APPENDIX D
STRUCTURAL EXAMPLE PROBLEMS

This appendix illustrates the implementation of the provisions of this document for the seismic evaluation and rehabilitation of military buildings. These examples presume that geologic hazard studies for each building have been performed and that any identified hazard has been mitigated or resolved. Guidelines for geologic hazard studies are presented in Appendix F of TI 809-04 and examples of geologic studies are provided in Appendix G of that document. The example problems in the following sections of this Appendix were selected to represent various structural systems in representative existing military buildings.

D1 Three-story Barracks Building

Building & Site Data.

Building Description.

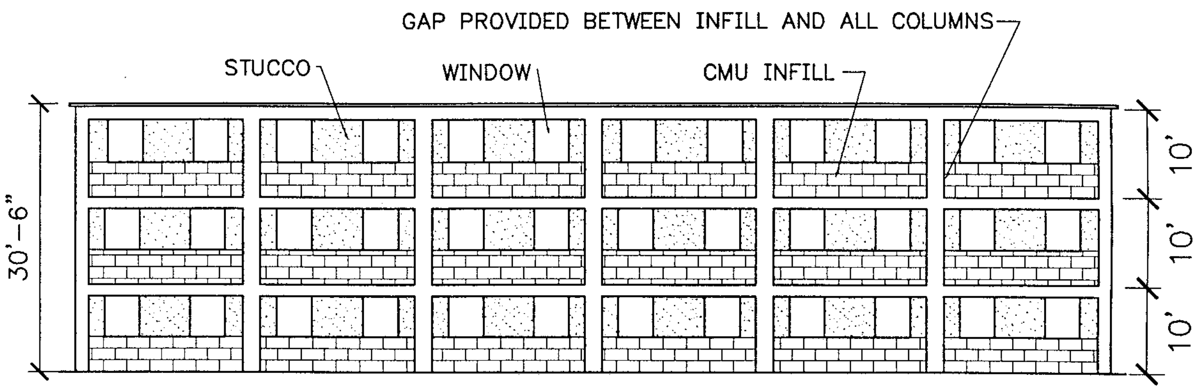
The H-shaped barracks (building 1452) is a three-story cast-in-place reinforced-concrete structure located at Fort Lewis, Washington (For this example only one of the two legs which form the H-shaped group of structures is considered). According to the available drawings obtained before and during the initial site visit, it was designed as a two-company barracks in 1956.

The barracks consist of four separate structures with 2-inch separation between adjacent structures. Dimensions of the structure considered in this example are approximately 39 feet by 117 feet (11.9 m by 35.7 m). The building is three stories with a story height of 10 feet each (3.1 m).

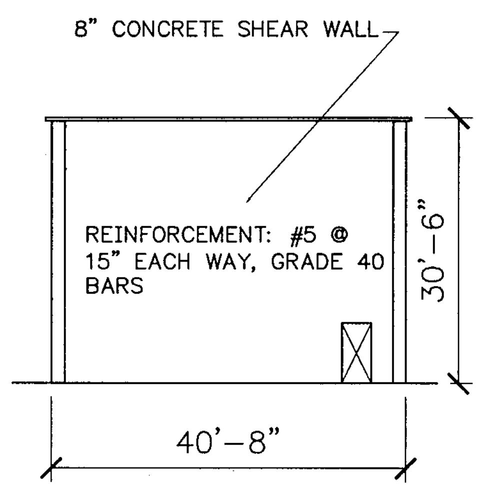
Vertical Load Resisting System. The vertical load resisting system consists of reinforced concrete flat slab and columns. The columns are nominally spaced at 19.5' (5.9 m) in both directions of building axes. The slab thickness is 7 inches (178 mm) at the roof, third, and second floor levels. The first floor slab is 4 inches (102 mm) thick concrete on grade. The footings consist of individual spread footings below the perimeter and interior columns. Strip footings support the transverse walls at the end of the structure and the partial CMU infills along the longitudinal walls.

The interior columns are 14-inch (356 mm) square with relatively light reinforcement and #3 ties at 12 inches (30.5 mm). The perimeter framing is a beam-column framing system. The perimeter columns at the ends of the frame are 12 inches by 18 inches (30.5 mm by 457 mm), while the interior columns of the perimeter frames measure 12 inches by 24 inches (30.5 mm by 610 mm) with the major axes oriented in the longitudinal axis of the structure. The beams at the roof level are 12 inches wide by 18 inches (30.5 mm by 457 mm) deep and the beams at the third and second floor levels are 10 inches wide by 15-1/2 inches (254 mm by 394 mm) deep.

Lateral Load Resisting System. The primary lateral-force resisting system consists of the concrete floors acting as diaphragms transmitting lateral forces to the perimeter frames. The lateral-force resisting frame system consists of rectangular columns and beams in the longitudinal direction. The transverse lateral-force resisting system consists of 8-inch (203mm) thick concrete shear walls at the ends of each structure. The spread and strip footing foundations resist shear forces through friction and passive soil pressure.

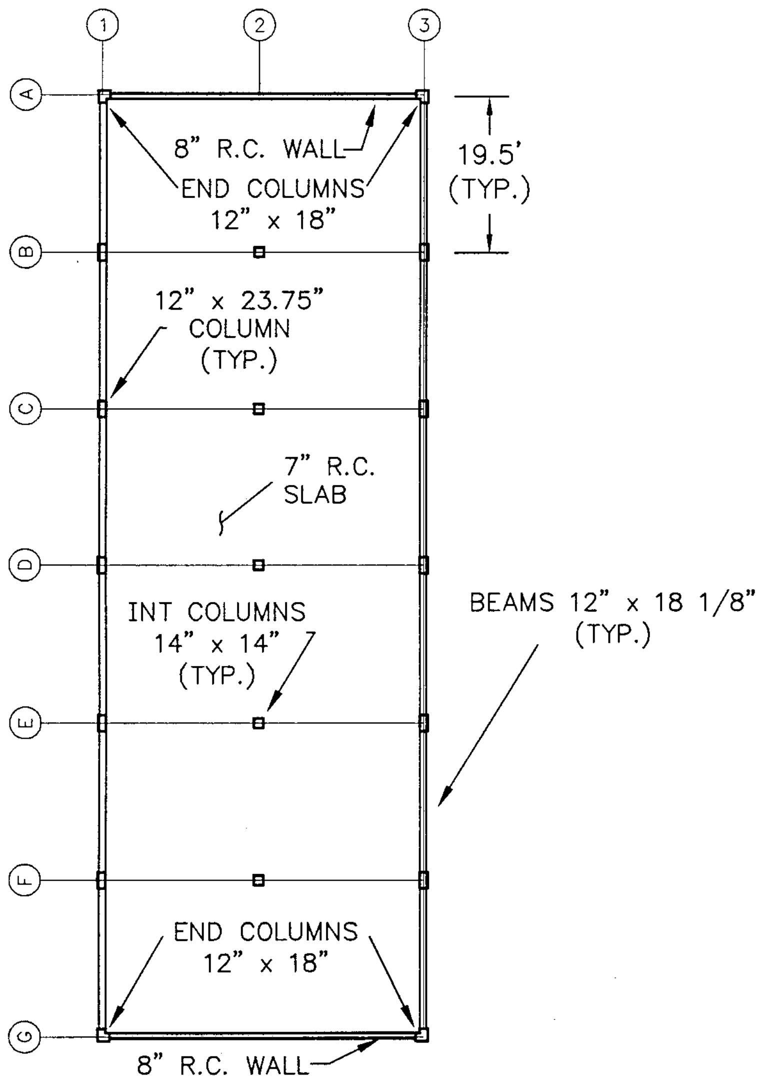


LONGITUDINAL BUILDING ELEVATION

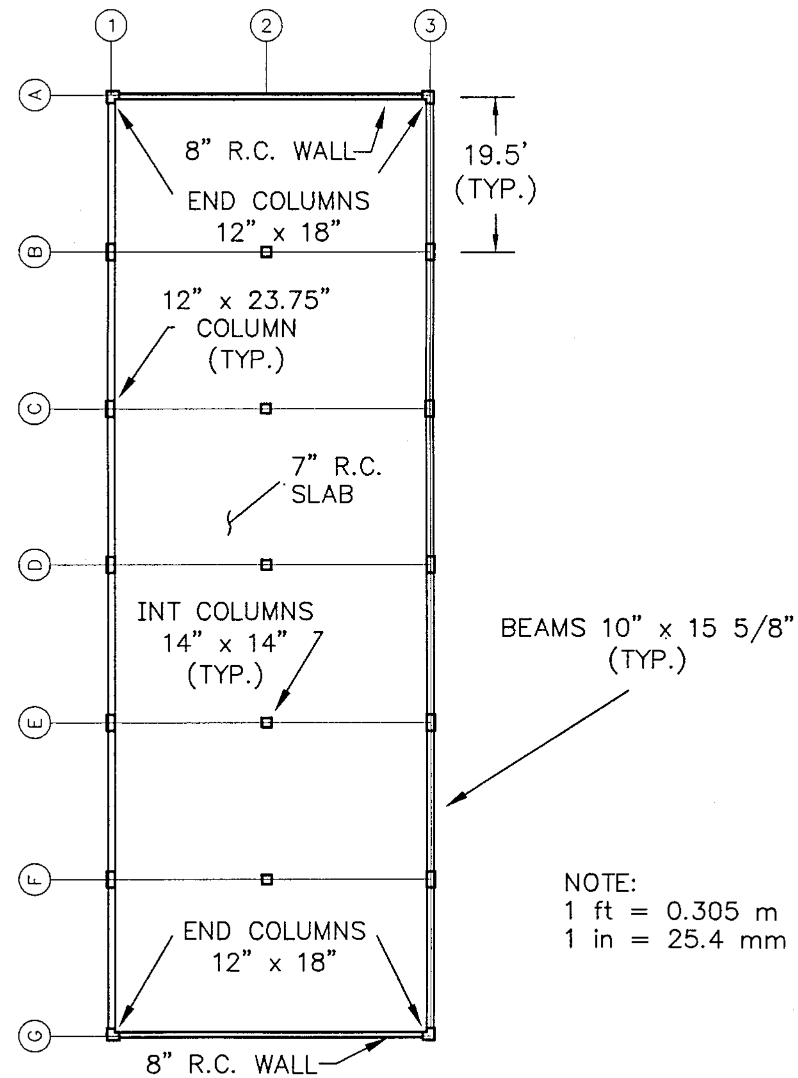


TRANSVERSE BUILDING ELEVATION

Note: 1 ft = 0.305 m
1 inch = 25.4 mm

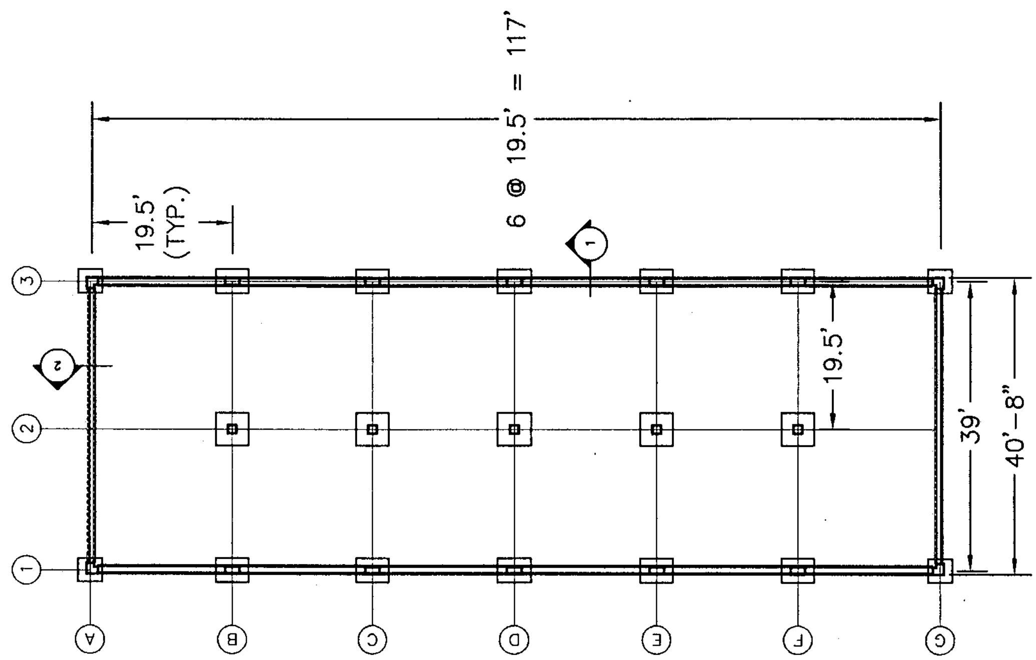


FRAMING @ ROOF LEVEL

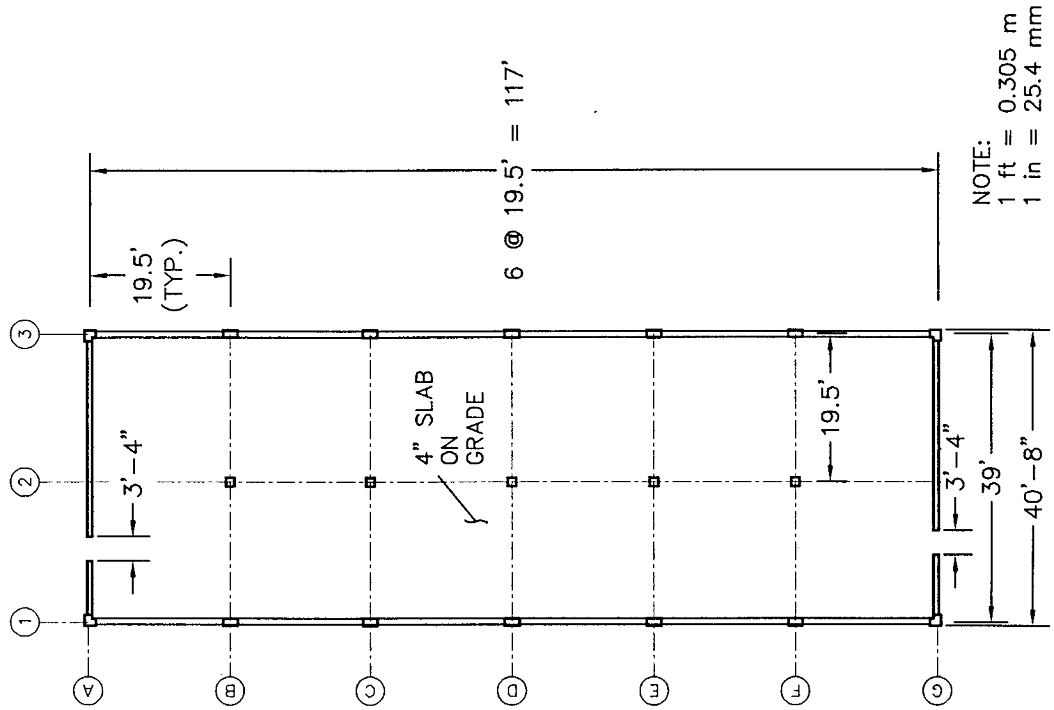


NOTE:
1 ft = 0.305 m
1 in = 25.4 mm

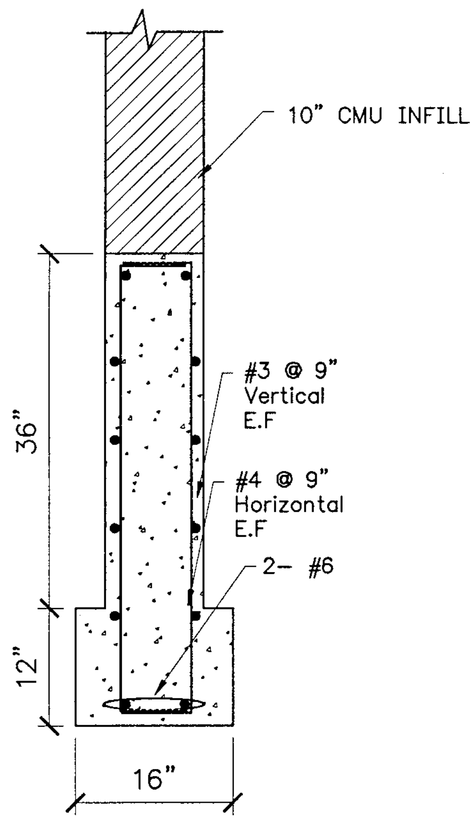
FRAMING @ SECOND & THIRD FLOOR LEVELS



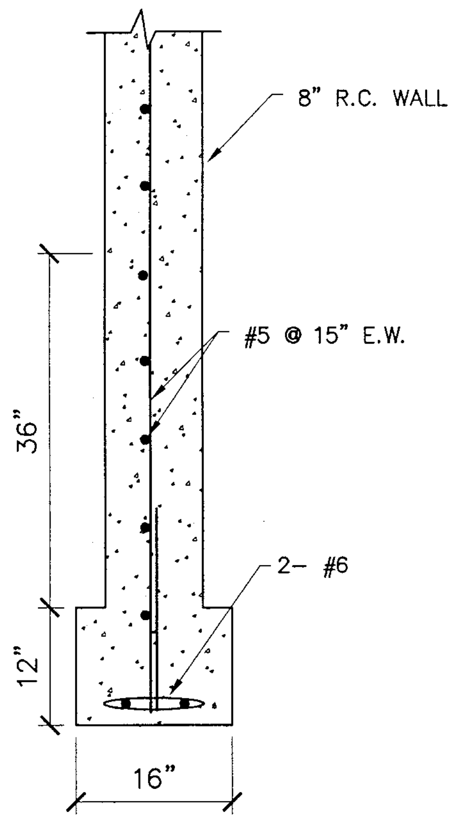
FOUNDATION PLAN



FIRST FLOOR PLAN SHOWING DOOR OPENINGS



SECTION (1) THROUGH FOOTING

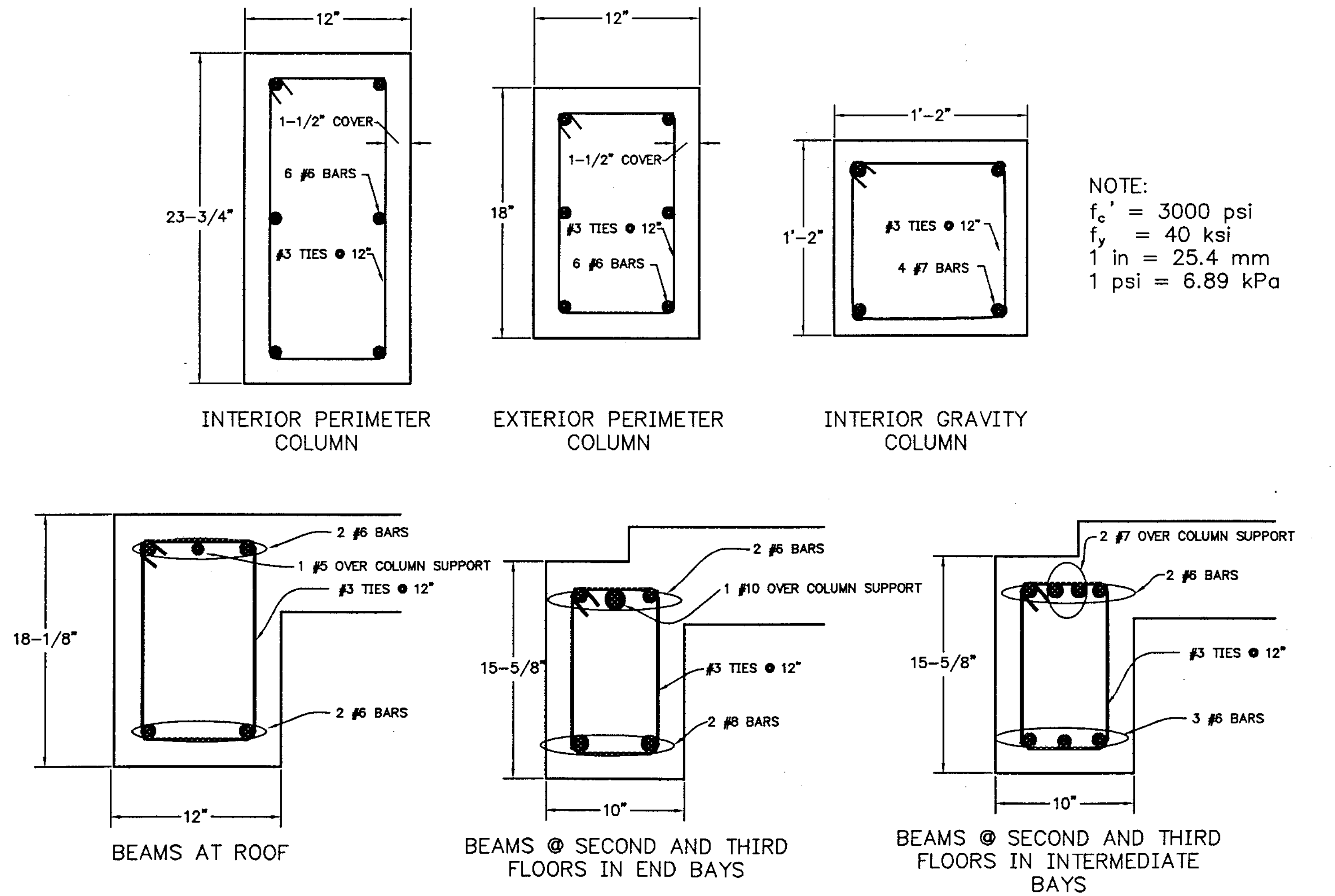


SECTION (2) THROUGH FOOTING

CONCRETE $f_c' = 3000$ psi

ALL STEEL $f_y = 40$ ksi

NOTE:
1 in = 25.4 mm
1 psi = 6.89 kPa



COLUMN AND BEAM CROSS SECTIONS

A. Preliminary Determinations (from Table 2-1)

1. Obtain building and site data:

a. Seismic Use Group. Since the building is not described by any of the occupancies in Table 2-2 for special, hazardous, or essential facilities, it is designated as a standard occupancy structure within Seismic Use Group I.

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

c. Applicable Ground Motions (Performance Objectives). Table 2-4 prescribes a ground motion of 2/3 MCE for the Seismic Use Group I, Life Safety 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:

$S_{MS} = F_aS_S = (1.02)(1.20) = 1.224$ (TI 809-04 Eq. 3-1)
$S_{M1} = F_vS_1 = (1.62)(0.39) = 0.632$ (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 checklists). 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. Table 4-3 of this document requires that the geologic site hazard and foundation checklists contained in FEMA 310 be completed. See Section C, Structural Screening (Tier 1), for the completed checklist.

3. Evaluate geologic hazards. Not necessary.

4. Mitigate geologic hazards. Not Necessary.

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

At this point, after reviewing the drawings and conducting an on-site visual inspection of the building, a judgmental decision is made as to whether the building definitely requires rehabilitation without further evaluation or whether further evaluation might indicate that the building can be considered to be acceptable without rehabilitation.

1. Determine if building definitely needs rehabilitation without further evaluation. It is not obvious if the building definitely needs rehabilitation or not. There is a continuous load path and no obvious signs of structural distress. In the longitudinal direction the building frame system lacks ductile detailing but could possibly possess enough strength and stiffness due to the large column depth. In the transverse direction, the shear walls likely have the capacity to resist the lateral force demands. The building must be evaluated to determine if it is acceptable or if it needs rehabilitation.

2. Determine evaluation level required. FEMA 310 provides three tiers of evaluation that are described in paragraph 4-2 of this document. Buildings in Seismic Use Group I may be evaluated using a Tier 1 evaluation, provided the structure meets the requirements of FEMA 310 Table 3-3. If deficiencies are found a Tier 2 or Tier 3 evaluation will determine if the building is acceptable or needs rehabilitation. For evaluations performed in accordance with this document, a Tier 2 or Tier 3 evaluation may be performed in lieu of the Tier 1 evaluation, when it is considered that the lower tier evaluation would not produce conclusive results.

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

1. Determine applicable checklists. Table 4-2 lists the required checklists for a Tier 1 evaluation based on Seismic Design Category. Seismic design category D buildings require completion of the Basic Structural, Supplemental Structural, Geologic Site Hazard & Foundation, Basic Nonstructural and Supplemental Nonstructural checklists. (Note: A nonstructural evaluation is not in the scope of this design example).

2. Complete applicable checklists. The checklists are taken from FEMA 310.

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). *This building is being evaluated for the Life Safety Performance Level only.*

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). *$0.6S_a = (0.6)(0.82) = 0.49$ (See the quick checks section following the checklists for determination of S_a. The height of the building ≈ 30 ft.*
Transverse: (base/height) = 39.67 / 30 = 1.32 > 0.49, OK
Longitudinal: (base/height) = 117 / 30 = 3.9 > 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). *This building is being evaluated for the Life Safety Performance Level only.*

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 being evaluated for the Life Safety Performance Level only.*

Basic Structural Checklist for Building Type C1: Concrete Moment Frames (FEMA 310, Section 3.7.8)

Building System

C NC N/A LOAD PATH: The structure shall contain one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (Tier 2: Sec. 4.3.1.1). *See Building Description.*

C NC N/A ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4% of the height for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.3.1.2). *There is an adjacent building 2" from this structure. However, both structures are the same height and have matching floors. Pounding damage is likely to result only in nonstructural damage. Therefore, the small separation is not a concern.*

C NC N/A MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure (Tier 2: Sec. 4.3.1.3). *The building does not have any mezzanine levels.*

C NC N/A WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety and Immediate Occupancy (Tier 2: Sec. 4.3.2.1). *There is no story with a lateral strength less than 80% of an adjacent story.*

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

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

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

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

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

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

C NC N/A POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used (Tier 2: Sec. 4.3.3.5).

Lateral-Force-Resisting System

C NC N/A REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy (Tier 2: Sec. 4.4.1.1.1). *There are two lines of moment frames in the longitudinal direction with 6 bays per frame line.*

C NC N/A INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements (Tier 2: Sec. 4.4.1.2.1). *Isolation joints of adequate dimensions provided between nonstructural wall at sides and top.*

C NC N/A SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or $2(f'_c)^{1/2}$ for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.1). *See Quick Checks section following checklists for calculation of shearing stress demand on columns of moment frames. Demand = 276 psi > Allowable = 100 psi*

C NC N/A AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10f'_c$ for Life Safety and Immediate Occupancy. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30f'_c$ for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.2). *See Quick Checks section following checklists for calculation of axial load demand. Demand = 300 psi < Allowable = 900 psi, OK*

Connections

C NC N/A CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the tensile capacity of the column for Immediate Occupancy (Tier 2: Sec. 4.6.3.2). *All of the concrete columns are doweled into the foundation.*

Supplemental Structural Checklist for Building Type C1: Concrete Moment Frames (FEMA 310, Section 3.7.8S)

Lateral-Force-Resisting System

C NC N/A FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams (Tier 2: Sec. 4.4.1.4.3). *The lateral-force-resisting systems consist of beam-column moment frames and shear walls.*

C NC N/A PRESTRESSED FRAME ELEMENTS: The lateral-load-resisting frames shall not include any prestressed or post-tensioned elements (Tier 2: Sec. 4.4.1.4.4). *There are no prestressed frame elements in the building.*

C NC N/A SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level for Life Safety and 75% for Immediate Occupancy (Tier 2: Sec. 4.4.1.4.5). *There are no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level.*

C NC N/A NO SHEAR FAILURES: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns (Tier 2: Sec. 4.4.1.4.6). *See the quick checks section for this compliance check.*

C NC N/A STRONG COLUMN/WEAK BEAM: The sum of the moment capacity of the columns shall be 20% greater than that of the beams at frame joints (Tier 2: Sec. 4.4.1.4.7). *The sum of the column moment capacities is more than 20% greater than that of the beams.*

C NC N/A BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment shall be continuous throughout the length of the members for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.8). *Bottom longitudinal bars are not continuous through joints.*

C NC N/A COLUMN-BAR SPLICES: All column bar lap splice lengths shall be greater than 35 d_b for Life Safety and 50 d_b for Immediate Occupancy and shall be enclosed by ties spaced at or less than 8 d_b for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.9). *Column bars are spliced for a length of* $20d_b$ *only.*

C NC N/A BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing shall not be located within $l_b/4$ of the joints and shall not be located within the vicinity of potential plastic hinge locations (Tier 2: Sec. 4.4.1.4.10). *The beam-bar splices are located at the beam midspan.*

C NC N/A COLUMN-TIE SPACING: Frame columns shall have ties spaced at or less than d/4 for Life Safety and Immediate Occupancy throughout their length and at or less than 8 d_b for Life Safety and Immediate Occupancy at all potential plastic hinge locations (Tier 2: Sec. 4.4.1.4.11). *Column-ties are spaced at 12" = d.*

C NC N/A STIRRUP SPACING: All beams shall have stirrups spaced at or less than d/2 for Life Safety and Immediate Occupancy throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of 8 d_b or d/4 for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.12). *Typical stirrup spacing @ 12" for both 12" and 10" wide beams.*

C NC N/A JOINT REINFORCING: Beam-column joints shall have ties spaced at or less than 8d_b for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.13). *No transverse ties in beam-column joints.*

C NC N/A JOINT ECCENTRICITY: There shall be no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.1.4.14). *This building is being evaluated for the Life Safety Performance Level only*

C NC N/A STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.1.4.15). *This building is being evaluated for the Life Safety Performance Level only.*

C NC N/A DEFLECTION COMPATIBILITY: Secondary components shall have the shear capacity to develop the flexural strength of the elements for Life Safety and shall have ductile detailing for Immediate Occupancy (Tier 2: Sec. 4.4.1.6.2). *See the quick checks section for compliance check of this statement.*

C NC N/A FLAT SLABS: Flat slabs/plates classified as secondary components shall have continuous bottom steel through the column joints for Life Safety. Flat slabs/plates shall not be permitted for the Immediate Occupancy Performance Level (Tier 2: Sec. 4.4.1.6.3).

Diaphragms

C NC N/A DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints (Tier 2: Sec. 4.5.1.1). *There are no split-level floors.*

C NC N/A PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.5.1.7). *This building is being evaluated for the Life Safety Performance Level only*

C NC N/A DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.5.1.8). *This building is being evaluated for the Life Safety Performance Level only*

Connections

C **NC** N/A LATERAL LOAD AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy (Tier 2: Sec. 4.6.3.10). *No pile foundations in structure.*

Basic Structural Checklist for Building Type C2: Concrete Shear Wall Buildings with Stiff Diaphragms (FEMA 310, Section 3.7.9)

Building System

C	NC	N/A	LOAD PATH: The structure shall contain one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (Tier 2: Sec. 4.3.1.1). *See Building Description.*
C	NC	N/A	MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure (Tier 2: Sec. 4.3.1.3). *There are no mezzanines in the structure.*
C	NC	N/A	WEAK STORY: The strength of the lateral-force-resisting-system in any story shall not be less than 80% of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.3.2.1). *There is no story with a lateral strength less than 80% of an adjacent story*
C	NC	N/A	SOFT STORY: The stiffness of the lateral-force-resisting-system in any story shall not be less than 70% of the stiffness in an adjacent story above or below, or less than 80% of the average stiffness of the three stories above or below for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.3.2.2). *The stiffness of the lateral-force resisting system in any story is not less than 70% of the stiffness in an adjacent story above or below or less than 80% of the average stiffness of the three stories above or below*
C	NC	N/A	GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses (Tier 2: Sec. 4.3.2.3). *There are no changes in the horizontal dimension of the lateral-force-resisting system.*
C	NC	N/A	VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation (Tier 2: Sec. 4.3.2.4). *All of the columns and shear walls are continuous to the foundation.*
C	NC	N/A	MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.3.2.5). *There are no changes in effective mass more than 50% from one story to the next.*
C	NC	N/A	TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.3.2.6). *The center of rigidity and the center of mass coincide due to the symmetry of the structure.*

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

C NC N/A POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used (Tier 2: Sec. 4.3.3.5). *None used in building.*

C NC N/A CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8" for Life Safety and 1/16" for Immediate Occupancy, shall not be concentrated in one location, and shall not form an X pattern (Tier 2: Sec. 4.3.3.9).

Lateral-Force-Resisting System

C NC N/A COMPLETE FRAMES: Steel or concrete frames classified as secondary components shall form a complete vertical load carrying system (Tier 2: Sec. 4.4.1.6.1). *See building description.*

C NC N/A REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.1.1). *There are two shear walls in the transverse direction.*

C NC N/A SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 100 psi or $2(f'_c)^{1/2}$ for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.2.1). *See quick checks section for calculations,* $v_{avg} = 75$ *psi < 100 psi*

C NC N/A REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18" for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.2.2). *See quick checks section for check of compliance*

Connections

C NC N/A TRANSFER TO SHEAR WALLS: Diaphragms shall be reinforced and connected for transfer of loads to the shear walls for Life Safety and the connections shall be able to develop the shear strength of the walls for Immediate Occupancy (Tier 2: Sec. 4.6.2.1). *The diaphragms are reinforced and doweled into walls and longitudinal beams.*

C NC N/A WALL REINFORCING: Walls shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the strength of the walls for Immediate Occupancy (Tier 2: Sec. 4.6.3.5). *The walls are doweled into the foundation*

Supplemental Structural Checklist for Building Type C2: Concrete Shear Wall Buildings with Stiff Diaphragms (FEMA 310, Section 3.7.9S)

Lateral-Force-Resisting System

C	NC	N/A	DEFLECTION COMPATIBILITY: Secondary components shall have the shear capacity to develop the flexural strength of the elements for Life Safety and shall have ductile detailing for Immediate Occupancy (Tier 2: Sec. 4.4.1.6.2). *See quick checks section for check of compliance*
C	NC	N/A	FLAT SLABS: Flat slabs/plates classified as secondary components shall have continuous bottom steel through the column joints for Life Safety. Flat slabs/plates shall not be permitted for the Immediate Occupancy Performance Level (Tier 2: Sec. 4.4.1.6.3).
C	NC	N/A	COUPLING BEAMS: The stirrups in all coupling beams over means of egress shall be spaced at or less than d/2 and shall be anchored into the core with hooks of 135° or more for Life Safety and Immediate Occupancy. In addition, the beams shall have the capacity in shear to develop the uplift capacity of the adjacent wall for Immediate Occupancy (Tier 2: Sec. 4.4.2.2.3). *No coupling beams*
C	NC	N/A	OVERTURNING: All shear walls shall have aspect ratios less than 4 to 1. Wall piers need not be considered. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.2.2.4). *This building is being evaluated for the Life Safety Performance Level only*
C	NC	N/A	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2.0, the boundary elements shall be confined with spirals or ties with spacing less than 8 d_b. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.2.2.5). *This building is being evaluated for the Life Safety Performance Level only*
C	NC	N/A	REINFORCING AT OPENINGS: There shall be added trim reinforcement around all wall openings. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.2.2.6). *This building is being evaluated for the Life Safety Performance Level only*
C	NC	N/A	WALL THICKNESS: Thickness of bearing walls shall not be less than 1/25 the minimum unsupported height or length, nor less than 4". This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.4.2.2.7). *This building is being evaluated for the Life Safety Performance Level only*

Diaphragms

C	NC	N/A	DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints (Tier 2: Sec. 4.5.1.1). *There are no split-level floors.*

C NC N/A OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length for Life Safety and 15% of the wall length for Immediate Occupancy (Tier 2: Sec. 4.5.1.4). *There are no openings immediately adjacent to the shear walls or openings greater than 25% of the wall length.*

C NC N/A PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.5.1.7). *This building is being evaluated for the Life Safety Performance Level only*

C NC N/A DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only (Tier 2: Sec. 4.5.1.8). *This building is being evaluated for the Life Safety Performance Level only*

Connections

C NC N/A LATERAL LOAD AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy (Tier 2: Sec. 4.6.3.10). *No pile foundations used in structure*

Quick Checks:

The pseudo lateral force is needed to complete some of the quick check statements (shearing stress check of columns, axial stress due to overturning in columns, and shear stress check of shear walls) triggered by the Tier 1 checklists.

Determination of Pseudo Lateral Force (per FEMA 310 Section 3.5.2.1)

Building Period (per FEMA 310 Section 3.5.2.4)

$T = C_t h_n^{3/4}$ (FEMA 310 Eq. 3-7)

h_n = 30.6' (9.3 m)

Transverse Direction:
C_t = 0.020 (Reinforced Concrete Shear Walls)
$T = 0.020(30.6)^{3/4} = 0.26$ seconds

Longitudinal Direction:
C_t = 0.030 (Reinforced Concrete Moment Frames)
$T = 0.030(30.6)^{3/4} = 0.39$ seconds

Determine Building Seismic Weight

	Unit Weight (psf)	Unit Wall Weight (plf)	Total Area (ft.^{2})	Total Wall Length (ft.)	Total Weight (kips)
Roof Diaphragm Level					
Weight of Roof	120.0	---	4641	---	557.2
Exterior Longitudinal Walls	---	378	---	210	79.4
Exterior Transverse Walls	---	500	---	79.3	39.7
Total Roof Tributary Weight					***676.2***
Third Floor Diaphragm Level					
Weight of Floor	140.6	---	4641	---	652.5
Exterior Longitudinal Walls	---	602	---	210	126.4
Exterior Transverse Walls	---	1000	---	79.3	79.3
Total Third Floor Tributary Weight					***858.2***
Second Floor Diaphragm Level					
Weight of Floor	140.6	---	4641	---	652.8
Exterior Longitudinal Walls	---	602	---	210	126.4
Exterior Transverse Walls	---	1000	---	79.3	79.3
Total Second Floor Tributary Weight					***858.5***

Total Seismic Weight of Building ***2393***

10645 kN

Determine Mapped Spectral Acceleration (per FEMA 310 Section 3.5.2.3.1)

$S_a = S_{D1} / T$, but Sa shall not exceed S_{DS} (FEMA 310 Eq. 3-4)

$S_{DS} = 0.82$ $S_{D1} = 0.42$ (previously calculated in Preliminary Determinations Section)

Transverse Direction:
$S_a = 0.42 / 0.26 = 1.62 > S_{DS} = 0.82$, use $S_a = 0.82$

Longitudinal Direction:
$S_a = 0.42 / 0.39 = 1.08 > S_{DS} = 0.82$, use $S_a = 0.82$

Determine Pseudo Lateral Force

$V = CS_aW$ (FEMA 310 Eq. 3-1)

Transverse Direction:
C = 1.1 (Shear wall building with 3 stories) (FEMA 310 Table 3-4)
V = (1.1)(0.82)(2393 kips) = 2158 kips (9599 kN)

Longitudinal Direction:
C = 1.0 (Moment frame building with 3 stories) (FEMA 310 Table 3-4)
V = (1.0)(0.82)(2393 kips) = 1962 kips (8727 kN)

Determine Story Shear Forces (per FEMA 310 Section 3.5.2.2)

$$V_j = \left(\frac{n+j}{n+1}\right)\left(\frac{W_j}{W}\right)V$$ (FEMA 310 Eq. 3-3)

	j	W_i (kips)	Transverse V_i (kips)	Longitudinal V_i (kips)
Third Story	3	676	915	832
Second Story	2	1534	1730	1573
First Story	1	2393	2158	1962

1 kip = 4.448 kN

Longitudinal Direction: Concrete Moment Frames

Shearing Stress Check of Columns (per FEMA 310 Section 3.5.3.2)

"SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or $2(f'_c)^{1/2}$ for Life Safety and Immediate Occupancy."

The average shear stress, v_{avg}, in the columns of concrete frames shall be computed as:

$$v_{avg} = \frac{1}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right)$$ (FEMA 310 Eq. 3-10)

$2\sqrt{f'_c} = 2\sqrt{3000} = 110\text{psi}$ (758 kPa) > 100 psi (689 kPa), use 100 psi for allowable stress

A_c = 10(23.75" x 12") + 4(18" x 12") = 3714 in^2 (2.40 m^2) ((Total cross-sectional column area)

$$v_{avg} = \frac{1}{2}\left(\frac{14}{14-2}\right)\left(\frac{1962\text{k}}{3714\,\text{in}^2}\right)\left(\frac{1000\text{ lb}}{1\text{kip}}\right) = 308\,\text{psi (2122 kPa)} > 100\text{ psi (689 kPa)},\ \textbf{No Good}$$

Axial Stress Due to Overturning (per FEMA 310 Section 3.5.3.6):

"AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10f'_c$ for Life Safety and Immediate Occupancy. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30f'_c$ for Life Safety and Immediate Occupancy."

Check exterior perimeter frame column (12" x 18" = 216 in^2, 1394 cm^2)

The axial stress of columns subjected to overturning forces, p_{ot}, shall be calculated as:

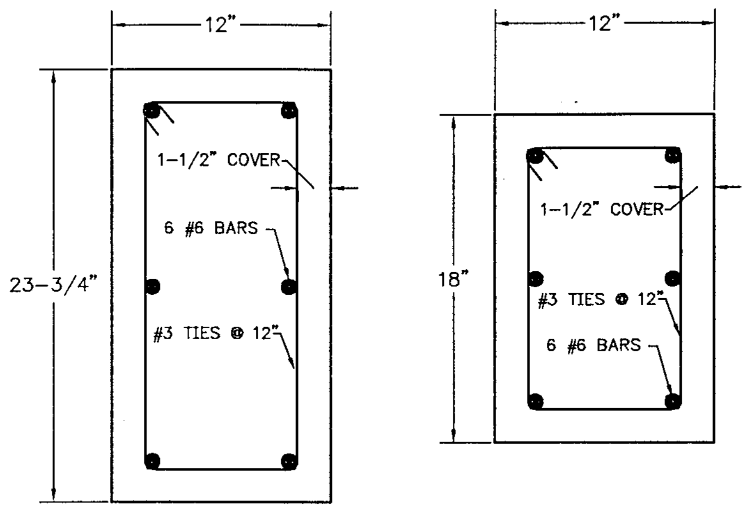
$$p_{ot} = \frac{1}{m}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right) = \frac{1}{2}\left(\frac{2}{3}\right)\left(\frac{(1962\,\text{k})(30.6')}{(117')(2\text{ frames})}\right) = 86\,\text{kips (383 kN)} \qquad \text{(FEMA 310 Eq. 3-14)}$$

Axial Stress = p_{ot} / Area = (86 kips)(1000 lb / kip) / (216 in^2) ≈ 400 psi (2756 kPa)
Allowable Stress = 0.3 f_c' = (0.3)(3000 psi) = 900 psi (6201 kPa)> 400 psi, **OK**

No Shear Failures of Columns

"NO SHEAR FAILURES: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns"

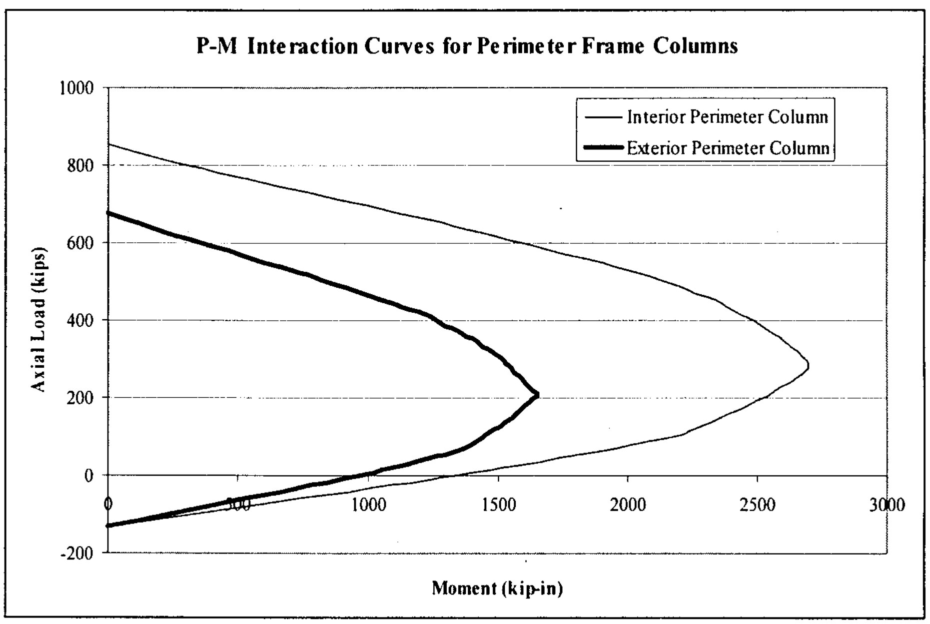
The probable flexural moment strength, M_{pr}, of typical interior columns of perimeter frame is calculated based on the nominal capacity (with a capacity reduction factor, ϕ, equal to unity), and with reinforcement exhibiting strain hardening to an ultimate strength = $1.25f_y$ = 1.25(40ksi) = 50 ksi (345 MPa) (FEMA 310 Section 4.2.4.4 gives guidance on determination of member component strengths.) Calculate maximum column shear, V_e, associated with formation of plastic hinges at both ends of the column (M_{pr} at each end). Compare V_e with the column shear capacity, ϕV_n. Note: This mechanism may not form due to strong column-weak beam condition but gives an upper bound to the shear demand.



INTERIOR PERIMETER COLUMN EXTERIOR PERIMETER COLUMN

NOTE:
f_c' = 3000 psi
f_y = 40 ksi
1 in = 25.4 mm
1 psi = 6.89 kPa

The columns are checked at the base level. The axial loads are highest in the first story columns, increasing their flexural capacities. The higher flexural capacities increase the flexural-shear demand on the columns.



Interior Perimeter Columns

The flexural strengths of the columns and beams are calculated with the computer program BIAX.

M_{pr} of interior perimeter column (from BIAX) = 190 k-ft (258 kN-m) (@ Axial load of 130 kips = gravity load on column; calculation of axial loads not shown)

$V_e = 2M_{pr} / L$ = 2(190 kip-ft) / (9')= 42.2 kips (188 kN) (Assuming a clear column height = 9')

(Note: This shear value, 42.2 kips, is greater than the column shear determined for the joint shear, 20 kips, in the Tier 2 analysis to follow. The 20 kips value corresponds to the formation of a beam hinging mechanism. At this point, it is unknown whether this condition is satisfied or not. Therefore, use the 42.2 kips value to be conservative.)

Determine Column Shear Capacity, ϕV_n (per ACI 318)

For this check, ϕ=1.0 (No strength reduction factor)

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

$$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d \qquad \text{(ACI 318 Eq. 11-4)}$$

$$= 2\left(1+\frac{130000}{2000(12"\text{x}23.75")}\right)\sqrt{3000}(12")(21.5") = 34.7\text{kips}\ \ (126\text{ kN})$$

$$V_s = \frac{A_v f_y d}{s} = \frac{2(0.11\text{in}^2)(40\text{ksi})(23.75"-1.5"-.375"-.75"/2)}{12"} = 15.8\text{kips}\ \ (70.3\text{ kN})\text{(ACI 318 Eq. 11-15)}$$

$V_n = V_c + V_s$ = 34.7 k + 15.8 k = 50.5 kips (225 kN) > V_e = 42.2 kips (188 kN), **OK**

Exterior Perimeter Columns

M_{pr} of exterior column (from BIAX) = 115 k-ft (156 kN-m) (@ Axial load of 70 kips, calculation of axial loads not shown)

$V_e = 2M_{pr} / L$ = 2(115 kip-ft) / (9')= 26 kips (116 kN) (Assuming a clear column height = 9')

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

$$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d \qquad \text{(ACI 318 Eq. 11-4)}$$

$$= 2\left(1+\frac{70000}{2000(12"\text{x}18")}\right)\sqrt{3000}(12")(15.5") = 23.7\text{kips}\ \ (105\text{ kN})$$

$$V_s = \frac{A_v f_y d}{s} = \frac{2(0.11\text{in}^2)(40\text{ksi})(18"-1.5"-.375"-.75"/2)}{12"} = 11.6\text{kips}\,(52\text{ kN}) \qquad \text{(ACI 318 Eq. 11-15)}$$

$V_n = V_c + V_s$ = 23.7 k + 11.6 k = 35.3 kips (157 kN) > V_e = 26 kips (116 kN), **OK**

Strong Column / Weak Beam

Compare the sum of the beam moment capacities to that of the column moment capacities.

$$\frac{\sum M_{pr(col)}}{\sum M_{pr(beam)}} \geq 1.2$$

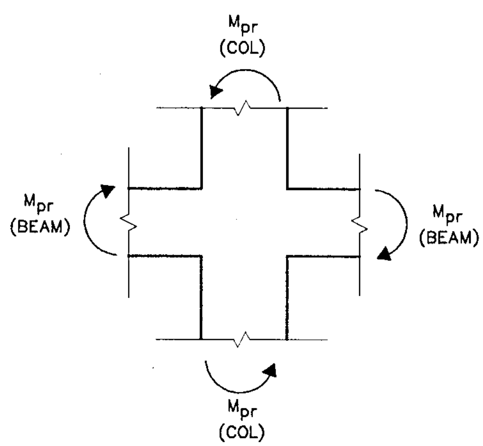
The moment capacities are determined assuming the steel exhibits elasto-plastic behavior with an expected yield strength of $f_{ye} = 1.25\ f_y = 1.25(40\ \text{ksi}) = 50$ ksi and an ultimate concrete strain capacity, $\varepsilon_{cu} = 0.003$. The beam and column capacities were determined using the BIAX computer program. The bottom steel in the beams is not continuous through the beam-column joints, however, for this check it is conservatively assumed that the bottom steel is able to be fully developed. The columns at the roof level are not checked since a column mechanism at the roof level will not lead to collapse of the structure. The column flexural capacities are calculated at an axial load equal to the tributary gravity loads.

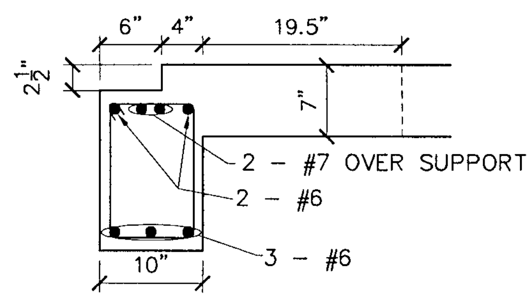
3rd floor level

Assume that the slab contributes to the moment strength of the beams. The effective compression zone width is determined per ACI 318 Section 8.10.3; The effective compression zone width shall not exceed:
(a) 1/12 the span length of the beam = 1/12 (19.5')(12"/1') = 19.5" (495 mm) (governs)
(b) six times the slab thickness = 6(7") = 42"
(c) one-half the clear distance to the next web = (1/2)(39'-8")(12"/1') = 238"

The beam flexural strengths were determined assuming no axial load in members.

Positive Moment Strength = 1237 kip-in (140 kN-m)
Negative Moment Strength = 1184 kip-in (134 kN-m)
M_{beam} = (1237 kip-in) + (1184 kip-in) = 2421 kip-in (274 kN-m)





3rd FLOOR BEAM TYPE 'A'

Flexural strength at bottom of column at 3rd floor level.

The column flexural capacity is determined assuming the members carry gravity loads from the roof. (See "No shear failure" section for column cross-section).

$M_{col@bottom}$ = 1699 kip-in (192 kN-m) (at an axial load of 31 kips)

The column flexural strength below the joint is higher due to the increased column axial loads imposed by the floor slab.

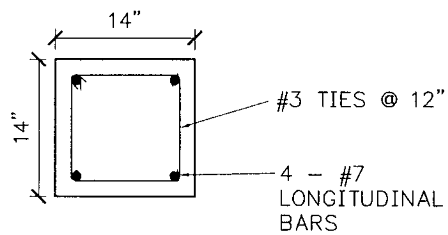
$M_{col@top}$ = 1993 kip-in (225 kN-m) (at an axial load of 75 kips)

$1.2\Sigma M_{beam}$ = 1.2(2421 kip-in) = 2905 kip-in (328 kN-m)
ΣM_{col} = (1699 kip-in) + (1993 kip-in) = 3692 kip-in (417 kN-m) > 2905 kip-in (328 kN-m), **OK**

Deflection Compatibility:

"DEFLECTION COMPATIBILITY: Secondary components shall have the shear capacity to develop the flexural strength of the elements for Life Safety and shall have ductile detailing for Immediate Occupancy (Tier 2: Sec. 4.4.1.6.2)."

The interior gravity columns are checked to determine if they have the shear capacity available to develop the flexural strength of the elements. Calculate probable flexural moment strength, M_{pr}, of typical interior columns with capacity reduction factor, ϕ, equal to unity and with reinforcement exhibiting strain hardening to an ultimate strength = $1.25f_y$ = 1.25(40ksi) = 50 ksi. Calculate maximum column shear, V_e, associated with formation of plastic hinges at both ends of the column (M_{pr} at each end). Compare V_e with the column shear capacity, ϕV_n. The program BIAX was used to calculate the flexural capacity of the column.



Axial Load on Column = 200 kips (890 kN)
From BIAX, M_p = 1248 kip-in = 104 kip-ft (141 kN-m)
$V_e = 2M_p / L$ = 2(104 kip-ft) / (9'-4") = 22.3 kips (99 kN) (Assuming a clear column height = 9'-4")

Determine Column Shear Capacity, ϕV_n (per ACI 318)

$\phi V_n = \phi(V_c + V_s)$ with ϕ=1.0 (ACI 318 Eq. 11-2)

$$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f'_c}\,b_w d \qquad \text{(ACI 318 Eq. 11-4)}$$

$$= 2\left(1+\frac{200000}{2000(14"\text{x}14")}\right)\sqrt{3000}(14")(11.5") = 26.6\text{kips (118 kN)}$$

$$V_s = \frac{A_v f_y d}{s} = \frac{2(0.11\text{in}^2)(40\text{ksi})(14"-1.5"-.375"-.875"/2)}{12"} = 8.6\text{kips} \qquad \text{(ACI 318 Eq. 11-15)}$$

$V_n = V_c + V_s$ = 26.6 k + 8.6 k = 35.2 kips (157 kN) > V_e = 22.3 kips (99 kN), OK

Transverse Direction: Reinforced Concrete Shear Walls

Shear stress in shear walls (per FEMA 310 Section 3.5.3.3)

"SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 100 psi or $2(f'_c)^{1/2}$ for Life Safety and Immediate Occupancy."

The average shear stress in shear walls, v_{avg}, shall be calculated as:

$$v_{avg} = \frac{1}{m}\left(\frac{V_j}{A_w}\right) \qquad \text{(FEMA 310 Eq. 3-11)}$$

The walls are checked at the base level;

V_j = 2158 kips (9599 kN) (base shear in the transverse direction)
A_w = 2(40.67'-3.33')(12"/1')(8") = 7169 in^2 (4.63 m^2)
m = 4.0 (FEMA 310 Table 3-7)

$$v_{avg} = \frac{1}{4}\left(\frac{2158\text{kips}}{7169\text{in}^2}\right)\left(\frac{1000\text{ lb}}{\text{kip}}\right) = 75\text{psi (517 kPa)} < 100 \text{ psi (689 kPa), OK}$$

Reinforcing steel

"REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18" for Life Safety and Immediate Occupancy."

The 8" thick concrete walls are reinforced with #5 bars at 15" each way.

A_b = 0.31 in.^{2}

$\rho_v = \rho_h$ = (0.31 in.^{2}) / (8")(15") = 0.0026 > 0.0025, OK

Spacing < 18", OK

3. *Evaluate screening results (Summary of Tier 1 deficiencies)*

ADJACENT BUILDINGS: *There is an adjacent building 2" from this structure. However, both structures are the same height and have matching floors. Pounding damage is likely to result only in nonstructural damage. Therefore, the small separation is not a concern.*

SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or $2(f'_c)^{1/2}$ for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.1.4.1). *See Quick Checks section following checklists for calculation of shearing stress demand on columns of moment frames. Demand = 276 psi > Allowable = 100 psi*

BEAM BARS: *The bottom longitudinal bars are not continuous through the joints. Therefore, the beams will be unable to develop their full positive strength at the joint interface.*

COLUMN BAR SPLICES: *The column longitudinal bar splices extend for a length of 20 d_b only. This is less than the required 35 d_b splice length. However, the column bars are spliced at midheight of the second floor. This area of the columns is not likely to see inelastic actions.*

COLUMN TIE SPACING: *The column ties are spaced at 12". This large spacing leads to lack of concrete confinement, increases the buckling length of the longitudinal steel and weakens laps. These conditions lead to reduced column ductility.*

STIRRUP SPACING: *The beam stirrups are spaced at 12". This large spacing leads to lack of concrete confinement and increases the buckling length of the longitudinal steel. These conditions lead to reduced beam ductility.*

JOINT REINFORCING: *There is no joint reinforcement at the beam-column joints.*

The above deficiencies indicate that the concrete moment frames are lacking ductile detailing and may not be able to carry gravity loads after being subjected to several cycles of inelastic deformations. The columns also fail the shear stress check. However, the structure may have sufficient capacity to resist the imposed seismic loading; therefore, the building will be subjected to a Tier 2 analysis to investigate the above deficiencies and determine if the building is acceptable or needs rehabilitation. Buildings designated for a Tier 2 evaluation based on results from a Tier 1 screening may be evaluated by a "deficiencies only" or "full-building evaluation" (per Section 5.1.b.(1)). FEMA 310 Section 3.4 states that for buildings not requiring a Full-Building Tier 2 evaluation, a Deficiency-Only Tier 2 evaluation may be conducted if potential deficiencies are identified by the Tier 1 evaluation. FEMA 310 Table 3-3 gives guidance on when a Full-Building Tier 2 evaluation is to be conducted. For this structure, the Tier 1 investigation identified the potential deficiencies, and a "Deficiencies Only" Tier 2 evaluation is conducted.

All of the statements pertaining to the concrete shear walls are found to be true. Therefore, the shear wall system in the transverse direction is acceptable.

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) (from Table 5-1)

1. Select appropriate analytical procedure. The building is analyzed using the linear static procedure described in Section 4.2.2 of FEMA 310 for ease of calculations. Limitations on the use of this procedure are found in paragraph 5-2 of TI 809-04.

2. Determine applicable ground motion. For Seismic Use Group I and the Life Safety Performance Level the ground motion specified in Table 2-4 is 2/3 MCE.

3. Perform structural analysis. The steps required for the LSP are laid out in Section 4.2.2.1 of FEMA 310.

- *Develop a mathematical model of the building in accordance with Sec. 4.2.3 of FEMA 310.*

The building is analyzed using a two-dimensional model with rigid diaphragms. Torsional effects resulting from the eccentricity between the centers of mass and the centers of rigidity are sufficiently small so as to be ignored. Therefore, only an accidental torsion of 5% of the horizontal dimension is considered. This analysis only considers the moment frames as the transverse walls were determined to have sufficient capacity based on the Tier 1 analysis. The walls are much stiffer than the frames. Therefore, it is assumed that the torsional forces are resisted entirely by the walls with no torsional forces resisted by the moment frames. Torsional effects are therefore neglected for the Tier 2 analysis of the concrete moment frames.

- *Calculate the pseudo lateral force in accordance with Sec. 4.2.2.1.1 of FEMA 310.*

(1) Period. The period in the longitudinal direction was determined previously to be 0.39 seconds (see Quick Checks section).

(2) Pseudo Lateral Force. The pseudo lateral force in the longitudinal direction was determined to be 1962 kips (8727 kN) (see Quick Checks section).

- *Distribute the lateral forces vertically in accordance with Sec. 4.2.2.1.2 of FEMA 310.*

The pseudo lateral force shall be distributed vertically in accordance with the equations:

$$F_x = C_{vx} V \qquad \text{(FEMA 310 Eq. 4-2)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(FEMA 310 Eq. 4-3)}$$

where $k = 1.0$ for a building period of 0.39 seconds.

Level	w_x (kips)	h_x (ft)	$w_x h_x$ (kip-ft)	C_{vx}	F_x (kips)	
Roof	676.2	30	20286	0.441	865	(3848 kN)
3rd Floor	858.2	20	17165	0.373	732	(3256 kN)
2nd Floor	858.5	10	8585	0.186	366	(1628 kN)
		$\Sigma =$	46036	1.0		

- *Determine the building and component forces and displacements.*

Modeling Assumptions:

- The building is modeled assuming rigid diaphragm action. (Equal deflections at top of each column at a particular level)

- Modulus of Elasticity of Concrete; (use the ACI 318 method for determining E)
  $E = 57000\sqrt{f_c^{'}} = 57000\sqrt{3000} = 3.12 \times 10^3 \text{ksi}$ (ACI 318 Sec. 8.5.1)
  Concrete strength indicated on drawings, $f_c^{'} = 3000$ psi (20.7 MPa)

- Effective Concrete Stiffness Values
  Stiffness of reinforced concrete components depends on material properties (including current condition), component dimensions, reinforcement quantities, boundary conditions, and stress levels. The calculation of a member's effective stiffness directly from principles of basic mechanics is impractical in most cases. FEMA 273 provides guidance for calculation of member stiffness for evaluation of concrete structures in Section 6.4.1.2. Table 6-4 of FEMA 273 provides effective stiffness values for a variety of reinforced concrete components, and is used here for ease of calculations.

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 = $0.4E_cA_w$

- Component Gravity Loads
  $Q_G = 1.2\ Q_D + 0.5\ Q_L + 0.2\ Q_S$ (Eq. 7-1)
  FEMA 310 Section 4.2.4.2 states that $Q_S = 0.0$ where the design snow load is less than 30 psf. (Note: Eq. 7-1 is different than FEMA 310 Equation 4-6. This document uses the gravity load combination specified in ASCE 7 rather than the FEMA equation.)

  $Q_G = 0.9\ Q_D$ (FEMA 310 Eq. 4-7)

  Q_D = Dead load
  $Q_{D\ roof}$ = 1568 plf (22.9 kN / m)
  $Q_{D\ 3rd}$ = 1997 plf (29.1 kN / m)
  $Q_{D\ 2nd}$ = 1997 plf (29.1 kN /m)

  Q_L = Design live load
  $Q_{L\ roof}$ = 198 plf (2.89 kN / m)
  $Q_{L\ 3rd}$ = 496 plf (7.24 kN / m)
  $Q_{L\ 2nd}$ = 496 plf (7.24 kN / m)

– Q_E = Earthquake load

Actions shall be classified as either deformation-controlled or force-controlled. Guidance for classifying components is given in FEMA 310 Section 4.2.4.3. Due to symmetry, each of the frames in the longitudinal direction resists ½ of the longitudinal base shear:

Deformation-controlled actions:
Actions controlled by deformations include the moment demand in the columns and beams.

$Q_{E\ roof}$ = ½(865 kips) = 433 kips
$Q_{E\ 3rd}$ = ½(732 kips) = 366 kips
$Q_{E\ 2nd}$ = ½(366 kips) = 183 kips

Force-controlled actions:
Force-controlled actions include beam and column shear, shear in joints, and column axial demand. FEMA 310 Section 4.2.4.3.2 lists two methods for determining the force-controlled demands on components (Three methods are actually specified, however, method 3 is only to be used when the pseudo lateral force is calculated using FEMA 310 Equation 3-2, which was not used for the pseudo lateral force in this design example.) Method 1 states that force-controlled actions, Q_{UF}, shall be calculated as the sum of forces due to gravity and the maximum force that can be delivered by deformation-controlled actions. This method is used to check joint shear. Method 2 states that force-controlled actions may be calculated according to:

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

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

Equation 4-9 shall be used when the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system. Equation 4-10 shall be used for all other cases. The beam and column shear forces, and the column axial loads are delivered by yielding components of the seismic framing system. For these actions Equation 4-9 is used, producing earthquake demands for these force-controlled actions equal to those for deformation controlled actions divided by the term CJ. Therefore, the force-controlled earthquake actions are evaluated as Q_E / CJ.

C = 1 in the longitudinal direction (See Quick Checks Section)
$J = 1.5 + S_{DS} < 2.5$; $J = 1.5 + 0.82 = 2.32 < 2.50$ (FEMA 310 Eq. 4-11)

$Q_{E\ roof}$ / CJ = (433 kips) / (1.0)(2.32) = 187 kips (832 kN)
$Q_{E\ 3rd}$ / CJ = (366 kips) / (1.0)(2.32) = 158 kips (703 kN)
$Q_{E\ 2nd}$ / CJ = (183 kips) / (1.0)(2.32) = 79 kips (351 kN)

<u>Analysis Results</u>

The structure was analyzed using a 2-D frame analysis with the RISA-3D computer software. Results of the analysis are tabulated below.

Deformation controlled actions:

Deformation-controlled design actions, Q_{UD}, are calculated according to:
$Q_{UD} = Q_G \pm Q_E$ (FEMA 310 Eq. 4-8)

Results from RISA 3-D computer analysis: (Only the most critical demands are shown from the load combinations $1.2Q_D + 0.5Q_L + 1.0Q_E$ and $0.9Q_D + 1.0Q_E$

Deformation-Controlled Actions

Element	Axial Load[1] (kips)	Positive Bending Demand (kip-ft)	Negative Bending Demand[2] (kip-ft)	Nominal Capacity for Positive Bending [3] (kip-ft)	Nominal Capacity for Negative Bending [3] (kip-ft)
Exterior Perimeter Columns in Tension					
1st Story	-62	682	682	24	24
2nd Story	-41	300	300	38	38
3rd Story	-22	302	302	51	51
Exterior Perimeter Columns in Compression					
1st Story	174	707	707	125	125
2nd Story	113	361	361	114	114
3rd Story	53	386	386	95	95
Typical Interior Perimeter Columns					
1st Story	136	1554	1554	178	178
2nd Story	88	700	700	153	153
3rd Story	41	687	687	121	121
Typical Beam at End Bay[4]					
1st Story	---	452	-302	81	-81
2nd Story	---	437	-286	81	-81
3rd Story	---	411	-288	53	-59
Typical Beam at Interior Bay[4]					
1st Story	---	452	-300	86	-80
2nd Story	---	438	-286	86	-80
3rd Story	---	412	-286	59	-59

Notes:

1. Axial load is neglected for check of beams in frames
2. Due to symmetry, negative and positive bending demands on columns are equal
3. Nominal bending capacities are calculated assuming the given axial load is present on the member. Capacities calculated using BIAX program. The columns are spliced at midheight of the second floor. The moments are low at the midheight of the columns so no reduction in flexural capacity is considered due to the short splice length.
4. The beam bottom steel is embedded into the joints for only 11 inches. This is less than the required tensile development length for the bars. Therefore, the flexural strength of the bottom bars at the joints is limited to a fraction of the specified yield strength of 40 ksi. FEMA 273 Section 6.4.5 gives guidance for this situation. The limit to the bottom steel stress from FEMA 273 Eq. 6-2 becomes:

$$f_s = \frac{2500}{d_b} l_e \le f_y \text{, where } l_e = 11\text{"}$$

#6 bars: $f_s = 37$ ksi
#8 bars: $f_s = 27.5$ ksi

Force-Controlled Actions

Element	Axial Load Demand (kips)	Shear Demand[3] (kips)	Nominal Axial Load Capacity (kips)[1]	Nominal Shear Capacity (kips)[2]
Exterior Perimeter Columns in Tension				
1st Story	-3	29	-95	12
2nd Story	-3	22	-95	12
3rd Story	-3	10	-95	12
Exterior Perimeter Columns in Compression				
1st Story	112	37	520	38
2nd Story	72	35	520	36
3rd Story	33	25	520	34
Typical Interior Perimeter Columns				
1st Story	135	71	660	50
2nd Story	87	58	660	48
3rd Story	40	30	660	46
Typical Beam at End Bay				
1st Story	---	40	---	10
2nd Story	---	40	---	10
3rd Story	---	35	---	12
Typical Beam at Interior Bay				
1st Story	---	40	---	10
2nd Story	---	40	---	10
3rd Story	---	35	---	12

1 kip = 4.448 kN

Notes:
1. Nominal Axial Load Capacity: The axial load capacities of the columns are calculated using ACI 318 equations (no strength reduction factors are used since the evaluation uses the nominal strength, not the reduced design strength).

$P_n = 0.80\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$ (for compression members) (ACI 318 Eq. 10-2)

$P_n = f_s A_{st}$ (for tension members)

where f_s is reduced from f_y to account for short lap splices.
The longitudinal column reinforcement is lapped 15". The development length of #6 bars is 16.5" (per ACI 318 Chapter 12).
$f_s = (l_b / l_d)(f_y) = (15" / 16.5")(40 \text{ ksi}) = 36 \text{ ksi}$ (248 MPa) (FEMA 273 Eq. 6-1)

Exterior perimeter columns: (12" x 18" with 6- #6, $A_s = 6 \times 0.44\text{in.}^2 = 2.64 \text{ in.}^2$, $A_g = 12" \times 18" = 216\text{in.}^2$)
Compression: $P_n = 0.80[0.85(3\text{ksi})(216\text{in}^2 - 2.64\text{in.}^2) + (40\text{ksi})(2.64\text{in}^2)] = 520$ kips (2330 kN)

Tension: P_n = (36 ksi)(2.64 in^2) = 95 kips (423 kN) (tension)

Interior perimeter columns: (12" x 23.75" with 6-#6 bars, A_s = 2.64 in.^{2}, A_g = 12" x 23.75" = 285 in.^{2})
Compression: P_n = 0.80[0.85(3ksi)(285in^2-2.64in.^{2}) + (40ksi)(2.64in^2)] = 660 kips (2936 kN)
Tension: No interior perimeter columns in tension

2. Nominal Shear Capacity: The shear capacities of the columns are calculated using ACI 318 equations (no strength reduction factors are used since the evaluation uses the nominal strength, not the reduced design strength).

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

For members subject to axial compression,

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\,b_w d$$ (ACI 318 Eq. 11-4)

For members subject to significant axial tension, (ACI 318 Sec. 11.3.1.3)
V_c = 0 kips
For members subject to shear and flexure only,

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

(Note: The above equations are used to determine the contribution of concrete to the shear capacity of the frame members. V_c should be taken = 0 in locations of potential plastic hinging in the beam members.)

$V_s = A_v f_y d / s$ (ACI 318 Eq. 11-15)
All members tied with #3 ties @ 12", A_s = 2(0.11in.^{2}) = 0.22in.^{2}

Exterior perimeter columns: (d = 18" – 1.5" – 0.375" – ½(6/8)" = 15.75")
V_s = (0.22 in.^{2})(40 ksi)(15.75") / 12" = 11.6 kips (51.6 kN)
Compression: Use ACI 318 Eq. 11-4 to determine V_c with given axial load (calcs not shown)
Tension: V_c = 0 kips

Interior perimeter columns: (d = 23.75" – 1.5" – 0.375" – ½(6/8)" = 21.5")
V_s = (0.22 in.^{2})(40 ksi)(21.5") / 12" = 15.8 kips (70.3 kN)
Compression: Use ACI 318 Eq. 11-4 to determine V_c with given axial load (calcs not shown)
Tension: V_c = 0 kips

Beams at roof level: (d = 18.125" – 1.5" – 0.375" – ½(6/8)" = 15.88")
V_s = (0.22 in.^{2})(40 ksi)(15.88") / 12" = 11.6 kips (51.6 kN)
V_n = 11.6 kips + 0 = 11.6 kips (51.6 kN)

Beams at 3rd and 2nd floor levels: (d = 15.625" – 1.5" – 0.375" – ½(6/8)" = 13.38")
V_s = (0.22 in.^{2})(40 ksi)(13.38") / 12" = 9.8 kips (43.6 kN)
V_n = 9.8 kips + 0 = 9.8 kips (43.6 kN)

3. The column shear forces are from the load combination $Q_{UF} = Q_G \pm \frac{Q_E}{CJ}$. The values shown in the table are different from the column shear values shown below for the determination of joint shear forces. The values in the table are more conservative and are used for the evaluation of the column shear.

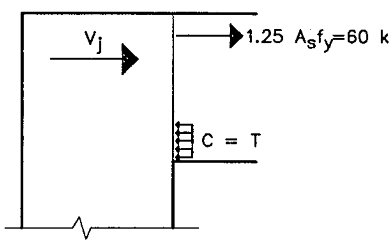
Joint Shear Forces

Joint shear forces are calculated based on development of flexural plastic hinges in adjacent framing members. Therefore, the longitudinal reinforcing steel in the beams is assumed to be stressed to $1.25f_y$. (Note: The bottom longitudinal bars are not capable of developing their full tensile capacity due to the short embedment length into the beam-column joints. However, it is conservatively assumed that they are capable of developing the 1.25 f_y value.) Calculation of the beam capacities, Mp_{beam}, were done using the computer program BIAX.

Shear strength of joint: $Q_{cl} = \lambda\gamma A_j\sqrt{f'_c}$, where: (FEMA 310 Sec. 4.4.1.4.13)

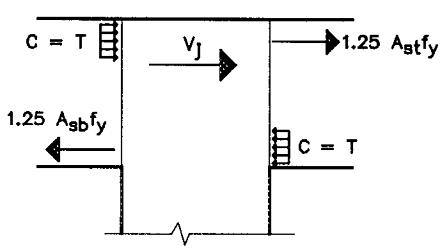
λ = 1.0 for normal weight concrete
γ = 10 for interior joints without transverse beams
γ = 6 for exterior joints without transverse beams
γ = 4 for corner joints
These γ values correspond to $\rho'' < 0.003$ since there is no transverse reinforcement in the beam-column joints.

Typical exterior joint at roof level: (A_{st} = 2-#6 and 1-#5 = 1.19 in.^{2})



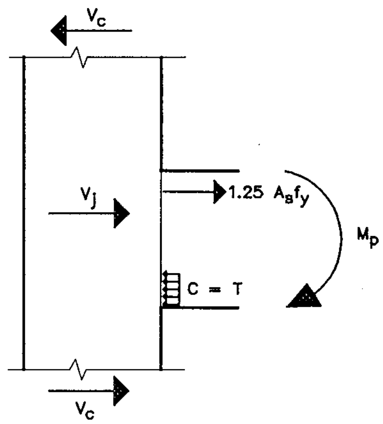
$V_j = 1.25A_{st}f_y = 1.25(1.19\text{in.}^2)(40\text{ksi}) = 60$ kips (267 kN)
$A_j = (18")(12") = 216$ in.^{2}
$Q_{cl} = (1.0)(4.0)\sqrt{3000}(216) = 47$ kips (209 kN)

Typical interior joint at roof level: (A_{st} = 1.19 in.^{2}, A_{sb} = 2-#6 = 0.88 in.^{2})



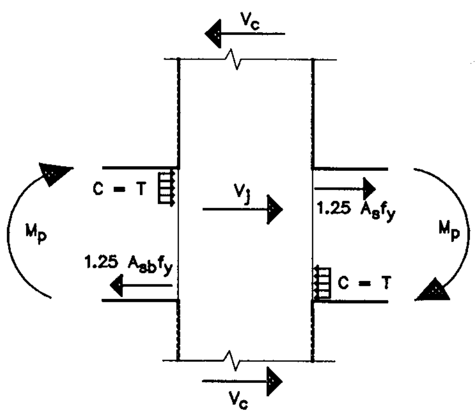
$V_j = 1.25A_{st}f_y + 1.25A_{sb}f_y = 1.25(1.19\text{in.}^2 + 0.88\text{in.}^2)$ (40 ksi) = 104 kips 463 kN)
$A_j = (23.75")(12") = 285$ in.^{2}
$Q_{cl} = (1.0)(10)\sqrt{3000}(285) = 156$kips (694 kN)

Typical ext. joint at 3rd floor level: (A_{st} = 2-#6 and 1-#10 = 2.11 in.^{2})



$V_{col} = M_{p\,beam}$ / col. clear ht. = 1203 kip-in / 120" = 10 kips (44.5 kN)
$V_j = 1.25f_y(A_{st}) - V_{col}$ = (1.25)(40 ksi)(2.11 in.^{2}) – 10 kips = 96 kips (427 kN)
A_j = (18")(12") = 216 in.^{2}
$Q_{cl} = (1.0)(6.0)\sqrt{3000}(216) = 71\text{kips}\,(316\text{ kN})$

Typical int. joint at 3rd floor level: (A_{st} =2-#6 and 2-#7 = 2.08 in.^{2}, A_{sb} = 3-#6 = 1.32 in.^{2})



$V_{col} = (Mp^+_{beam} + Mp^-_{beam})$/ col. clear ht. = (1237 kip-in + 1184 kip-in) / 120" = 20 kips (89 kN)
$V_j = 1.25f_y(A_{st} + A_{sb}) - V_{col}$ = (1.25)(40 ksi)(2.08 in.^{2} + 1.32 in.^{2}) – 20 kips = 150 kips (667 kN)
A_j = (23.75)(12") = 285 in.^{2}
$Q_{cl} = (1.0)(10)\sqrt{3000}(285) = 156\,\text{kips}\,(694\text{ kN})$

4. *Acceptance criteria*

a. *Linear static procedure, LSP.* A deficiency only evaluation is completed for the building.

(1) Deformation-controlled actions. The deformation-controlled actions include bending of the frame members. Acceptance of deformation-controlled elements is based on:

$Q_{CE} \geq \dfrac{Q_{UD}}{m}$, where: (FEMA 310 Eq. 4-12)

Q_{UD} = Action due to gravity and earthquake loading per FEMA 310 Section 4.2.4.3.1.
m = Component demand modifier from FEMA 310 Table 4-4
Q_{CE} = Expected strength of the component at deformation level under consideration.

Q_{CE} = The nominal strength of the element multiplied by 1.25 per FEMA 310 Section 4.2.4.4

Beam flexure (primary components): m = 2.5
Column flexure (primary components): m = 1.5

Element	Q_{UD}		Q_{CE}		$(Q_{UD}/m) / Q_{CE}$[2]	
	Positive Bending Demand (kipft)	Negative Bending Demand (kipft)	Expected Strength for Positive Bending[1] (kipft)	Expected Strength for Negative Bending[1] (kipft)	Acceptance for Positive Bending	Acceptance for Negative Bending
Exterior Columns in Tension, m = 1.5						
1st Story	682	682	30	30	14.9	14.9
2nd Story	300	300	47	47	4.2	4.2
3rd Story	302	302	63	63	3.2	3.2
Exterior Columns in Compression, m = 1.5						
1st Story	707	707	156	156	3.0	3.0
2nd Story	361	361	143	143	1.7	1.7
3rd Story	386	386	119	119	2.2	2.2
Typical Interior Columns, m = 1.5						
1st Story	1554	1554	222	222	4.7	4.7
2nd Story	700	700	191	191	2.4	2.4
3rd Story	687	687	151	151	3.0	3.0
Typical Beam at End Bay, m = 2.5						
1st Story	452	-302	101	-101	1.8	1.2
2nd Story	437	-286	101	-101	1.7	1.1
3rd Story	411	-288	66	-74	2.5	1.6
Typical Beam at Interior Bay, m = 2.5						
1st Story	452	-300	107	-100	1.7	1.2
2nd Story	438	-286	107	-100	1.6	1.1
3rd Story	412	-286	74	-74	2.2	1.5

Notes:
1. Expected strength, Q_{CE} = Nominal Strength x 1.25
2. An acceptance value greater than 1.0 implies non-compliance for the component action.

All of the elements lack the required strength.

(2) Force-controlled actions. The force-controlled actions include beam, column and joint shear and axial loads on the columns. Acceptance of the force-controlled components is based on:

$Q_{CN} \geq Q_{UF}$, where: (FEMA 310 Eq. 4-13)

$Q_{CN} = Q_N$ = Nominal strength of the component at the deformation level under consideration.

Q_{UF} = Action due to gravity and earthquake loading calculated in accordance with FEMA 310 Section 4.2.4.3.2.

Element	Q_{UF}		Q_{CN}		Q_{UF} / Q_{CN}	
	Axial Load Demand (kips)	Shear Demand (kips)	Nominal Axial Load Capacity (kips)	Nominal Shear Capacity (kips)	Acceptance for Axial Load[1]	Acceptance for Shear Load[1]
Exterior Columns in Tension						
1st Story	-3	29	-106	12	0.03	2.50
2nd Story	-3	22	-106	12	0.03	1.90
3rd Story	-3	10	-106	12	0.03	0.86
Exterior Columns in Compression						
1st Story	112	37	520	38	0.22	0.97
2nd Story	72	35	520	36	0.14	0.97
3rd Story	33	25	520	34	0.06	0.74
Typical Interior Columns						
1st Story	135	71	660	50	0.20	1.42
2nd Story	87	58	660	48	0.13	1.21
3rd Story	40	30	660	46	0.06	0.65
Typical Beam at End Bay						
1st Story	---	40	---	10	---	4.08
2nd Story	---	40	---	10	---	4.08
3rd Story	---	35	---	12	---	3.02
Typical Beam at Interior Bay						
1st Story	---	40	---	10	---	4.08
2nd Story	---	40	---	10	---	4.08
3rd Story	---	35	---	12	---	3.02

1 kip = 4.448 kN

Notes:
1. An acceptance value greater than 1.0 implies non-compliance for the component action.

From the above table it is seen that all the columns have sufficient axial capacity for the imposed seismic loading. The columns and beams lack sufficient shear capacity in various areas throughout the frames. This may lead to non-ductile shear failure of some elements.

Joint reinforcing (4.4.1.4.13): The joint shear demands and capacities were calculated previously to determine if the joint is able to develop the adjoining members forces.

Joint	Joint Shear Demand (kips)	Joint Shear Capacity (kips)
Typical Exterior Joint at Roof Level	60	47
Typical Interior Joint at Roof Level	104	156
Typical Exterior Joint at 3rd Floor Level	96	71
Typical Interior Joint at 3rd Floor Level	150	156

1 kip = 4.448 kN

It is seen from the Table that the beam-column joints at the exterior of the roof and 3rd floor levels lack the required shear strength.

5. *Evaluation results.* It is clear from observation of the results that the structure lacks the required strength and ductility imposed on the building from the design earthquake. The building needs major work to add strength, stiffness and ductility if it is going to be continued to be used as living quarters. Therefore, it is recommended that this structure "Definitely needs rehabilitation."

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 assessment is not in the scope of this 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 (Structural Evaluation Tier 2).

- *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:

The building lacks the required strength and ductile detailing to resist the calculated seismic demands. The column footings have a small footprint (3'-6" x 4'-6") and are also likely to cause overstress in the soil when subjected to the imposed lateral forces. Strengthening the existing footings, columns, and beams is an obvious alternative; but experience has indicated that this approach is costly and disruptive. Additionally, this type of strengthening generally makes the building stiffer and results in increased seismic demand on the existing frames.

The addition of shear walls is a better alternative as they have both high strength to resist the large lateral demands and significant stiffness to reduce drift. Shear walls may either be placed at the building interior or around the perimeter.

At this point a relative cost analysis would be completed to determine which rehabilitation strategy would be most cost efficient.

The tentative rehabilitation concept is to place shear walls around the building perimeter as it is less intrusive than placing new walls at the building interior. Perimeter walls will also provide better torsional resistance.

The intent of the rehabilitation concept is to maintain the present appearance of the exterior walls with a cast-in-place infill. New concrete shear walls could be added adjacent to both sides of the columns or in between the window openings. The space between the window openings is 88" long and the total length available for panels adjacent to the columns is only 48". Using walls between the windows requires less flexural and shear steel due to the larger moment arm and cross-sectional area.

New shear wall panels will be added at each bay on both sides of the building. This will reduce the force concentrations on the foundations below the new wall segments. The walls will extend for the full height of the building. The partial CMU infill panels must be removed in order to accommodate the walls.

3. Structural rehabilitation concept

The purpose of the concept is to define the nature and extent of the rehabilitation in sufficient detail to allow the preparation of a preliminary cost estimate. The rehabilitation strategy chosen for this building is the addition of 12 perimeter shear walls at the existing stucco panel locations.

As a first approximation for the steel required, assume that each of the 12 new shear wall segments will resist 1/12 of the forces from the Tier 2 analysis (neglecting the added weight of the walls). The demands on each wall are:

$V_r = 1/12(865 \text{ kips}) = 72 \text{ kips } (320 \text{ kN})$

$V_{3rd} = 1/12(732 \text{ kips}) = 61 \text{ kips } (271 \text{ kN})$

$V_{2nd} = 1/12(366 \text{ kips}) = 31 \text{ kips } (138 \text{ kN})$

$V_{total} = 72 \text{ k} + 61 \text{ k} + 31 \text{ k} = 164 \text{ kips } (729 \text{ kN})$

M_{base} = (30')(72) + (20')(61 k) + (10')(31 k) = 3690 kip-ft (5004 kN-m)

Assuming an m-factor of 2.5 (from TI 809-04 Table 7-2 with low axial loads and no confined boundary);
M_{trial} = 1/2.5(3690 kip-ft) = 1476 kip-ft (2001 kN-m)
Assume f_y = 1.25 x 60 ksi = 75 ksi and a 70" lever arm, the flexural steel requirements:
= (1476 kip-ft) / (75 ksi)(70" arm) = 3.4 in.^{2} (21.9 mm^2) (approximately 6-#7 bars at each wall end).

It will likely take two curtains of steel to fit all of the longitudinal steel into the wall. Try 10" (254 mm) thick walls to match the beam width of the first two floors.

Holes will need to be drilled through the beams to allow for the longitudinal wall steel to be continuous throughout the wall height.

The existing wall strip footings have minimal steel and most likely will lack the required flexural and shear capacities to resist the anticipated demands imposed by the new shear walls. Therefore, at the ground level all of the CMU infill will be removed and will be replaced with reinforced concrete. This will reduce the flexural demands at the base of the walls as they will be four feet shorter due to the footing height extension.

At this point a programming level estimate of material quantities associated with the selected structural rehabilitation concept would be developed.

4. Nonstructural evaluation assessment:

Nonstructural assessment is not in the scope of this example.

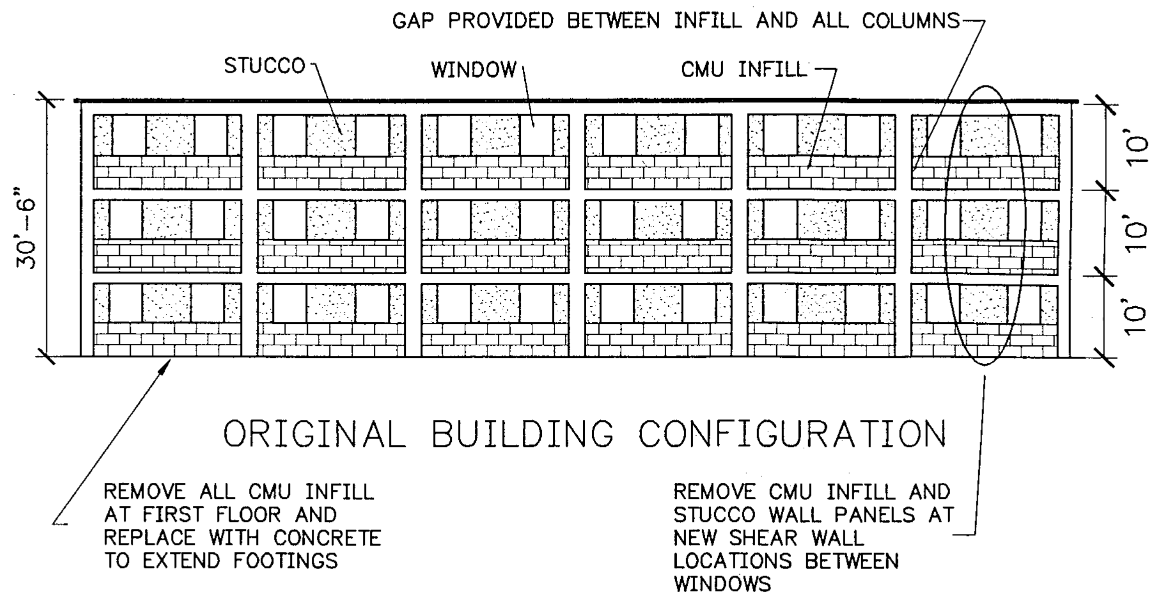
5. Nonstructural rehabilitation strategy:

Nonstructural assessment is not in the scope of this example.

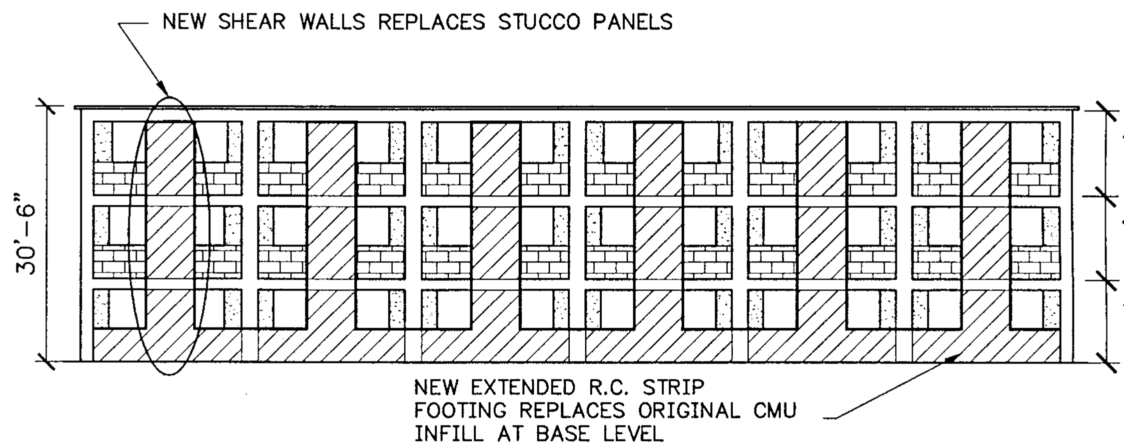
6. Nonstructural rehabilitation concept:

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.



ORIGINAL BUILDING CONFIGURATION



BUILDING REHABILITATION CONCEPT

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)

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

5. & 6. *Rehabilitation design and confirming evaluation:* These two steps are combined since the design and confirmation is an iterative process. 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 new shear walls is based on a new pseudo lateral force per FEMA 273 and detailed in accordance with FEMA 302. Following the design of the new shear walls, the capacities of the existing concrete frame elements are checked to make sure they can resist the new loads. Finally, the capacities of the foundation and soil are checked to ensure that they can resist forces equal to the development of the superstructure element capacities.

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 FEM 273 Section 3.3.1.3)*

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

Determination of C_1 factor:

$C_1 = 1.5$ 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 2.6.1.5 for discussion of T_0).

Building Period (per FEMA 273 Section 3.3.1.2): The building period is determined using Method 2;

$T = C_t h_n^{3/4}$ (FEMA 273 Eq. 3-4)

Longitudinal Direction: ($C_t = 0.02$ for concrete shear walls, $h_n = 30.6'$)

$T = (0.02)(30.6')^{3/4} = 0.26$ seconds

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

Linearly interpolate to obtain $C_1 = 1.5 + \frac{(0.26-0.10)}{(0.51-0.10)}(1.0-1.5) = 1.30$

Determination of C_2 factor:
The C_2 factor is determined from FEMA 273 Table 3-1. The rehabilitation plan calls for the use of non-shear critical shear walls. Therefore, assume framing type 2.

$C_2 = 1.0$ for the Life Safety Performance Level and Framing Type 2.

Determination of C_3 factor:
The C_3 factor is dependent on the stability coefficient, θ, described in FEMA 273 Section 2.11.2. The shear walls are very rigid, and therefore, low drifts are expected. The low drifts will lessen the P-Δ effects so it is assumed that the stability coefficient is less than 0.1. This condition is checked later when constructing the mathematical model of the structure.

$C_3 = 1.0$

Determination of S_a:

S_a is the response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration. The value of Sa is obtained from the procedure in FEMA 273 Section 2.6.1.5.

$T = 0.26$ seconds $< T_0 = 0.51$ seconds, use FEMA 273 Equation 2-8.

For building periods between $0.2T_0 = 0.2(0.51) = 0.10$ and $T_0 = 0.51$, $S_a = S_{XS} / B_S = 0.82/1.0 = 0.82$ (see FEMA 273 Figure 2-1 for a graphical description of the general response spectrum)

$S_a = 0.82$

Determination of Building Weight, W:
The building weight must be updated to reflect the added weight of the new longitudinal shear walls. The new weight is W = 2575 kips (calculations not shown)

V = (1.30)(1.0)(1.0)(0.82) (2575 kips) = 2745 kips (12210 kN)

- *Determine Vertical Distribution of Seismic Forces:*

$F_x = C_{vx}V$ (FEMA 273 Eq. 3-7)

$C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$, where k = 1.0 for T=0.26 sec < 0.5 seconds (FEMA 273 Eq. 3-8)

Level	w_x (kips)	h_x (ft)	w_xh_x (kipft)	C_{vx}	F_x (kips)	
Roof	712.7	30	21380	0.434	1190	(5293 kN)
3rd Floor	931.1	20	18623	0.378	1037	(4613 kN)
2nd Floor	931.4	10	9314	0.189	518	(2304 kN)

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

– The building is modeled assuming rigid diaphragm action. (Equal deflections at top of each column at a particular level)

– Horizontal Torsion (per FEMA 273 Section 3.2.2.2)

The total horizontal torsional effect is made up of the actual and accidental torsion. There is no actual torsion for this structure. Due to symmetry, the centers of mass and eccentricity coincide. The accidental torsion is produced by a horizontal offset in the centers of mass 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 need only 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%. For this regular, symmetrical structure the accidental torsion does not need to be considered. Therefore, no horizontal torsion, either actual or accidental, needs to be considered in the building model.

- The structure is analyzed using a two-dimensional model with RISA 3D software. The new shear walls are designed assuming that they resist the entire seismic force demand in the longitudinal direction. Therefore, they are evaluated as primary components. The columns and beams are evaluated as secondary components.

- Modulus of Elasticity of Concrete; (use the ACI 318 method for determining E)

  $E = 57000\sqrt{f'_c} = 57000\sqrt{3000} = 3.12 \text{ x } 10^3 \text{ksi}$ (ACI 318 Sec. 8.5.1)

  Concrete strength indicated on drawings, f'_c = 3000 psi (20670 kPa)

- Effective Concrete Stiffness Values

  Stiffness of reinforced concrete components depends on material properties (including current condition), component dimensions, reinforcement quantities, boundary conditions, and stress levels. The calculation of a member's effective stiffness directly from principles of basic mechanics is impractical in most cases. FEMA 273 provides guidance for calculation of member stiffness for evaluation of concrete structures in Section 6.4.1.2. Table 6-4 of FEMA 273 provides effective stiffness values for a variety of reinforced concrete components, and is used here for ease of calculations. The walls are assumed to be cracked at the design deformation levels (see FEMA 273 Section 6.8.2.2 for discussion on wall stiffness for analysis.)

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 (assume cracked)	Flexural Rigidity = $0.5E_cI_g$	Shear Rigidity = $0.4E_cA_w$

- Component Gravity Loads

  The walls are assumed to carry no gravity loads since the gravity loads are already in place and being resisted by the concrete frames when the walls are being constructed.

  $Q_G = 1.2\ Q_D + 0.5\ Q_L + 0.2\ Q_S$ (Eq. 7-1)

  FEMA 273 Section 3.2.8 states that $Q_S = 0.0$ where the design snow load is less than 30 psf. (Note: Eq. 7-1 is different than FEMA 273 Equation 3-2. This document uses the gravity load combination specified in ASCE 7 rather than the FEMA equation.)

  $Q_G = 0.9\ Q_D$ (FEMA 273 Eq. 3-3)

  Q_D = Dead load

  $Q_{D\ roof}$ = 1568 plf (22.9 kN / m)
  $Q_{D\ 3rd}$ = 1997 plf (29.1 kN / m)
  $Q_{D\ 2nd}$ = 1997 plf (29.1 kN /m)

  Q_L = Design live load

  $Q_{L\ roof}$ = 198 plf (2.89 kN / m)
  $Q_{L\ 3rd}$ = 496 plf (7.24 kN / m)
  $Q_{L\ 2nd}$ = 496 plf (7.24 kN / m)

  Q_E = Earthquake load (for each longitudinal line of framing)

  ½ of the forces go to each longitudinal framing line on each side of the building.

  $Q_{E\ roof}$ = ½(1190 kips) = 595 kips (2647 kN)
  $Q_{E\ 3rd\ floor}$ = ½(1037 kips) = 519 kips (2309 kN)
  $Q_{E\ 2nd\ floor}$ = ½(518 kips) = 259 kips (1152 kN)

- P-Δ Effects

Two types of P-Δ effects are considered, static and dynamic.

<u>Static P-Δ effects:</u> For linear procedures, the stability coefficient, θ, should be evaluated for each story in the building using FEMA 273 Eq. 2-14. If the coefficient is less than 0.1 in all the stories, static P-Δ effects will be small and may be ignored.

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \qquad \text{(FEMA 273 Eq.2-14)}$$

The lateral forces, V_i, are placed on the structure to determine the story lateral drifts, δ_i. The calculation of the gravity loads, Pi, is not shown. The story heights are 10' = 120" for the upper two floors, however, since the footings are to be extended upward 4', the first story height is only 6' = 72". The drifts were determined by placing the lateral loads on computer model described above.

3rd Story: $\theta_3 = \frac{(713k)(0.73")}{(1190k)(120")} = 0.004 < 0.1$

2nd Story: $\theta_2 = \frac{(1644k)(0.57")}{(2227k)(120")} = 0.004 < 0.1$

1st Story: $\theta_1 = \frac{(2575k)(0.132")}{(2745k)(72")} = 0.002 < 0.1$

All of the θ values are less than 0.1, therefore, static P-Δ effects are ignored.

<u>Dynamic P-Δ effects:</u> The dynamic P-Δ effects are indirectly evaluated for the linear procedures by using the coefficient C_3, which has been done in the calculation of the pseudo lateral force.

Check of Deformation-Controlled Components

The deformation-controlled actions for the structure include wall and column flexural demands. The spandrel beams are allowed to hinge at the columns and walls as they are not relied on to act as coupling beams. The beams only need to have the capacity to sustain the imposed shear loads after they form hinges. The check of the shear capacity of the beams is done in the force-controlled component checks.

The acceptance criteria for deformation-controlled components is:

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

where:

$$Q_{UD} = Q_G \pm Q_E \qquad \text{(FEMA 273 Eq. 3-14)}$$

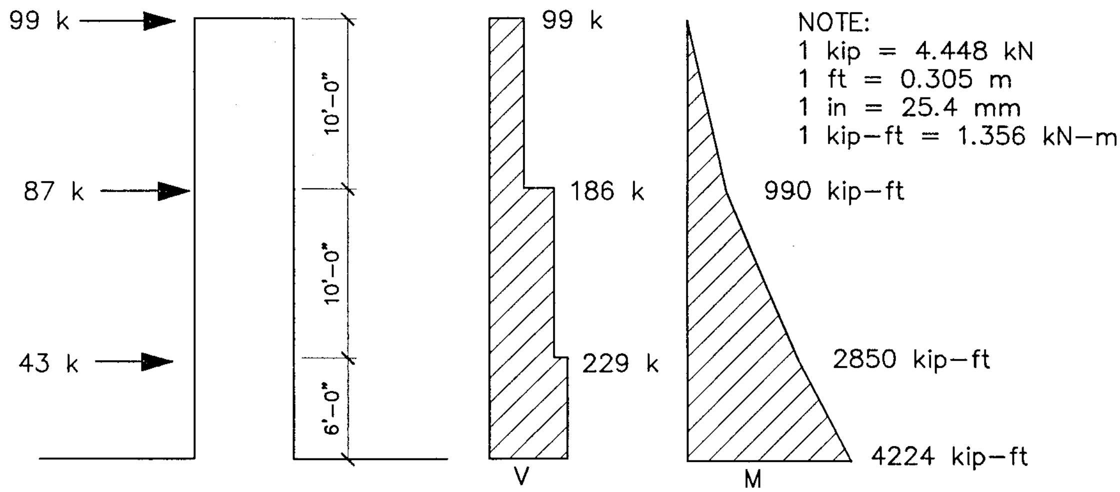
Per paragraph 7-2.e(5)(d)1.i, the m-factors used to account for expected ductility of the action shall be taken from Chapter 7 of TI 809-04. These m-factors are the same as the ones in FEMA 273, except that they have values for the Safe Egress Performance Level not contained in the FEMA 273 tables.

Design of shear wall flexural steel and boundary zone detailing

Paragraph 7.2.e(4) states that the primary references for structural detailing of new construction associated with the rehabilitation of existing buildings are the applicable requirements of FEMA 302 and its incorporated reference documents, which for this example includes ACI 318. The design and detailing of the shear walls follows FEMA 302 requirements since the walls are new structural members. The demands on the shear wall segments are determined from the FEMA 273 forces. FEMA 273 Section 6.8 gives guidance on the modeling and acceptance criteria of concrete shear walls.

The beams are assumed to develop hinges at the walls and columns due to the strong column/weak beam condition. The walls are assumed to carry no gravity loads since the gravity loads are already in place and being resisted by the concrete frames when the walls are being constructed.

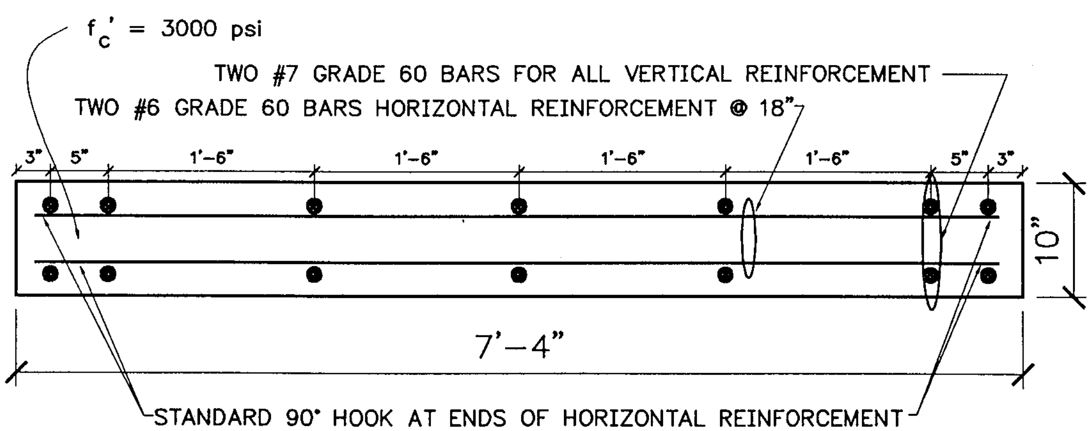
Shear wall forces assuming the walls resist the entire lateral force demand: (The shear forces are equal to the story forces on each framing line divided by the number of shear walls per framing line.)



SHEAR WALL FORCES

Flexural demands and longitudinal reinforcement requirements:

The flexural design of the walls follows FEMA 273. The demands on the wall are checked by taking the FEMA 273 forces on the wall and dividing them by the appropriate m-factor. This demand is checked against the expected strength assuming the steel strength = $1.25f_y$ (per FEMA 273 Section 6.8.2.3). Therefore, for 60 ksi reinforcing steel, $1.25f_y = 75$ ksi.



TYPICAL WALL CROSS-SECTION

The moment demand from the analysis is $Q_{UD} = M = 4224$ kip-ft (5728 kN-m)

Assuming the wall reinforcement details will force the wall to act as a flexure-controlled element. The m-factor is determined from TI 809-04 Table 7-2. The walls carry no axial force, have no boundary zones (see section on need for boundary zones below), and their shear ratio is:

$$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}} = \frac{229000\ \text{lbf}}{10''\text{x}88''\sqrt{3000\,\text{psi}}} = 4.75$$

Interpolate in the table for the Life Safety performance level to obtain m = 2.21

$Q_{UD} = 4224$ kip-ft (5728 kN-m) = moment demand on wall from elastic analysis.

The flexural strength of the wall is determined using the program BIAX. Design assumptions are that the ultimate concrete strain capacity = 0.003 and $f'_c = 3000$ psi (20.7 MPa). The expected strength of the wall is determined assuming a reinforcing yield strength = $1.25f_y$ = 1.25(60 ksi) = 75 ksi (517 MPa)

Wall capacity from BIAX = Q_{CE} = 1956 kip-ft (2652 kN-m)

mQ_{CE} = (2.21)(1956 kip-ft) = 4323 kip-ft (5862 kN-m) > Q_{UD} = 4224 kip-ft (5728 kN-m), OK

– Determine need for boundary zones (per FEMA 302 Sec. 9.1.1.13)

The FEMA 302 design requirements assume that the lateral forces are computed from the equation

$V = C_sW$, (FEMA 302 Eq. 5.3.2)

where $C_s = S_{DS}/R$ (TI 809-04 Eq. 3-7)

R = 6 for building frame systems with special reinforced concrete shear walls (TI 809-04 Table 7-1)

The forces shown above were determined from $V=C_1C_2C_3S_aW$ with $S_a = S_{DS} = 0.82$.
Therefore, to check the FEMA 302 requirements for boundary zone details, the forces above must be modified.
Modification factor, $\alpha = 1 / C_1C_2C_3R = 1 / (1.3)(1.0)(1.0)(1.0)(6.0) = 0.13$
Therefore, the demands on the walls will be multiplied by 0.13 to determine boundary zone requirements.

$P_u = 1.2D + 0.5L + E$ (per FEMA 302 Sec. 9.1.1.2)
The only gravity loads carried by the walls are their self-weight. The walls are assumed to act as cantilevers with no coupling action by the beams. Therefore, the E term equals zero.

1st story: P_u = 1.2(26')(10/12)(88/12)(0.150 kcf) = 29 kips
2nd story: P_u = 1.2(20')(10/12)(88/12)(0.150 kcf) = 22 kips
3rd story: P_u = 1.2(10')(10/12)(88/12)(0.150 kcf) = 11 kips

1.) $P_u \le 0.10A_g f'_c$ check

0.10(88")(10")(3ksi) = 264k > 29k, OK for all segments

2.) $M_u / V_u l_w$ <1.0 or 3.0 check (α cancels out here)

1st: (4224 kip-ft) / (229 k x 7.33') = 2.5 < 3.0 but > 1.0, check if it passes the shear check

$3A_{cv}\sqrt{f'_c} = 3(880\text{in.}^2)\sqrt{3000\text{psi}} = 145\text{k}$

Shear = α(229 k) = (0.13)(229 k) = 30 k < 145 k, no boundary zones required

2nd: (2850 kip-ft) / (186 k x 7.33') = 2.1 < 3.0 but > 1.0, check shear
Shear = α(186 k) = 24.2 k < 145 k, no boundary zones required

3^{rd}: (990 kip-ft) / (99 k x 7.33') = 1.4 < 3.0 but > 1.0, check shear
Shear = α(99 k) = 12.9 k < 145 k, no boundary zones required

Check the minimum vertical reinforcement requirement (per ACI 318 Sec. 21.6.2.1):
ρ_{vmin} = 0.0025 along the longitudinal and transverse axes

In each wall there is a total of 14- #7 bars, A_{st} = 8.4 in.^{2}

ρ = (8.4 in.^{2}) / (10" x 88") = 0.0095 > 0.0025, OK

Design lap splices for longitudinal reinforcement:
FEMA 273 Section 6.8.2.3 states that splice lengths for primary longitudinal reinforcement shall be evaluated using the procedures given in FEMA 273 Section 6.4.5. FEMA 273 Section 6.4.5 states that the development strength of straight bars and lap splices shall be calculated according to the general provisions of ACI 318-95, with the following exceptions: within yielding regions of components with moderate or high ductility demands, details and strength provisions for new straight developed bars and lap spliced bars shall be according to Chapter 21 of ACI 318-95; within yielding regions of components with low ductility demands, and outside yielding regions, details and strength provisions for new construction shall be according to Chapter 12 of ACI318-95, except requirements and strength provisions for lap splices may be taken as equal to those for straight development of bars in tension without consideration of lap splice classifications.

The shear walls are expected to yield at the base since their design strength is less than the design seismic forces. The ductility demand of the wall must be calculated to determine the lap splice requirements. The DCR for the walls is equal to 4224 kip-ft / 1956 kip-ft = 2.2. For a DCR = 2.2, FEMA 273 Table 6-5 classifies the wall components as having a moderate ductility demand. Therefore lap splices at the base of the wall between the longitudinal steel in the wall and the dowels into the foundation are designed per Chapter 21 of ACI 318-95. Lap splices at intermediate locations along the height of the wall are designed according to Chapter 12 of ACI 318-95 since these intermediate splices are at locations of low ductility demands.

Development length of longitudinal reinforcing steel at the base of the wall per ACI 318 Chapter 21;

ACI 318 Section 21.6.2.4 states that all continuous reinforcement in structural walls shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in ACI 318 Section 21.5.4. ACI 318 Section 21.5.4.2 states that the development length of straight bars shall be 2.5 or 3.5 times the development length required for a standard 90 degree hook as specified in ACI 318 Section 21.5.4.1. Per ACI 318 Section 21.5.4.1, the development length l_{dh} for a bar with a standard 90 degree hook in normal weight aggregate concrete shall not be less than 8db, 6 in. and the length required by;

$$l_{dh} = f_y d_b / \left(65\sqrt{f'_c}\right) \qquad \text{(ACI 318 Eq. 21-5)}$$

For #7 bars $l_{dh} = (60000)(7/8)/\left(65\sqrt{3000}\right) = 14.7" > 8"$ and > $8d_b$ = 7", use l_{dh} = 15"

The development length of straight bars is equal to 3.5 times l_{dh} (assuming that the depth of the concrete cast in one lift beneath the bar exceeds 12 in. to be conservative).

$l_d = 3.5\ l_{dh}$ = 3.5(15") = 52.5", use 53" (135 cm) lap splice length for longitudinal bar to dowel laps at base of wall.

Lap splices of longitudinal steel at intermediate heights of wall;

These lap splices are designed per ACI 318 Chapter 12;

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\frac{(c+K_{tr})}{d_b}} \qquad \text{(ACI 318 Eq. 12-1)}$$

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{60000}{\sqrt{3000}}\frac{(1.0)(1.0)(1.0)(1.0)}{2.5} = 32.9$$

For #7 bar development length = 32.9 (7/8) = 29"

Lap splice length = l_d = 29" (74 cm) (no increase for lap splice Class)

Check of Column Members

The column members are evaluated as secondary components. The column demands are determined by subjecting the RISA 3D model to the forces determined earlier with the stiffness of the columns included in the model.

The column flexural capacities are determined at the given axial load. When determining the expected flexural strength of the columns, the expected yield strength of the reinforcement is taken as $1.25f_y$ = 1.25(40 ksi) = 50 ksi (per FEMA 273 Section 6.4.2.2). The flexural demands on the columns are determined by assuming that the beam-column and beam-wall joints are continuous. The frame action tends to brace the walls at the upper floors, while the reverse is true at the bottom floors. Also, the flexural capacity of the columns is a function of the axial load, which is lowest at the upper floors. The higher flexural demands, coupled with the lower flexural capacities of the columns at the upper floors make the top stories critical for the check of column acceptance; however, all columns are checked for the structure.

The m-factors for the columns are taken from Table 7-15 of TI 809-04 for secondary components. There are three factors that are needed to determine the m-factor to use for columns controlled by flexure:

1. The axial load ratio $= \frac{P}{A_g f'_c}$; the m-factor is determined by linearly interpolating for axial load ratios between 0.1 and 0.4 . All of the columns in the structure have axial load ratios less than 0.4, with most falling below 0.1. Linear interpolation is used to determine the intermediate values.

2. Stirrup conformance; if the stirrups in areas of possible plastic hinging are spaced at d/3 or less they are in conformance. All of the columns in the structure have stirrup spacing that is greater than d/3, and are therefore Non-Compliant.

3. The shear ratio $= \frac{V}{b_w d\sqrt{f'_c}}$; the m-factor is determined by linearly interpolating for shear ratios between 3 and 6.

Only one column check is shown to illustrate the check of acceptance.

Q_{UD} = M_u from RISA 3D elastic analysis = 210 kip-ft (285 kN-m)
Axial load on column = 28.3 kips (126 kN)
Shear in column = 33.7 kips (150 kN)

– The axial load ratio $= \frac{28.3\,\text{kips}}{(12"\text{x}23.75")(3\,\text{ksi})} = 0.03 < 0.1$

Stirrups are Non-conforming

The shear ratio $= \dfrac{33.7\ \text{kips}(1000\ \text{lbf/kip})}{(12"\text{x}21.5")\sqrt{3000}} = 2.38 < 3.0$

The m-factor corresponding to this load state = 2.0 for the Life Safety Performance Level.

The flexural capacity of the column was determined by the program BIAX for an axial load = 28.3 kips (126 kN).

Flexural capacity = Q_{CE} = 134 kip-ft (182 kN-m)

mQ_{CE} = (2.0)(134 kft) = 268 kip-ft (363 kN-m) > Q_{UD} = 210 kip-ft (285 kN-m), OK

All of the columns were found to be acceptable.

Check of Force-Controlled Components

The force-controlled actions for the structure include wall, column, beam, and joint shear, and foundation forces.

The acceptance criteria for force-controlled components is:

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

where Q_{UF} is determined from capacity limit analysis of the members delivering forces to the element being evaluated or from either FEMA 273 Equation 3-15 or 3-16. Equation 3-16 can always be used. Equation 3-15 may only be used when the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system.

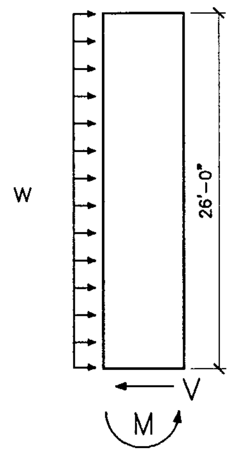
$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad \text{(FEMA 273 Eq. 3-15)}$$

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

Shear forces in new wall segments

For a cantilever shear wall, 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 (per FEMA 273 Sec. 6.8.2.3).

Determine shear reinforcement requirement based on flexural capacity of the wall:



$M = wH^2/2$
$w = 2M / H^2 \quad w = 2(1956 \text{ kft}) / (26')^2 = 5.8 \text{ kips / ft.}$
$V = wH = (5.8 \text{ kips / ft})(26') = 151 \text{ kips}$
$\therefore$ Use $Q_{UF} = V = 151$ kips (672 kN) for design

Determine amount of horizontal shear reinforcement needed to develop V = 151 kips (672 kN);

FEMA 273 Section 6.8.2.3 states that the nominal shear strength of a shear wall is determined based on the principles and equations given in Section 21.6 of ACI 318-95. For all shear strength calculations, 1.0 times the specified reinforcement yield strength should be used.

$V_u = V = 151 \text{ kips}$

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

$$\rho_n = \frac{V_n / A_{cv} - 2\sqrt{f_c'}}{f_y}, \text{ set } V_n = V_u = 151 \text{ kips}$$

$$\rho_n = \frac{151000/(10"\text{x}88") - 2\sqrt{3000}}{60000} = 0.001$$

Minimum reinforcement (per ACI 318 Sec. 21.6.2.1)
$\rho_{vmin} = 0.0025$ along the longitudinal and transverse axes

Try two #6 bars @ 18", $A_{st} = 0.88 \text{ in.}^2$

$\rho = (0.88 \text{ in.}^2) / (10" \text{ x } 18") = 0.0049 > 0.0025$, OK

$$Q_{CN} = V_n = (10"\text{x}88")\left[2\sqrt{3000} + 0.0049(60000)\right] = 355 \text{kips} \ (1579 \text{ kN})$$

Check maximum wall shear strength (per ACI 318 Sec 21.6.5.6)

Individual piers: $V_{max} = 10A_{cv}\sqrt{f_c'} = 10(10"x88")\sqrt{3000} = 482\text{kips / wall} > 355\text{ k}$

Average for wall: $V_{max} = 8A_{cv}\sqrt{f_c'} = 8(10"x88")\sqrt{3000} = 386\text{kips / wall} > 355\text{ k}$

Q_{CN} = 355 k (1579 kN) > Q_{UF} = 151 k (672 kN), OK

Check shear transfer between walls & foundation;
The transfer of shear forces between the walls and the foundation is evaluated using the shear-friction design method of ACI 318 Section 11.7.4

V from capacity analysis of wall = 151 kips = Q_{UF}

$V_n = A_{vf} f_y \mu$ (ACI 318 Eq. 11-25)

$$A_{vf} = \frac{V_n}{f_y \mu}$$

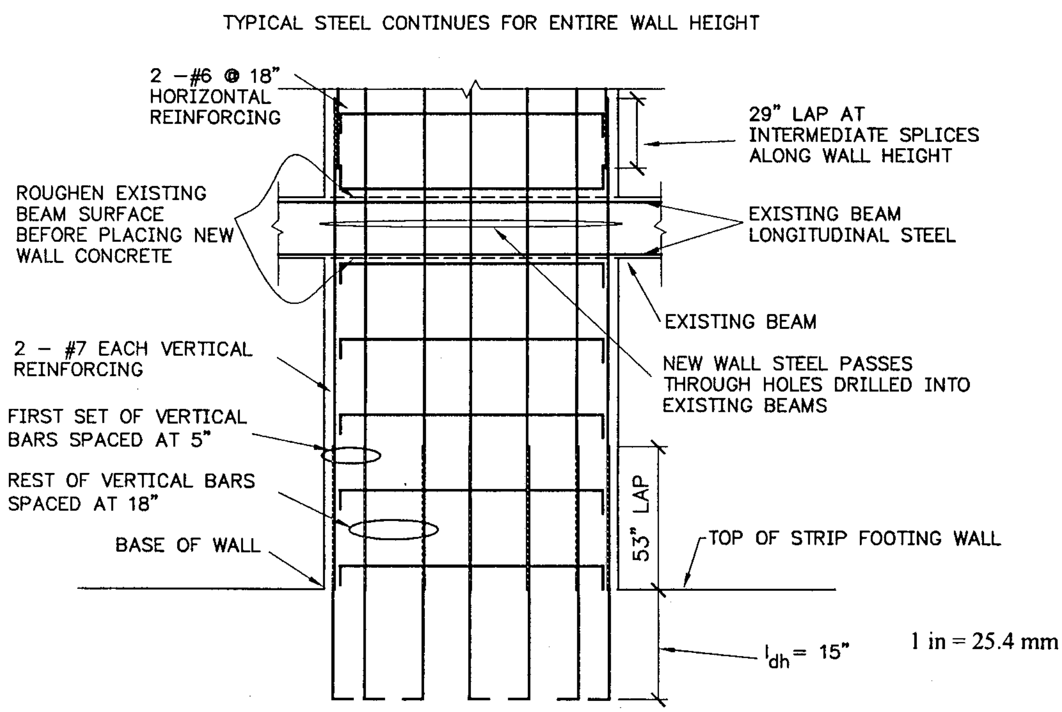
$\mu = 1.0\lambda,\quad \lambda = 1.0,\quad \mu = 1.0(1.0) = 1.0$ (ACI 318 Sec. 11.7.4.3)

$A_{vf} = V_n / f_y$

A_{vf} = 151 k / 60 ksi

A_{vf} = 2.52 in.^{2} / wall (16.3 cm^2 / wall)

This is much less than the vertical steel already in the wall. Lap dowels at each of the vertical bars.
∴ Steel reinforcement is adequate for design.



TYPICAL SHEAR WALL ELEVATION

Column Shear

The shear strength of columns is determined using procedures in FEMA 273 Section 6.4.4. To use these procedures, the column demand / capacity ratio must be calculated to classify the member as having high, moderate, or low ductility demand (per FEMA 273 Section 6.4.2.4). For this structure, all of the columns have flexural demand / capacity ratios that are less than 2.0. Therefore, per FEMA 273 Table 6-5, all of the columns are classified as having a low ductility demand.

The shear strength of the columns is calculated as:

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

The concrete contribution to the shear strength, V_c, is calculated using the method described in FEMA 273 Section 6.5.2.3.

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

where $\lambda = 1.0$ for normal weight concrete, $k = 1.0$ in areas of low ductility demand, and N_u is the axial load determined in accordance with FEMA 273 Section 6.5.2.3.

The shear reinforcement contribution to strength is calculated as:

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

Check of a typical column (interior perimeter column used for check):

Q_{UF} = V from RISA output using FEMA 273 Equation 3-16 = 26.3 kips (117 kN)

N_u = Axial load from RISA output = 35.4 kips (157 kN)

$$V_c = 3.5(1.0)\left(1.0 + \frac{35400}{2000(12"\text{x}23.75")}\right)\sqrt{3000}(12")(21.5") = 52.5\,\text{kips}\ \ (234\text{ kN})$$

$$V_s = \frac{(0.22\,\text{in.}^2)(40\,\text{ksi})(21.5")}{12"} = 15.8\,\text{kips}\,(70.3\text{ kN})$$

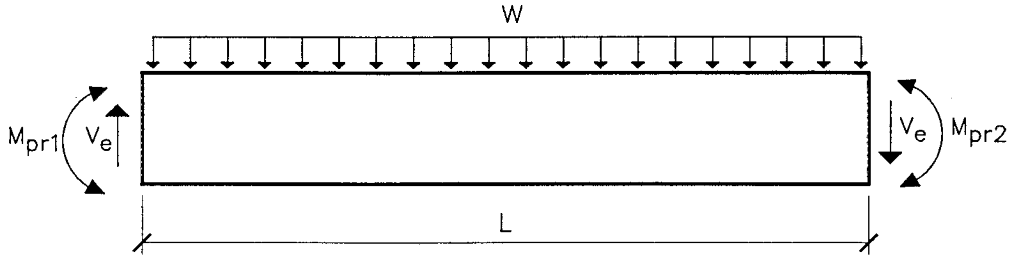
$V_{CN} = V_n$ = 52.5 kips + 15.8 kips = 68.3 kips (304 kN) > Q_{UF} = 26.3 kips (117 kN), OK

All of the columns were found to be acceptable.

Beam Shear

The beams develop flexural hinges at the beam-column and beam-wall interfaces. The shear demand on the beams is calculated 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 moment of opposite sign 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 forces;



Beam moment capacities (from BIAX): Side 1 is the left end, Side 2 is the right end
M_{pr1}^{+} = 55.2 kip-ft M_{pr1}^{-} = 44.4 kip-ft M_{pr2}^{+} = 55.2 kip-ft M_{pr2}^{-} = 157.0 kip-ft

w = Gravity loads = 1.0(D + L) = 1.0(1997 plf + 496 plf) = 2.5 klf

The beam shears must be evaluated for seismic forces in both directions since the beam flexural strengths are not symmetrical. Only one direction is shown here for illustration.

$V_e = (M_{pr1}^{+} + M_{pr2}^{-}) / L + wL/2$ = (55.2 kip-ft + 157.0 kip-ft) / 5.1' + (2.5 klf)(5.1') / 2 = 48 kips (214 kN)

The beams have high ductility demands at their ends due to the hinging at the walls and columns. FEMA 273 Section 6.4.4 states that within yielding regions of components with moderate or high ductility demand, shear strength shall be calculated according to Chapter 21 of ACI 318-95. ACI 318 Section 21.3.4.2 states that V_c should be taken = 0 in plastic hinge zones. FEMA 273 Section 6.4.4 further states that within yielding regions of components with moderate or high ductility demands, transverse reinforcement shall be assumed ineffective in resisting shear where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measure in the direction of shear. The transverse ties in the beams are spaced at 12" which is greater than d/2, therefore, the transverse reinforcement is assumed ineffective in resisting shear. With no concrete or transverse reinforcement contributions to the shear strength, FEMA 273 predicts that the beams will have no shear capacity and they must be rehabilitated to resist the design forces.

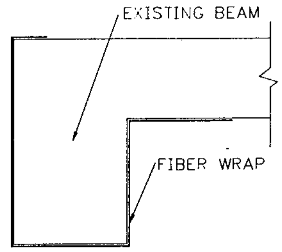
$V_n = V_c + V_s$ (ACI 318 Eq. 11-2)

$V_n = 0 + 0 = 0$ kips

$V_{UF} = V_e = 48$ kips (214 kN)

$V_n = 0$ kips $< V_{UF} = 48$ kips, No Good.

All of the beams fail this check considering reversal of the seismic forces. The beams must be strengthened to provide greater shear capacities. The shear capacities of the beams may be increased by increasing the beam size and adding additional transverse reinforcement or by adding fiber wrapping. Adding additional transverse reinforcement is very difficult due to the presence of the slab. Therefore, fiber wrapping is chosen. The wrap design is detailed per the manufactures' specs (no capacity calcs shown here).



(Note: There are no established military or industry standards for the materials and application techniques used for this upgrade method, so manufacture's information must be relied upon. The manufacturer's claims should be viewed with skepticism and certified conformation of their validity should be required. Also, dealing with one fiber-wrapping manufacturer could constitute proprietary procurement, which is generally not allowed in Government contracts.)

Joint shear

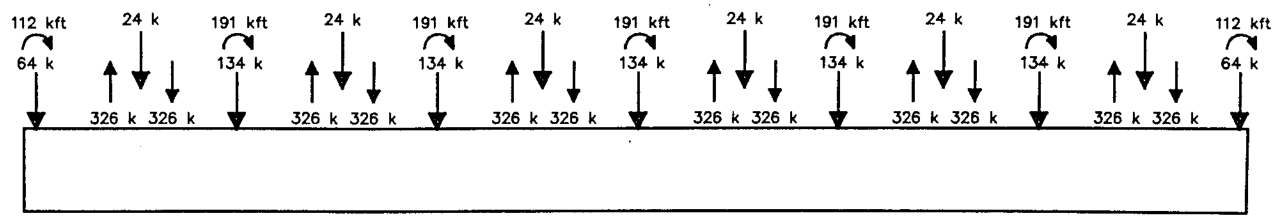
The joint shear was checked in the Tier 2 analysis. The exterior joints were found to be unacceptable, while the interior joints were found to have adequate shear capacity. Therefore, the exterior joints must be strengthened. Like the beams, fiber wrapping is chosen to strengthen the joints. The wrapping is detailed per the manufacture's specs (no capacity calcs shown here). See note above for beam fiber wrapping.

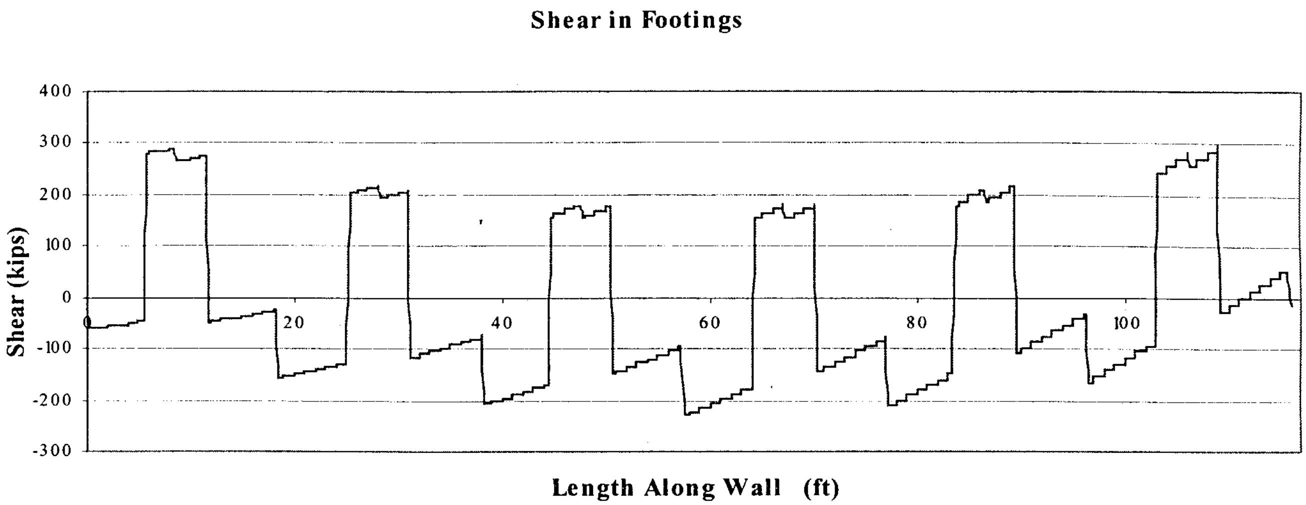
Foundation

Concrete strip footings:

The foundation demands are based on the gravity loads and flexural capacities of the columns and walls. At column locations, the foundation is loaded with a point load equal to the design gravity load, and a moment equal to the flexural capacity of the column at the design gravity load. At wall locations, the foundation is loaded with a force couple equal to the flexural capacity of the wall, with a distance between the forces (lever arm of wall) equal to 6', and an axial load equal to the weight of the wall. The strip footing is modeled as a beam on an elastic medium by supporting the beam with closely spaced compression only springs. The axial loads are from the tributary gravity loads. The moment capacity of the members is determined using the computer program BIAX.

Axial load on exterior columns:	64 kips (285 kN)
Moment capacity of columns @ axial load:	112 kipft (152 kN-m)
Axial load on interior columns:	134 kips (596 kN)
Moment capacity of columns @ axial load:	191 kip-ft (259 kN-m)
Weight of wall: (10"/12)(88"/12)(0.150 kcf)(26'):	24 kips (107 kN)
Moment capacity of wall:	1956 kip-ft (2652 kN-m)
Forces for couple = M / lever arm = 1956 kft / 6'	326 kips (1450 kN)





$V_{max} = V_{UF} = 300$ kips (1334 kN)

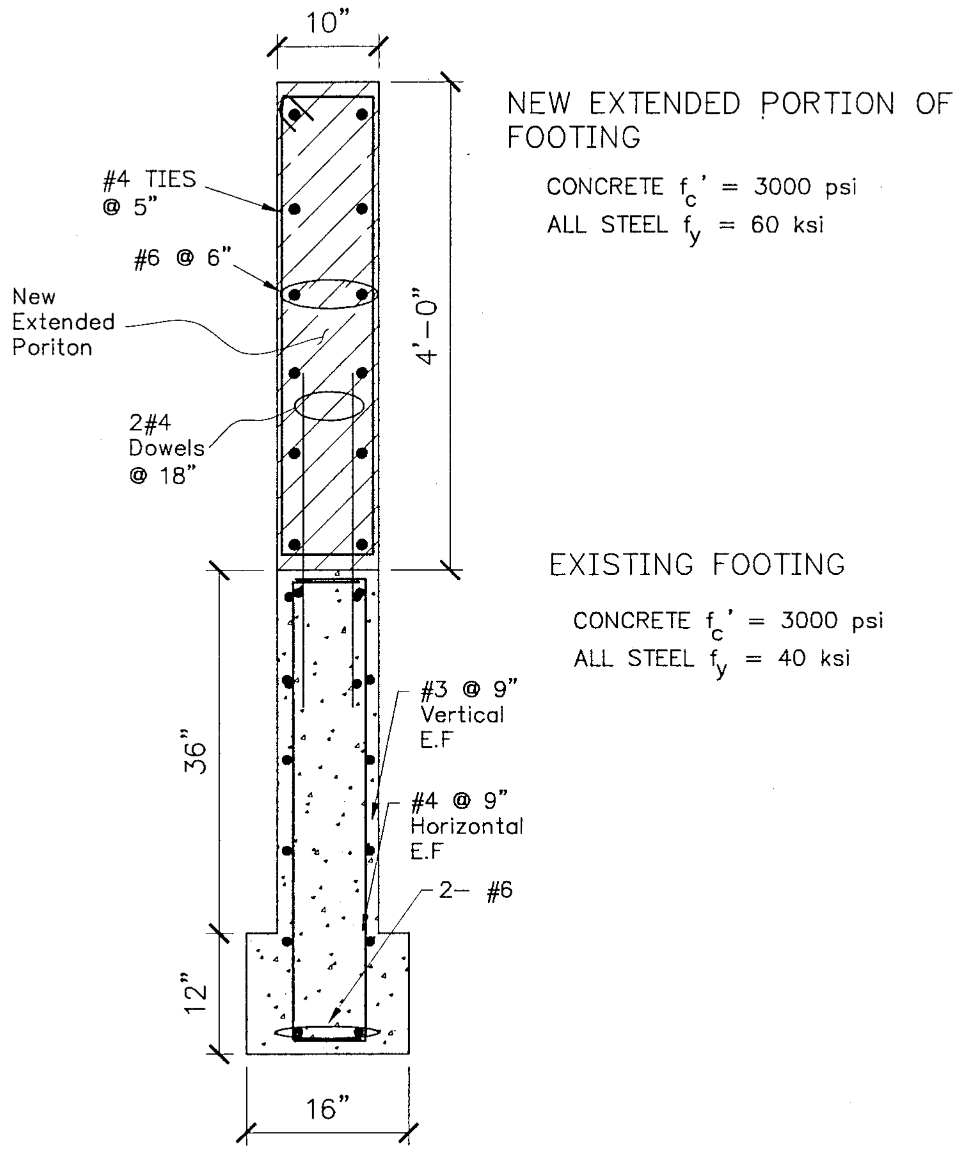
The shear strength of the new expanded footing is made up of contributions from the concrete and reinforcing steel. The concrete contribution is calculated assuming that both the existing and new concrete portions resist shear. The reinforcing steel contribution is calculated assuming that only the new steel resists shear, neglecting the existing reinforcement.

$V_n = V_c + V_s$

$V_c = 2\sqrt{f_c^{'}}b_w d = 2\sqrt{3000}(10")(70") = 77\text{kips}$ d = 70" from BIAX output

$$V_s = \frac{A_s f_y d}{s} = \frac{(0.39\text{in.}^2)(60\text{ksi})(48")}{(5")} = 225\text{kips}$$

$V_n = V_{CN} = 77$ kips + 225 kips = 302 kips (1343 kN) > $V_{UF} = 300$ kips (1334 kN), OK



CROSS SECTION OF NEW EXTENDED FOOTING

The shear forces from the columns and walls must be transferred through the strip footing and across the interface between the new and existing concrete. The new extended portion of the wall is doweled into the existing portion with 2#4 bars at every 18". The shear friction capacity is calculated using ACI 318 Section 11.7.4. The shear demand is calculated using the flexural-shear capacities of the walls and columns at the design gravity loads.

Moment capacity of exterior columns @ axial load:	112 kip-ft
Shear at base = 2M / L = 2(112 kft)/6':	37 kips
Moment capacity of interior columns @ axial load:	191 kip-ft
Shear at base = 2M / L = 2(191 kft)/6':	64 kips

Moment capacity of wall: 1956 kip-ft
Shear at base (see shear wall design): 151 kips

V_{UF} = 2(37 kips) + 5(64 kips) + 6(151 kips) = 1300 kips (5782 kN)

Check horizontal shear capacity of strip footing:
Assume the footing acts as a squat wall.

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right), \text{ where } \alpha = 3 \qquad \text{for } h/l < 1.5 \qquad \text{(ACI 318 Eq. 21-7)}$$

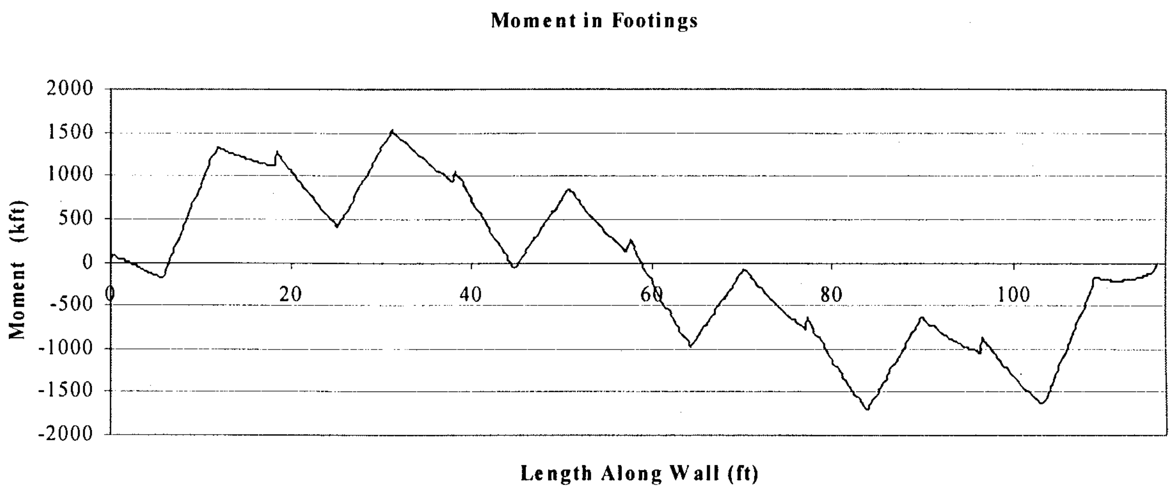
neglect the steel contribution;

$$V_n = V_{CN} = V_n = (10'')(117' \text{x} 12''/')\left(3\sqrt{3000}\right) = 2307 \text{ kips } (10262 \text{ kN}) > V_{UF} = 1300 \text{ kips } (5782 \text{ kN}), \text{ OK}$$

Check shear friction:

$V_n = A_{vf} f_y \mu$ = (0.39 in.^{2})(60 ksi)(1.0) = 23.4 kips per set of 2 # 4 dowels (ACI 318 Eq. 11-25)
Dowels spaced at 18" 117' / 18" spacing per set = 78 sets of dowels
V_{CN} = 78(23.4 kips) = 1825 kips (8118 kN) > V_{UF} = 1300 kips (5782 kN), OK

Check moment capacity of footings:



M_{CL}^+ = 1543 kip-ft > M_{UF}^+ = 1510 kip-ft, OK (Capacities from BIAX)
M_{CL}^- = 3400 kip-ft (4610 kN-m) > M_{UF}^- = 1650 kip-ft (2237 kN-m), OK

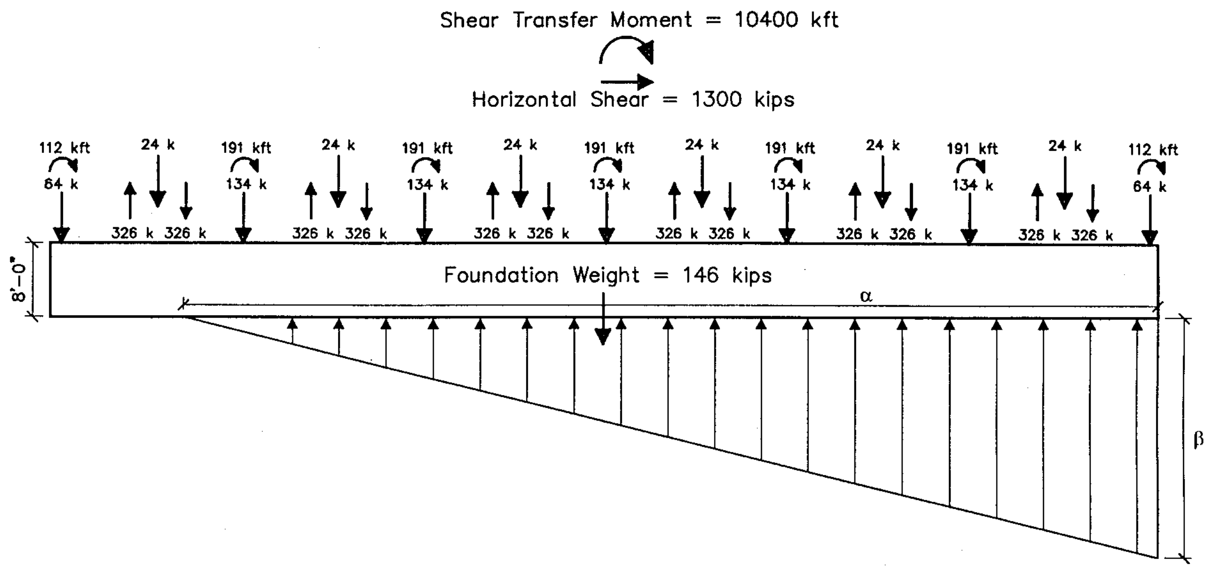
Check soil capacity:

The allowable stress for the soil indicated on the drawings = 8 ksf. FEMA 273 Section 4.4.1.2 states that the expected strength of the soil for seismic effects using the presumptive method may be taken as two times the allowable stress value. Therefore $q_c = 2q_{all}$ = 2(8 ksf) = 16 ksf. The soil is loaded with the foundation loads shown above as well as the weight of the foundation and a moment created by the shears at the column and wall bases acting over a length equal to the new expanded footing depth (8').

Shear at base = 1300 kips (5782 kN)

Moment due to shear = V(depth of footing) = 1300 kips(8') = 10400 kip-ft (14102 kN-m)
Weight of foundation: [(16"/12)(1')+(12"/12)(7')](0.150 kcf) = 1.25 klf (18.2 kN/m)
Total weight of foundation = 1.25 klf(117') = 146 kips (649 kN)

Calculate soil stress distribution below footing:



Total moment about right end of footing due to superstructure forces, M = 40333 kip-ft (54692 kN-m) ccw
Total axial force due to superstructure and foundation weight, P = 1088 k (4839 kN)

Determine if 16 ksf stress is violated by assuming linear soil force distribution:
$P = 1/2\alpha\beta$
$M = 1/2\alpha\beta(1/3\alpha) = 1/6\alpha^2\beta$

Solving for α and β,
$\alpha = 111.2' < 117'$
$\beta = 19.6$ klf

Footing is 16" wide at base,
$\sigma_{soil} = \beta/w = 19.6$ klf / (16"/12) = 14.7 ksf < 16 ksf, OK

Shear strength of transverse walls

The shear strength of the transverse walls is checked to see if they have enough capacity based on the new base shear. The base shear is higher since the building weight has been increased by the wall additions.

Pseudo lateral force in the transverse direction is equal to that in the longitudinal direction since the weight (W), C-coefficients ($C_1C_2C_3$) and spectral acceleration (S_a) are the same for both directions.

$V_{pseudo} =$ 2745 kips / 2 walls = 1373 kips / wall (6107 kN)
$V_{UF} = V_{pseudo} / C_1C_2C_3 = 1373$ kips / (1.0)(1.0)(1.3) = 1056 kips (4697 kN)
$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right)$, where $\alpha = 3$ h/3 = 30.6' / 39' = 0.8 <1.5 (ACI 318 Eq. 21-7)

The total wall area, A_{cv} = thickness x (wall length); thickness = 8", length = 40'-8", openings = 3'-4"
$A_{cv} = (8")(40.67'-3.33')(12"/') = 3585$ in.^{2}

$V_n = 3585\text{in.}^2\left(3.0\sqrt{3000} + (0.0026)(40000)\right) = 962\,\text{kips}\ (4279\ \text{kN})$

$V_{CN} = 962$ kips (4279 kN) < $V_{UF} = 1056$ kips (4697 kN)

The walls D/C = 1056/962 = 1.1

The walls are shown to be 10% overstressed. However, per paragraph 7-2.f.(5)(d), a 10 to 15 percent reduction in the seismic demand of a deficient component is permitted in the structural evaluation if such reduction can preclude the rehabilitation of an otherwise deficient building. The walls are slightly overstressed, but not enough to warrant additional rehabilitation. Therefore, assume the walls are acceptable.

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.