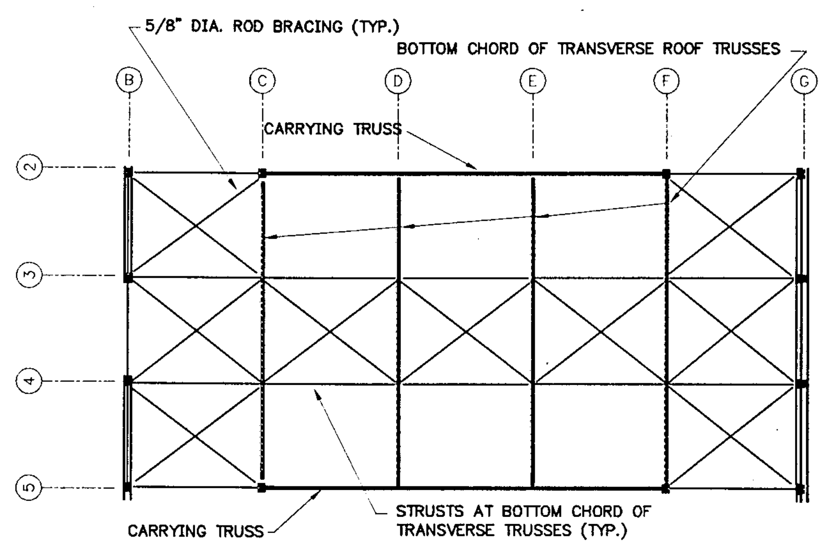
ELEVATION OF WALL LINE 6

1 ft =0.305 m

1 in = 25.4 mm



ROOF FRAMING



HIGH ROOF FRAMING
(AT BOTTOM CHORD OF ROOF TRUSSES)

1 ft = 0.305 m
1 in = 25.4 mm

A. Preliminary Determinations (from Table 2-1)

1. Obtain building and site data:

a. Seismic Use Group. The theater is a Special Occupancy Structure due to its occupancy (covered structures whose primary occupancy is public assembly with a capacity greater than 300 persons). Therefore, from Table 2-2, the building falls into Seismic Use Group II.

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

c. Applicable Ground Motions (Performance Objective). Table 2-4 prescribes a ground motion of 2/3 MCE for the Seismic Use Group II, Safe Egress Performance Level. The derivations of the ground motions are described in Chapter 3 of TI 809-04. 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:

$S_S = 1.20$ g	(MCE Map No. 9)
$S_1 = 0.39$ g	(MCE Map No. 10)

(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.

$F_a = 1.02$	(TI 809-04 Table 3-2a)
$F_v = 1.62$	(TI 809-04 Table 3-2b)

(3) Determine the adjusted MCE spectral response accelerations:

$S_{MS} = F_a S_S = (1.02)(1.20) = 1.224$	(TI 809-04 Eq. 3-1)
$S_{M1} = F_v S_1 = (1.62)(0.39) = 0.632$	(TI 809-04 Eq. 3-2)
$S_{MS} \leq 1.5F_a = (1.5)(1.02) = 1.53 > 1.224$, use 1.224	(TI 809-04 Eq. 3-5)
$S_{M1} \leq 0.6F_v = (0.6)(1.62) = 0.96 > 0.632$, use 0.632	(TI 809-04 Eq. 3-6)

(4) Determine the design spectral response accelerations:

$S_{DS} = 2/3\ S_{MS} = (2/3)(1.224) = 0.82$	(TI 809-04 Eq. 3-3)
$S_{D1} = 2/3\ S_{M1} = (2/3)(0.632) = 0.42$	(TI 809-04 Eq. 3-4)

Enter FEMA 310 Table 2.1 with these values to determine the region of seismicity (this information is needed when completing the FEMA 310 Geologic Site Hazards and Foundations Checklist). It is determined that the site is in a region of high seismicity.

d. Determine seismic design category:

Seismic design category: D (based on S_{DS})	(Table 2-5a)
Seismic design category: D (based on S_{D1})	(Table 2-5b)

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 at the site. Table 4-3 of this document requires that the geologic site hazard and foundation checklist contained in FEMA 310 be completed. See Section C, Structural Screening (Tier 1), for the completed checklist.

FEMA 310 only defines two Performance Levels; Life Safety and Immediate Occupancy. Paragraph 4-2.a of this document states that for evaluations performed in accordance with this document the Immediate Occupancy Performance Level in Table 3-3 of FEMA 310 will be interpreted as representing the Safe Egress Performance Level for Seismic Use Group II structures. This information is needed since some of the statements in the Geologic Hazards Checklist apply only to Immediate Occupancy structures.

3. *Evaluate geologic hazards.* Not necessary.

4. *Mitigate geologic hazards.* Not Necessary.

B. Preliminary Structural Assessment (from Table 4-1)

This building was evaluated as Example Problem H3 in EI 01S103, dated 01 October 1997. It was determined from the evaluation that the structure definitely needs rehabilitation without further evaluation. Components failing the evaluation were:

- The wood-sheathed diaphragms lack stiffness and strength.
- Horizontal roof bracing does not possess adequate strength for diaphragm forces
- The unreinforced masonry shear walls are overstressed and the walls along grid lines B & G exceed the allowable height as URM shear walls.

C. Structural Screening (Tier 1) (from Table 4-2)

This step has already been completed in Example Problem H3 of TI 809-51. It was determined that the building definitely needs rehabilitation.

1. *Determine applicable checklist.* Table 4-3 requires that the Basic Structural, Supplemental Structural, Basics Nonstructural, Supplemental Nonstructural, and Geologic Site Hazard and Foundations Checklists be completed for Seismic Design Category D structures being evaluated by the Tier 1 procedures. It has already been determined that the structure definitely needs rehabilitation, and therefore, no Tier 1 evaluation is completed. However, for every building being evaluated by this document it is required to complete the Geologic Site Hazards Checklist from FEMA 310.

2. *Complete applicable checklist*

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

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). *Geotechnical report states that there is no liquefaction hazard.*
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). *Geotechnical report states that there is no slope failure hazard.*
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). *Geotechnical report states that there is no surface fault rupture hazard.*

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). *No evidence of excessive foundation movement or settlement.*

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). *No evidence of deterioration.*

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). *There are no pole foundations.*

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).
$0.6S_a = (0.6)(0.82) = 0.49$ ($S_a = S_{DS}$) The height of the building ≈ 30 ft.
Transverse: (base/height) = 80 / 30 = 2.67 > 0.49, OK
Longitudinal: (base/height) = 100 / 30 = 3.33 > 0.49, OK

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). *Footings are restrained by slabs.*

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). *No piles or piers used in this structure.*

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). *This building is not located on a sloping site*

3. Evaluate screening results. There are no 'Noncompliant' statements from the Geologic Hazards checklist. The design of the rehabilitation may now be completed.

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

Nonstructural components are not addressed in this design example.

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

Nonstructural components are not addressed in this design example.

F. Structural Evaluation (Tier 2) (from Table 5-1)

The scope of this problem states that seismic evaluation completed in "EXAMPLE PROBLEM H3" in EI 01S103, dated 01 October 1997 is to be used as the starting point for this example. That evaluation found that the building definitely requires rehabilitation. Therefore, no Tier 2 evaluation is necessary.

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

No Tier 3 evaluation is completed for this structure (see statements in step F above).

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

Nonstructural components are not addressed in this design example.

I. Final Assessment (from Table 6-1)

1. *Structural evaluation assessment*

    - *Quantitative:* Deficiencies in the structural components have been identified and quantified (see the evaluation results completed for Step F above).

    - *Qualitative:* The building is a serious life safety hazard and rehabilitation is feasible. The structure contains adequate load paths, however, the structural systems require strengthening.

2. *Structural rehabilitation strategy: Since the seismic hazard evaluation was completed previously, the structural rehabilitation strategy and structural rehabilitation concept steps in Table 6-1 will not be completed here. These issues will be addressed in Paragraph K, Rehabilitation, below.*

3. *Structural rehabilitation concept: (See statement above)*

4. *Nonstructural evaluation assessment:*

    Nonstructural components are not considered for this example.

5. *Nonstructural rehabilitation strategy:*

    Nonstructural components are not considered for this example.

6. *Nonstructural rehabilitation concept:*

    Nonstructural components are not considered for this example.

J. Evaluation Report (from Table 6-2)

At this point, an evaluation report would be compiled to summarize the results of the evaluation of structural systems and nonstructural components. An evaluation report is not shown for this design example; however, the items to be included in the report are:

1. *Executive summary*

2. *Descriptive narrative*

- Building and site data
- Geologic hazards
- Structural evaluations
- Nonstructural evaluations

3. *Appendices*

- Prior evaluations
- Available drawings and other construction documents
- Geotechnical report
- Structural evaluation data
- Nonstructural evaluation data

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)*

The existing wood plank diaphragm lacks the strength to resist the lateral forces. A quick calculation shows that a plywood overlay would not add enough strength capacity to resist the lateral forces.

Check of diaphragm shear in transverse direction: (Weights taken from Example Problem H3 in EI 01S103)

Weight of existing roof = 320 kips (1423 kN)
Weight of longitudinal walls = 60 kips (267 kN)
Total weight = 380 kips (1690 kN)

Approximate lateral force to be resisted by diaphragm ≈ S_aW = 0.82(380 kips) = 312 kips (1388 kN)
Shear to each transverse wall = ½(312 kips) = 156 kips (694 kN)
Diaphragm shear = 156 kips / 80 ft ≈ 2 klf (29 kN / m)

FEMA 273 Section 8.5.8.2 states that the yield capacity for wood panel overlays over straight wood sheathing is approximately 450 plf and that the yield capacity is approximately 65% of the ultimate strength of the diaphragm. Therefore, the ultimate capacity ≈ 450 plf / 0.65 = 692 plf (10.1 kN / m). The ultimate capacity (692 plf) is much less than the required capacity (2 klf); therefore, a wood panel overlay will not work and the roof will need to be replaced with a stronger material.

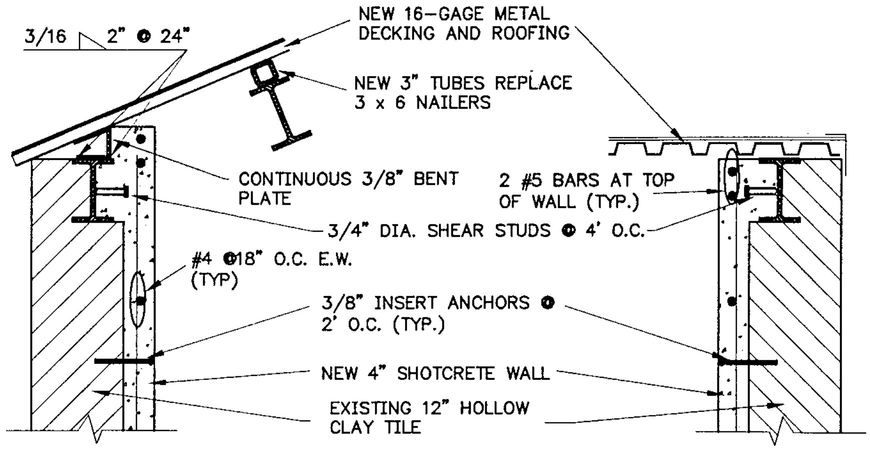
4. *Rehabilitation concept selection*

The existing roof will be replaced with a much stronger metal deck diaphragm and the existing heavy Spanish tiles will be replaced by lighter roofing materials. The original 3 x 6 nailers that the wood decking was connected to will be replaced with 3" steel tubing. The new deck is to welded to the tubing, and the tubing is to be welded to the existing WF purlins.

The unreinforced masonry walls will be strengthened by adding a 4" layer of reinforced shotcrete to the walls. The shotcrete is to be reinforced with #4 bars at 18" in both the vertical and horizontal directions and anchored to the existing walls with insert anchors spaced at 24" each way.

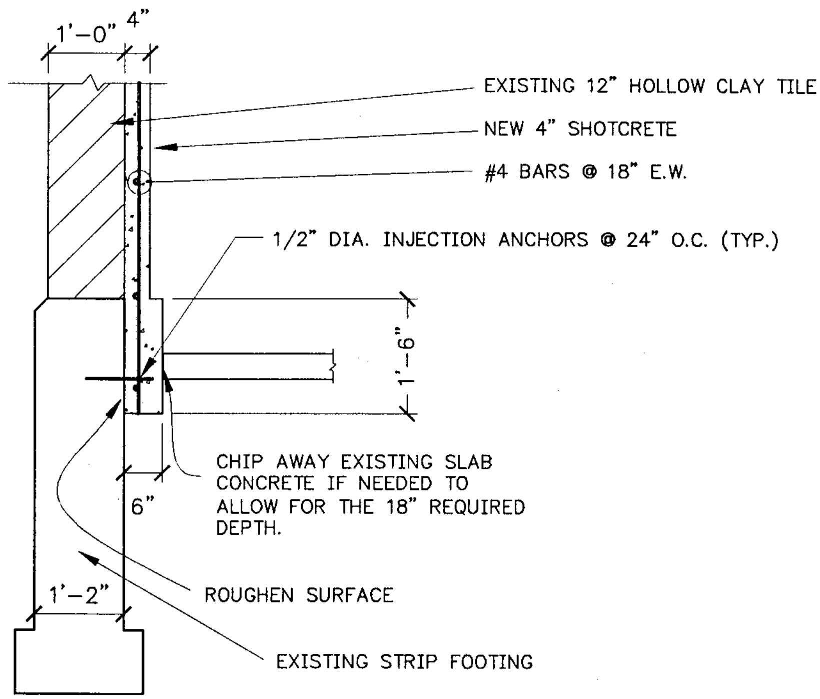
5. *Rehabilitation design*

(See figures below for details of rehabilitation)



TYPICAL SECTION THROUGH REHABILITATED LONGITUDINAL WALL

TYPICAL SECTION THROUGH REHABILITATED TRANSVERSE WALL



TYPICAL SECTION THROUGH REHABILITATED FOOTING

1 ft = 0.305 m
1 in = 25.4 mm

6. Confirming evaluation

a. Analytical procedures: The structure is analyzed with the Linear Static Procedure in accordance with Section 3.3.1 of FEMA 273. Limitations on the use of the procedure are addressed by paragraph 5-2b of TI 809-04 and Section 2.9 of FEMA 273. The design of the shotcrete addition to the infill panels is based on a pseudo lateral force per FEMA 273.

Analysis of Structure using the Linear Static Procedure (LSP) (per Section 3.3.1 of FEMA 273)

In the LSP, the building is modeled with linearly elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. For this structure, 5% viscous damping is assumed. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral force defined by FEMA 273 Equation 3-6.

- *Determine pseudo lateral load (per FEMA 273 Section 3.3.1.3)*

$$V = C_1C_2C_3S_aW \qquad \text{(FEMA 273 Eq. 3-6)}$$

Determine seismic weight, W: (per FEMA 273 Sec. 3.3.1.3 A.)

The structure does not have partitions in the main theater area. Therefore, the requirement of using a minimum 10 psf partition is used for determination of the lobby seismic weights only.

Roof

Tar and Gravel Roofing	6.0 psf	
2" Rigid Insulation	3.0 psf	
16 Gage Metal Decking	3.5 psf	
Steel Framing	6.0 psf	
Hung Ceiling, Mech. & Elec.	10.0 psf	
Total =	**28.5 psf**	*1365 Pa*

Exterior Strengthened Walls

12" Hollow Clay Tile w/ Brick Veneer	60.0 psf	
4" New Shotcrete	40.0 psf	
Total =	**100.0 psf**	*4788 Pa*

Lobby Walls 2A-2B and 5A-5B

12" Hollow Clay Tile w/ Brick Veneer	60.0 psf	
Total =	**60.0 psf**	*2873 Pa*

Live Load

Roof Live Load	20 psf	
Total =	**20.0 psf**	*958 Pa*

Component	Length (ft.)	Width or Tributary Height (ft.)	% Solid	Total Area (ft.^{2})	Unit Weight (psf)	Total Weight for Longit. Forces (kips)	Total Weight for Trans. Forces (kips)
Roof Level							
Main theater area between gridlines B & G							
Roofing and Framing	100.0	80	100.0%	8000	28.5	228.0	228.0
Entrance area between gridlines A & B (for loads tributary to wall line B)[2]							
Roofing and Framing	52.0	15	100.0%	780	28.5	0.0	22.2
10 psf Partition loads	52.0	15	100.0%	780	10.0	0.0	7.8
Walls							
Walls of the theater section							
Wall 1B-1G	100.0	6	95.0%	570	100.0	57.0	57.0
Wall 6B-6G	100.0	6	95.0%	570	100.0	57.0	57.0
Wall B1-B6[1]	---	---	---	543	100.0	54.3	54.3
Wall G1-G6[1]	---	---	---	680	100.0	68.0	68.0
Walls of the lobby area[2]							
Wall 5A-5B (only 1/2 length)	15	9.45	95.0%	135	60.0	0.0	8.1
Wall 2A-2B (only 1/2 length)	15	9.45	95.0%	135	60.0	0.0	8.1
					Total Wt. =	**464 kips** (2064 kN)	**511 kips** (2273 kN)

Notes:
1. The area of the transverse walls was determined graphically from the wall elevations.
2. The weight from the lobby area tributary to wall line B includes the lobby wall and roof areas for seismic forces in the transverse direction. No mass from the lobby area is used in the longitudinal direction since the shear forces tributary to the lobby area are resisted directly by the lobby longitudinal shear walls.

Determine Building Period:

The fundamental period of a one-story building with a single span flexible diaphragm is calculated using Method 3 described in FEMA 273 Section 3.3.1.2. (Note: The period calculated using Method 3 will be compared to the period calculated using Method 3 at the end of this section.)

$$T = \sqrt{0.1\Delta_w + 0.078\Delta_d} \qquad \text{(Method 3 Period)}$$

where Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm.

Determine Δ_d, the diaphragm deflection:

The diaphragm deflection consists of two parts; the flexural deflection and the shear deflection. The flexural deflection is determined in the same manner as that of a simply supported beam. The shotcrete walls are assumed to act as the flanges. The effective depth of the shotcrete wall assumed to act as a flange is estimated to be six times the shotcrete thickness = 6 * 4" = 24" (The assumption that the effective flanges are equal to six times the wall thickness is taken from paragraph 4-2.B of the "Reinforced Masonry Engineering Handbook", Fifth Edition by James Amrhein.) The area of each of the flanges is thus taken as = (24")(4") = 96 in^2 (619 cm^2). The diaphragm shear deflections are estimated using the flexibility factors contained in the deck manufacture's catalog.

Flexural Deflection, Δ_{flex};
The diaphragm is modeled as a simply supported beam subject to uniform transverse loading, w.

$$\Delta_{flex} = \frac{5wL^4}{384EI} \text{ at midspan}$$

f_c' = Compressive strength of shotcrete = 3000 psi

$$E = w_c^{1.5} 33\sqrt{f_c'} = (120\,\text{pcf})^{1.5} 33\sqrt{3000\,\text{psi}} = 2376\,\text{ksi} \quad (16371\text{ MPa})$$ (ACI 318 Section 8.5.1)

$I = 2A_{flange}(\text{depth of diaphragm} / 2)^2$

Determine weight tributary to the diaphragm in each direction.

Transverse Direction:

		Length (ft.)	Width or Tributary Height (ft.)	% Solid	Total Area (ft.^{2})	Unit Weight (psf)	Total Weight (kips)
Roof Level							
Main theater area between gridlines B & G							
Roofing and Framing		100.0	80	100.0%	8000	28.5	228.0
Walls							
Walls of the theater section							
Wall 1B-1G		100.0	6	95.0%	570	100.0	57.0
Wall 6B-6G		100.0	6	95.0%	570	100.0	57.0
						Total Wt. =	**342 kips** *(1521 kN)*

Longitudinal Direction:

		Length (ft.)	Width or Tributary Height (ft.)	% Solid	Total Area (ft.^{2})	Unit Weight (psf)	Total Weight (kips)
Roof Level							
Main theater area between gridlines 1 & 6							
Roofing and Framing		80.0	100.0	100.0%	8000	28.5	228.0
Walls							
Walls of the theater section							
Wall B1-B6[1]		---	---	---	543	100.0	54.3
Wall G1-G6[1]		---	---	---	680	100.0	68.0
						Total Wt. =	**350 kips** (1557 kN)

w_{trans} = (342 kips) / (100 ft) = 3420 plf
w_{long} = (350 kips) / (80 ft) = 4375 plf

$I_{trans} = 2A(d/2)^2 = 2(96\text{ in.}^2)(80\text{ ft} / 2)^2 = 44236800\text{ in.}^4$
$I_{long} = 2A(d/2)^2 = 2(96\text{ in.}^2)(100\text{ ft} / 2)^2 = 69120000\text{ in.}^4$

$$\Delta_{\text{flex trans}} = \frac{5wL^4}{384EI} = \frac{5(3420\text{ plf})(1'/12")(100\text{ ft} * 12"/')^4}{384(2376000\text{ psi})(44236800\text{ in}^4)} = 0.07\text{ in (1.8 mm)}$$

$$\Delta_{\text{flex long}} = \frac{5wL^4}{384EI} = \frac{5(4375\text{ plf})(1'/12")(80\text{ ft} * 12"/')^4}{384(2376000\text{ psi})(69120000\text{ in}^4)} = 0.02\text{in (0.5 mm)}$$

Determine shear deflection, Δ_{shear}

$$\Delta_{\text{shear}} = \frac{q_{ave} L_1 F}{10^6} \qquad \text{(TI 809-04 Eq. 7-6)}$$

L_1 = Distance in feet between vertical resisting element (shear wall) and the point to which the deflection is to be determined (diaphragm midspan).
$L_{1\text{ trans}}$ = (100 ft / 2) = 50 ft
$L_{1\text{ long}}$ = (80 ft / 2) = 40 ft

q_{ave} = Average shear in diaphragm in pounds per foot over length L_1
The diaphragm shear force is resisted evenly between the walls on each end of the building in both the longitudinal and transverse directions.

Transverse:
$V_{\text{walls B \& G}}$ = 342 kips / 2 = 171 kips (761 kN)
v_{max} = V / diaphragm depth = 171 kips / 80 ft = 2125 plf
$q_{ave} = v_{max}$ / 2 = 1063 plf

Longitudinal:
$V_{\text{walls 1 \& 6}}$ = 350 kips / 2 = 175 kips (778 kN)
v_{max} = V / diaphragm depth = 175 kips / 100 ft = 1750 plf
$q_{ave} = v_{max}$ / 2 = 875 plf

F = Flexibility factor: The average microinches a diaphragm web will deflect in a span of 1 foot under a shear of 1 pound per foot (This value is taken from manufacture's catalog.)

F = 6.73 μin / ft / plf (from deck manufacture's catalog)

$$\Delta_{\text{shear trans}} = \frac{(1063\text{plf})(50\text{ft})(6.73\mu\text{in / ft / plf})}{10^6} = 0.36\text{in (9.1 mm)}$$

$$\Delta_{\text{shear long}} = \frac{(875\text{plf})(40\text{ft})(6.73\mu\text{in / ft / plf})}{10^6} = 0.24\text{in (6.1 mm)}$$

Total diaphragm deflections;

$\Delta_{\text{total trans}} = \Delta_{flex} + \Delta_{shear}$ = 0.07 in. + 0.36 in. = 0.43 in.
$\Delta_{\text{total long}} = \Delta_{flex} + \Delta_{shear}$ = 0.02 in. + 0.24 in. = 0.26 in.

Determine wall deflections from tributary loads;

The walls each resist ½ of the shear force tributary to the main theater diaphragm and their self-inertial forces.

Shears to walls;
Wall B: Shear from diaphragm = 171 kips, Self-weight = 54.3 kips, V_B = (171 k) + (54.3 k) = 225 kips
Wall G: Shear from diaphragm = 171 kips, Self-weight = 68.0 kips, V_G = (171 k) + (68.0 k) = 239 kips
Wall 1: Shear from diaphragm = 175 kips, Self-weight = 57.0 kips, V_1 = (175 k) + (57.0 k) = 232 kips
Wall 6: Shear from diaphragm = 175 kips, Self-weight = 57.0 kips, V_6 = (175 k) + (57.0 k) = 232 kips

Determine wall rigidities;

The rigidity of the shear walls is made up of contributions from both the original 12" hollow clay tiles and the new 4" shotcrete. Since no testing information is available, the mechanical properties of the hollow clay tile are taken as the default values from FEMA 273 Section 7.3.2. The default values for masonry in good condition are:

- Compressive strength, f'_m (FEMA 273 Sec. 7.3.2.1)
  f'_m = 900 psi
  f'_{me} = 1.25(900 psi) = 1125 psi (7.75 MPa)
  (expected strength = nominal strength x 1.25 per Section 7-2.f.(5)(d)1.i)

- Modulus of elasticity, E_m (FEMA 273 Sec. 7.3.2.2)
  $E_m = 550\, f'_{me}$ = 550(1125 psi) = 6.19×10^5 psi (4265 MPa)

- Shear modulus, G_{me} (FEMA 273 Sec. 7.3.2.5)
  $G_{me} = 0.40E_m$ = 0.40(6.19×10^5psi) = 2.48×10^5 psi (1709 MPa)

- Tensile strength (FEMA 273 Sec. 7.3.2.3)
  f'_{mt} = 20 psi
  f'_{mte} = 1.25(20 psi) = 25 psi (172 kPa)

- Shear strength (FEMA 273 Sec. 7.3.2.4)
  v_m = 27 psi
  v_{me} = 1.25(27 psi) = 33.75 psi (233 kPa)

- Equivalent solid thickness for 12" ungrouted hollow clay tile = 5.5" (140 mm)

The mechanical properties of the concrete are:

- Compressive strength of the new shotcrete, f'_c = 3000 psi (20.7 Mpa)

- Modulus of elasticity, E_c (ACI 318 Sec. 8.5.1)
  E_c = 2376000 psi (16371 MPa)

- Shear modulus, G_{me} (FEMA 273 Table 6-4)
  $G_{me} = 0.40E_c$ = 0.40(2.376×10^6psi) = 9.50×10^5 psi (6548 MPa)

The deflections of a cantilever and fixed-fixed wall pier element are determined from the equations:

$$\Delta_c = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG}$$, cantilever pier deflection

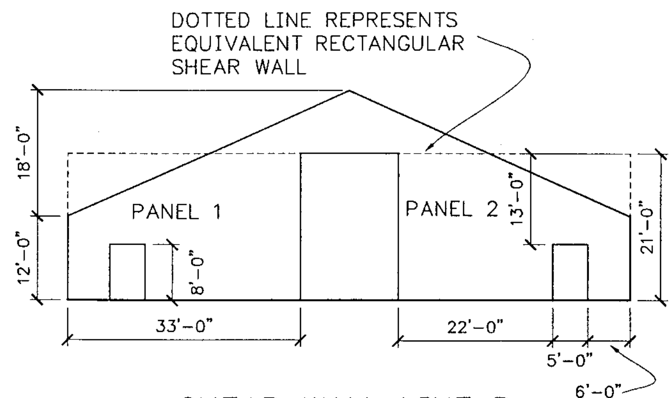
$$\Delta_f = \frac{Ph^3}{12EI} + \frac{1.2Ph}{AG}$$, fixed-fixed pier deflection

Cracked section properties are used to determine the wall rigidities. The flexural rigidity of the piers is estimated to be 0.5I (FEMA 273 Table 6-4).

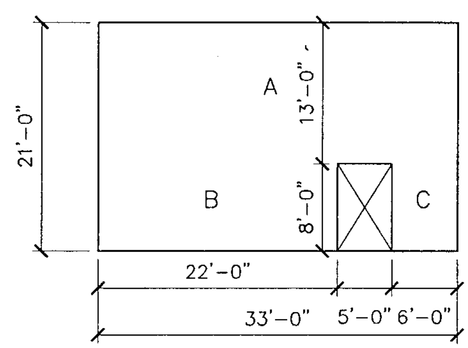
The total rigidity of the walls is equal to the sum of the masonry and concrete contributions.

Transverse walls: The rigidity of the transverse walls is estimated by assuming that the triangular portion is replaced with an equivalent rectangular section.

- **Wall B**: Wall line B is assumed to act as two separate cantilever panel sections. The stiffness of each of the two panels is added to determine the total wall stiffness.



SHEAR WALL LINE B



PANEL 2 SHEAR WALL LINE B

Masonry contribution:

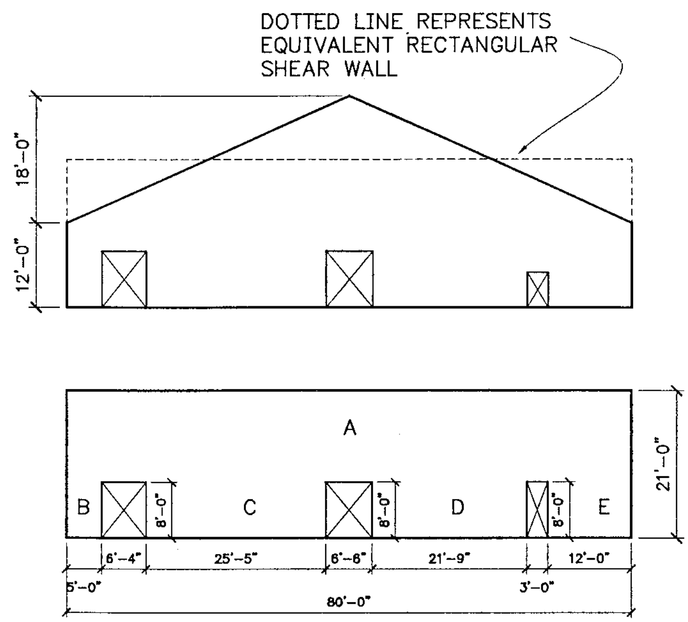
Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(21 \cdot ft, 33 \cdot ft, 5.5 \cdot in)$	$\Delta_{solid} = 0.001 \, in$
Subract Bottom Strip	$\Delta_{strip} := \Delta_c(8 \cdot ft, 33 \cdot ft, 5.5 \cdot in)$	$\Delta_{strip} = 0 \, in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0.001 \, in$
Add Back in Piers B & C	$\Delta_B := \Delta_f(8 \cdot ft, 22 \cdot ft, 5.5 \cdot in)$	$\Delta_B = 0 \, in$
	$\Delta_C := \Delta_f(8 \cdot ft, 6 \cdot ft, 5.5 \cdot in)$	$\Delta_C = 0.003 \, in$
	$R_{BC} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C}$	$R_{BC} = 3256.15 \, \frac{1}{in}$
	$\Delta_{BC} := \frac{1}{R_{BC}}$	$\Delta_{BC} = 0 \, in$
	$\Delta_{panel\_2} := \Delta_A + \Delta_{BC}$	$\Delta_{panel\_2} = 0.001 \, in$
Total rigidity = 2 R	$R_{panel\_2} := \frac{1 \cdot kip}{\Delta_{panel\_2}}$	$R_{panel\_2} = 815.201 \, \frac{kip}{in}$
	$R_{wall} := 2 \cdot R_{panel\_2}$	$R_{wall} = 1630.401 \, \frac{kip}{in}$

Concrete contribution:

Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(21 \cdot ft, 33 \cdot ft, 4 \cdot in)$	$\Delta_{solid} = 0 \, in$
Subract Bottom Strip	$\Delta_{strip} := \Delta_c(8 \cdot ft, 33 \cdot ft, 4 \cdot in)$	$\Delta_{strip} = 0 \, in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0 \, in$
Add Back in Piers B & C	$\Delta_B := \Delta_f(8 \cdot ft, 22 \cdot ft, 4 \cdot in)$	$\Delta_B = 0 \, in$
	$\Delta_C := \Delta_f(8 \cdot ft, 6 \cdot ft, 4 \cdot in)$	$\Delta_C = 0.001 \, in$
	$R_{BC} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C}$	$R_{BC} = 9093.54 \, \frac{1}{in}$
	$\Delta_{BC} := \frac{1}{R_{BC}}$	$\Delta_{BC} = 0 \, in$
	$\Delta_{panel\_2} := \Delta_A + \Delta_{BC}$	$\Delta_{panel\_2} = 0 \, in$
Total rigidity = 2 R	$R_{panel\_2} := \frac{1 \cdot kip}{\Delta_{panel\_2}}$	$R_{panel\_2} = 2276.633 \, \frac{kip}{in}$
	$R_{wall} := 2 \cdot R_{panel\_2}$	$R_{wall} = 4553.266 \, \frac{kip}{in}$

Total wall rigidity = masonry + concrete = 1630 k / in. + 4553 k / in = 6183 kips / in (10826 kN / cm)

- **Wall G**



SHEAR WALL LINE G

Masonry contribution:

Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(21\cdot ft, 80\cdot ft, 5.5\cdot in)$	$\Delta_{solid} = 0 \circ in$
Subract Bottom Strip BCDE	$\Delta_{strip} := \Delta_c(8\cdot ft, 80\cdot ft, 5.5\cdot in)$	$\Delta_{strip} = 0 \circ in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0 \circ in$
Add Back in Piers BCDE	$\Delta_B := \Delta_f(8\cdot ft, 6\cdot ft, 5.5\cdot in)$	$\Delta_B = 0.003 \circ in$
	$\Delta_C := \Delta_f(8\cdot ft, 25.5\cdot ft, 5.5\cdot in)$	$\Delta_C = 0 \circ in$
	$\Delta_D := \Delta_f(8\cdot ft, 21.75\cdot ft, 5.5\cdot in)$	$\Delta_D = 0 \circ in$
	$\Delta_E := \Delta_f(8\cdot ft, 12\cdot ft, 5.5\cdot in)$	$\Delta_E = 0.001 \circ in$
	$R_{BCDE} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C} + \frac{1}{\Delta_D} + \frac{1}{\Delta_E}$	$R_{BCDE} = 7924.083 \circ \frac{1}{in}$
	$\Delta_{BCDE} := \frac{1}{R_{BCDE}}$	$\Delta_{BCDE} = 0 \circ in$
	$\Delta_{wall} := \Delta_A + \Delta_{BCDE}$	$\Delta_{wall} = 0 \circ in$
	$R_{wall} := \frac{1\cdot kip}{\Delta_{wall}}$	$R_{wall} = 3229.785 \circ \frac{kip}{in}$

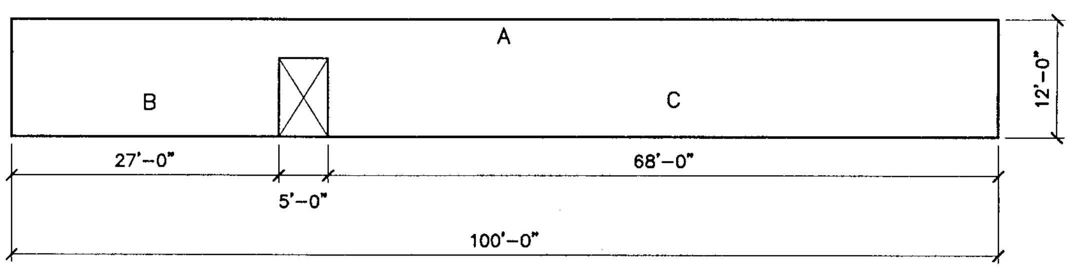
Concrete contribution:

Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(21\cdot ft, 80\cdot ft, 4\cdot in)$	$\Delta_{solid} = 0 \circ in$
Subract Bottom Strip BCDE	$\Delta_{strip} := \Delta_c(8\cdot ft, 80\cdot ft, 4\cdot in)$	$\Delta_{strip} = 0 \circ in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0 \circ in$
Add Back in Piers BCDE	$\Delta_B := \Delta_f(8\cdot ft, 6\cdot ft, 4\cdot in)$	$\Delta_B = 0.001 \circ in$
	$\Delta_C := \Delta_f(8\cdot ft, 25.5\cdot ft, 4\cdot in)$	$\Delta_C = 0 \circ in$
	$\Delta_D := \Delta_f(8\cdot ft, 21.75\cdot ft, 4\cdot in)$	$\Delta_D = 0 \circ in$
	$\Delta_E := \Delta_f(8\cdot ft, 12\cdot ft, 4\cdot in)$	$\Delta_E = 0 \circ in$
	$R_{BCDE} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C} + \frac{1}{\Delta_D} + \frac{1}{\Delta_E}$	$R_{BCDE} = 22129.802 \circ \frac{1}{in}$
	$\Delta_{BCDE} := \frac{1}{R_{BCDE}}$	$\Delta_{BCDE} = 0 \circ in$
	$\Delta_{wall} := \Delta_A + \Delta_{BCDE}$	$\Delta_{wall} = 0 \circ in$
	$R_{wall} := \frac{1\cdot kip}{\Delta_{wall}}$	$R_{wall} = 9019.908 \circ \frac{kip}{in}$

Total wall rigidity = masonry + concrete = 3230 k / in + 9020 k / in = 12250 kips / in (21450 kN / cm)

Longitudinal Walls

- **Walls 1 & 6** (These walls have identical elevations, and therefore, the same rigidities)



SHEAR WALL LINES 1 & 6

Masonry contribution:

Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(12 \cdot ft, 100 \cdot ft, 5.5 \cdot in)$	$\Delta_{solid} = 0 \circ in$
Subract Bottom Strip	$\Delta_{strip} := \Delta_c(8 \cdot ft, 100 \cdot ft, 5.5 \cdot in)$	$\Delta_{strip} = 0 \circ in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0 \circ in$
Add Back in Piers B & C	$\Delta_B := \Delta_f(8 \cdot ft, 27 \cdot ft, 5.5 \cdot in)$	$\Delta_B = 0 \circ in$
	$\Delta_C := \Delta_f(8 \cdot ft, 68 \cdot ft, 5.5 \cdot in)$	$\Delta_C = 0 \circ in$
	$R_{BC} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C}$	$R_{BC} = 13170.861 \circ \frac{1}{in}$
	$\Delta_{BC} := \frac{1}{R_{BC}}$	$\Delta_{BC} = 0 \circ in$
	$\Delta_{wall} := \Delta_A + \Delta_{BC}$	$\Delta_{wall} = 0 \circ in$
	$R_{wall} := \frac{1 \cdot kip}{\Delta_{wall}}$	$R_{wall} = 8768.435 \circ \frac{kip}{in}$

Concrete contribution:

Deflection of Solid Wall	$\Delta_{solid} := \Delta_c(12 \cdot ft, 100 \cdot ft, 4 \cdot in)$	$\Delta_{solid} = 0 \circ in$
Subract Bottom Strip	$\Delta_{strip} := \Delta_c(8 \cdot ft, 100 \cdot ft, 4 \cdot in)$	$\Delta_{strip} = 0 \circ in$
	$\Delta_A := \Delta_{solid} - \Delta_{strip}$	$\Delta_A = 0 \circ in$
Add Back in Piers B & C	$\Delta_B := \Delta_f(8 \cdot ft, 27 \cdot ft, 4 \cdot in)$	$\Delta_B = 0 \circ in$
	$\Delta_C := \Delta_f(8 \cdot ft, 68 \cdot ft, 4 \cdot in)$	$\Delta_C = 0 \circ in$
	$R_{BC} := \frac{1}{\Delta_B} + \frac{1}{\Delta_C}$	$R_{BC} = 36782.624 \circ \frac{1}{in}$
	$\Delta_{BC} := \frac{1}{R_{BC}}$	$\Delta_{BC} = 0 \circ in$
	$\Delta_{wall} := \Delta_A + \Delta_{BC}$	$\Delta_{wall} = 0 \circ in$
	$R_{wall} := \frac{1 \cdot kip}{\Delta_{wall}}$	$R_{wall} = 24487.848 \circ \frac{kip}{in}$

Total wall rigidity = masonry + concrete = 8768 k / in. + 24488 k / in = 33256 kips / in (58231 kN / cm)

The wall deflections due to the lateral loads are:

Wall Line	Shear to Wall (kips)	Wall Rigidity (kips / in)	Wall Deflection (in)	Wall Deflection (mm)
B	225	6183	0.036	0.924
G	239	12250	0.020	0.496
1	232	33256	0.007	0.177
6	232	33256	0.007	0.177

The in-plane diaphragm and wall deflections are used to determine the building period from the equation:
$T = \sqrt{0.1\Delta_w + 0.078\Delta_d}$

Transverse Direction: The walls in the transverse direction, B and G, have different rigidities. The shorter the period of a structure, the higher the shear forces. Therefore, the smaller wall deflection is used (Wall line G) to produce a shorter building period.

$$T = \sqrt{0.1(0.020") + 0.078(0.43")} = 0.19\,\text{sec}$$

Longitudinal Direction: The longitudinal walls have the same rigidity, and therefore, produce the same building period.

$$T = \sqrt{0.1(0.007") + (0.078)(0.26")} = 0.14\,\text{sec}$$

Compare with period using Method 1 with $C_t = 0.020$ and h = 30', T = 0.26 sec > 0.19 and 0.14 sec
The periods calculated using Method 3 are shorter than the period calculated using Method 1. A shorter building period produces higher pseudo lateral forces due to the higher C1 coefficient calculated below. Therefore, the periods calculated using Method 3 are used.

Determination of C_1 factor:

$C_1 = 1.5$ for $T < 0.10$ second, $C_1 = 1.0$ for $T \geq T_0$. Linear interpolation is used for intermediate values of T.

$C_1 = 1.5$ for $T < 0.10$ seconds
$C_1 = 1.0$ for $T \geq T_0$ seconds

The building period, T, and the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum, T_0, are needed to calculate C_1 (see FEMA 273 Section 2.615 for discussion of T_0).

Determination of T_0 (per FEMA 273 Section 2.6.1.5)

$T_0 = (S_{X1}B_S) / (S_{XS}B_1)$ (FEMA 273 Eq. 2-10)

For determination of T_0, use S_{D1} (= 0.42) and S_{DS} (= 0.82) determined for the building evaluation for S_{X1} and S_{XS}, respectively.

From FEMA 273 Table 2-15, B_S and $B_1 = 1.0$ for 5% damping

$T_0 = (0.42 \times 1.0) / (0.82 \times 1.0) = 0.51$ seconds

Transverse Direction: Linearly interpolate to obtain $C_1 = 1.5 + \frac{(0.19 - 0.10)}{(0.51 - 0.10)}(1.0 - 1.5) = 1.39$

Longitudinal Direction: Linearly interpolate to obtain $C_1 = 1.5 + \frac{(0.14 - 0.10)}{(0.51 - 0.10)}(1.0 - 1.5) = 1.45$

Determination of C_2 factor: (from FEMA 273 Table 3-1)

Footnote 1 of FEMA 273 Table 3-1 states that structures in which more than 30% of the story shear at any level is resisted by elements whose strength and stiffness may deteriorate during the design earthquake be classified as framing type 1. This structure resists loads through a combination of the original unreinforced hollow clay tile and the new shotcrete. Both of these materials are subject to strength and stiffness degradation, and are therefore classified as framing type 1.

Linearly interpolate for the Safe Egress Performance Level between the Life Safety and Immediate Occupancy Performance Levels to obtain:

Transverse: $C_2 = 1.16$
Longitudinal: $C_2 = 1.18$

Determination of C_3 factor:

The C_3 coefficient is a modification factor to represent increased displacements due to dynamic P-Δ effects. The structure being evaluated has squat shear walls, which produce very small deflections. Therefore, P-Δ effects are neglected for this example.

Transverse: $C_3 = 1.0$
Longitudinal: $C_3 = 1.0$

Determination of spectral acceleration, S_a:

For periods less than T_0, $S_a = S_{DS}$ (TI 809-04 Eq. 3-13)
$S_a = 0.82$ for both longitudinal and transverse directions.
Determine pseudo lateral forces:

$V_{trans} = (1.39)(1.16)(1.0)(0.82)(511\text{ k}) = 676$ kips (3007 kN)
$V_{long} = (1.46)(1.18)(1.0)(0.82)(464\text{ k}) = 655$ kips (2913 kN)

- *Mathematical Modeling Assumptions (per FEMA 273 Section 3.2.2):*

– The metal deck diaphragm is modeled as a flexible diaphragm relative to the stiff shear walls. The seismic masses are assigned to shear walls based on tributary area.

– Horizontal torsion: Torsion is neglected since the building has a flexible diaphragm.

– The new 4" shotcrete walls are assumed to resist all of the shear forces. The existing 12" hollow clay tile walls are assumed to act as anchored veneer that resists no loads, either in plane or out-of-plane.

- P-Δ effects are neglected due to the squatnness of the walls.
- The new shotcrete walls are assumed to have no gravity loading other than their self-weight.

Determine seismic effects on building components, Q_E

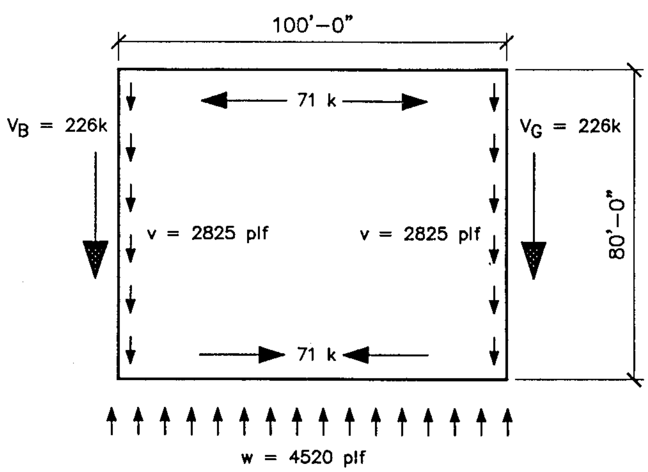
Diaphragm forces:

Total shear force to the main theater diaphragm = $C_1C_2C_3S_aW$

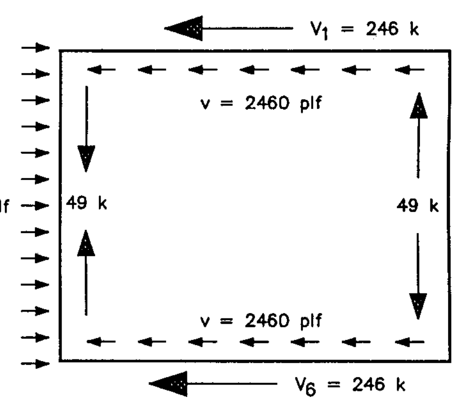
Transverse direction:
Weight tributary to the main theater diaphragm: W_{trans} = 342 kips
Total shear force acting on the diaphragm: F_d = (1.39)(1.16)(1.0)(0.82)(342 kips) = 452 kips
Diaphragm span, L = 100 ft. Diaphragm depth, d = 80 ft.
Running load to diaphragm, w = F_d / L = 452 kips / 100 ft. = 4520 plf
Shear force resisted by each transverse wall line, B and G = ½ F_d = ½(452 kips) = 226 kips per wall
Maximum diaphragm shear (at the shear walls) = v_{trans} = (226 kips) / (80 ft) = 2825 plf
Moment at diaphragm midspan = wL^2 / 8 = (4520 plf)(100 ft.)^2 / 8 = 5650 kip ft
Chord force = M / d = 5650 kip-ft / 80 ft. = 71 kips (316 kN)

Longitudinal direction:
Weight tributary to the main theater diaphragm: W_{long} = 350 kips
Total shear force acting on the diaphragm: F_d = (1.45)(1.18)(1.0)(0.82)(350 kips) = 491 kips
Diaphragm span, L = 80 ft. Diaphragm depth, d = 100 ft.
Running load to diaphragm, w = F_d / L = 491 kips / 80 ft. = 6138 plf
Shear force resisted by each longitudinal wall line, 1 and 6 = ½ F_d = ½(491 kips) = 246 kips per wall
Maximum diaphragm shear (at the shear walls) = v_{long} = (246 kips) / (100 ft) = 2460 plf
Moment at diaphragm midspan = wL^2 / 8 = (6138/ plf)(80 ft.)^2 / 8 = 4910 kip ft
Chord force = M / d = 4910 kip-ft / 100 ft. = 49 kips (218 kN)

Note:
1 kip = 4.448 kN
1 plf = 14.59 N / m
1 ft = 0.305 m



DIAPHRAGM FORCES FOR TRANSVERSE SHEAR FORCE



DIAPHRAGM FORCES FOR LONGITUDINAL SHEAR FORCE

Shear Wall Forces

Transverse Direction:

Wall line B:

Shear wall line B resists forces tributary to both the main theater and lobby area diaphragms, in addition to self-inertial forces. Note: A partition load of 10 psf is included since there are partitions in the lobby area.

Determine shear forces from lobby area:

Tributary weight:

	Tributary Length (ft.)	Width or Tributary Height (ft.)	% Solid	Total Area (ft.^{2})	Unit Weight (psf)	Total Weight (kips)
Roof Level						
Entrance area between gridlines A & B (for loads tributary to wall line B)						
Roofing and Framing	52	15	100.0%	780	28.5	22.2
Partition Load (default 10 psf)	52	15	100.0%	780	10.0	7.8
Walls						
Walls of the lobby area						
Wall 5A-5B (only 1/2 length)	15	9.45	95.0%	135	60.0	8.1
Wall 2A-2B (only 1/2 length)	15	9.45	95.0%	135	60.0	8.1

Total Wt. = 46.2 kips
(205kN)

Shear from lobby area = $C_1C_2C_3S_aW$ = (1.39)(1.16)(1.0)(0.82)(46.2 k) = 61 kips (271 kN)

Shear from main theater diaphragm = 226 kips (determined previously)

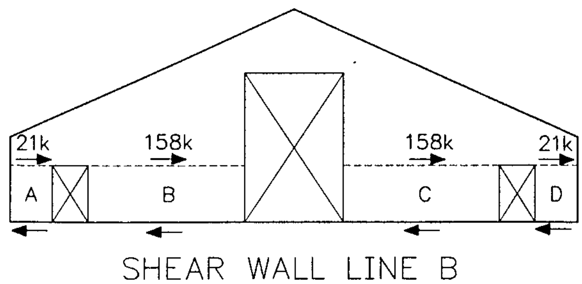
Self-weight of Wall line B = 54.3 kips

Shear from self-weight = $C_1C_2C_3S_aW$ = (1.39)(1.16)(1.0)(0.82)(54.3 k) = 72 kips

Total shear to Wall line B = 61 kips + 226 kips + 72 kips = 359 kips (1597 kN)

Distribute the wall shear to the individual piers based on relative rigidities:

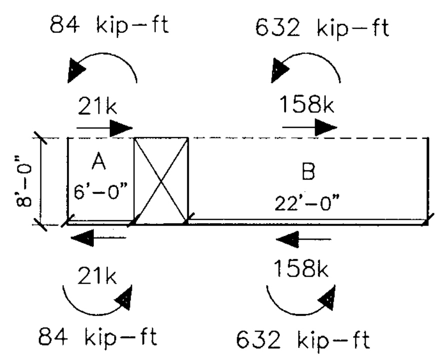
Wall Pier	Rigidity (kips / in)	Shear to Pier (kips)	Length of Pier (ft.)	Width of Pier (in.)	Area of Pier, A_{cv} (in.^{2})	Pier Shear Stress (psi)
Pier A	1476	21	6	4	288	74
Pier B	10873	158	22	4	1056	150
Pier C	10873	158	22	4	1056	150
Pier D	1476	21	6	4	288	74



Determine moments to piers:

Piers A and D are similar and piers B and C are similar.

M_A = (21 kips)(8 ft) / 2 = 84 kip-ft (114 kN-m)
M_B = (158 kips)(8 ft) / 2 = 632 kip-ft (857 kN-m)



Wall line G:

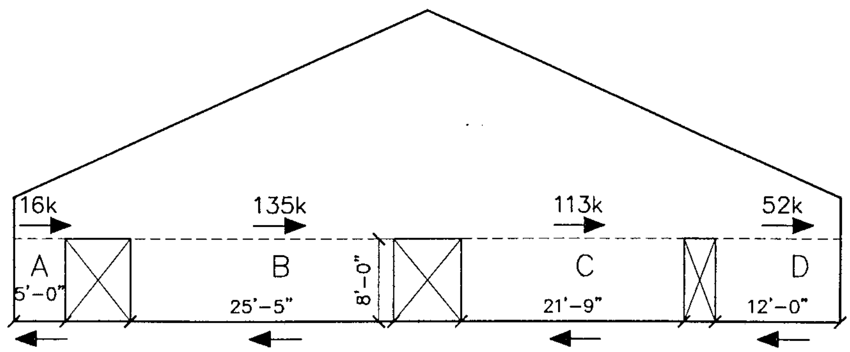
Shear wall line G resists forces from the main roof diaphragm and self-inertial forces.

Shear from main theater diaphragm = 226 kips (determined previously)
Self-weight of Wall line G = 68.0 kips
Shear from self-weight = $C_1C_2C_3S_aW$ = (1.39)(1.16)(1.0)(0.82)(68.0 k) = 90 kips

Total shear to Wall line G = 226 kips + 90 = 316 kips (1406 kN)

Distribute the wall shear to the individual piers based on relative rigidities:

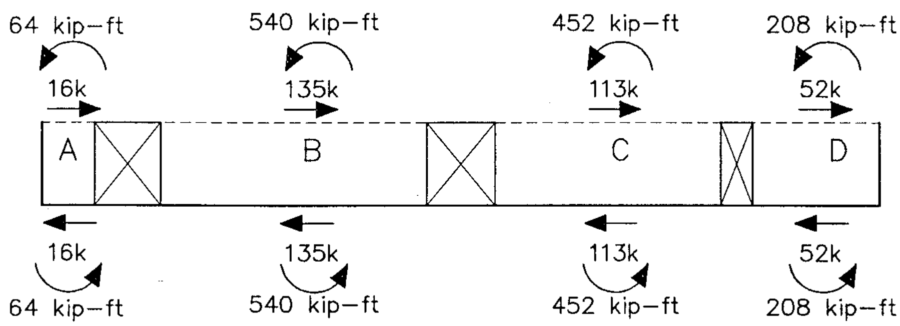
Wall Pier	Rigidity (kips / in)	Shear to Pier (kips)	Length of Pier (ft.)	Width of Pier (in.)	Area of Pier, A_{cv} (in.^{2})	Pier Shear Stress (psi)
Pier A	1476	16	5	4	240	65
Pier B	12869	135	25.5	4	1224	111
Pier C	10729	113	21.75	4	1044	108
Pier D	4979	52	12	4	576	91



SHEAR WALL LINE G

Determine moments to piers:

M_A = (16 kips)(8 ft) / 2 = 64 kip-ft (87 kN-m)
M_B = (135 kips)(8 ft) / 2 = 540 kip-ft (732 kN-m)
M_C = (113 kips)(8 ft) / 2 = 452 kip-ft (613 kN-m)
M_D = (52 kips)(8 ft) / 2 = 208 kip-ft (282 kN-m)



Longitudinal Direction:

Wall lines 1 and 6 are similar:

Shear wall lines 1 and 6 resist forces tributary to the main theater diaphragm and self-inertial forces.

Shear from main theater diaphragm = 246 kips (determined previously)

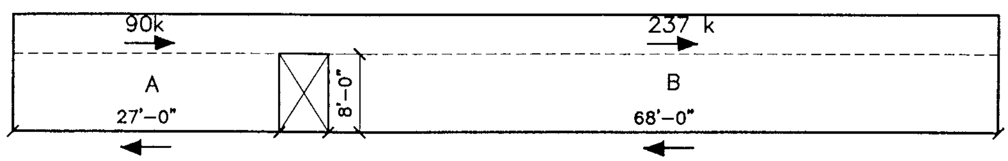
Self-weight of Wall line 1 = 57 kips

Shear from self-weight = $C_1C_2C_3S_aW$ = (1.45)(1.18)(1.0)(0.82)(49.1 k) = 80 kips

Total shear to Wall lines 1 & 6 = 246 kips + 80 kips = 326 kips (1450 kN)

Distribute the wall shear to the individual piers based on relative rigidities:

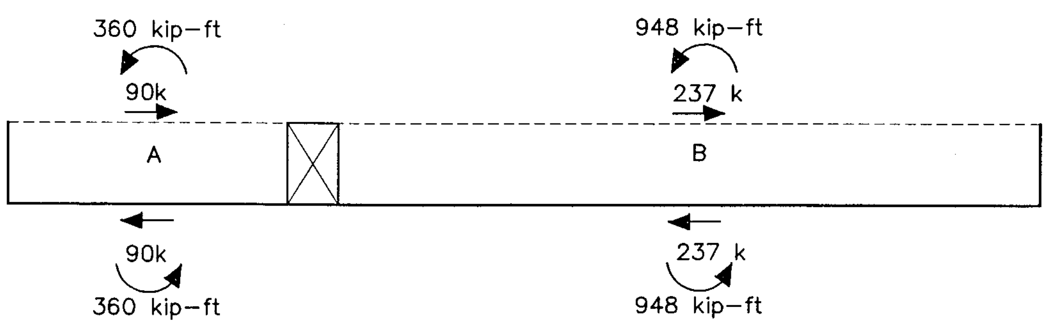
Wall Pier	Rigidity (kips / in)	Shear to Pier (kips)	Length of Pier (ft.)	Width of Pier (in.)	Area of Pier, A_{cv} (in.^{2})	Pier Shear Stress (psi)
Pier A	13718	89.5	27	4	1296	69
Pier B	36236	236.5	65	4	3120	76



SHEAR WALL LINES 1 & 6

Determine moments to piers:

M_A = (90 kips)(8 ft) / 2 = 360 kip-ft (488 kN-m)
M_B = (237 kips)(8 ft) / 2 = 948 kip-ft (1285 kN-m)



Out-of-plane wall forces

Wall anchorage:

Wall-to-diaphragm:

The walls must be anchored to the diaphragm for the larger off 400 S_{XS} pounds per foot of wall or χS_{XS} times the weight of the wall tributary to the anchor (per FEMA 273 Sec. 2.11.7)

$S_{XS} = S_{DS} = 0.82$
$\chi = 0.5$ (χ value for Safe Egress Performance Level is taken as the average of the Life Safety and Immediate Occupancy values listed in FEMA 273 Table 2-18.)

$400S_{XS} = 400(0.82) = 328$ plf
$\chi S_{XS} = (0.5)(0.82) = 0.41$
W_{trib} = (trib area)(unit weight)
trib area = ½ of average wall ht. x 1' wide strip = (21'/2)(1') = 10.5 ft.^{2}
$W_{trib} = (10.5\ ft^2)(110\ psf) = 1.155$ kips / ft. of length
$\chi S_{XS} W_{trib} = (0.41)(1.155\ klf) = 474\ plf > 328\ plf$

∴ 474 plf governs (6.92 kN / m)

Out-of-plane forces to be resisted by shotcrete:

The shotcrete walls must resist out-of-plane flexural and shear forces. The force level is determined in the same manner as the hollow clay tile-to-shotcrete anchorage forces.

Transverse walls:

For the transverse direction, the 'x' term is taken as the average wall height = 21'.

$$F_p = \frac{0.4a_p S_{XS} I_p W_p\left(1+\frac{2x}{h}\right)}{R_p},\quad F_p = \frac{0.4(1.0)(0.82)(1.0)(100psf)\left(1+\frac{2(21')}{21'}\right)}{(1.5)} = 66\,psf\ (3.16\ kPa)$$

Longitudinal walls:

$$F_p = \frac{0.4a_p S_{XS} I_p W_p\left(1+\frac{2x}{h}\right)}{R_p},\quad F_p = \frac{0.4(1.0)(0.82)(1.0)(100psf)\left(1+\frac{2(12')}{12'}\right)}{(1.5)} = 66\,psf\ (3.16\ kPa)$$

All of the walls are assumed to act as simply supported beams. The longitudinal walls span from the roof to the grade level (span = 12'). The transverse walls are assumed to span horizontally between the steel gravity columns (span = 15'-6").

$M = wL^2/8$
$M_{long} = (66\ psf)(12')^2 / 8 = 1.2$ kip-ft / ft of wall (5.34 kN-m / m)
$M_{trans} = (66\ psf)(15.5')^2 / 8 = 2.0$ kip-ft / ft of wall (8.90 kN-m / m)

$V = wL/2$
$V_{long} = (66\ psf)(12') / 2 = 0.4$ kips / ft (5.84 N / m)
$V_{trans} = (66\ psf)(15.5') / 2 = 0.5$ kips / ft (7.30 N / m)

Combination of load effects

- Gravity loads, Q_G: The structural components being evaluated do not resist any gravity loads other than their self-weight. Therefore, the gravity load effects are neglected.

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

b. Acceptance criteria

Deformation-controlled actions

The deformation-controlled actions that need to be checked include in-plane wall flexure and shear, out-of-plane wall flexure and diaphragm shear.

The design actions Q_{UD} are calculated according to $Q_{UD} = Q_G \pm Q_E$ (FEMA 273 Eq. 3-14)
Gravity effects are negligible so the design actions reduce to Q_E only.

The acceptance criteria for deformation-controlled components is:
$mQ_{CE} \geq Q_{UD}$ (Eq. 7-2)

Diaphragm shear

The diaphragms can be either deformation-controlled for panel buckling, or force-controlled for connection capacity. Therefore, they are checked for both conditions.

The diaphragm connection along wall line B must transmit the transverse diaphragm forces from both the main theater and entrance lobby. The diaphragm shear from the main theater was previously determined to be 2825 plf (see diaphragm forces section). The transverse shear force from the lobby diaphragm that is transmitted to wall line B was previously determined to be 61 kips (see shear wall forces section). This force from the lobby area is equal to a shear of 61 kips / 80 ' = 763 plf. The total shear that must be transmitted across the roof deck-to-wall line B connection = 2825 plf + 763 plf = 3588 plf (52.4 kN / m)

The allowable shear listed in the manufacture's catalog for this deck is 2420 plf (16 gage metal decking, side-seam welds 1-1/2" long @ 24", span = 5', with 7 welds per support). This value is multiplied by 1.5 to bring it to ultimate strength (FEMA 273 Sec. 5.8.1.3 states that allowable shear values may be multiplied by 2.0 to bring them to ultimate strength. However, the catalog values already have the 1/3 increase for allowable stress included. Therefore, the allowable stresses are multiplied by (2.0)(3/4) = 1.5.)

Diaphragm strength, Q_{CE} = 2420 plf * 1.5 = 3630 plf (53.0 kN/m)
(Note: The expected diaphragm strength does not have the 1.25 factor applied. The deformation-controlled failure of a diaphragm is due to buckling of the deck. Buckling will not allow for much strain hardening, and therefore, the 1.25 factor is not used.)

Q_{UD} = 3588 plf (52.4 kN / m)

The m-factor for bare metal deck diaphragms for the Safe Egress Performance Level from TI 809-04 paragraph 7-7.e.(4)(b)2.ii is 1.5.

mQ_{CE} = 1.5(3630 plf) = 5445 plf (79.4 kN / m) > Q_{UD} = 3588 plf (52.4 kN / m), OK.

Shear Walls
It is assumed that the seismic forces are resisted entirely by the new shotcrete.

The new shotcrete is 4" (102 mm) thick and is reinforced with #4 bars at 18" (457 mm) in both the horizontal and vertical directions. This is the minimum steel allowed based on ACI 318 Section 21.6.2.1.

Shear strength of wall piers

Footnote 1 of Table 7-3 in TI809-04 states that for shear wall segments to be considered as deformation-controlled components the maximum shear stress must be $\leq 6\sqrt{f_c'} = 6\sqrt{3000\,\text{psi}} = 329\,\text{psi}$ (2267 kPa) and the axial load must be $\leq 0.15A_g f_c'$. The highest shear stress in the shotcrete piers is equal to 150 psi (1034 kPa) < 329 psi (2267 kPa) and axial loads are negligible. Therefore, the shear wall segments are deformation-controlled components.

The nominal shear strength V_n of structural wall segments is determined by:

$$V_n = A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right) \qquad \text{(ACI 318 Eq. 21.6.5.2)}$$

The capacity of deformation-controlled components is based on the component's expected strength. For all shear strength calculations, 1.0 times the specified reinforcement yield strength should be used to determine the nominal strength of the member(per FEMA 273 Sec. 6.8.2.3). The nominal shear strength of the walls is used to check acceptance rather than the expected strength to be conservative.

Area of #4 bar = 0.20 in.^{2}
$\rho_n = A_{st}$ / (thickness)(spacing) = (0.20 in.^{2}) / (4")(18") = 0.0028
The nominal shear strength =

$$V_n = A_{cv}\left(2\sqrt{f_c'} + \rho_n f_y\right) = A_{cv}\left(2\sqrt{3000\,\text{psi}} + 0.0028(60000\,\text{psi})\right) = A_{cv}(278\,\text{psi})$$

The m-factor from Table 7-3 of TI 809-04 for shear wall segments is equal to 2.0.

$mQ_{CE} = mV_n$ = (2.0)(A_{cv})(278 psi) = A_n (556 psi) = A_n (3831 kPa)
Q_{UD} = 150 psi (1034 kPa) (highest shear stress in any of the shotcrete piers occurs in piers B and C along wall line B)

556 psi (3831 kPa) > 150 psi (1034 kPa), OK

Moment strength of wall piers

The expected moment strength for the wall piers is conservatively estimated to be equal to the strength of the steel at the pier boundaries times the moment arm between the steels at the boundaries. The trim steel at each pier edge consists of two #4 bars. The moment arm between the trim steel is estimated to be equal to the length of the pier minus 1'. For moment calculations, the yield strength of the flexural reinforcement is taken as 125% of the specified yield strength to account for material overstrength and strain hardening (per FEMA 273 Sec. 6.8.2.3).

Expected reinforcing steel strength = $f_{ye} = 1.25f_y$ = 1.25(60 ksi) = 75 ksi
Moment lever arm = L' = Length of pier –1'
Expected moment strength = $Q_{CE} = A_{st}f_{ye}L'$
Area of reinforcing steel = A_{st} = 2 - #4 bars = 0.40 in.^{2}

Wall line B:

Wall Pier	Shear to Pier (kips)	Moment on Pier Q_{UD} (kip-ft)	Moment Arm of Pier (ft.)	Expected Moment Strength of Pier Q_{CE} (kip-ft)	m-factor	Q_{UD} / mQ_{CE}	Acceptance
Pier A	21	84	5	150	2	0.3	OK
Pier B	158	632	21	630	2	0.5	OK
Pier C	158	632	21	630	2	0.5	OK
Pier D	21	84	5	150	2	0.3	OK

Wall line G:

Wall Pier	Shear to Pier (kips)	Moment on Pier Q_{UD} (kip-ft)	Moment Arm of Pier (ft.)	Expected Moment Strength of Pier Q_{CE} (kip-ft)	m-factor	Q_{UD} / mQ_{CE}	Acceptance
Pier A	16	64	4	120	2	0.3	OK
Pier B	135	540	24.5	735	2	0.4	OK
Pier C	113	452	20.75	623	2	0.4	OK
Pier D	52	208	11	330	2	0.3	OK

Wall lines 1 & 6 (these walls are similar):

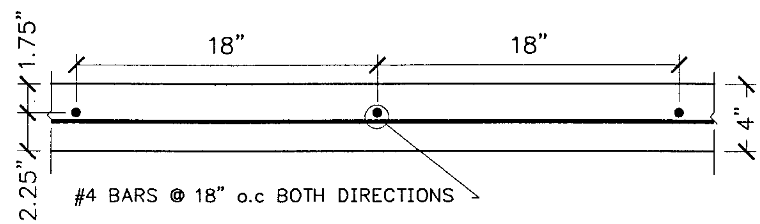
Wall Pier	Shear to Pier (kips)	Moment on Pier Q_{UD} (kip-ft)	Moment Arm of Pier (ft.)	Expected Moment Strength of Pier Q_{CE} (kip-ft)	m-factor	Q_{UD} / mQ_{CE}	Acceptance
Pier A	90	360	26	780	2	0.2	OK
Pier B	237	948	64	1920	2	0.2	OK

All of the piers have adequate moment strength.

Out-of-plane wall flexural forces:

The flexural demand on the walls = 2.0 kip-ft (transverse walls are more critical than longitudinal walls).
Q_{UD} = 2.0 kip-ft (2.71 kN-m)

Determine flexural strength of the walls:



CROSS-SECTION OF SHOTCRETE WALL

Flexural strength calculations use the expected reinforcement strength, $f_{ye} = 1.25\ f_y = 1.25(60\text{ ksi}) = 75$ ksi
Distance to tension steel, d = 1.75"
Area of steel in a one foot wide strip, $A_s = 0.20$ in.^{2} / (18" / 12) = 0.13 in.^{2} / foot

$$a = \frac{A_s f_{ye}}{0.85 f'_c b} = \frac{(0.13\text{in}^2)(75\text{ksi})}{0.85(3\text{ksi})(12")} = 0.32"$$

$$M_{CE} = A_s f_{ye}\left(d - \frac{a}{2}\right) = \left(0.13\,\text{in}^2\right)(75\,\text{ksi})\left(1.75" - \frac{0.32"}{2}\right) = 15.5\,\text{kipin} = 1.3\,\text{kip} - \text{ft}\ (1.76\text{ kN-m})$$

$Q_{CE} = M_n$ = 1.3 kip-ft

It is assumed that the walls are flexure-controlled for out-of-plane forces. The m-factor from TI 809-04 Table 7-2 = 2.3.

mQ_{CE} = (2.3)(1.3 kft) = 3.0 kip-ft (4068 kN-m) > Q_{UD} = 2.0 kip-ft (2.71 kN-m), OK

Force-controlled actions

The force-controlled actions that need to be checked include diaphragm shear (strength of connections), diaphragm chord forces, connections of the diaphragm to new shotcrete walls, out-of-plane wall shear strength and anchorage, shear transfer to strip footings, and shear capacity of the existing strip footings.

The design actions Q_{UF} are calculated according to $Q_{UF} = Q_G \pm \dfrac{Q_E}{C_1C_2C_3}$ (FEMA 273 Eq. 3-15)

Gravity effects are negligible so the design actions reduce to $Q_{UF} = \dfrac{Q_E}{C_1C_2C_3}$ only.

For transverse seismic forces, $C_1C_2C_3$ = (1.39)(1.16)(1.0) = 1.61
For longitudinal seismic forces, $C_1C_2C_3$ = (1.45)(1.18)(1.0) = 1.71

Therefore, the force-controlled demands are:

Transverse: $Q_{UF} = Q_E$ / 1.61
Longitudinal: $Q_{UF} = Q_E$ / 1.71

The acceptance criteria for force-controlled components is:
$Q_{CN} \geq Q_{UF}$ (Eq. 7-3)

Diaphragm shear

The connection capacity of the metal deck diaphragm is considered a force-controlled action.

The governing shear is the total shear that must be transmitted across the roof deck-to-wall line B connection determined earlier = 3588 plf (52.4 kN / m) for seismic forces in the transverse direction.

$Q_{UF} = Q_E / 1.61 = 3588$ plf / 1.61 = 2229 plf (32.5 kN / m)
Diaphragm strength, $Q_{CN} = Q_{CE} = 3630$ plf (53.0 kN / m) (determined previously)

$Q_{CN} = 3630$ plf (53.0 kN / m) > $Q_{UF} = 2229$ plf (32.5 kN / m), OK

Diaphragm chord forces

The diaphragm chord elements are the edge beams along the wall lines and the reinforcing at the top of the new shotcrete. The 2-#5 reinforcing bars in the top of the shotcrete are made continuous by passing the bars through holes drilled in the interfering columns. The chord members are checked for the reinforcing only, neglecting the capacity of the edge beam, to be very conservative.

$Q_{CN} = Af_{ye}$, where $f_{ye} = 1.25f_y = 1.25(60 \text{ ksi}) = 75$ ksi
$Q_{CN} = (0.61 \text{ in.}^2)(75 \text{ ksi}) = 46$ kips (205 kN)

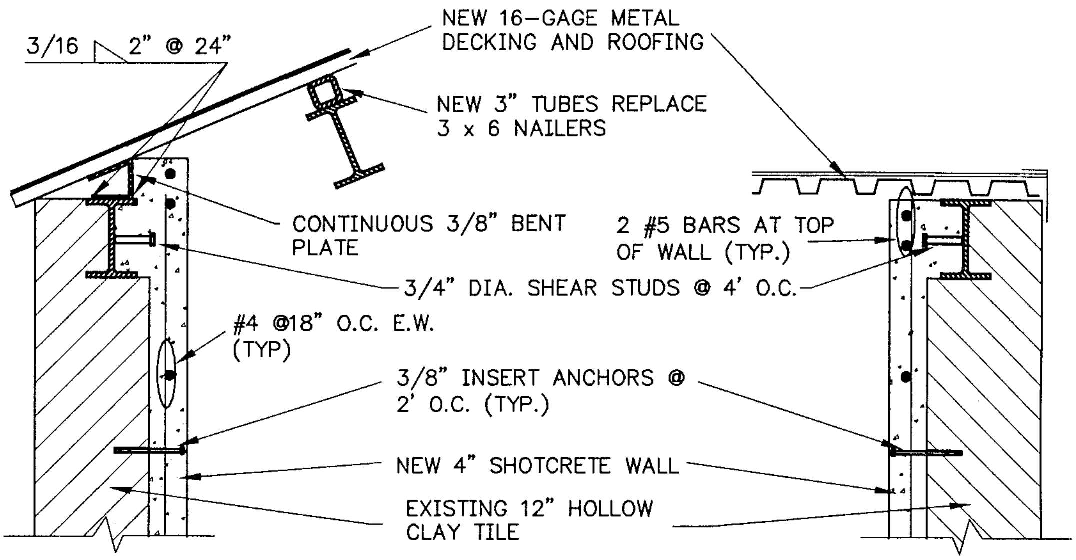
Maximum Chord force, $Q_E = 71$ k (for seismic forces in the transverse direction)

$Q_{UF} = Q_E / 1.61 = 71 \text{ k} / 1.61 = 44$ k (196 kN)

$Q_{CN} = 46$ kips (205 kN) > $Q_{UF} =$ 44 k (196 kN), OK

Connection of diaphragm to shear walls for in-plane shear transfer

The deck is welded to 3/8" (9.5 mm) bent plate (longitudinal walls) or directly to the edge beam (transverse walls) which are anchored to the wall with ¾" (19 mm) studs at 4' (1.22 m) on center. The welding of the decking to the plate and the edge beam is per the manufacture's specs. At the longitudinal walls, the bent plate is welded to the beams with 2" (51 mm) fillet welds at every 2' (61 cm) on both sides of the plate.



TYPICAL LONGITUDINAL METAL DECKING–TO–WALL CONNECTION

TYPICAL TRANSVERSE METAL DECKING–TO–WALL CONNECTION

1 in = 25.4 mm
1 ft = 0.305 m

Check of shear stud capacity:

The maximum shear force transferred to the shotcrete walls occurs at wall line B. This shear was determined to be 2229 plf in the previous step (scaled to be force-controlled).

Shear demand, Q_{UF} = 2229 plf (32.5 kN / m)

Q_n for a ¾" headed stud = 17.7 kips / stud (78.7 kN) (AISC LRFD Table 5-1)

Studs are place at every 4'

Q_{CN} = Q_n / 4' = 17.7 kips / 4' = 4.3 kips / ft (62.8 kN / m)
Q_{CN} = 4.3 kips / ft (62.8 kN / m) > Q_{UF} = 2229 plf (32.5 kN / m), OK

Check of intermittent welds of plate to beam at longitudinal walls (use Q_{UF} = 2229 plf (32.5 kN / m) to be conservative):

Strength of weld = 0.60E70xx = 0.60(70 ksi) = 42 ksi
Weld size = 0.707(3/16") =0.133"
Length of each weld = 2"
Q_n = (42 ksi)(0.133")(2") = 11.2 kips / weld
2 welds (one on each side); Q_n = (2)(11.2 kips) = 22.4 kips
Welds spaced at 2' O.C.;
Q_{CN} = 22.4 kips / 2' = 11.2 kips / ft (163 kN / m) > Q_{UF} = 2229 plf (32.5 kN / m), OK

Connection of diaphragm to shear walls for out-of-plane bracing

The walls load the anchor connection in tension. The design tensile strength of an anchor bolt in concrete from FEMA 302 Section 9.2.4.1 is:

$P_s = 0.9A_bF_un$ (strength governed by steel) (FEMA 302 Eq. 9.2.4.1-1)
$P_s = 0.9(0.44\text{ in.}^2)(60\text{ ksi})(1\text{ bolt}) = 24\text{ kips / bolt}$

$\phi P_c = \phi\lambda\sqrt{f_c'}(2.8A_s)n$ (strength governed by concrete failure) (FEMA 302 Eq. 9.2.4.1-2)
For this document $\phi = 1.0$
$A_s = \pi l_e^2 = \pi(3")^2 = 28.3\text{ in.}^2$
$\phi P_c = (1.0)(0.85)\sqrt{3000\,\text{psi}}\left(2.8(28.3\text{in.}^2)\right)(1\,\text{bolt}) = 3.7\text{kips / bolt}$ - governs
$Q_{CN} = (3.7\text{ kips / bolt}) / 4\text{ ft.} = 925\text{ plf}$ (13.5 kN / m) $> Q_{UF} = 474$ plf (6.92 kN / m), OK

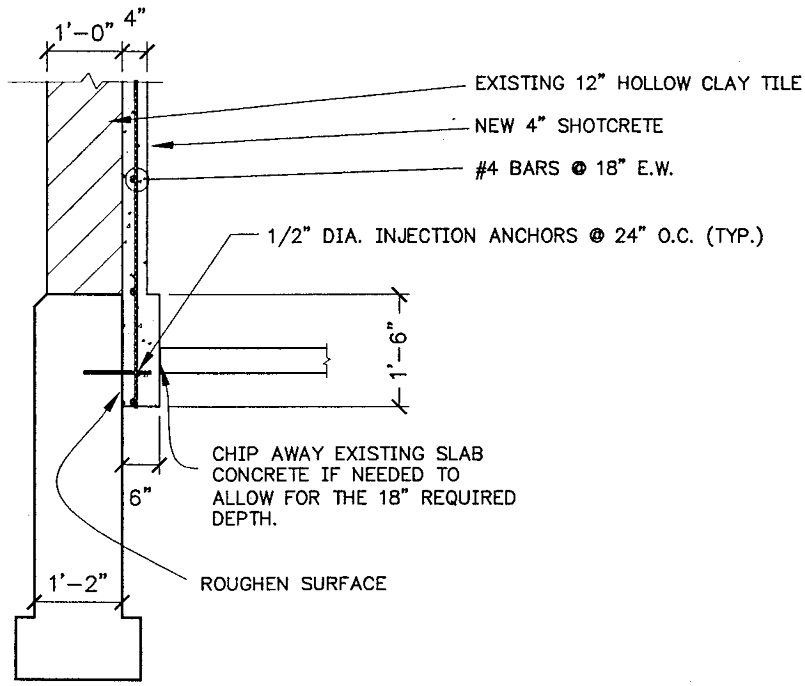
Out-of-plane wall shear strength

$Q_{UF} = V_{trans} = 0.5$ kips / ft (7.3 kN / m)

The shear strength of the wall is conservatively taken to be $2\sqrt{f_c'} = 2\sqrt{3000} = 110\text{psi}$
$Q_{CN} = (110\text{ psi})(d)(b) = (110\text{ psi})(1.75")(12" / \text{ft}) = 2.3$ kips / ft (33.6 kN / m)
$Q_{CN} = 2.3$ kips / ft (33.6 kN / m) $> Q_{UF} = 0.5$ kips / ft (7.3 kN / m), OK

Shear transfer from new shotcrete to existing footings

The shear force is transferred from the new shotcrete walls to the existing strip footings through 1/2" diameter injection adhesive anchors from a manufacturer's catalog. The anchors are placed at 24" on center.



TYPICAL REHABILITATED FOOTING

Shear strength of one anchor form catalog = 8.30 kips (37 kN)
Q_{CN} = 8.3 k / 24" = 4.2 kips / ft (61.3 kN / m)

Wall Line	Deformation-Controlled Shear Demand Q_E (kips)	Scale Factor $C_1C_2C_3$	Force-Controlled Shear (kips)	Length of Wall - Openings (ft)	Shear Demand Q_{UF} (klf)
B	359	1.62	222	56	4.0
G	316	1.62	195	64	3.0
1	326	1.70	192	95	2.0
6	326	1.70	192	95	2.0

$Q_{CN} > Q_{UF}$ for all wall lines, OK

Shear in existing concrete strip footings

The shear strength of the existing footings is calculated based on the concrete contribution only while neglecting the steel reinforcement contribution. Therefore, the strength of the footings is conservatively calculated as $A_{cv}2\sqrt{f_c'}$.
The footings are 14" wide and the concrete strength, f_c', is assumed to be 2500 psi

Footing shear strength = (Width)(Length) $2\sqrt{2500}$ = (14")(Length)(100 psi) = 1.4 k / inch = 16.8 kips / ft

Q_{CN} = 16.8 kips / ft > Q_{UF} for all wall lines, 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.