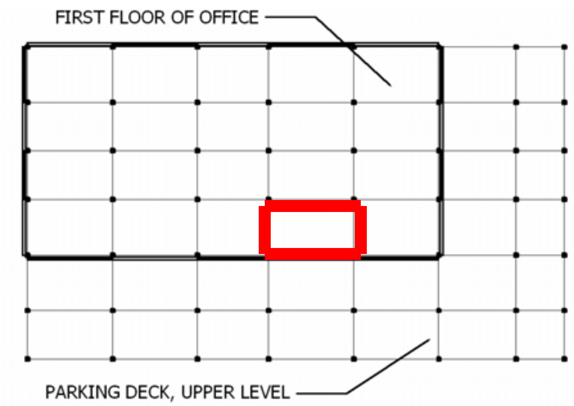
DEPTH ANALYSIS: CONCRETE DESIGN

Two-Way Floor Slab

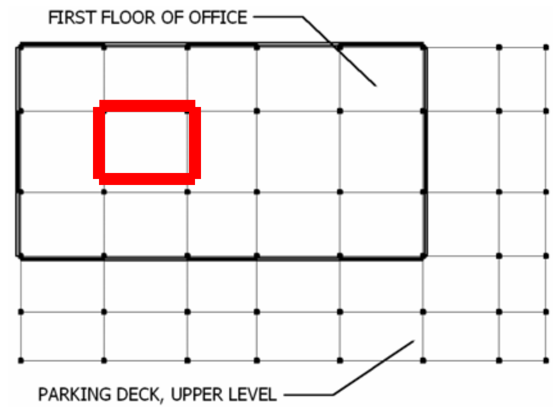
Alternatives. An initial estimate of differing floor systems using the CRSI Manual found that a two-way slab, though heavy, would effectively reduce floor section thickness, and provide for easiest construction. However, this estimate assumed a new column layout with square bays rather than the current 30'-0"x17'-6" size. In addition, to more effectively control the large live loads in both the office and parking areas, differing two-way slab systems were considered, including:

- Flat Plate
- Flat Plate with Edge Beams
- Flat Slab with Drops
- Flat Slab with Drops and Edge Beams
- Flat Slab with Beams between all Columns

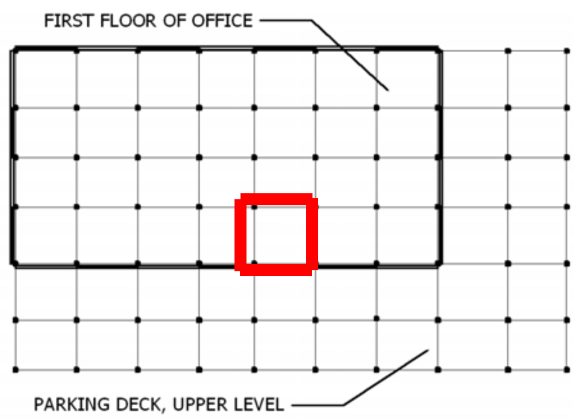
Four differing column layouts were considered, making sure to provide a column-free entry centered on the north and south building façade, as shown in Figure 4.



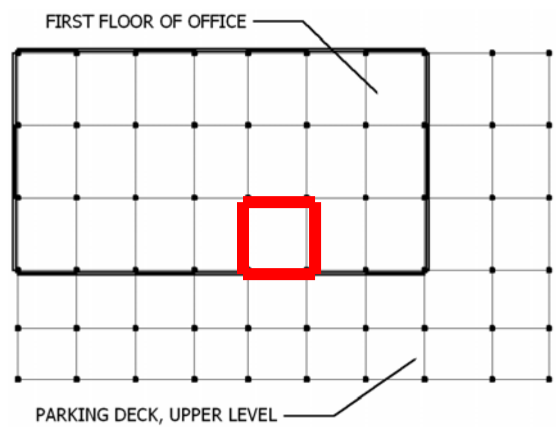
LAYOUT 1: Existing Column Layout
Maximum Bay Size: 20'-0"x30'-0"



LAYOUT 2: Adjusted in N-S Direction
Maximum Bay Size: 30'-0"x30'-0"



LAYOUT 3: Adjusted in E-W Direction
Maximum Bay Size: 20'-0"x21'-0"



LAYOUT 4: Adjusted in Both Directions
Maximum Bay Size: 25'-0"x21'-0"

Figure 4. Column Layout Overview

Procedure. Before using a more exact analysis, the **Direct Design Method** was used to find approximate values of positive and negative moments in the column and middle strips of the two way slabs. The Direct Design Method can be used throughout the entire structure because [ACI 318-05 13.6.1]:

- In each condition, there are at least three spans in all directions.
- The most drastic rectangular bay is 17'-6" x 30'-0", which has a l_2/l_1 = 1.72<2.0.
- The most drastic shift in span length between two adjacent spans is 5'-0", or 16%, less than one-third of the larger span.
- Columns are minimally offset from the basic building grid.
- Only in a few situations are separate concentrated or line loads presented (ie. Bank Vault, HVAC equipment). These panels will be assessed individually. Even in the parking structure, due to the large dead weight of concrete, live loads should not be greater than two times the dead load.

The minimum slab depths given by Table 13.5 of *Design of Concrete Structures* (436) are used to ensure satisfactory deflections.

Using results from the Direct Design Method, acceptable designs and layouts were then assessed in **ADOSS** at six different sections, as shown in Figure 5:

- On an interior column line in the East-West direction in the office.
- On an interior column line in the North-South direction in the office.
- On an interior column line in the East-West direction passing between the first floor of the office and the parking deck.
- On an interior column line in the East-West direction passing entirely through the parking deck.
- On an interior column line in the North-South direction passing between the first floor of the office and the parking deck.
- On an interior column line in the North-South direction passing entirely through the parking deck.

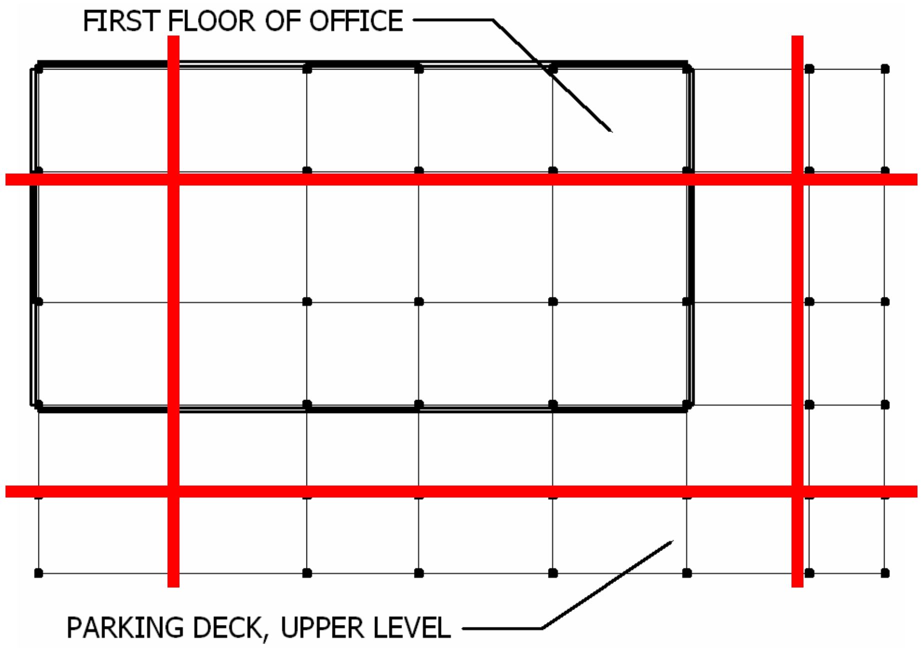


Figure 5. Frame Sections Analyzed by ADOSS in Red

Findings, Direct Design Method. Results are summarized in the Table 1. For comparison purposes, worst case reinforcement requirements at the interior support of the exterior span are presented.

Slab Type, Bay Size	Design Estimate	Notes
Office Flat Plate, 30'-0"	12" thick #7@6", As=1.20 in^2 (worst case)	Largest slab moment (417 ft-k) at interior support, column strip, end span
Office Flat Slab with Drops, 30'-0"	11" thick 3.5" thk 6'-8"x10'-0" drops #6@6", As=0.88 in^2 (worst case)	Moment distribution largely unaffected, weight reduction
Office Flat Plate with 12'x20" edge beam, 30'-0"	11" thick #7@6", As=1.20 in^2 (worst case)	Interior moment in end span effectively reduced by 40 ft-k, interior spans generally unaffected
Office Flat Slab with 12"x20" beams between all columns, 30'-0"	8" thick #5@4", As=0.91 in^2 (worst case)	Moments in slabs drastically reduced (by over 350 ft-k at interior support, column strip, end span), steel larger from smaller slab
Parking Flat Plate, 30'-0"	14" thick #6@4", As=1.32 in^2 (worst case)	Largest slab moment (632 ft-k) at interior support, column strip, end span
Parking Flat Slab with Drops, 30'-0"	14" thick 3.5" thick drops #5@3", As=1.24 in^2 (worst case)	Similar moment distribution to flat plate, larger drops required
Parking Flat Slab Slab with 14"x24" beams between all columns, 30'-0"	10" thick (slab) #5@3", As=1.24 in^2 (beam) 4-#9, As=4.0 in^2 (worst case)	Slab moment effectively reduced to 345 ft-k at interior support, column strip, end span
Office Flat Plate, 25'-0"	10" thick #6@6", As=0.88 in^2 (worst case)	Largest slab moment (298 ft-k) significantly reduced from 30'-0" span condition
Office Flat Plate with 12"x20" edge beam, 25'-0"	9.5" thick #6@6", As=0.88 in^2 (worst case)	Moment distribution not largely affected
Office Flat Slab with 12"x20" beams between all columns, 25'-0"	7" thick (slab) #5@12", As=0.31 in^2 (beam) 4-#9, As=4.0 in^2 (worst case)	Drastically reduced moments throughout al slab sections

Table 1. Summary of Estimates for Concrete Size and Required Steel Area

Initial estimates found that:

- When estimating sizes for the larger 30'-0" span, deflections came to control slab thickness; as the span reduced in length, thickness reduced significantly. However, this is using conservative deflection guidelines.
- The constructability of a flat plate system outweighs its larger thickness than with other systems; the 12" thick plate needed for the existing office area layout could be reduced to 10" if the maximum bay length were reduced to 25'-0". However, in the parking structure, a 14" slab combined with a 4" asphalt topping seems less effective.
- 12"x20" edge beams serve mostly the purpose of reducing positive midspan moment in the exterior bays, which does not significantly affect slab thickness at the more critical negative moment areas, but may affect deflection.
- 3.5" thick drop panels do not significantly affect moment distribution, but rather increase effective slab depths to reduce steel sizes.
- 12"x20" beams between all columns serve to reduce enhance flexural resistance and to reduce deflection, requiring slab thicknesses as small as 7". Though these beams will affect plenum space, they will be hidden by a drop ceiling in the office area, and are significantly smaller than existing girders in the parking structure.

Findings, ADOSS Analysis. Through changing values in ADOSS at each of the six sections, it was easy to adjust design parameters, concrete sizes, and ascertain whether each size is feasible. Three problems not completely considered in the Direct Design Method became immediately apparent:

- **Excessive Deflection.** While economizing slab depth, deflection came to control especially with larger 30'-0" spans, with two apparent solutions. A first solution would be edge beams, which are able to absorb negative moment at the exterior edge to reduce positive moment at midspan and therefore deflection. Another solution would be placing beams between all columns, which effectively absorb most midspan moment.
- **Flexure and Unbalanced Moments.** Since the smaller spans throughout the first floor of the office area in layouts 3 and 4 are more capable of absorbing unbalanced moments from the adjacent parking area, they experience deflection and flexure problems that can only be solved by a thicker slab.
- **Shear and Moment Transfer.** At the exterior edge of the floor slab, smaller column sizes provided for large shear from moment transfer through alternating load patterns. To combat this problem, larger columns in conjunction with drops were used despite relatively small compressive loads; larger column dimensions produced greater shear areas and torsional moments of inertia, reducing shear transfer. Therefore, column sizes increased to a minimum of 20" square, and since the transverse column direction affected shear transfer more than the parallel direction, rectangular columns up to 20"x30" were used.

Therefore, only two-way slab systems with edge beams and drop panels or beams between all columns were analyzed, with results summarized in the following table. As it became apparent that Layout 2 was most likely the best choice, further analysis produced varying column sizes. Results are summarized in the Table 2.

Reinforcement sizes are presented at the interior support of the exterior span, and serve as a comparison to direct design method findings. Under the first floor and parking deck, using drops instead of beams increased steel requirements within reason. Reinforcement layouts for a typical 30'-0"x30'-0" bay are shown in Figures 6 and 7.

Slab Type, Layout	ADOSS Design Summary	Notes
Office Flat Slab with 3.5" drops with 15"x15" edge beam Layout 1, 20'-0"x30'-0" bay	9.5" slab, 15" columns #7@7", As=1.02 in^2 (worst case)	Drops at edges should be thicker to combat shear moment transfer
Office Flat Slab with 3.5" drops with 20"x20" edge beam Layout 2, 30'-0"x29" bay	10" slab, varying columns 4.5" drops at ext columns #7@8", As=0.92 in^2 (worst case)	Column and edge beam sizes increased to combat moment shear transfer; ext column sizes limited by exterior wall panel size and windows
Office Flat Slab with 3.5" drops with 15"x15" edge beam Layout 3, 21'-0"x20'-0" bay	7" slab, 15" columns #5@7", As=0.53 in^2 (worst case)	
Office Flat Slab With 3.5" drops With 15"x15" edge beam Layout 4, 21'-0"x25'-0" bay	8" slab, 15" columns #6@9", As=0.52 in^2 (worst case)	Drops at edges should be thicker to combat shear moment transfer
Parking Flat Slab with beams between all columns Layout 1, 20'-0"x30'-0" bay	(office) 8" slab, 15" columns, 15"x15" beams #5@8", As=0.46 in^2 (parking) 10" slab, 18" columns, 18"x18" beams #6@7", As=0.79 in^2 (worst case)	Edge beam used between office and parking areas, shear transfer a concern in north-south direction
Parking Flat Slab with 3.5"/7" drops with 20"x20" edge beam Layout 2, 30'-0"x31'-0" bay	(office) 11" slab, varying columns, 20"x20" edge beam #7@8", As=0.68 in^2 (parking) 11" slab, varying columns, 20"x20" edge beam #9@12", As=0.96 in^2 (worst case)	Edge beam used between office and parking areas; increased drop depth at interior columns in parking area combats flexure without thicker slab
Parking Flat Slab with beams between all columns Layout 3, 21'-0"x20'-0" bay	(office) 7" slab, 15" columns, 15"x15" beams #4@9", As=0.28 in^2 (parking) 9" slab, 18" columns, 18"x18" beams #6@7", As=0.78 in^2 (worst case)	Shear moment transfer at columns a concern in east-west direction
Parking Flat Slab with beams between all columns Layout 4, 21'-0"x25'-0" bay	(office) 8"/9" slab, 15" columns, 15"x15" beams #5@8", As=0.47 in^2 (parking) 10" slab, 18" columns, 18"x18" beams #6@8", As=0.63 in^2 (worst case)	Thicker slab at office bay adjoining parking structure to combat flexure from unbalanced moment transfer, shear transfer a concern in north-south direction

Table 2. Summary of Results for Concrete Size and Required Steel Area

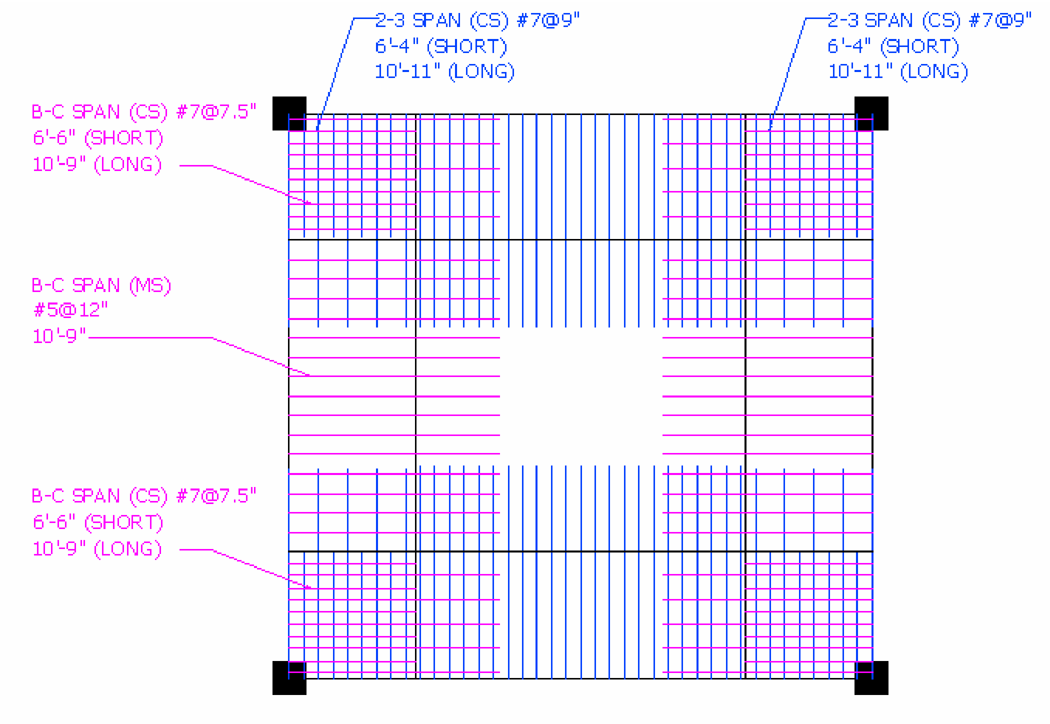


Figure 6A. Negative Reinforcement Layouts in Bay bounded by Column Lines B and C, 2 and 3

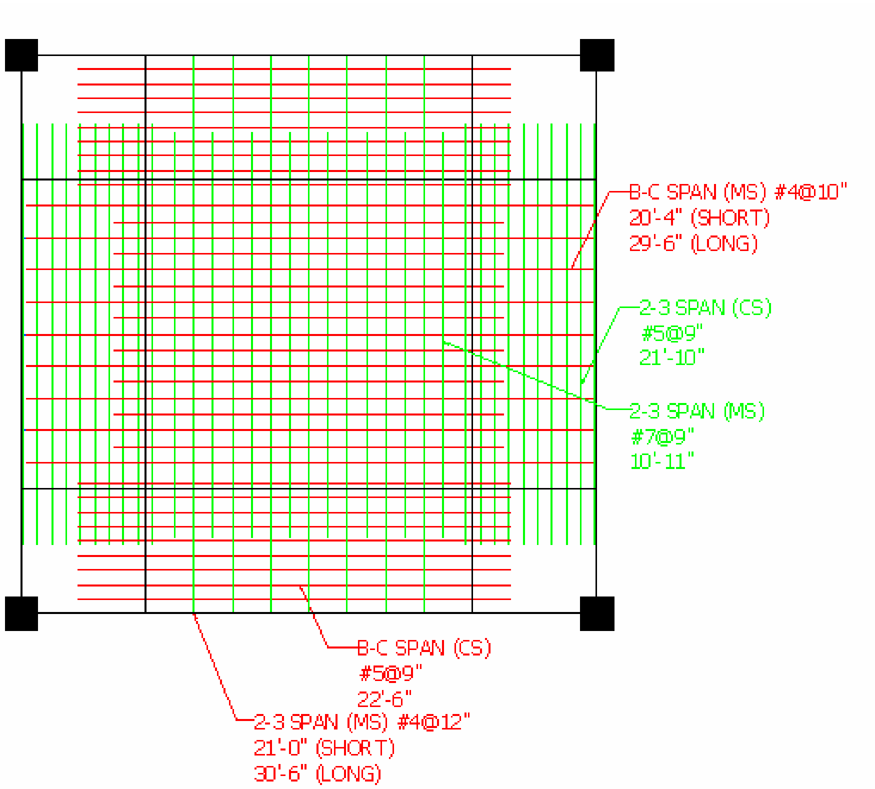


Figure 6B. Positive Reinforcement Layouts in Bay bounded by Column Lines B and C, 2 and 3

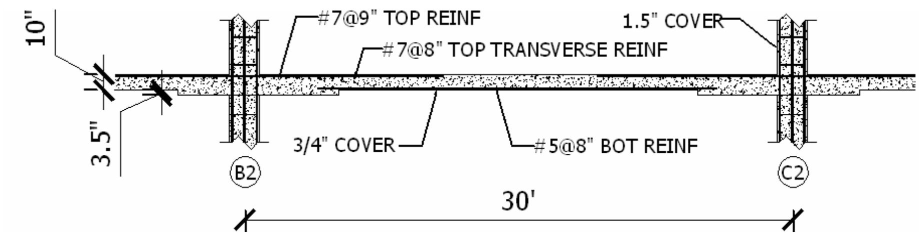


Figure 7. Sample Slab/Drop/Column Section along Column Line 2

Superimposed Dead Loads. When the 55,000-lb bank vault and two 10,230-lb air handling units on the roof were added to the ADOSS input for this design, the concrete slab design proved to be more forgiving to load irregularities than the steel system, as reinforcement areas changed to accommodate irregularities rather than the entire floor thickness. For example, when the bank vault load was applied, required steel areas in the column strip increased from 4.6 in^2 to 7.5 in^2 at midspan and from 9.24 in^2 to 12 in^2 and from 12 in^2 to 13.43 in^2 at each support, respectively. This added load served to only slightly increase moments and therefore required steel areas at supports in adjacent spans, while midspan steel areas reduced from 5.58 in^2 to 4.96 in^2 and from 7.92 in^2 to 7.48 in^2 in adjacent office and parking spans, respectively. From the perspective of moment transfer, larger 7" drops at columns adjacent to the vault would be sufficient to resist shear.

Undulating Parking Structure. This entire design assumed that the parking structure was flat when it actually fluctuates in elevation by 35" from one side to another. Though this will not significantly affect the actual slab design, the connection from the slab under the parking area to slab under the first floor of the office must be reviewed. The edge beam dividing the two areas will therefore be enlarged to provide a connection between two different elevations, and will need to be designed to torsion in addition to flexure and shear.

Shear, torsion, and moment output from the initial ADOSS analysis revealed that alternating load patterns between the parking and office span caused large unbalanced moments and therefore large torsion. Per ACI code 11.6.3.1, the size of each beam was expanded to a minimum of 20x26 along column line 4 and 24x32 along column line F to prevent cracking, while larger beam sizes accommodate variations in elevation between the office slab and parking deck. See Table 3 for a design summary, and Figure 8 for a sample detail.

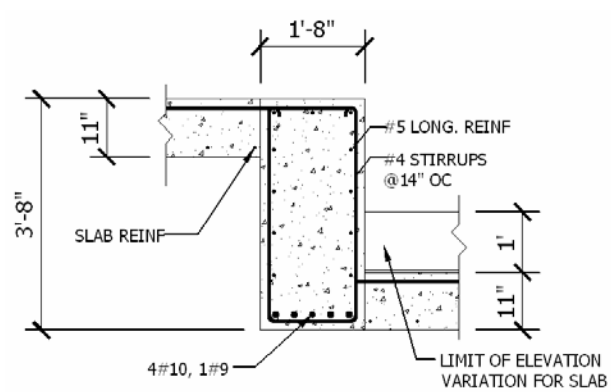


Figure 8. Sample Slab and Reinforcement Layout for Beam Spanning Column A4 to B4

Beam	Size	Max Shear	Max Torsion	Max Moment	Steel Design Summary
A4-B4	20x44	80.7	107.9	888.6	(shear) #4 stirrups @ 14" 11#5 long. Distributed on three sides (flexure) 4#10, 1#9
B4-C4	20x36	80.7	107.9	863.0	(shear) #4 stirrups @ 12" 9 # 5 long. Distributed on three sides (flexure) 4#11, 1#10
C4-D4	20X30	80.7	107.9	516.9	(shear) #4 stirrups @ 10" 7 #5 long. Distributed on three sides (flexure) 4#10, 1#9
D4-E4	20x26	80.7	107.9	7367	(shear) #4 stirrups @ 9.5" 5 #5 long. Distributed on three sides (flexure) bottom row: 4#10, 1#1 top row: 5#9
E4-F4	20x28 +2" elev.	80.7	107.9	7367	(shear) #4 stirrups @ 9.5" 5 #5 long. Distributed on three sides (flexure) bottom row: 4#10, 1#1 top row: 5#9
F1-F2	24x34 +2" elev.	96.0	151.0	606.4	(shear) #4 stirrups @ 10" 7 # 5 long. Distributed on three sides (flexure) 4#11, 4#10
F2-F3 F3-F4	24x32	96.0	151.0	606.4	(shear) #4 stirrups @ 10" 7 # 5 long. Distributed on three sides (flexure) 4#11, 4#10

Table 3. Summary of Design Considerations for Transverse Beams

Floor System Design Summary. Layout 2 was determined to be the most effective because:

- Slab section depth did not increase dramatically as the north-south spans expanded; it increased by 0.5" in the office area, and by 1" in the parking area from the existing layout.
- It reduced the number of interior columns from 12 in the existing layout to 8. Meanwhile, Layout 3 used 18 columns while Layout 4 used 12. This provides for more unobstructed open office areas.
- The reduced east-west span length in Layouts 3 and 4 conflicted with the parking layout in the floor below; a 30'-0" wide entrance ramp in the existing layout would need to be moved so it could be evenly divided by a column, which would reduce the number of parking spaces.
- 22'-6" and 30'-0" spans in the north-south direction easily accommodate precast panels for the façade in increments of 3'-9" and 5'-0", as discussed further in the architectural breadth section.

See Figure 11 for a final design drawing.

Lateral System Design

Alternatives. Since this building design is only five stories tall, and since Northern Virginia experiences mild wind and seismic loads, it was proposed that the given structure could be modeled as a system of concrete moment frames. Therefore, there is no need for shear walls or additional lateral load resistance as long as drift and lateral stresses in slabs, columns, and beams are acceptable.

The given concrete frames, as optimized for the floor system, will therefore be evaluated based on:

- Shear and flexural capacity in the slab when loaded with lateral loads, and
- Total drift of the structure.

Procedure. Using new seismic loads derived from a greater building weight, a building model was created on ETABS and new loads were placed on the floor diaphragms. Assumptions for this model include:

- All floor areas are rigid diaphragms with columns rigidly attached. These are meshed at all column lines and drops, and lateral loads are directly applied to the centroid of each diaphragm.
- All columns are considered part of a concrete frame system.
- There are five total stories, and since the first floor is a basement, lateral loads are only applied to the top four. No restraint is provided at the first level to represent ground pressures, however, because some sides of the basement area will be excavated for access to underground parking and there will be no resisting compressive ground force.

The model, shown in Figure 9, was then checked for drift in each direction.

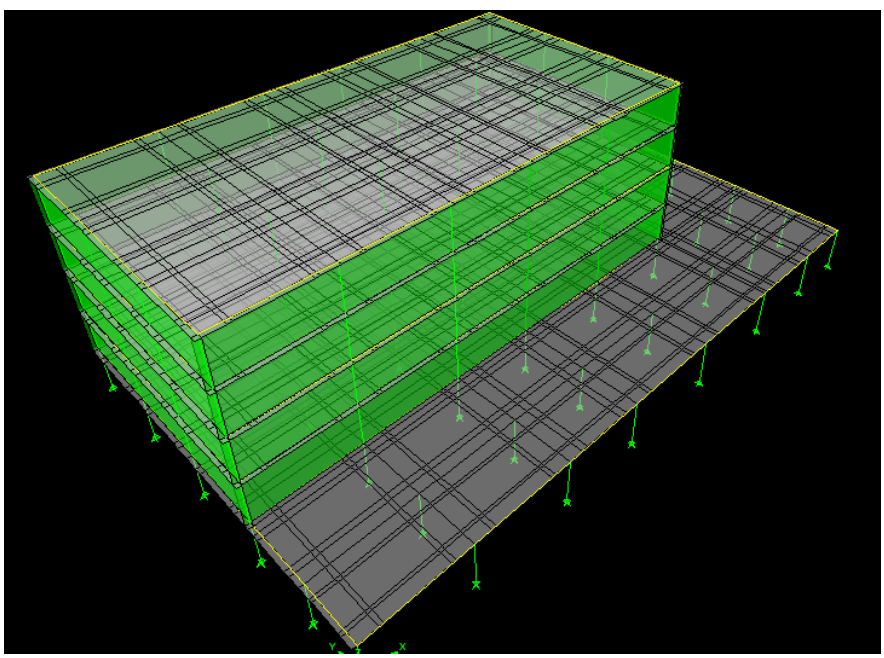


Figure 9. ETABS Model, Viewed from Southwest Corner

To assess flexural and shear capacity of the slab, first moments determined from the ETABS model were compared to a portal analysis of the concrete frames, assuming that exterior frames were half as stiff as interior frames and therefore resisted half the lateral forces. Then more conservative lateral loads were applied to the ADOSS model; since ADOSS calculates lateral loads using a simplified procedure similar to a portal analysis, this comparison ensures that larger and more conservative loads are used for the frame analysis.

Analysis Findings. Seismic loads dramatically increased due to much larger building weights than in the original steel design as shown in Table 4. With a base shear of 354 kips, these are almost double the seismic loads associated with steel construction, and these values in turn will control. For the serviceability requirement of drift, these values were then adjusted by a factor of 0.7 to bring them from ultimate to service values.

Diaphragm	Wind Load (NS) *critical wind load	Wind Load (EW)	Seismic Load
Roof	15.8k	8.3k	131k
Floor 4	31.1k	16.4k	111k
Floor 3	29.1k	15.3k	75k
Floor 2	26.4k	13.9k	37k

Table 4. Summary of New Seismic Loads

Final drift values are summarized in Table 5, and deflection in both directions is shown in Figure 10. Allowable drift is H/400, or 1.57". Therefore, these drift values are acceptable and there is no need for further lateral resisting elements than the slab and rigidly attached columns.

Load Case	Diaphragm	Drift (in)
0.7Ex	Roof	0.876
	4	0.773
	3	0.607
	2	0.394
	1	0.186
0.7Ey	Roof	0.818
	4	0.734
	3	0.605
	2	0.439
	1	0.253
Wind	Roof	0.292
	4	0.274
	3	0.237
	2	0.179
	1	0.105

Table 5. Drift Values in Both Directions Under Seismic and Wind Loads

Moments in the slab calculated using the portal frame analysis were generally greater than moments found in the ETABS model, revealing that the exterior frames may actually absorb more than half the lateral load. This more greatly affected resistance in the east-west direction, where there were only four frames.

Therefore, the same lateral loads used for the portal analysis were applied to the ADOSS model, which would analyze eight different loading patterns including both gravity and lateral loads. Results show that flexure in the slab was satisfactory; however, critical shear stresses from moment transfer in the interior columns were exceeded. Therefore, interior columns under the third floor, where lateral loads are greater, were upsized to 20x24 to increase the shear perimeter and reduce shear stresses.

See Figure 11 for a final design drawing.

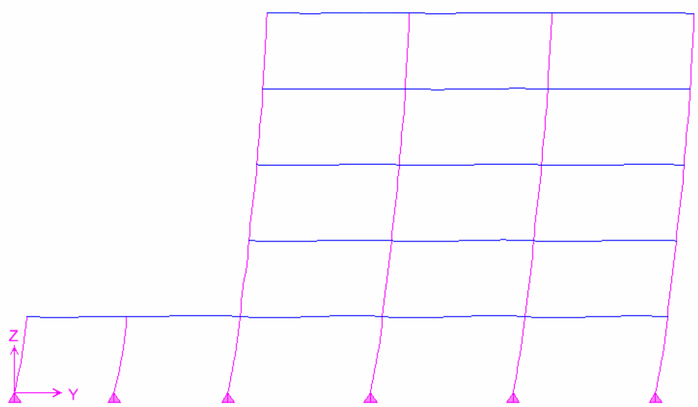


Figure 10A. Displacement from 0.7Ex

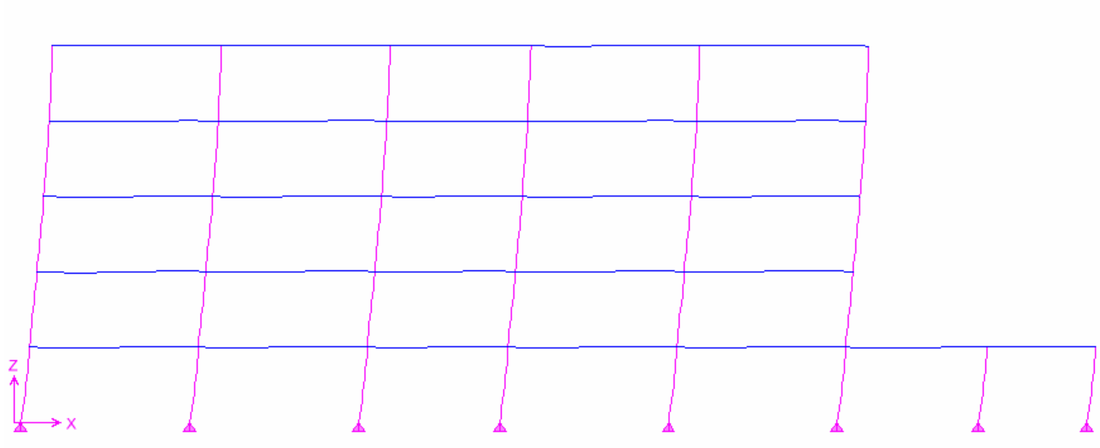


Figure 10B. Displacement from 0.7Ey

Column Design

Procedure. Initial column sizes were governed by shear transfer in the slabs and axial loads were determined directly from the ETABS model and then hand checked using tributary area. Moments in the columns were determined from the same ADOSS model used for the lateral load analysis; this way, unbalanced moments transferred to columns from both lateral loads and unbalanced gravity loading could be considered.

Upon determining moments and axial loads applied to representative columns along grid lines 3 and 5, rough steel design estimates were determined using the CRSI Handbook. For simplicity, the 1988 CRSI Handbook, with comparable load factors to ADOSS was used.

Analysis Findings. Column design considerations are summarized in Table 6. Results generally showed that:

- Moments determined on ETABS were generally less than as determined through a portal analysis. This can be attributed to an inaccurate assumption that the exterior frames only resist half as much lateral load as the interior frames; this assumption affects moments in the east-west direction more severely, as there are less frames. Larger and therefore more conservative loads from the portal analysis were used for the ADOSS analysis.
- Due to the relatively short 13'-4" unbraced length of each column and double curvature, slenderness effects could be neglected.
- While moments from lateral loads controlled in most columns, load patterns featuring only gravity loads controlled in select cases for exterior columns and columns supporting the parking deck. At these locations, unbalanced moment from large live load fluctuations between spans would be a key consideration.

See Figure 11 for a final design drawing.

Grid	Floor	Moments Top/Bottom	Axial Load	Final Design
A2	Floors 3-4	262.0/-235.1	171.1	20”x24”, 4#10
	Floors 1-2	265.4/-238.1	602.8	20”x24”, 4#10
	Parking	278.5/0	777.9	20”x30”, 6#11
B2	Floors 3-4	139.8/-103.6	407.1	20”x20”, 4#9
	Floors 1-2	153.4/-135.8	940.3	20”x24”, 4#10
	Parking	86.6/0	1234.1	24x24”, 8#10
C2	Floors 3-4	170.2/-126.8	363.3	20”x20”, 4#9
	Floors 1-2	181.3/-177.5	838.4	20”x24”, 4#10
	Parking	129.8/0	1092.9	24x24”, 8#10
E2	Floors 3-4	see B2		
	Floors 1-2	see B2		
	Parking	see B2		
F2	Floors 3-4	see A2		
	Floors 1-2	see A2		
	Parking	279.2/0	962.6	24”x24”, 4#11
G2	Parking	324.7/0	431.0	24”x24”, 8#8
H2	Parking	123.5/0	156.0	20”x20”, 4#9
A1	Floors 3-4	262.0/-235.1	154.9	20”x20”, 4#9
	Floors 1-2	265.4/-238.1	358.5	20”x20”, 4#9
	Parking	281.3/0	455.8	20”x20”, 4#9
A5	Parking	581.5/0	199.8	20”x20”, 8#18
B5	Parking	328.1/0	441.2	20”x20”, 8#10
C5	Parking	315.3/0	369.8	20”x20”, 8#8
D5	Parking	328.0/0	375.9	20”x20”, 8#8
E5	Parking	288.0/0	419.6	20”x20”, 8#7
F5	Parking	255.1/0	378.4	20”x20”, 8#7
G5	Parking	209.2/0	304.2	20”x20”, 8#7
H5	Parking	138.4/0	117.6	20”x20”, 8#7
B6	Parking	155.7/0	172.4	20”x20”, 4#9

Table 6. Summary of Representative Column Design Details

Effects on Foundation System

Procedure. By using basement level column loads from the original steel analysis and the given 5000 psf soil bearing capacity, the original factor of safety can be determined. Using this factor of safety, new column takedown loads were used to size new footings. Since the original building was modeled to have pinned connections at the footings, any possible moment is determined to be minimal and only axial loads were considered.

Analysis Findings. Using a general factor of safety of 2, it was determined that though the spread footings under each column will drastically enlarge to offset heavier axial loads, the new sizes are still reasonable for the given design. See Table 7 for a summary of design conditions and Figure 12 for a design detail.

Column	New/Old Axial Loads	Old Size	New Size	New Size Reinforcement
A3	(new) 579k (old) 198k	9'x9'x28"	13.5'x20'x28"	(long) 41#6 (short) 40#6
B5	(new) 305k (old) 110k	6.5'x6.5'x20"	11.5'x11.5'x28"	23#6 each direction
D2	(new) 810k (old) 251k	8'x8'x24"	16.5'x20'x34.5"	(long) 50#6 (short) 40#6
D4	(new) 639k (old) 273k	8'x8'x24"	15'x18'x30"	(long) 45#6 (short) 36#6
F4	(new) 538k (old) 254k	9'x9'x28"	15'x15'x28"	30#6 each direction
G2	(new) 303k (old) 94k	6'x6'x18"	11.5'x11.5'x28"	12#6 each direction
D1	(new) 532k (old) 226k	8'x8'x24"	12'x18'x26"	36#6 each direction
D6	(new) 104k (old) 57k	6.5'x6.5'x20"	9'x9'x12"	9#6 each direction
H3	(new) 103k (old) 54k	6'x6'x18"	9'x9'x12"	9#6 each direction

Table 7. Summary of Representative Footing Design Details

See Figure 11 for a final design drawing.

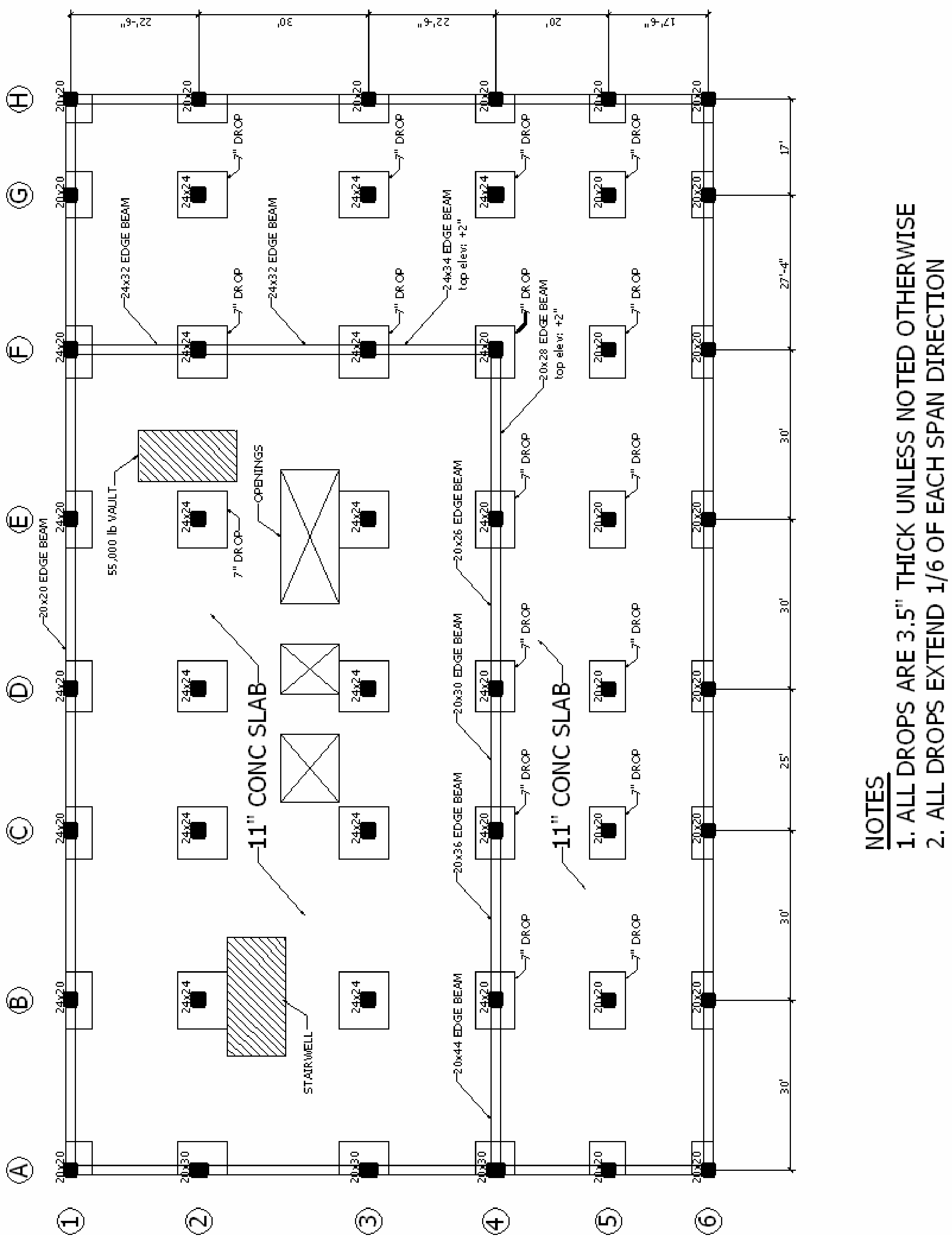


Figure 11A. First Floor/Parking Deck Final Design

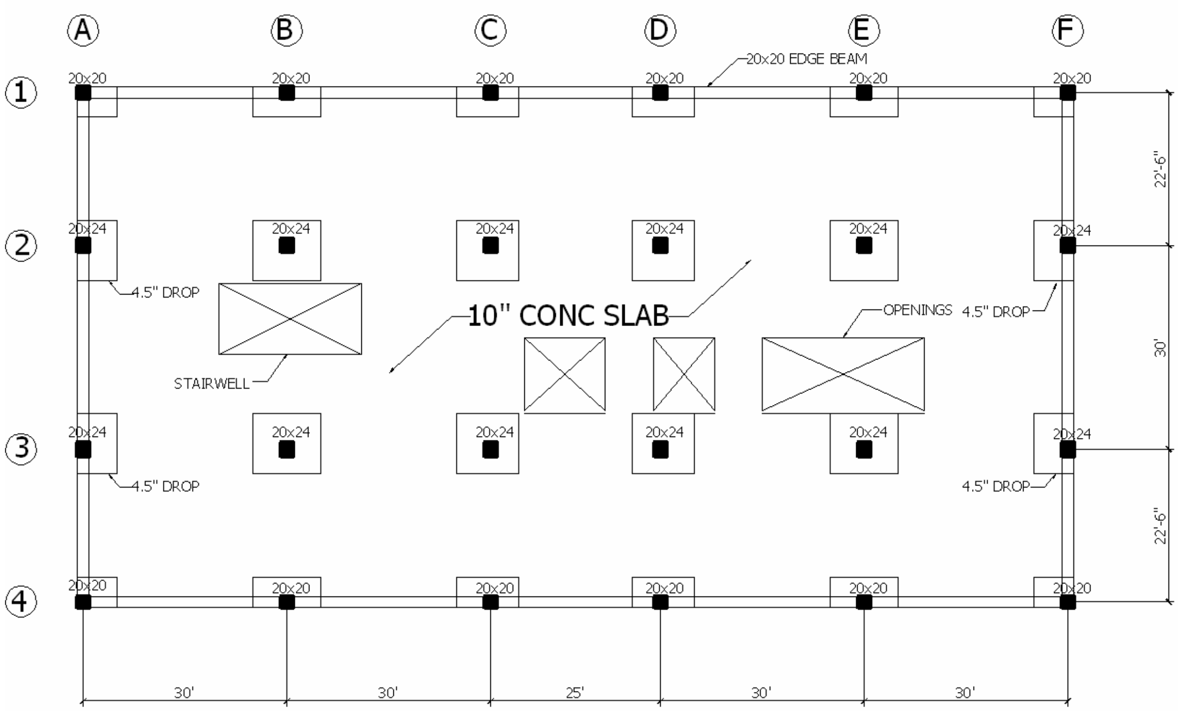


Figure 11B. Second, Third, and Fourth Floor Final Design

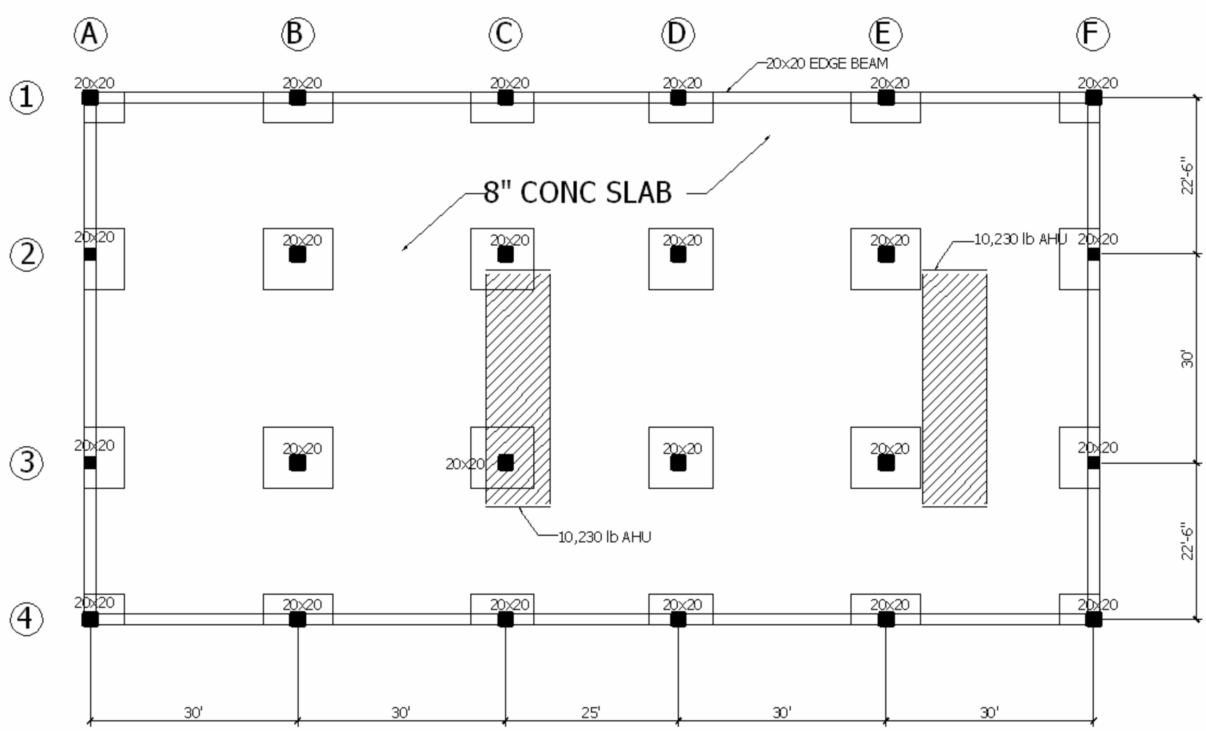


Figure 11C. Roof Floor Final Design

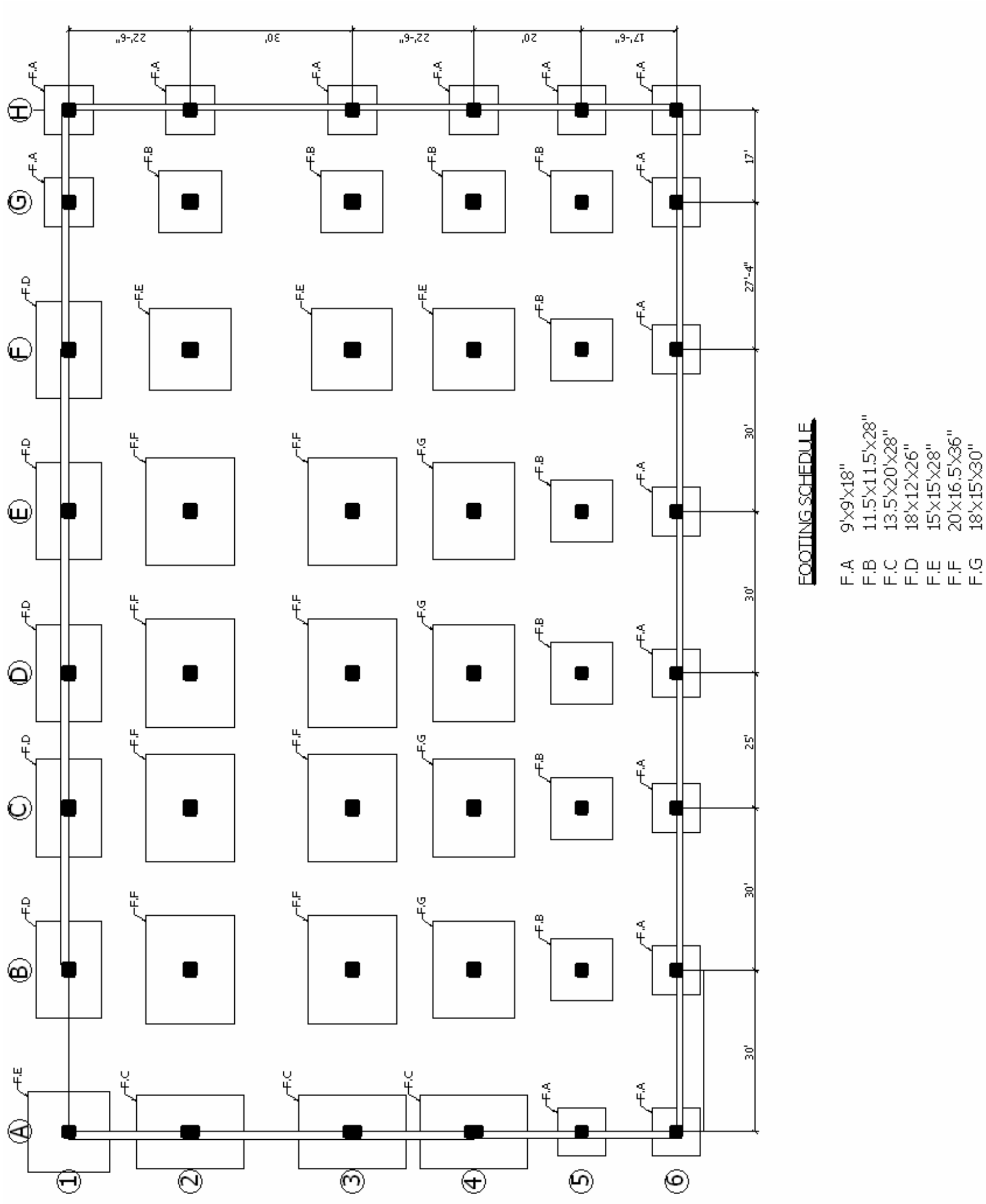


Figure 11D. Revised Footing Layout and Schedule

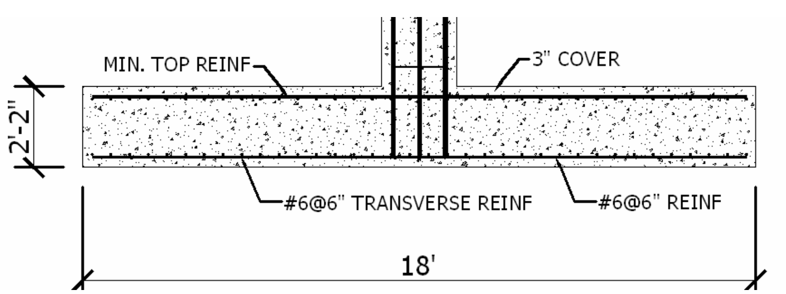


Figure 12. Sample Footing Detail