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1.0 EXECUTIVE SUMMARY

Gateway Plaza is a 15-story office tower located in the Central Business District of downtown Wilmington, Delaware. The $52 million project began in July 2005 and is projected for completion in December 2006. The 16-story, 210'-6" tower will offer 387,000 square feet of rentable office space for tenant fit-out. The architect of design is Gensler and the property owner is Buccini Pollin Group. The ground floor is a public plaza level including a restaurant, post-office, and WSFS branch bank. In the rear of the building there is a 5-story parking garage housing 600 car parking spaces for the building's employees.

Figure 1-Rendering of the front of Gateway Plaza. (Photo courtesy of Gensler).

This report is an in-depth study of the structural system of Gateway Plaza. The objective of the report is to gain knowledge and experience in post-tensioned concrete design and methods for reducing floor depth. RAM Conce*pt* software was utilized to determine deflection and stress plans for various loading conditions. The design resulted in a one-way post-tensioned slab that is 8" thick, with 6-strand unbonded tendons spaced approximately 6.25' o.c. The beams supporting this slab are also post-tensioned and range in size from 20"x24" using 12 strands to 36"x24" using 39 strands. The columns and floor system use 6000 psi concrete. Additionally, concrete shearwalls were designed to be 12" thick using 4000 psi concrete and work in conjunction with the concrete frames to resist lateral loads, which were developed using ASCE 7-02.

Two breadth studies were also performed to understand design from the point of view of the other building systems. These studies focus on how concrete design has impacted the mechanical system, specifically the layout of ducts, and the cost and schedule factors in managing the construction.

Through these feasibility studies it has been determined that the post-tensioned concrete redesign of the structure is a good alternative, but not likely to be utilized in the given building market of Wilmington, Delaware.

2.0 BUILDING INFORMATION

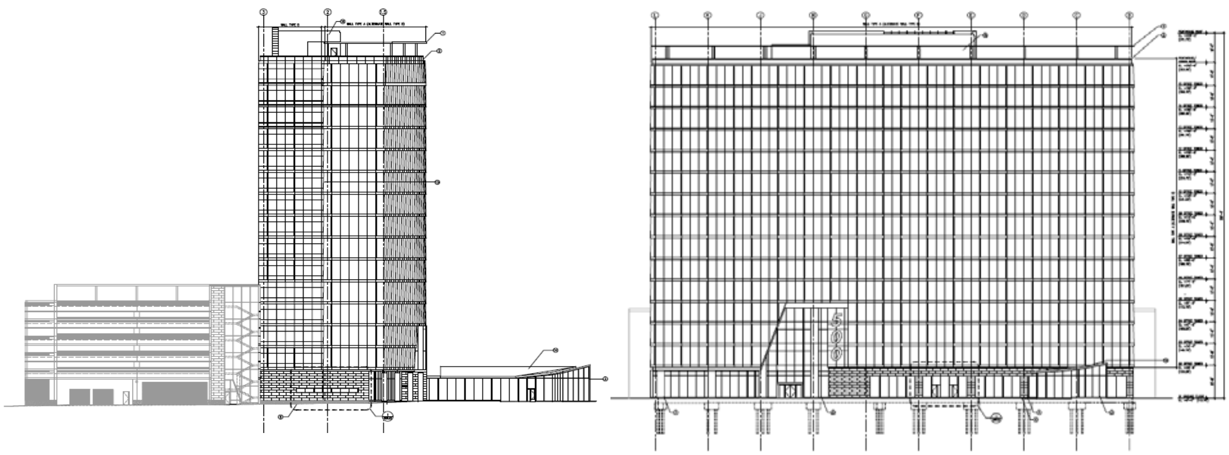


Figure 2-Front of side elevations of the office tower and parking garage. (Drawings courtesy of Gensler)

Gateway Plaza is a 15-story office tower located in the Central Business District of downtown Wilmington, Delaware. The $52 million project began in July 2005 and is projected for completion in December 2006. The 16-story, 210'-6" tower will offer 387,000 square feet of rentable office space for tenant fit-out. Two of the building's major occupants include the law firm of Morris, James, Hitchens & Williams and the WSFS Financial Corporation. The ground floor will play host for public retail including an indoor/outdoor café, post-office, and WSFS branch bank. In the rear of the building there is a 5-story parking garage housing 600 car parking spaces for the building's employees.

Primary Project Team

- Owner/Developer: Buccini/Pollin Group
- General Contractor: Gilbane
- Architect: Gensler
- Engineers:
  - *Civil*: Landmark Engineering
  - *Geotech*: Duffield Associates, Inc.
  - *MEP*: BALA Consulting Engineers, Inc.
  - *Structural*: O'Donnell, Naccarato & MacIntosh
- Project Delivery Method: Design-Bid-Build

Zoning

- Office tower
  - C-4 Use Group
  - Business Use Group B
- Garage
  - S-2 Garage
  - Construction Type: IB
  - Type 1 Protected

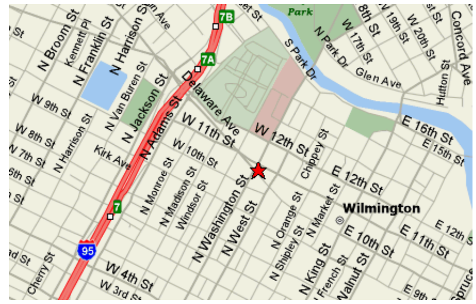


Figure 6-Map of downtown Wilmington and location of Gateway Plaza. (Photo courtesy of Mapquest)


Figure 6-Projected site plan.

Location and Site

Gateway Plaza is located at 500 Delaware Avenue, between West and Washington Streets. The building and its amenities are situated in the northwestern quadrant of the Wilmington's Central Business District, which is home to many of the nation's largest financial corporations. It is on the top of a hill with views in all directions of the city, including the waterfront in the south. To get a better view of the location, see the map on the right.

The urban site on which Gateway Plaza is constructed is currently a privately owned parking lot. To determine the soil composition, borings were taken by a subcontractor to the geotechnical engineer, Duffield Associates. Ten samples were taken, and the results of laboratory tests are summarized in the soils report. Such an urban site generally consists of contains a layer of miscellaneous fill (i.e. concrete, asphalt, brick, metal, etc.) that ranges in depth from 12'-16' below the existing ground surface. 'Medium to dense, coarse


Figure 6-Rendering of northwest corner from ground level.

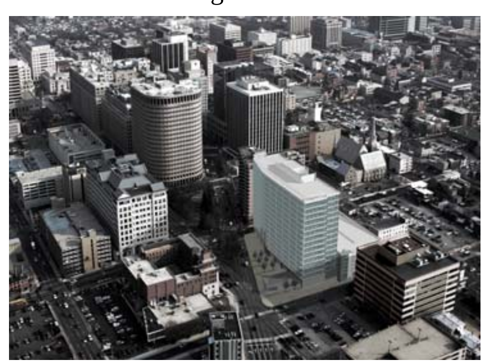


Figure 6-Aerial view of Wilmington with rendering of Gateway Plaza superimposed. (All three renderings courtesy of Gensler)

grained soils and medium to stiff consistency silt and clay soils lay beneath the miscellaneous fill layer. Their depths range from approximately 57'-63' below the existing ground surface." Finally, dense to very dense sand soils, evidence of weathered bedrock, lay below the sandy silt layer.

Architecture

Gateway Plaza is the first new building to be constructed in Wilmington's Central Business District (CBD) in 15 years. The tower will fill in the gap between neighboring office towers. It is predicted to be a landmark among all of the CBD's office towers because it will have one of the few all-glass curtain walls in town. The curtain wall will give the façade a more modern feel than the dated 1970s architecture of the DuPont Hotel and the "cookie-cutter" appearance of the Sheraton Hotel, which both have concrete facades. The northeast quadrant of the tower features a corner that appears to be "sliced" off at an angle that imitates the angle of Delaware Ave.

The main entrance to the lobby has a 5-story cut-out of the front façade. The public café on the ground level adds interest to the building because it protrudes out of the footprint of the main tower and features a kinked, standing seam, metal roof. The protrusion creates a courtyard in front of the entrance and allows for shaded outdoor seating, a new feature to the CBD. The remaining 14 stories of the tower remain office space for tenant fit-outs.

Building Envelope

The bulk of Gateway Plaza's primary office tower is enclosed with a glass curtain wall system featuring a reflective glazing. It is characterized by the overlapping, shingle-type construction of blue-tinted glazing. The first floor, however, is a flat lock, zinc wall panel system which is also carried up the office tower on the spine of the south end. The parking garage is clad with pre-cast concrete panels. There is a painted, metal panel screen wall on the roof serves to hide the mechanical penthouse. The roof is an EPDM system using polyisocyanurate insulation over fiberglass sheathing on metal deck.

Figure 7-Mock up of curtain wall system. (Photo courtesy of Gilbane).

2.1 Primary Engineering Systems

Mechanical

The variable volume mechanical system of Gateway Plaza is located on the roof in a mechanical penthouse. Here, the 3-cell water cooling tower and heat pumps supply conditioned air to all 15 floors of the building. On floors 2-15, there are direct exchange air conditioning units using R-22 refrigerant. In the building's IT/Telephone room on the first floor, there is a Liebert Challenger 3000 air conditioning unit to provide up to 5 tons of cooling control the temperature and humidity required by the sensitive computing equipment.

Since the building is to be fit out, space for the risers has been designated to a mechanical room found on each floor in the tower's service core. Return air from each floor is dumped into the mechanical room and is exhausted. There are outdoor air louvers in each room to replace the exhausted air and provide ventilation.

Electrical

Gateway Plaza's electrical system is powered by both 480/277V and 208/120V panel boards. Power is supplied to the building by two 2500/3325 kVA transformers. There are two main distribution panels that service floors 1-8 and floors 9-roof, respectively. The voltage is stepped down through transformers on the second, fifth, eighth, eleventh, and fourteenth floors in electrical rooms in the building’s core. A 1600A bus riser supplies power to the typical office levels for lighting and receptacles. There is a 750kW, 938 kVA emergency generator which services the fire pumps.

Lighting

Lighting for the service core of the typical office floors is provided with 6" open reflector down lights with compact fluorescent lamps. For the tenant spaces, the lighting plans will be finalized upon fit-out. On the exterior plaza area, metal halide downlights flood the walls of the first floor with light. Illuminating strips with tempered glass lenses are located under trees to light the plaza and seating areas.

2.2 Additional Engineering Systems

Fire Protection

Almost every area of Gateway Plaza is protected by a wet sprinkler system with a 4" wet standpipe in each of the two stairwells. The water is supplied to the standpipes by a fire pump and a jockey pump for the higher floors; both are located on the 1st floor in the water service room. The fire pump has a capacity of 750 GPM and the jockey pump has a capacity of 109 GPM. These can run on emergency power supplied by a diesel generator.

Transportation

Gateway Plaza has a central service core servicing most of the building's transportation needs. There are 5 public elevators, 1 service elevator, and two stairwells in the core. The public elevators and stairwells run to all floors of the office tower and the service elevator will also run to the mechanical penthouse.

The main entrance to the public is on Delaware Ave. and will have a vestibule to control air loss. The service core can be accessed from this main entrance, while the public areas will have separate entrances also along Delaware Ave. including one to the café and one to the WSFS bank.

Telecommunications

The building has a main technology/data center on the ground floor. It is located in the parking garage and will service all of the security systems and telephone lines in the building.

3.0 STRUCTURAL SYSTEM DESCRIPTION

Gateway Plaza has two distinct buildings and two distinct building systems. The office tower is composite steel construction with braced frames and the parking garage is pre-cast concrete with pre-cast shearwalls. The two structures are separated using a 2” expansion joint. The structural engineer is responsible for designing the office tower and the pre-cast supplier is responsible for the design of the garage. For this reason, focus will be centered on the office tower design.

Foundations

Due to the poor soil conditions on site, deep foundations were deemed necessary for the high gravity column loads and overturning moments from lateral resisting elements. The soils report recommended end-bearing caissons, but due to designer preference, concrete filled, steel tube piles were chosen. The piles are 12” in diameter and use high strength concrete to develop 120 tons of end-bearing capacity per pile. Most of the piles are drilled 70’ down to bedrock. The clusters are topped with pile caps that range from 40”-65” thick. Grade beams span the pile caps around the entire building’s perimeter, and a 5” slab on grade span the grade beams in much of the foundation and are intermittently supported by single piles.

The office tower’s steel columns sit on pile caps of various shapes. The columns from the lateral load resisting system generally sit on clusters of 18 piles or more where those from the gravity system sit on clusters of 6-12. The larger foundations under the lateral frames are to resist the overturning moment from wind and seismic loading. The pile clusters in the office tower are on a 30‘x52’ grid on the north side and a 30’x36’ grid on the south side (see the foundation plan on next page).

The vertical support members in the parking garage sit on pile caps that are 40-50” thick and are generally square. The lateral load resisting system of the garage, pre-cast concrete shearwalls, sits on 7-pile clusters while the cast-in-place concrete columns rest on 4-pile clusters. The clusters in the parking garage are on a 30’x62’ grid (not shown).

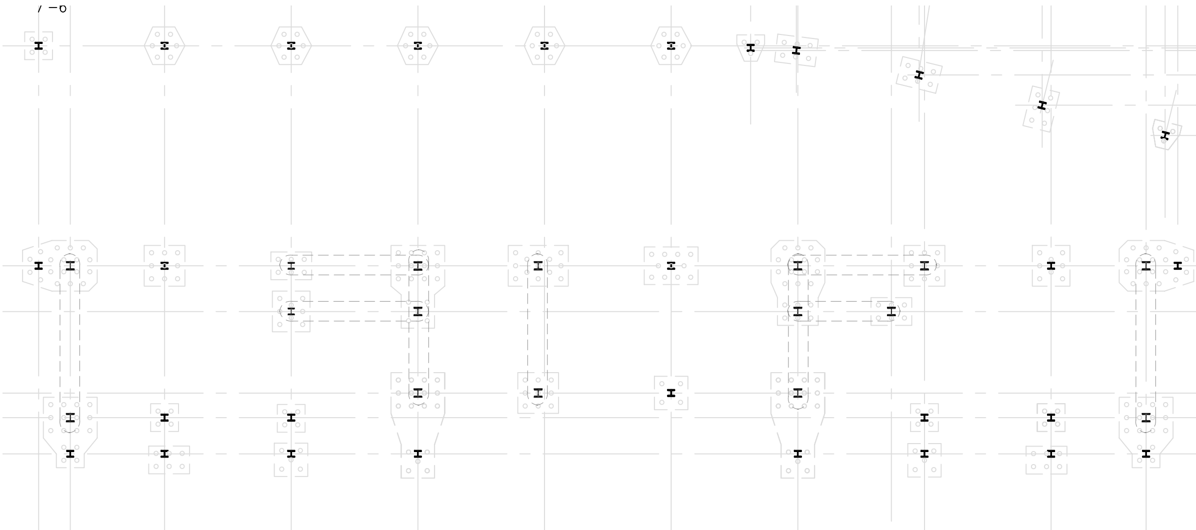


Figure 8-Foundation plan

Framing

Figure 9-Steel framing during construction. (Photo courtesy of Gilbane)

Gateway Plaza uses two types of framing systems for its two types of use: composite steel for the office tower and precast/cast-in-place concrete for the parking garage.

The composite steel in the office tower uses two grid systems: one orthogonal and one rotated. As aforementioned, the grids are 30'x52' and 30'x36'. The rotated grid is an adaptation of the orthogonal grid that is turned 14° clockwise from plan north, and is used to create the sliced surface on the northeast face. The columns are spliced every other floor or 27'. All framing members on the office floors use wide-flange shapes of A992, Gr. 50 steel where the framing of the penthouse and screen-wall on the roof use HSS tube shapes of A36 steel. All of the columns, in both the lateral and gravity systems, are W14 shapes of various sizes. The girders and beams range in size but are usually W18 or W24 shapes. The plan below shows the framing for a typical floor. The dashed lines indicate the locations of the braced frames.

The garage is typical construction of cast-in-place concrete columns and pre-cast concrete beams. The pre-cast double tee beams span girders that are pre-cast L-beams. Sizing of these members is left to the pre-cast contractor.

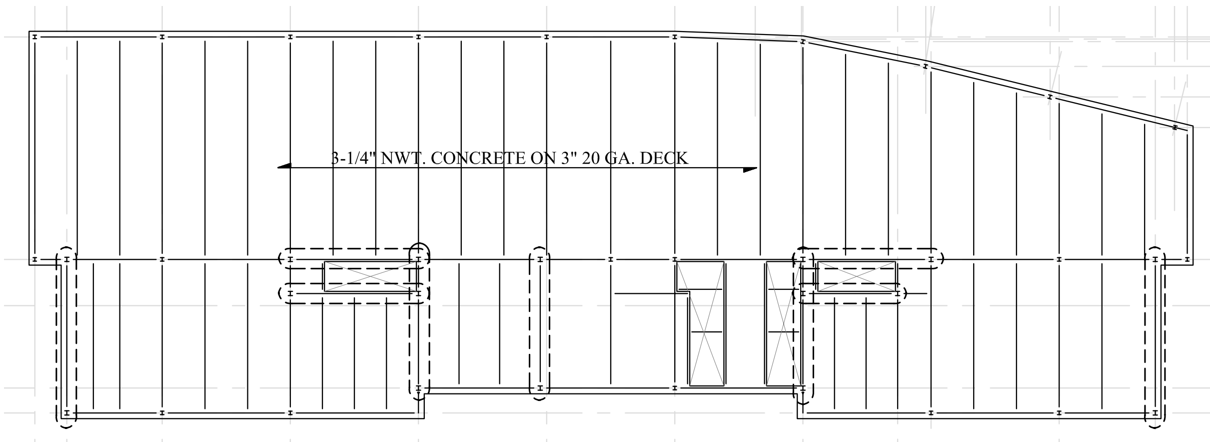


Figure 10-Framing plan of a typical floor.

Structural Slabs

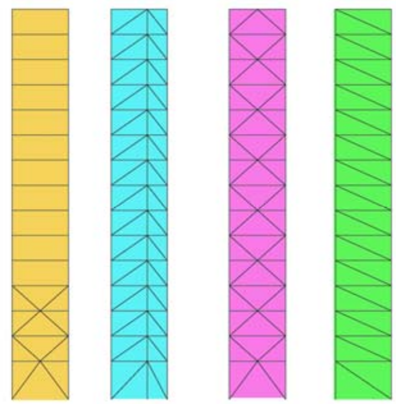
There are 6 types of structural slabs used in Gateway Plaza: three slabs on grade and three supported metal deck slabs.

Each type of slab on grade is normal-weight concrete cast over 6” crushed stone and compacted fill. The first type of slab on grade (SOG) is used in the on a majority of the ground level where there will be retail space and parking. This type is 5” of concrete with 6x6-W2.9x2.9 WWF reinforcing. Another type of slab on grade is 6” of concrete with 6x6-W2.0x2.0 WWF reinforcing. This SOG can be found in the loading area for the retail occupants. The final type of SOG is 12” concrete reinforced with #8 @ 12” o.c. each way in the top and #6 @ 12” o.c. each way in the bottom. This larger slab is used in the loading dock that is accessible by the entire building.

All three types of supported slabs on deck utilize ¾”ϕ shear studs and concrete with a compressive strength of 3000 psi. Two types of supported slabs are 3-1/4” light-weight concrete on 3” Lok-Floor composite deck and use 6x6 W1.4x1.4 WWF reinforcing. The difference between them is the gage of the deck. One type is 20 gage and used in the office area of all the elevated floors, the other is 16 gage which will be shored during construction and will be used in the mechanical areas of each floor. The third type of supported slab is found on the penthouse floor and is 2-1/2” of normal-weight concrete on 1-1/2” Lok-Floor composite deck. It, too, uses 6x6 W1.4xW1.4 WWF reinforcing.

Lateral Load Resisting System

The lateral load resisting system for Gateway Plaza utilizes ordinary moment frames and concentrically braced frames. The bracing members are not the traditional, A36, angles but are A992 wide flange shapes. There are five frames resisting load in the north-south direction, and four in east-west direction. The braced frames are located around the service core of the building, where bracing is not a concern. The moment frames are located on the east and west edges of the building, which is exposed by curtain wall. The location of these frames does a good job at preventing torsion by keeping the floor’s center of rigidity very close to its center of mass. However, this is only the case for loads in the north-south direction. The structure may be subjected to torsion in the case of loads in the east-west direction because the center of rigidity is further from the center of mass.



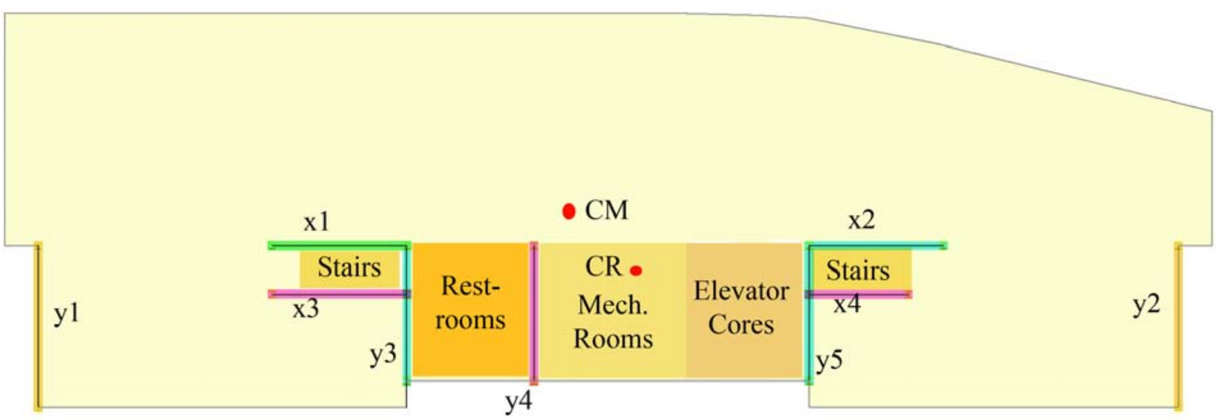


Figure 11-Location of braced frames and centers of mass and rigidity.

4.0 PROBLEM STATEMENT AND SOLUTION PROPOSAL

An area for redesign was difficult to pinpoint with Gateway Plaza. After the research and analysis performed during Technical Assignments 1-3, it has been determined that the current structural systems--steel framing, concentrically braced frames, and deep foundations--was the best given building type, local conditions and accepted practice. Though no feasible framing alternatives were found during research for *Technical Report 2: Pro-con Study of Alternate Framing Systems*, further research and faculty consultation has suggested that a one-way post-tensioned concrete slab system is worth considering. There are no height restrictions dictated by the architect or zoning ordinances, but the post-tensioned system could add a significant amount of ceiling space. The purpose behind this design is to gain knowledge and experience in the design of post-tensioned concrete systems in buildings.

Proposed Problem Solution

As mentioned in the problem statement above, post-tensioned concrete slabs and beams will be designed to replicate the architectural requirements of the building by adhering to the given column grids. To achieve this, cast-in-place columns, post-tensioned beams and slabs, and shearwalls will be designed. Although foundations will not be explicitly designed, they will be sized approximately for end bearing strength. Making an appropriate comparison of the concrete system to the steel system will require the consideration of the following factors: cost, project duration, and impact on foundations.

Method for Solution

In order to redesign the building using post-tensioned concrete slabs, research must initially be performed to gain knowledge in how to design such a system. By researching texts, journals, and code manuals, a good deal of technical knowledge should be gained. By talking with students whose thesis buildings use post-tensioned concrete in their existing designs and professionals in the industry, significant knowledge of practical design shall be gained.

Once a significant amount of information has been gathered, schematic designs will commence. A preliminary framing plan will be laid out and the office floors will be designed using loads obtained in *Technical Report 1: Existing Building Conditions*. The first round of analysis for all members will be performed by hand calculations using ACI 318-05 *Building Code Requirements for Reinforced Concrete* and *The Post-tensioning Manual*. Further analysis will be completed using structural software including: ENERCALC, RAM Structural System, and Conce*pt*. Finally, wind and seismic loads will be computed by hand and applied to the structure. A design for the lateral system will be designed and checked in RAM Frame for all possible loading cases and combinations as laid out in Chapter 2 of ASCE 7-02.

Once the design has been finalized, a proper comparison shall be made between the composite steel framing and the post-tensioned concrete framing based on the factors mentioned above.

5.0 DESIGN CRITERIA

5.1 Design Objectives

The main objective of this project is to find an alternative structural system that will perform as well as steel in achieving long-span bays. Additionally, the chosen type of system should allow greater control in determining floor depths. Therefore, keeping these depths to a minimum will be of great importance. In order to meet this goal, the following standards must be met:

- Long bay spans must be preserved.
- Service spaces in the building's core must remain unchanged.
- Limit the overall floor depth to 24".
- Keep shearwalls in locations similar to those of the braced frames.
- Design must be in compliance with model codes set forth by ACI 318-05, IBC 2003, and ASCE 7-05.

5.2 Design Procedure

To make this project manageable, one of the main assumptions considered is that floors 2-5 are one type of typical floor and floors 6-15 are a second type of typical floor. Since the office tower is the responsibility of the structural designer, it alone will be considered for the redesign. The garage will be omitted from design because it is the responsibility of the precast manufacturer.

All schematic designs will be performed by traditional hand calculations using procedures outlined in concrete design texts by Antoine Naaman and Charles Nilson as well as ACI 318-05. Spreadsheets containing the calculations embedded in these procedures will be created to ease repetition. The preliminary designs yielded by the spreadsheets (available in *Appendix B: Preliminary Member Design*) will be entered into ENERCALC to confirm hand calculations. A structure of these preliminary members will be modeled in RAM structural design software for refinement. The gravity system will be checked and refined using RAM Concrete and post-tensioned elements including slabs and beams will use RAM Conce*pt*. Conce*pt* will be used to check concrete stresses and deflections under service conditions as well as design minimum reinforcing. The lateral system will be analyzed further using RAM Frame to scrutinize lateral loads and compare them to hand calculations, as well as obtain building drifts under various load cases and combinations.

5.3 Loading Conditions

5.3.1 Gravity Loads

5.3.1.1 Dead and Live

LOADING IN POUNDS/SQUARE FOOT	Office Floors	Mechanical Rooms	Penthouse Floor	Main roof	Penthouse Roof
Concrete Slab	100	100	100	50	50
Roof & Insulation				5	5
Ceiling	5	5	5	5	5
Collateral	5	5	5	5	5
Mechanical	5	5	5	10	10
Total Dead Load	115	115	115	65	65
Total Live Load	80	125	150	60	60

Table 1-Gravity loading information.

Dead loads include the self-weight of the structure and any additional loads accounting for sprinklers, MEP, and collateral loading. It is evident that the total dead load is greater for the concrete design than it was for the composite steel design, 115 psf compared to 65 psf. The live loading conditions comply with those set forth in IBC 2003.

A majority of the office tower is classified as office occupancy which results in a live load of 60 psf, plus an additional 20 psf for partitions. Although the loads are not consistent with those of the original designer, they are still conservative and comply with IBC 2003. There are portions on each floor considered to be service spaces which house HVAC and electrical equipment in mechanical rooms. These spaces, subjected to heavy equipment loads, will be designed for a 125 psf live load. This area is illustrated on the typical floor in the diagram above. The table above summarizes the gravity loading conditions in each type of occupancy.

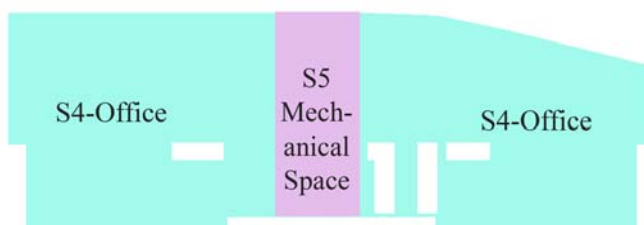


Figure 12-Diagram of loading conditions.

5.3.1.2 Snow Loads

The roof will inevitably be designed for snow loading. But since there is a building setback where the main roof meets the penthouse, drifting has the possibility of becoming an issue. Drifting can occur on the north and west sides the penthouse indicated by the lavender areas labeled 1 and 2 in the diagram to the right. A spreadsheet was developed according to the ASCE 7-02 guidelines set out in Chapter 7. In section 1, the maximum drift load was found to be 49 psf and in section 2, the load was found to be 63 psf. For further details on the calculations, please refer to *Appendix A.1: Snow Loading*.

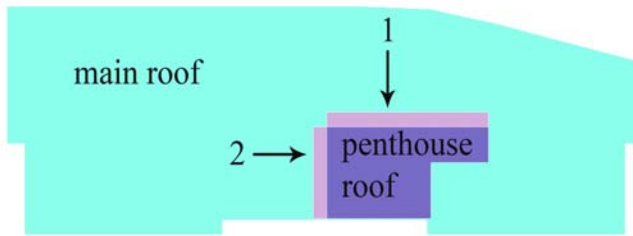


Figure 13-Areas of concern for snow drift.

5.3.2 Lateral Loads

The lateral loads, both wind and seismic, for the building were found using the guidelines set forth in IBC 2003 and ASCE7-02. Complete calculations were found using spreadsheets; please refer to *Appendix A.2: Lateral Loading* for intermediate steps. Although the wind loads for Gateway Plaza did not change due the redesign, seismic loads increased dramatically due to the increase in the structure's weight.

The wind load is distinctly greater in the north-south direction because the building dimension perpendicular to this direction is 270', which is three times larger than that in the opposite direction, and collects a great deal more pressure. In the east-west direction, however, seismic loads were found to control due to the increase in building weight. Considering both load types--seismic and all four cases of wind--and including accidental eccentricity, the only cases that resulted in unfavorable results were those that included eccentricity. In addition to the existing eccentricity between the center of rigidity and the center of mass, the accidental eccentricity created unfavorable rotations. Load combinations checked by RAM Frame include those from ASCE 7-02 in Chapter 2.0 for strength design:

1. $1.4(D)$
2. $1.2(D)+1.6(L)+0.5(L_r orS)$
3. $1.2(D)+1.6(L_r orS)+(0.5Lor0.8W)$
4. **$1.2(D)+1.6(W)+L+0.5(L_r orS)$ Controls in N-S**
5. $1.2D+1.0E+0.5L+0.2S$
6. $0.9(D)+1.6(W)$
7. $0.9(D)+1.0(E)$ **Controls in E-W**

5.3.2.1 Wind Loading

Wind Loads on the Main Wind Force Resisting System were found according to the Analytical Procedure, outlined in Section 6.5. To find the story forces and shears, a tributary area approach was taken. The pressure at each floor level was distributed over an area equal to half the floor height above and below the level. As would be expected from a building on the coastal Northeastern United States, wind is the controlling load case in the north-south direction. The basic wind loading characteristics are:

- Basic Wind Speed: 90 mph
- Wind Load Importance: 1.0
- Exposure Category: B
- Internal Pressure Coefficient: +/- 0.18
- Height: 210.5”
- Maximum wind pressure at roof: 23.3 psf

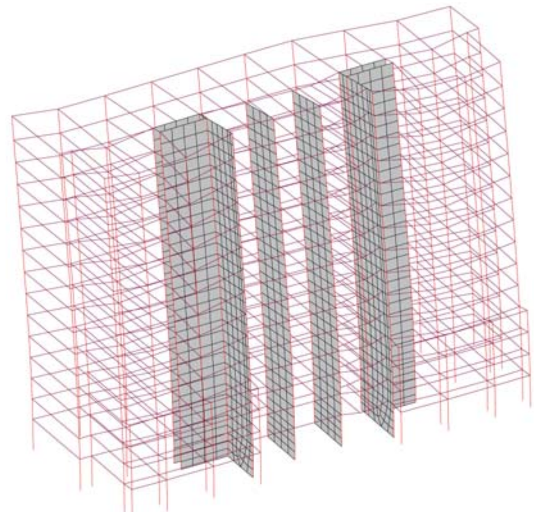


Figure 14-Structure under wind loads in north direction.

These characteristics were used to find the following loads on the building in both characteristic directions. The loads and controlling drift tabulated on page 20 are summaries of RAM output, which are similar to the hand calculations performed according to ASCE 7-02. ASCE 7’s Case 4 resulted in a building drift of 2.96”, approximately h/850, which is considered very acceptable.

5.3.2.2 Seismic Loading

Seismic Loads were found using the Equivalent Lateral Force Procedure as laid out in Section 9.5.5 of ASCE 7-02. Seismic loading characteristics include:

- Site Class: D
- Spectral Response: 0.3
- 1-second Spectral Response: 0.075
- Design Spectral Response: 0.32

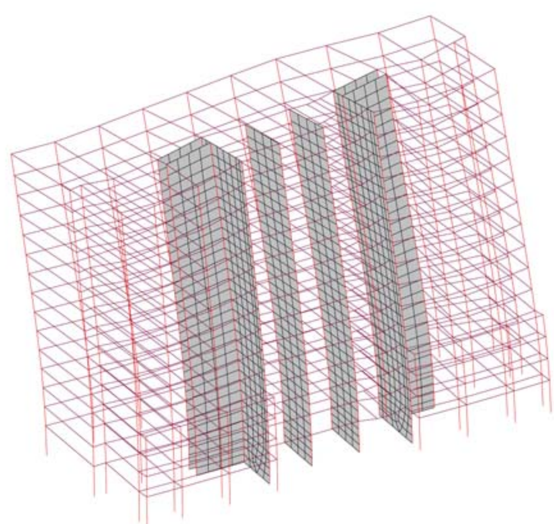


Figure 15-Structure under seismic loads in the east-west direction.

- 1-second Design Spectral Response: 0.12
- Seismic Use Group: II
- Seismic Design Category: B
- Seismic Importance Factor: 1.0
- Response Modification Factor: 3
- Base shear: 709 k

These characteristics were used to find the following loads on the building in both characteristic directions. The loads and controlling drift are tabulated on the next page. Case 7 was shown to control drift in the east-west direction, 2.25". Again, this is an acceptable drift limit.

The table on the next page summarizes the story shears in both, north-south and east-west, directions according to RAM Frame and hand calculations performed according to ASCE 7-02. Additionally, it tabulates the maximum story drifts according to the controlling load combinations. The diagrams below depict how these forces act on the building in both directions.

Story Shear due to Wind					Story Shear due to		Story Drift	Story Drift
	X		Y		Both X & Y		**X**	**Y**
	ASCE 7	RAM	ASCE 7	RAM	ASCE 7	RAM	**1.2D+1.0E+.5L$_r$**	**1.2D+1.6W+.5L$_r$**
R	24.6	15 k	90 k	48 k	55 k	88 k	2.255 in	2.963 in
15	46.8	42 k	172 k	139 k	156 k	205 k	2.383 in	2.703 in
14	68.5	69 k	253 k	225 k	251 k	306 k	2.193 in	2.457 in
13	90	95 k	332 k	310 k	335 k	393 k	1.998 in	2.212 in
12	111.1	121 k	411 k	393 k	408 k	467 k	1.799 in	1.967 in
11	131.6	146 k	488 k	476 k	472 k	528 k	1.596 in	1.726 in
10	151.9	170 k	564 k	556 k	526 k	579 k	1.392 in	1.488 in
9	171.7	193 k	639 k	634 k	572 k	619 k	1.190 in	1.259 in
8	190.9	216 k	712 k	709 k	610 k	679 k	0.992 in	1.038 in
7	209.4	237 k	783 k	782 k	639 k	672 k	0.801 in	0.829 in
6	227.3	256 k	852 k	852 k	662 k	686 k	0.620 in	0.634 in
5	244.4	274 k	919 k	918 k	680 k	695 k	0.446 in	0.458 in
4	260.7	289 k	984 k	980 k	692 k	698 k	0.301 in	0.304 in
3	275.9	301 k	1045 k	1036 k	699 k	695 k	0.178 in	0.177 in
2	281.6	311 k	1068 k	1110 k	702 k	695 k	0.082 in	0.081 in

Table 2-Comparison of hand calculations to computer analysis results and worst case story drifts.

NS-direction wind forces EW-direction wind forces Seismic Forces

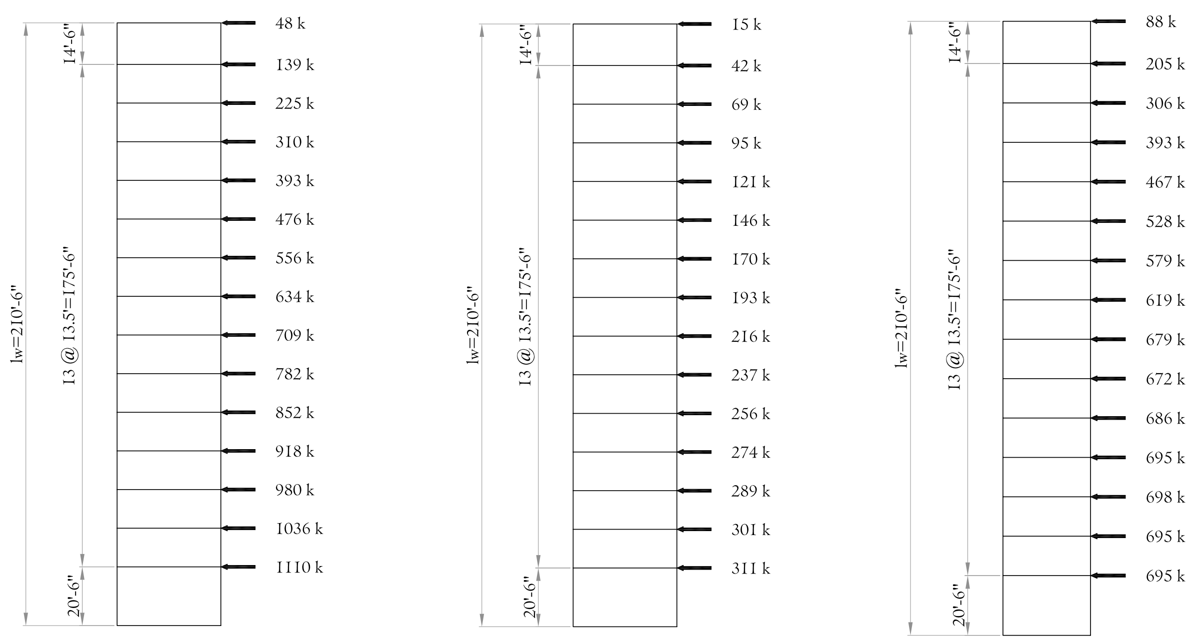


Figure 16-Story forces on the building due to lateral loads.

6.0 STRUCTURAL DEPTH

6.1 Post-tensioning

6.1.1 Slab

Spanning 52.5’ with regular reinforced concrete is very difficult, and nearly impossible to stay within reasonable floor depths. For this reason, it has been concluded that a one-way post-tensioned floor system is the best candidate for redesign. The 52.5’ span also requires the concrete to have a high compressive strength to withstand the large stress imposed by the prestressing tendons. For this reason, the monolithically cast slab and beam floor system will be designed using 6000 psi concrete. The slab spans the 30’-0” direction and is framed out by post-tensioned beams along column grid lines. Initial designs and hand calculations were performed following an example published by the Portland Cement Association. The example conforms to the concrete and steel stress limits provided by Chapter 18 in ACI 318-02 and is classified as Class U, uncracked. For detailed calculations, see *Appendix B.1: Post-tensioned Slab*.

An 8” thick slab was initially chosen according to an l/44 guideline set forth by the Post-tensioning Institute and accepted practice. The tendon profile was laid out in order to preserve the 2-hour fire rating of the existing system, requiring 1.75” of cover for prestressing tendons. This restricted the strands to a depth of 6.75” from the top of the slab at mid-span, 1.75” at the interior supports, and 4” at exterior supports (see **Figure 17**-Tendon profile in slab). With this profile, the effective prestressing force is found to be 1303 k. This translates into 49 tendons that need to be evenly distributed across the 52’-6” span. For constructability purposes, 8 ducts with 6 wires in each were distributed evenly across the bay. The ducts were routed around any slab openings to preserve the continuity of the prestressing force. In the angled northeast corner of the building where the slab area decreases, every other tendon was removed to prevent over-stressing the concrete (see **Figure 18**-Tendon layouts in slab).

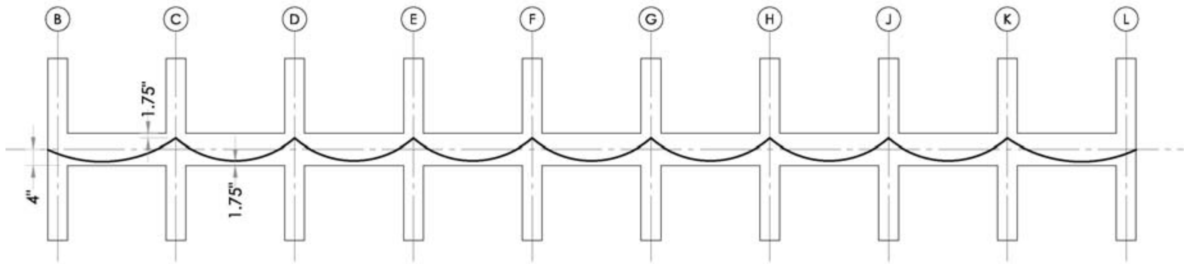


Figure 17-Tendon profile in slab.

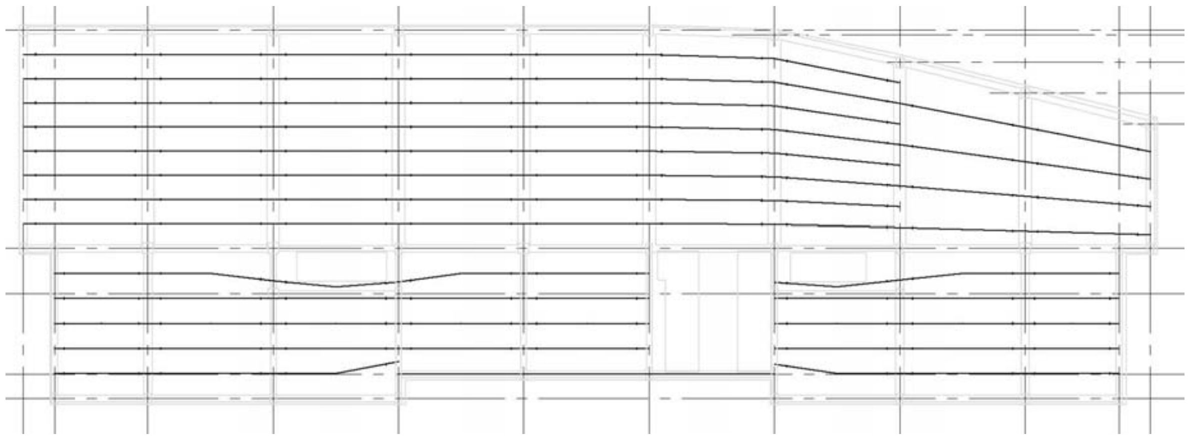


Figure 18-Tendon layouts in slab.

6.1.2 Post-tensioned Beams

The beams are included in the design because the aspect ratio of the bay is greater than the 2:1 ratio necessary for a two-way flat slab system. Therefore, post-tensioned beams will need to frame out the slab. They will need to be massive because of the large spans and heavy slab loads that they must support. To keep with the original goal of decreasing floor depth, the beams were kept to a maximum of 24" deep, including slab depth. Therefore, the beams are unconventionally wide and utilize a large amount of slab to aide in compression. The beams were initially designed according to Chapter 18 in ACI 318 and designed as Class T, the transition between uncracked and cracked. They are analyzed as T-beams to account for the additional compressive strength found in the slab. Detailed hand calculation that account for prestress losses can be found in *Appendix B.3: Beams*. These calculations consider the tendon profile to have a single drape, rather than a parabolic profile, which simplifies the calculation and provides a sufficient initial design. A feasible domain of acceptable initial forces and tendon eccentricities was constructed where an initial force, number of strands, and tendon profile were chosen.

The desired beam geometries were entered into ENERCALC to determine section properties including: area, moment of inertia, section modulus, and neutral axis. The geometry and loading for the 52.5' beam require 39 tendons with a profile of 14" at the ends and 5" at mid-span. The 36' beam requires 16 strands with the same profile. Because these calculations do not consider the beams to be continuous, they are just approximations and require closer evaluation. When the tendons from interior beams span shearwalls, they require a straight tendon profile.

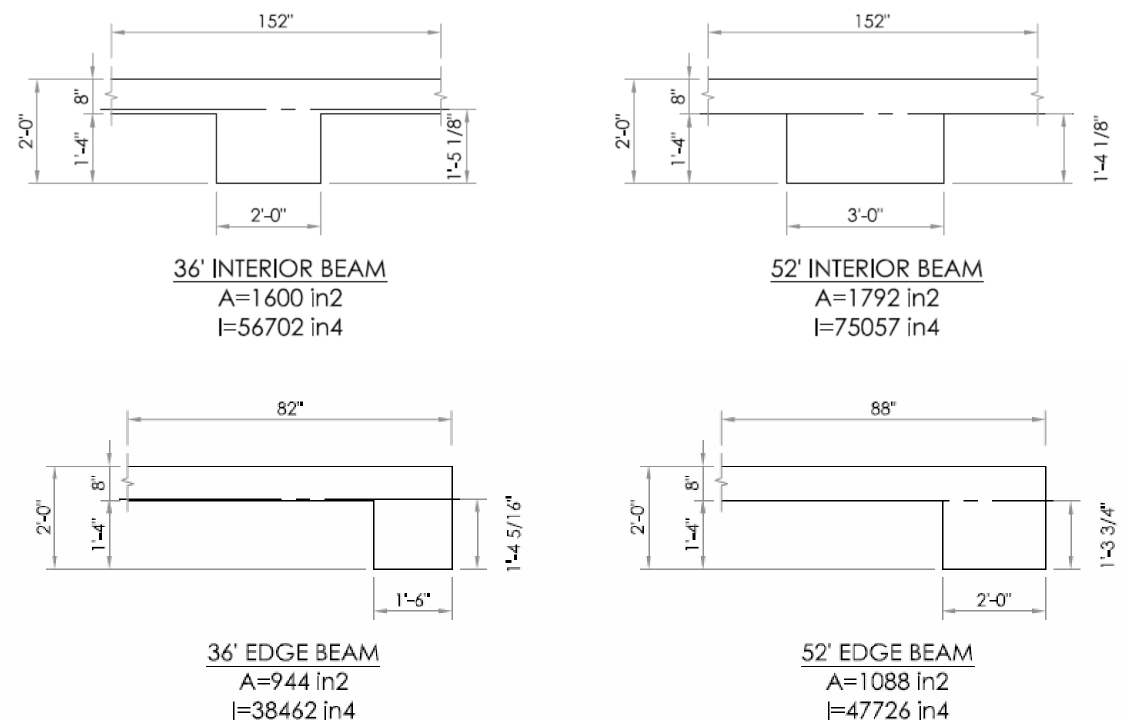


Figure 19-Cross sections of beams with section properties found in ENERCALC.

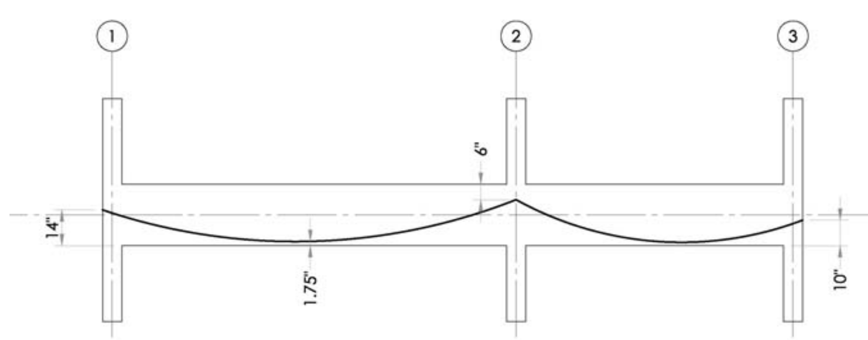


Figure 20-Tendon layout in continuous beams.

6.1.3 RAM Concept

The slab and beams, designed by hand calculations, were modeled in RAM Conce*pt* and are considered to be typical of all floors in the building. The program was used to determine concrete stresses and deflections for multiple load cases including: initial, sustained, and long-term service loading. Transverse shear reinforcement for beams and minimum required reinforcement for the slab were also determined with RAM Conce*pt*.

When modeled, column and middle design strips were generated according the ACI 318-02 for the maximum flange width of beams. Minimum reinforcement in the beams and slab was also indicated to be 0.0018 using #4 bars (see illustrations below for design strips).

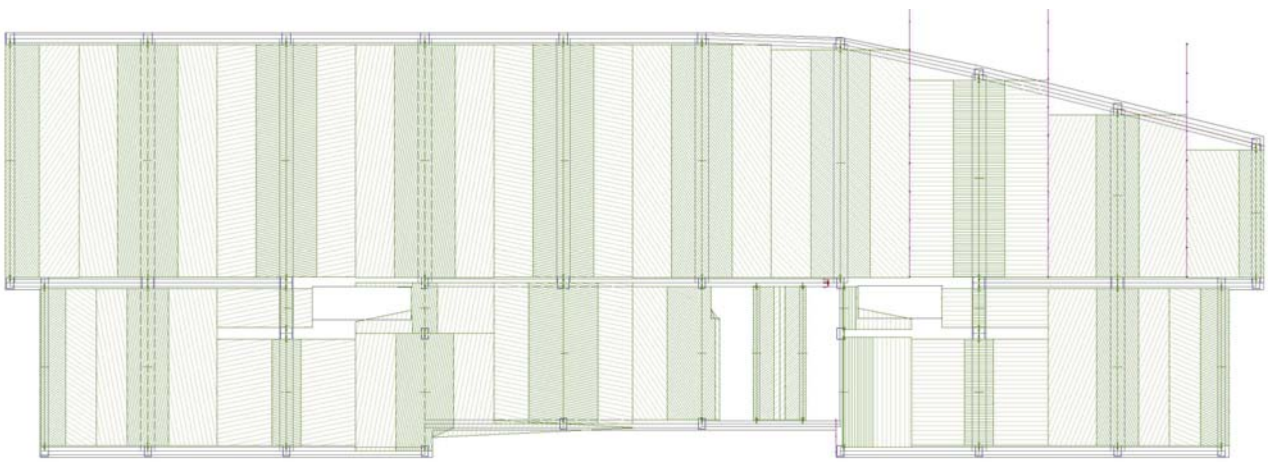


Figure 21-Longitude design strips generated by RAM Concept.

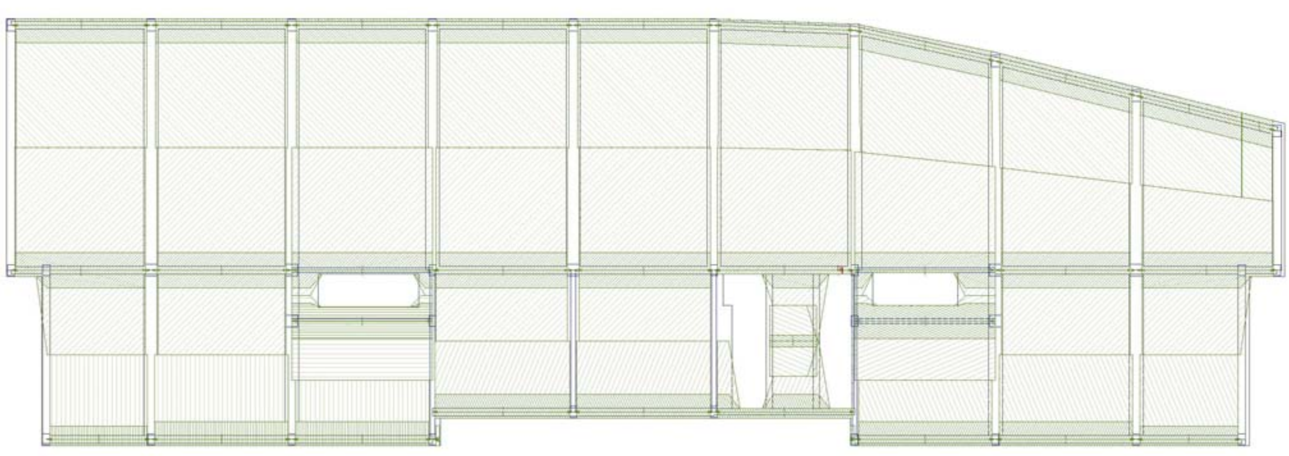


Figure 22-Latitude design strips generated by RAM Concept.

The preliminary designs worked well when modeled, and required only a few minor adjustments. The following beam schedule summarizes the location of beams, sizes, initial prestressing force, and number of strands. See the following two pages for transverse shear reinforcing details and deflection plans for various loading conditions. For minimum slab reinforcement plans, see *Appendix C: Plans*.

Properties	B-1	B-2	B-3	B-4	B-5	B-6
Dimensions	24x 24	36x 24	18x 24	24x 24	20x 24	24x 24
Shape	L	T	L	T	L	T
Fi	638 k	1064 k	266 k	1064 k	319 k	1064 k
# Strands	24	40	10	40	12	40
$d_{supports}$	17in 18in	14in 18in	18in 17in	18in 17in	17in 17in	18in 17in
d_{mid}	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in

Properties	B-7	B-8	B-9	B-10	B-11	B-12
Dimensions	18x 24	18x 24	20x 24	20x 24	20x 24	36x 24
Shape	L	L	T	T/L	L	T
Fi	319 k	319 k	319 k	319 k	319 k	1064 k
# Strands	12	12	12	12	12	40
$d_{supports}$	17in	14in 17in	17in 20in	14in 17in	14in 20in	14in 20in
d_{mid}	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in	1.75 in

Properties	B-13	B-14	B-15	B-16
Dimensions	36x 24	36x 24	24x 24	16x 12
Shape	T	T	L	L
Fi	718 k	718 k	559 k	0 k
# Strands	27	27	21	0
$d_{supports}$	14in 18in	14in 18in	17in 18in	-
d_{mid}	1.75 in	1.75 in	1.75 in	-

Table 3-Beam schedule including tendon profile, dimensions, and number of strands.

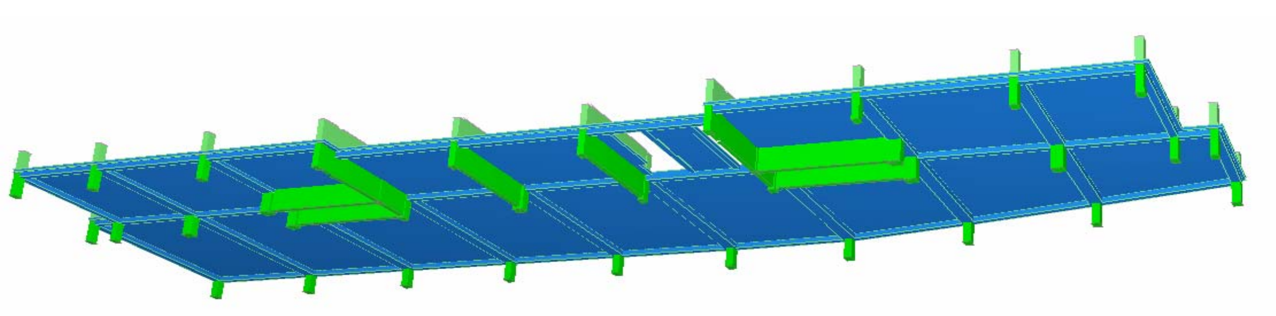


Figure 23- 3D view of underside of typical floor.

Transverse Reinforcing Details

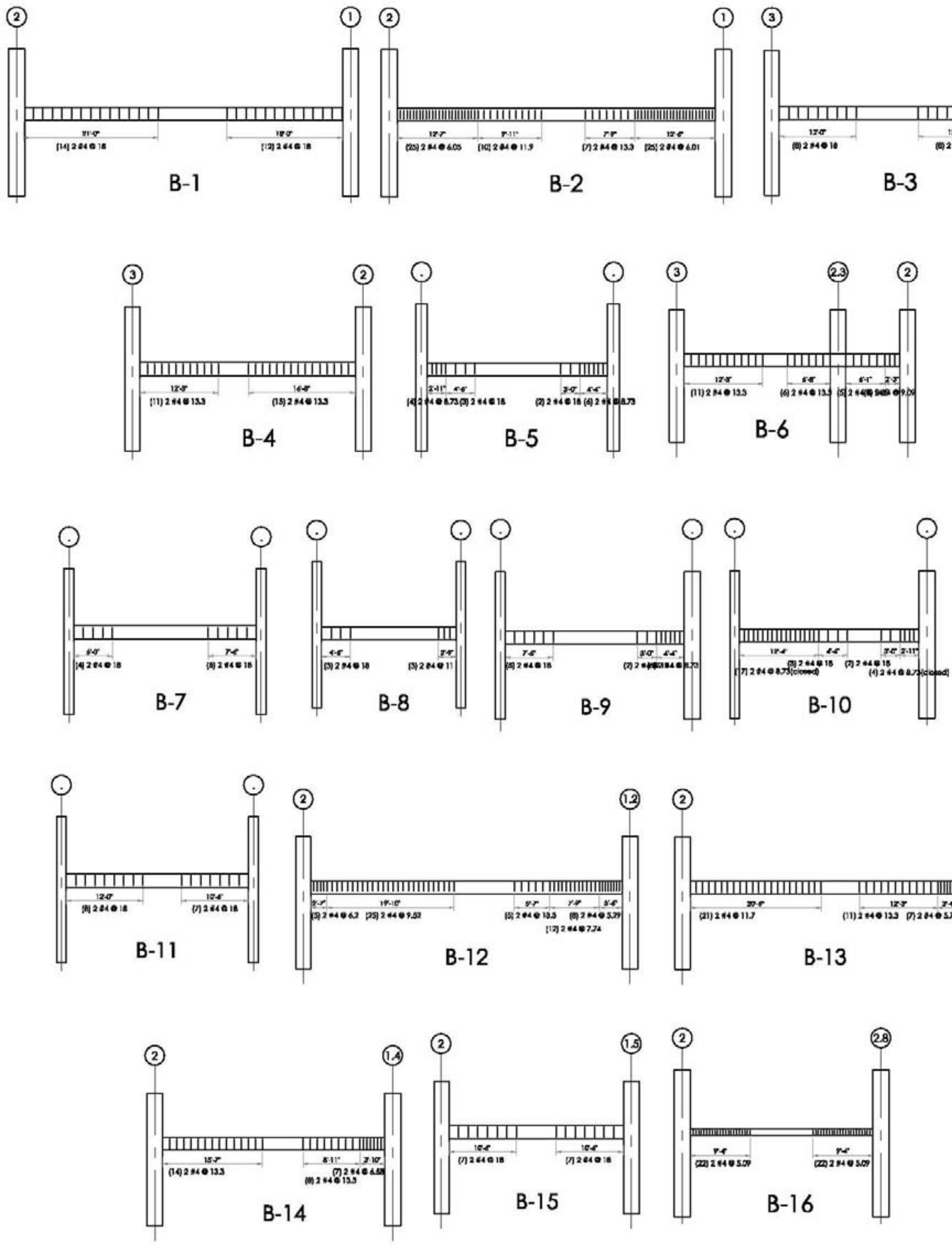


Figure 24-Transverse, shear reinforcement layouts for beams as designed by RAM Conce*pt*. See framing plan in *Appendix C.4: Post-tensioned concrete typical floor framing.*

Deflection Plans

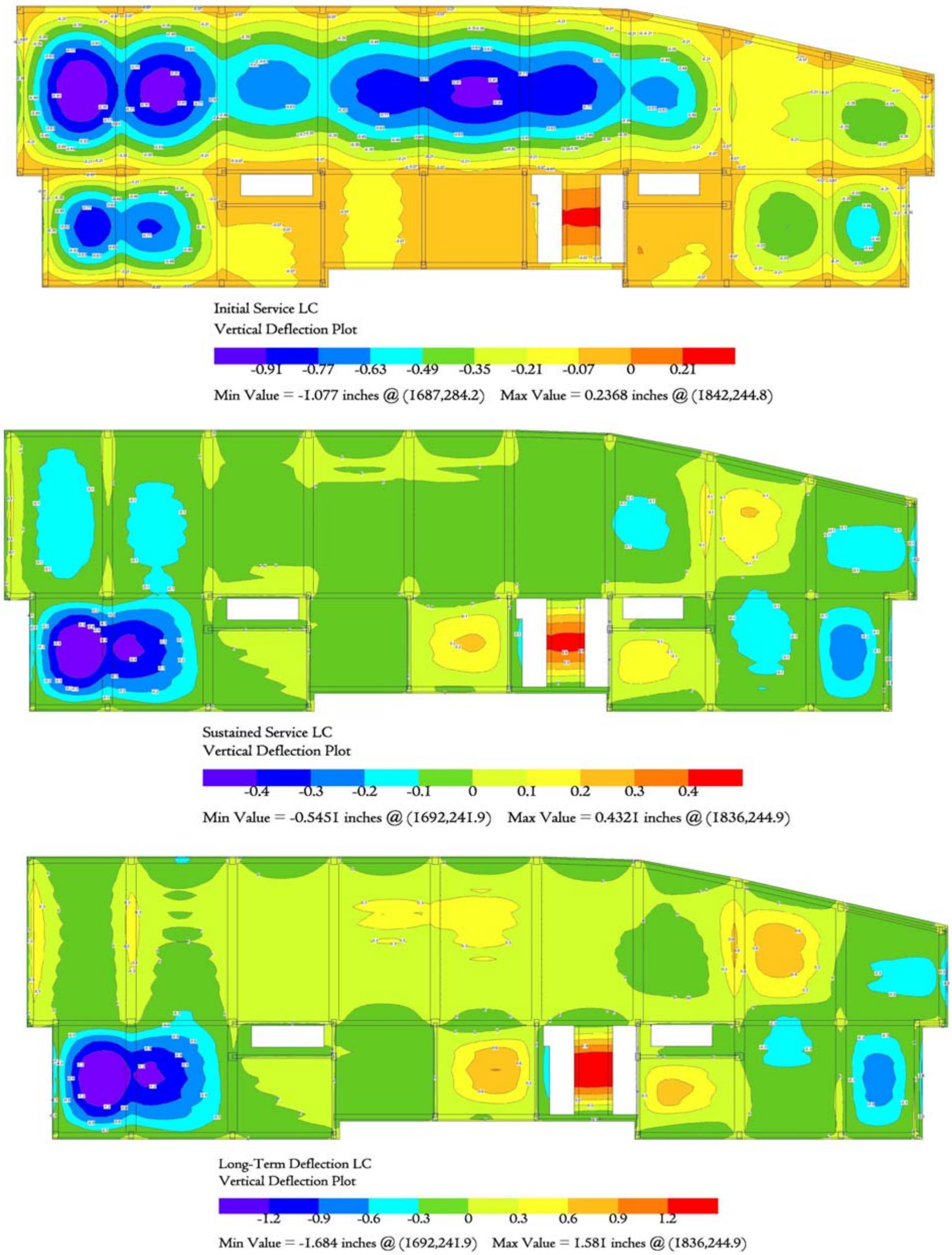


Figure 25-Deflection plans for three loading conditions from RAM Conce*pt*.

6.2 Regularly Reinforced

6.2.1 Columns

Columns were designed in a traditional manner by determining axial forces at each level and approximating moments applied to the top and bottom of the column from beams framing into it. The axial forces were found based on tributary area where live loads were reduced based on Chapter 4.8 in ASCE 7-02. Preliminary calculations can be found in *Appendix B.2: Columns*. With these axial loads and approximate moments, interaction diagrams were used to determine initial reinforcing details.

These initial column sizes were modeled in RAM Concrete and their reinforcing was analyzed more closely. The reinforcing is spliced at every other level, and patterns and bar sizes have been narrowed down for constructability purposes. All columns have transverse shear reinforcing of #3 closed bars at 9" o.c. The following page contains the final column schedule, and the foundation plan can be found in *Appendix C.3: Post-tensioned concrete foundation*.

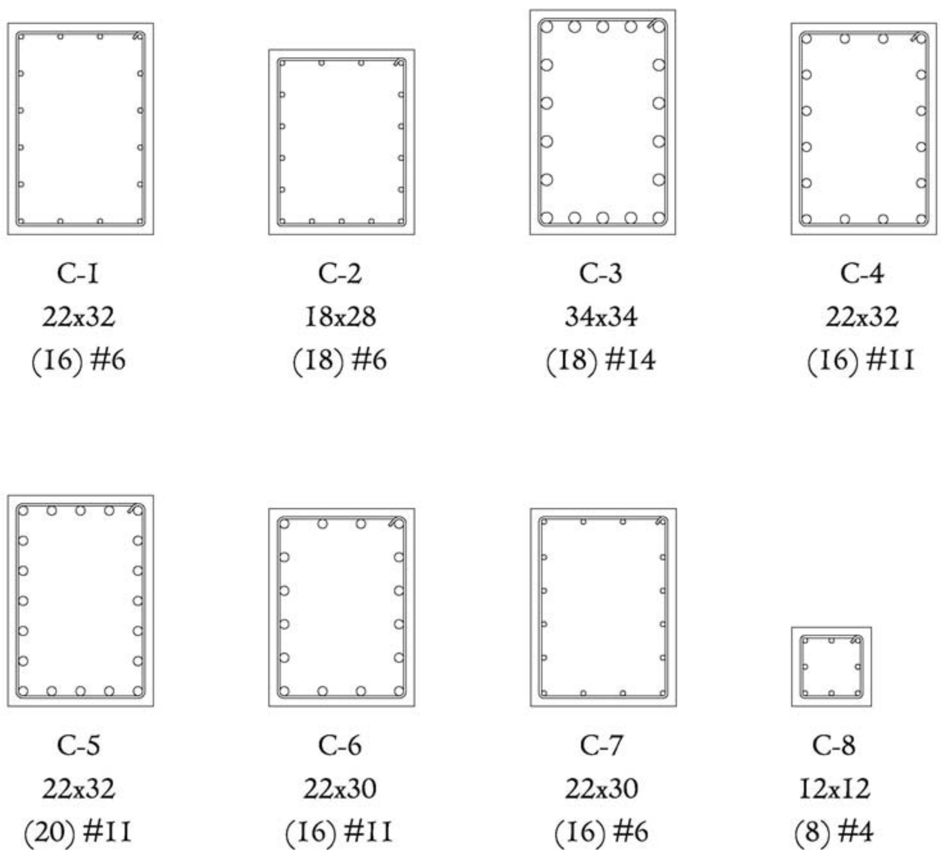


Figure 26-Reinforcing for columns (see column schedule on next page).

Floor	C-1	C-2	C-3	C-4	C-5	C-6	C-7	C-8
R	22x 22 (16)- #9 2228k	18x 18 (18)- #4 1212k	24x 24 (18)- #6 2218k	24x 24 (16)- #9 2533k	22x 22 (20)- #8 2221k	22x 26 (16)- #8 2389k	22x 26 (16)- #6 2171k	
15	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #9 2520k	22x 26 (20)- #7 2364k	22x 26 (16)- #9 2520k	22x 26 (16)- #6 2171k	
14	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
13	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
12	22x 26 (16)- #6 2170.740k	18x 24 (18)- #5 1647.594k	32x 32 (18)- #7 3815.760k	22x 26 (16)- #6 2170.740k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2170.740k	22x 26 (16)- #6 2170.740k	
11	22x 26 (16)- #6 2171k	18x 24 (18)- #5 1648k	32x 32 (18)- #7 3816k	22x 26 (16)- #6 2171k	22x 26 (20)- #5 #N/A	22x 26 (16)- #6 2171k	22x 26 (16)- #6 2171k	
10	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
9	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
8	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
7	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
6	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	32x 32 (18)- #7 3816k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	
5	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #8 4387k	22x 32 (16)- #6 2608k	22x 32 (20)- #6 2677k	22x 30 (16)- #6 2462k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
4	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #9 4534k	22x 32 (16)- #8 2827k	22x 32 (20)- #8 2950k	22x 30 (16)- #8 2681k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
3	22x 32 (16)- #6 2608k	18x 28 (18)- #5 1886k	34x 34 (18)- #11 4927k	22x 32 (16)- #10 3126k	22x 32 (20)- #10 3324k	22x 30 (16)- #10 2980k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k
2	22x 32 (16)- #6 2608k	18x 28 (18)- #6 1980k	34x 34 (18)- #14 5412k	22x 32 (16)- #11 3307k	22x 32 (20)- #11 3551k	22x 30 (16)- #11 3161k	22x 30 (16)- #6 2462k	12x 12 (8)- #4 539k

Table 4-Column schedule. See foundation plan in *Appendix C.3: Post-tensioned concrete-typical floor framing.*

6.2.2 Lateral System Design

Initially, the lateral system was planned to be cast-in-place shearwalls in similar locations to the braced frames in the composite steel structure. After running lateral load analyses in RAM considering all of the load combinations discussed in the section *5.3.2 Lateral Loads*, story drifts were too large in certain load combinations. Without the freedom to add more shearwalls, the concrete frames needed to be included. Because this building is designed as a cast-in-place concrete structure with connections similar to moment connections, every frame that has been designed for gravity loading can be considered in resisting lateral load. However, these frames are not enough to resist all of the lateral loads alone and need to be incorporated with the shearwalls. Concrete shearwalls are used as the main structural elements that resist lateral forces and concrete frames supplement in resisting lateral loads.

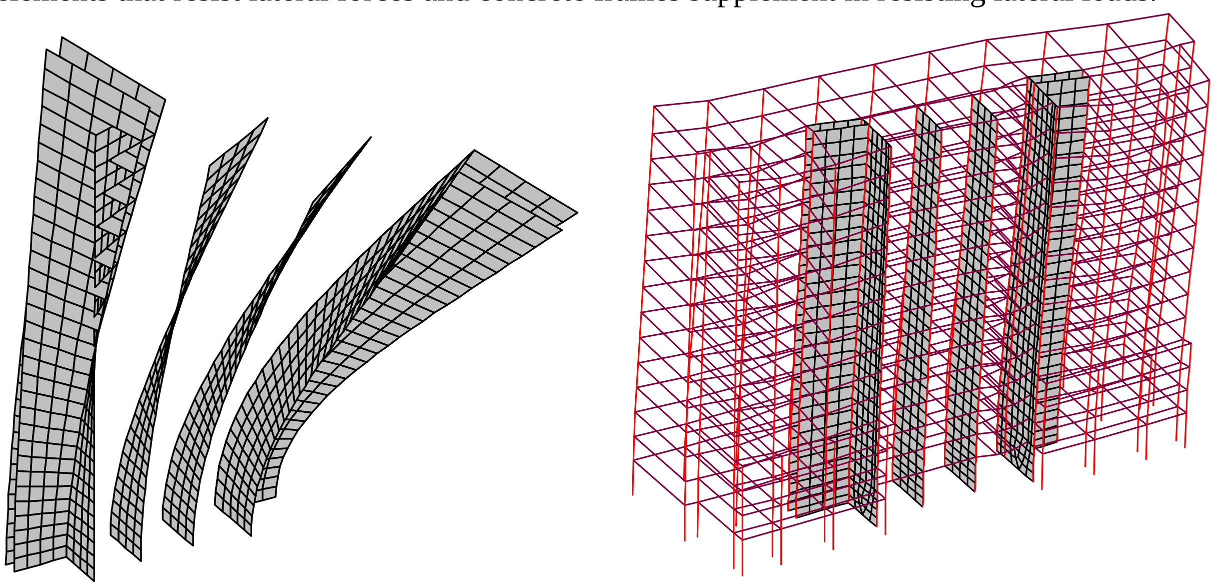


Figure 27-Lateral systems under 1.2D+0.5L+1.4E when seismic loading is in the east-west direction and considers 5% accidental eccentricity.

The shearwalls are sufficient in resisting load combinations considering dead, live, and most cases of wind loading. However, the load combinations considering seismic loading introduce large deflections, around 12” when load is applied in the east-west direction, and torsional problems, around 1.15°. Since this drift was not drastically decreased by increasing wall thickness or material strength, the introduction of concrete moment frames to help in resisting loads is necessary.

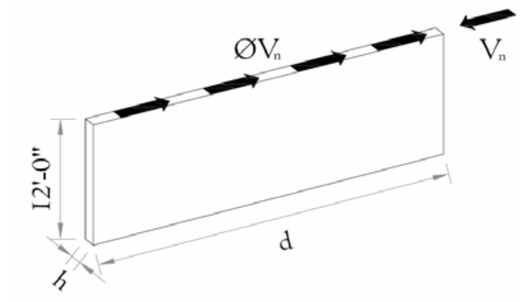


Figure 28-Shear wall dimension and loading diagram.

Shearwall Design: The reinforcement was designed to resist only shear forces, and not checked in bending for overturning. Initial designs for shearwalls were performed using the controlling load cases in each direction that were found by hand calculations. Wind loads were found to control in the north-south direction while seismic loads were found to control in the east-west direction. Shearwalls were designed using basic strength principles of $V_u \leq \phi V_n$ and $V_n = V_c + V_s$ and modeled as a cantilevered beam with a series wall shears acting as point loads.

The walls were designed for the most heavily loaded level in shear and bending. The greatest loading, shear of 308 k and moment of 270 ft-k, exists in walls 1 and 4 (see diagram below) during wind loading in the north-south direction. A trial wall size of 12” thick with a compressive strength of 4000 psi were used for both directions and found to work. The steel used is #6 @ 14” in both horizontal and vertical directions.

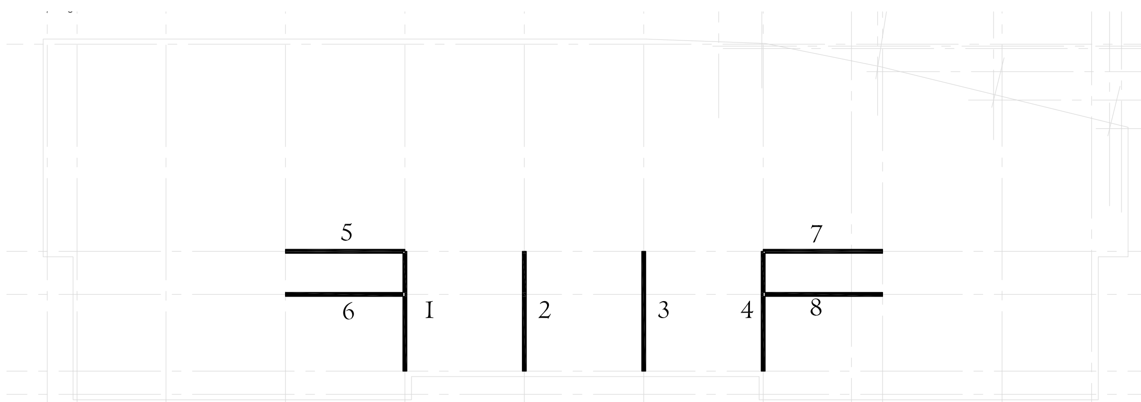


Figure 29-Location of shearwalls in plan.

To determine the distribution of lateral forces to the shearwalls, their stiffnesses were calculated by the following equation: $k = \dfrac{Et}{4(h/l)^3 + 3(h/l)}$ and the story shears were distributed accordingly. From these forces, the overturning moments and uplift at the base of each wall were found. All of these values are tabulated in the table on the next page and shown acting on the shearwalls. Refer to *Appendix B.4: Shearwalls* for further calculations on shearwalls.

Direction Shear on Shear Walls Due to Controlling Lateral Forces								
Floor	N-S				E-W			
	1	2	3	4	5	6	7	8
R	29 k	17 k	17 k	29 k	14 k	14 k	14 k	14 k
15	54 k	32 k	32 k	54 k	39 k	39 k	39 k	39 k
14	80 k	47 k	47 k	80 k	63 k	63 k	63 k	63 k
13	105 k	61 k	61 k	105 k	84 k	84 k	84 k	84 k
12	130 k	76 k	76 k	130 k	102 k	102 k	102 k	102 k
11	154 k	90 k	90 k	154 k	118 k	118 k	118 k	118 k
10	178 k	105 k	105 k	178 k	132 k	132 k	132 k	132 k
9	201 k	119 k	119 k	201 k	143 k	143 k	143 k	143 k
8	223 k	133 k	133 k	223 k	152 k	152 k	152 k	152 k
7	245 k	147 k	147 k	245 k	160 k	160 k	160 k	160 k
6	265 k	161 k	161 k	265 k	166 k	166 k	166 k	166 k
5	284 k	175 k	175 k	284 k	170 k	170 k	170 k	170 k
4	300 k	191 k	191 k	300 k	173 k	173 k	173 k	173 k
3	312 k	210 k	210 k	312 k	175 k	175 k	175 k	175 k
2	308 k	226 k	226 k	308 k	176 k	176 k	176 k	176 k
Overturning	**249,327**	**150,208**	**150,208**	**249,327**	**172,127**	**172,127**	**172,127**	**172,127**
Uplift	**6,926 k**	**5,007 k**	**5,007 k**	**6,926 k**	**5,738 k**	**5,738 k**	**5,738 k**	**5,738 k**

Table 5-Shear on walls due to direct shear.

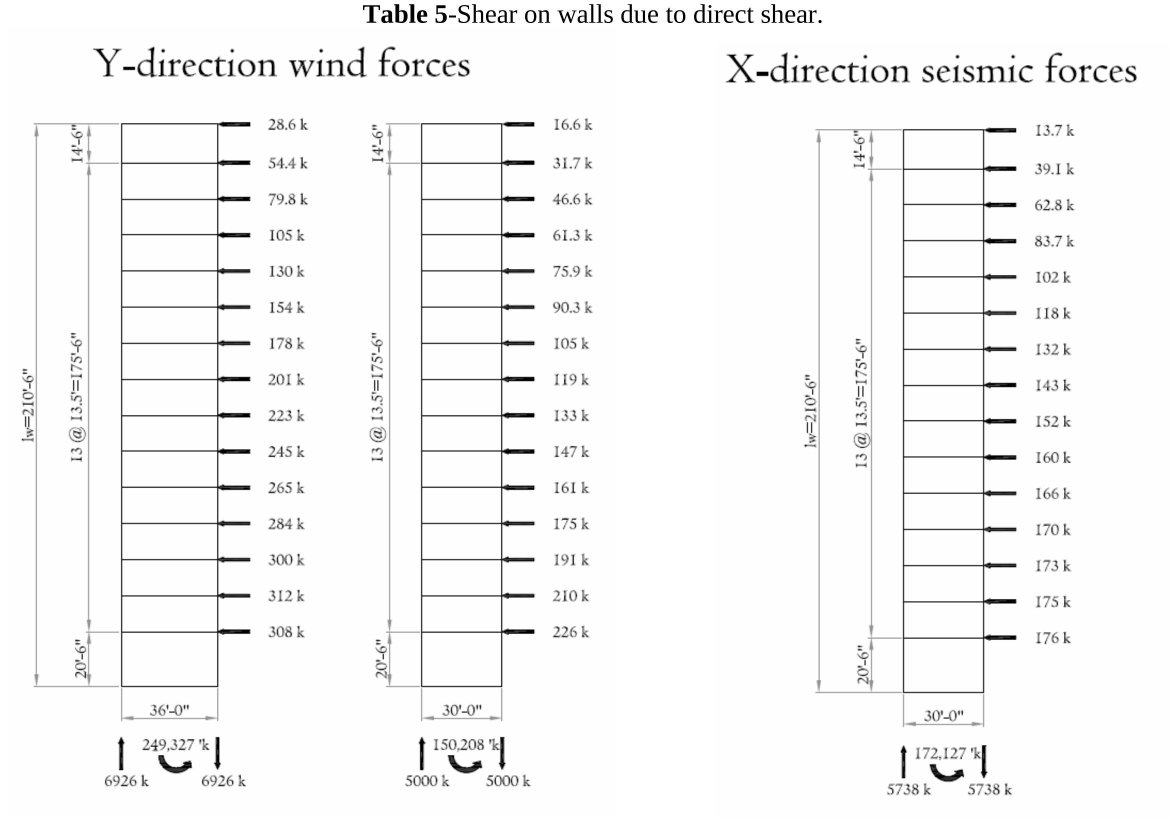


Figure 30-Direct shear acting on walls with resulting uplift and overturning moment.

6.3 Foundations

The concrete structure adds 30% more weight to the foundation when compared to the composite steel structure. The undesirable soil conditions in the city of Wilmington warrant deep foundations to support the gravity loads and overturning moment imposed on them. Since the clusters of auger-cast piles are already 10-18, increasing the size of clusters will begin to compromise the already poor soil. The objective of changing the foundations to caissons is to preserve the integrity of the soil, to limit settlement, and eliminate the need for a 60” pile cap. Their capacity was found based on the end soil bearing pressure of the caisson. For those underneath shearwalls, they will be reinforced to take tension due to overturning.

Caissons						
		Amount	**Material**	**Labor**	**Equip-ment**	**Cost**
A1020-310	4'-0" dia. x 100'	20 Ea	4358	70459		$1,496,329
	5'-0" dia. x 100'	6 Ea	8064	144990		$918,324
	6'-0" dia. x 100'	10 Ea	11730	172277		$1,840,069
					TOTAL	**$4,254,722**
Concrete Filled, Drilled Piers						
A1020-130	End Bearing Steel Piles					**Cost**
2380	4 pile cluster	5	5625	3325		$44,750
2460	6 pile cluster	8	8425	5025		$107,600
2480	7 pile cluster	7	9825	5850		$109,725
2500	8 pile cluster	5	12600	7525		$100,625
2560	12 pile cluster	9	15400	9200		$221,400
03310-240	Pile caps, incl. forms and reinf.	612	108	49	0.31	$96,309
					TOTAL	**$680,409**

Table 6-Cost estimate and comparison between caissons and concrete filled, steel piles.

In addition to the caissons, grade beams will be provided to engage all of the deep foundations when the shearwalls and frames are forced into action. Although the caissons and the grade beams have not been explicitly designed for, the impact that the structure has on them has been estimated. The foundation plan can be found in **Figure 29**-Foundation Plan.

7.0 STRUCTURAL SUMMARY

The concrete in the super-structure including: columns, girders, and slabs, will have a compressive strength of 6000 psi, but the shearwalls will be 4000 psi. The need for such high strength concrete in the gravity framing comes from the long-span feature of the office floors. In order to preserve the 52'-6" spans, the concrete needed to have enough strength to withstand the amount of stress caused by the post-tensioning force.

Post-tensioned Slab: The post-tensioned slab will be 8" thick and contribute 100 psf of dead load to the structure. The ungrouted tendons in the slab will be spaced about 6' o.c. and span the 30' direction of each bay. The tendons will be banded in groups of (6) ½" diameter strands and have yield strength of 270 ksi. They will have a parabolic profile of 6.75" above the bottom of the slab at supports and 1.75" from the bottom of the slab at mid-span.

Columns: The columns in the building range in size from 18"x28" to 30"x30". See page 28 for a full column schedule, and refer to the foundation plan in *Appendix C.3: Post-tensioned concrete-foundation* for column locations.

Beams: The beams will use post-tensioning steel with yield strength of 270 ksi. The steel strands will be grouted solid. Interior beams spanning 52'-6" will be 16"x36" and have approximately 35 strands in them, and interior beams spanning 36'-0" will be 16"x24" and have approximately 20 strands in them. In order to develop the full compressive capacity in the slab, the beams will be analyzed as T-beams. Beam designs have achieved the objective of decreasing floor depth by 7.25" over the composite steel system, when fire-proofing is considered.

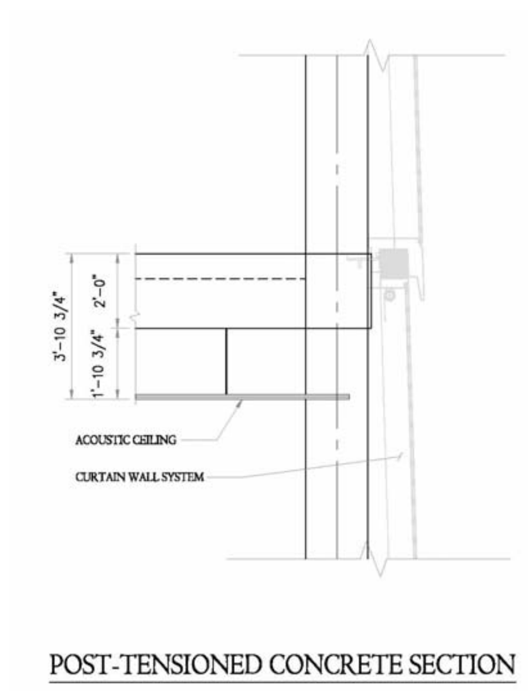


Figure 31-Section of concrete floor system.

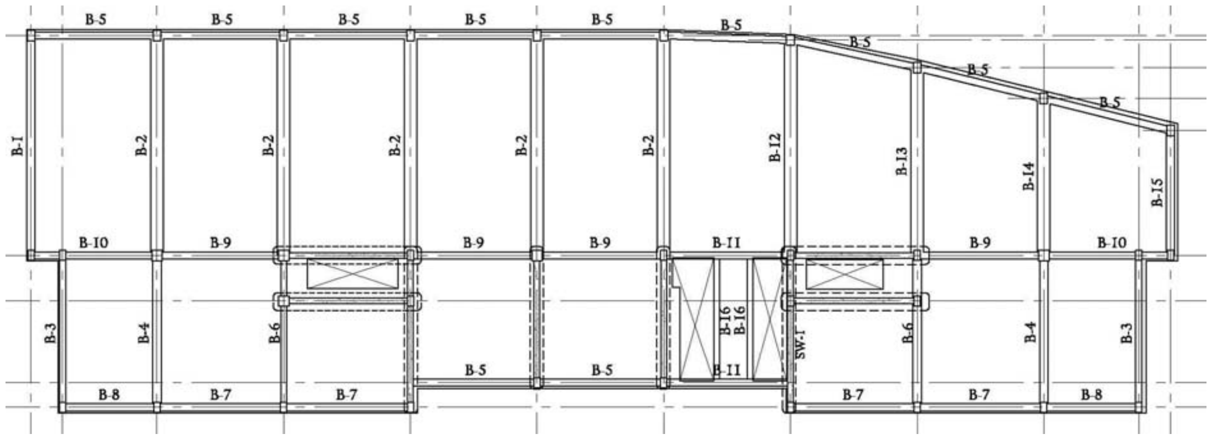


Figure 32-Post-tensioned concrete framing plan for typical floor.

Shearwalls: Shearwalls will be identical in each direction: 12” thick, 4000 psi, and #6 @ 14” both vertically and horizontally. Though shearwalls will resist a majority of the lateral loads, they are not the only lateral resisting elements.

Foundations: The foundations will be caissons to support the additional weight of the concrete frame. Their sizes range from 4’-0” to 6’-6” in diameter. The caissons under shearwalls will be reinforced at the top to prevent overturning. They will be connected by a network of grade beams to more evenly distribute load and prevent differential settlement. The foundation plan is pictured below.

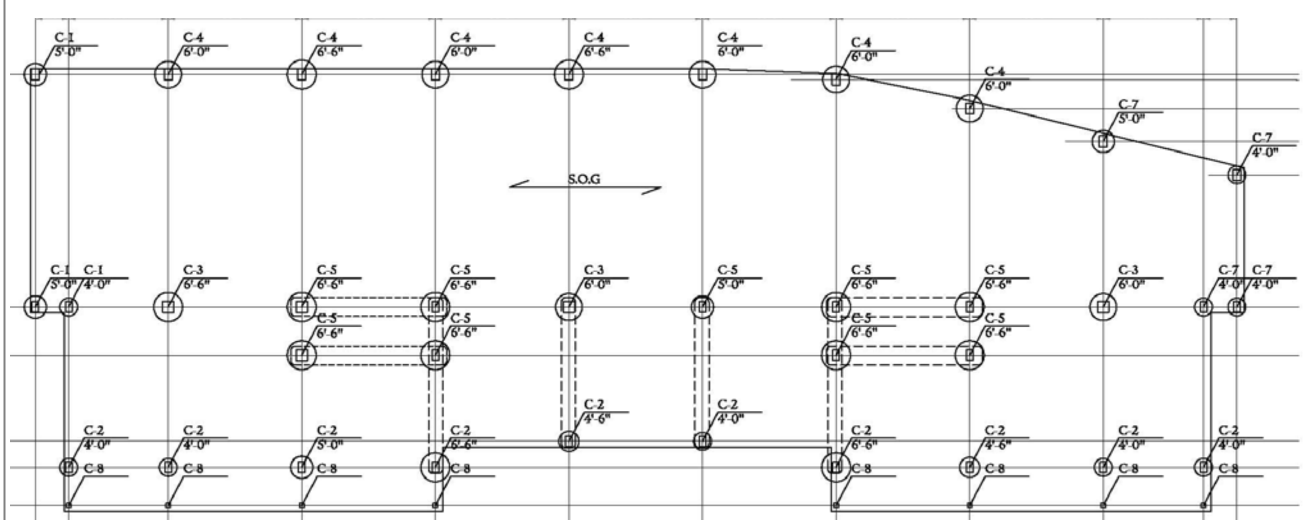


Figure 33-Foundation plan.

8.0 BREADTH STUDIES

The post-tension concrete design of Gateway Plaza will have an impact on all of the systems in the building, including the construction of the project. In particular, this report will focus on how supply air ducts, part of the mechanical system, can be designed according to ASHRAE Standard 62.1 and how the project can be scheduled efficiently.

8.1 Mechanical Study

The main goal of the mechanical study is to design supply and return air ducts in a typical floor in Gateway Plaza. To keep with the objective of the structural design, minimizing floor depth, these ducts will be designed to minimize their depth. Since the building is for tenant fit-out, there is no existing duct plan to compare. However, the riser on each floor has a main supply duct with a depth of 26". To handle air return, the ceiling will be used as a plenum for collecting return air, a system that works well when drop ceilings are used. The mechanical room is equipped with two 90"x32" grilles to pull in air from the plenum, and the room has a louver to exhaust air outside.

8.1.1 Indoor Air Quality

Before ducts could be laid out, it is necessary to check that the building is receiving the required amount of outdoor air for proper ventilation. The layout for the new structural system was designed according to *ASHRAE Standard 62.1: Ventilation for Acceptable Indoor Air Quality* using the Ventilation Rate Procedure laid out in section 6.2. This procedure "determines outdoor air intake rates based on space type, occupancy level, and floor area." The actual amount of outdoor air supplied by the mechanical equipment has been compared to the minimum amount of outdoor air intake typical of the office floor (shown on the next page) as determined by the above procedure.

Since the building is for tenant fit-out, a basic design was established where the typical office floor was subdivided into five separate offices, seen in the diagram below. To estimate the occupancy of each office, it was assumed that each person occupied a 10'x10' cubicle, or 100 ft^2, including hallway circulation space.

Office	A_z (ft²)
1	2911
2	3738
3	3513
4	2137
5	2600
6	2592

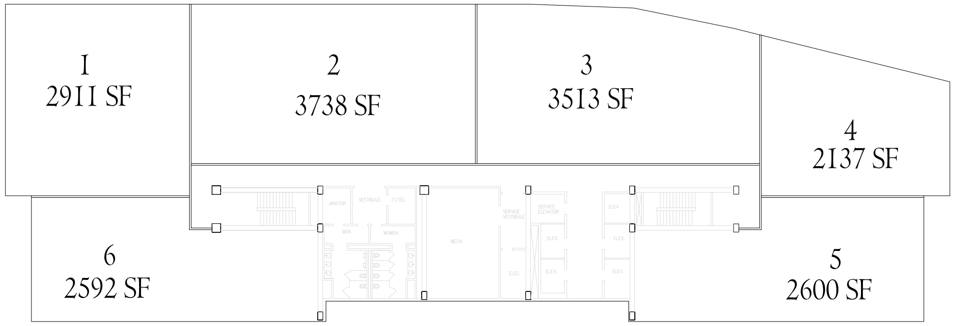


Figure 34-Typical layout of offices, with square footage, for tenant fit-out.

The ventilation rate procedure has been outlined below for the office type 1.

Step 1: Breathing Zone Outdoor Airflow

$$V_{bz} = R_p P_z + R_a A_z \qquad \text{(Eq. 6-1)}$$

- $R_p = 5cfm$ (Table 6-1)
- $R_a = 0.06cfm/sf$

$$V_{bz} = 5cfm(30\,people) + 0.06cfm/sf\,(2911sf) = 325cfm$$

Step 2: Zone Outdoor Airflow

$$V_{oz} = \frac{V_{bz}}{E_z} \qquad \text{(Eq. 6-2)}$$

- $E_z = 1.0$ (From Table 6-2 for ceiling supply of cool air.)

$$V_{oz} = \frac{325cfm}{1.0} = 325cfm$$

Step 3: Multiple-Zone Recirculation System

- Primary Outdoor Air Fraction

$$Z_p = \frac{V_{oz}}{V_{pz}} \qquad \text{(Eq. 6-5)}$$

$$Z_p = \frac{325cfm}{\frac{2911sf}{17{,}491sf}(22{,}725sf)} = .0859cfm$$ where V_{pz} is taken to be the ratio of the area of the office being supplied to the total area of all of the offices multiplied by the total area of the typical floor.

- Uncorrected Outdoor Air Intake

$$V_{ou} = D\Sigma_{allzones} R_p P_z + D\Sigma_{allzones} R_a A_z \qquad \text{(Eq. 6-6)}$$

- The occupancy diversity is taken to be 1.0, which is conservative, because each office is being designed as having the same occupancy requirements.

$V_{ou} = 325cfm$

- Outdoor Air Intake

$$V_{ot} = \frac{V_{ou}}{E_v}$$

- Ventilation Efficiency: $E_v = 1.0$ because $Z_p < 0.15$.

$V_{ot} = 325cfm$

Step 4: Outdoor Air Comparison

After all of the spaces have been calculated, the total amount of outdoor air that must be supplied to office space #1 is found to be 1942 cfm (see *Appendix D.1 Mechanical* for calculation of all spaces). This value is greatly less than the 4545 cfm that is supplied to the area by the existing equipment. Therefore, the typical office floor is capable of handling the ventilation requirements. This oversize is to be expected for a tenant fit-out space where space requirements are unknown.

8.1.2 Diffuser Layout

Since the requirements for each office space have been found according to ASHRAE Standard 62.1, the ducts can be laid out to achieve the necessary supply loads. Assuming that each supply diffuser will have a throw range of a 6'-16' radius, the diffusers can be laid out for each office space, taking care to cover the entire area. The diffusers specified by the architect are 24"x24" so that they will fit into the 24"x48" acoustic ceiling grid. See the diagram below for the preliminary diffuser lay-out and the throw area.

Return grills have been laid out in such a manner to create a natural circulation of air. The returns, about three supplies to each return, have been positioned between rows of supply diffusers. While the air will be supplied to the offices through forced air, ceiling plenum return will bring air back to the mechanical room.

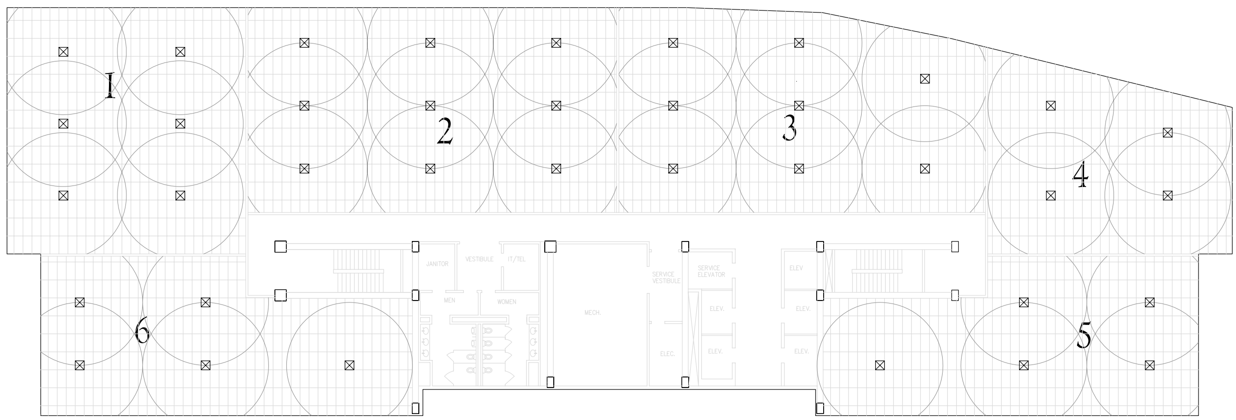


Figure 35-Diffuser layout and ranges of throw.

8.1.3 Duct Layout

The diffusers were connected using the shortest run of ducts from the riser originating in the mechanical room in each floor. The only obstacle that needed to be avoided was the shearwalls flanking the mechanical room and stairwells. Therefore, the runs to the ducts in office spaces 5 and 6 were unusually long. The ducts originated in the mechanical room at a size of 96x26 and eventually branched off to a size of 10x10 at their smallest. To size the ducts, the Duct Designer (duct-o-lator) supplied from the Loren Cook Company was used. A friction loss of 0.08” of water per 100’ of duct was assumed.

As was previously mentioned, air will be returned to the mechanical room through the ceiling plenum. To better direct the air in such a large ceiling plenum, duct stubs will be connected to the return grilles to direct the air to the mechanical room.

The plan below shows the layout of ductwork, both supply and return, to each office.

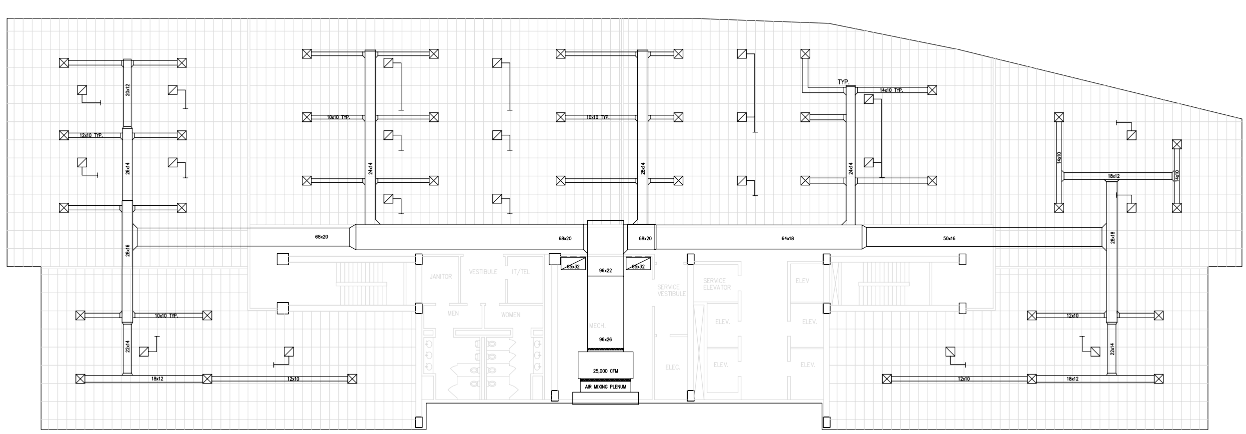


Figure 36-Duct layout.

8.2 Construction Study

8.2.1 Scheduling Impact

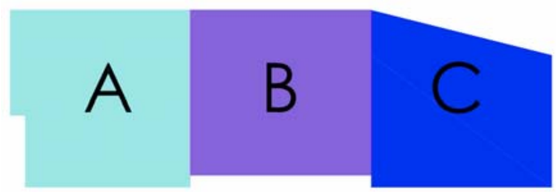


Figure 37-Lifts in the construction sequence.

There is a significant difference in tasks and sequence of tasks that take place when building a concrete structure and building a composite steel structure. There are more trades on site with concrete construction due to the necessary formwork, reinforcement, post-tensioning, and placing of concrete. This leads to the necessity of coordinating these trades to minimize down-time and preventing trades from interfering with one another.

Since there is virtually no lead time for concrete, compared to steel, a post-tensioned concrete floor can be produced from start to finish in around 5 days, so the structure can be erected quickly.

Due to the large size of the elevated slabs, approximately 24,000 ft^2 and 640 cubic yards of concrete per floor, and the limited capacity of concrete trucks, about 10 cubic yards per truck, tasks will be completed in the 3 sections. The diagram below depicts the three sections the building has been divided into: column lines B-E, E-H, and H-L. The areas of these sections are more manageable for crews and for delivery coordination: A=7650ft^2, B=7400 ft^2, C=7000 ft^2.

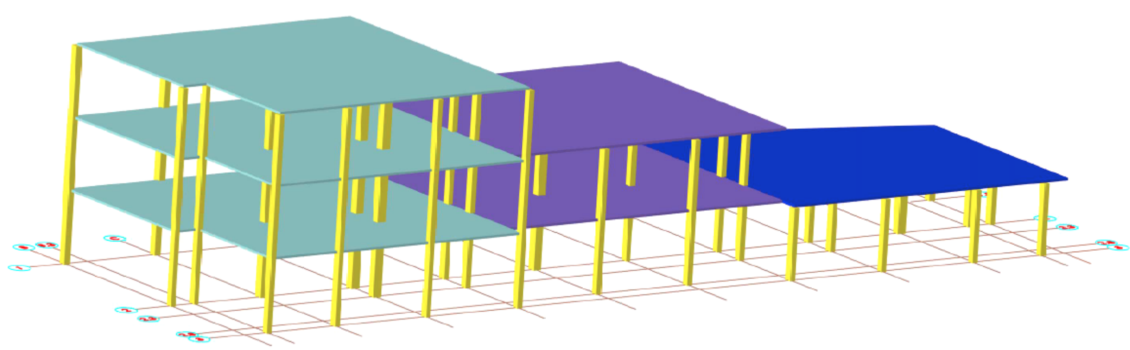


Figure 38-Schematic of how construction will take place with lift sequencing.

The following events and their durations, on a floor by floor and section by section basis, were considered when scheduling this project:

- Columns (F/R/P): 2 days
  - Forming

    - Tying Reinforcement
    - Pouring and curing
- Construct/Erect shoring and formwork: 3 days
- Tie reinforcement for slab and beams: 1 day
- Rough-in for MEP: 1 day
- Concrete (for each section):
    - Pour and finish: 1 day
    - Cure: 2 days
- Strip formwork and install reshoring: 2 days
- Jack the post-tensioning tendons: 1 day

The previous tasks were scheduled using Primavera project management software and compared to the Primavera schedule obtained from the Gilbane Construction Manager for the composite steel structure. For the full schedule, refer to *AppendixD.2: Construction*. The composite steel construction lasted 140 days while the post-tensioned concrete construction lasted 156 days. Though these construction times are comparable, it does not include the lead time needed to obtain and fabricate steel.

8.2.2 Cost Analysis

Structural Framing Costs: The following cost comparisons take into consideration only the structural systems including: slabs, beams, columns, and lateral elements, and do not include foundation costs. Estimates for both systems were compiled using material takeoffs for a typical floor and finding unit prices for each material in RS Means 2005. Although, this is not the year when the project was bid, the estimates are both done using RS Means 2005, so they are in direct comparison. See *Appendix D.2: Construction*.

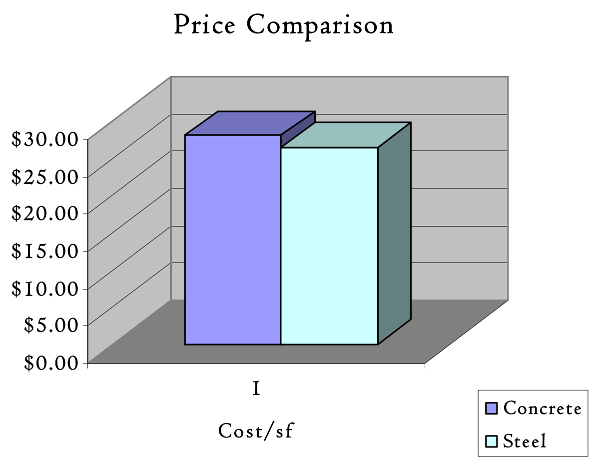
Cost Comparison		
	Cost/Floor	cost/sf
Concrete	$ 654,702	$ 27.98
Steel	$ 616,892	$ 26.36

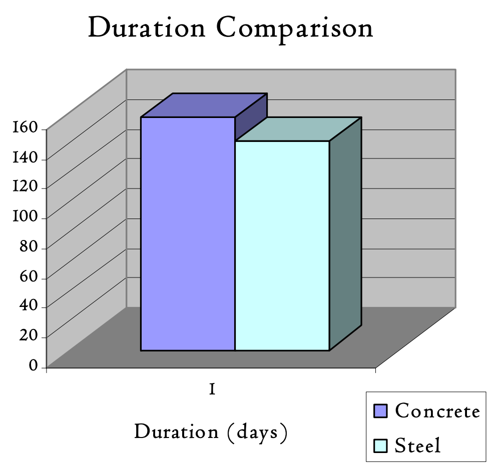
Table 7-Cost comparison between post-tensioned concrete structure and composite steel structure.

As evidence by the cost comparison for the two systems, the post-tensioned concrete system is more expensive by $1.62/ft^2$ and approximately $626,940 for the whole project. This is a 7% increase over the existing composite steel structure.

Foundations Costs: Since the foundations were changed as part of the post-tensioned concrete design, the change needs to be accounted for when considering project feasibility. Again, material takeoffs, available in *Appendix D.2: Construction*, were compiled and then assembly costs were taken from RS Means 2005. Caissons proved to be much more expensive than the existing concrete filled steel piles. The price for caissons is over $4 million where the price for the piles was only $700,000.

The charts below accurately illustrate differences in price, duration, and foundation costs between the composite steel and post-tensioned concrete systems.





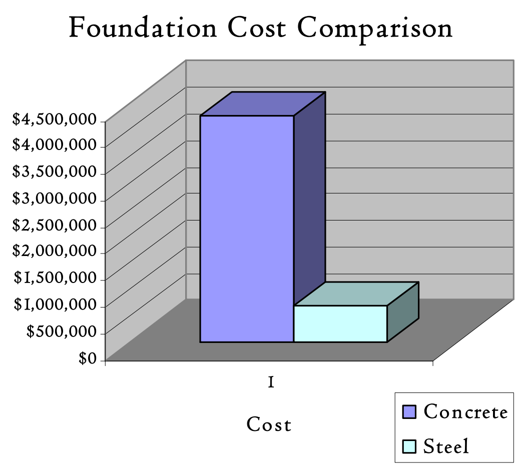


Figure 40-Comparison charts.

9.0 CONCLUSIONS

This report contains a thorough investigation of the existing composite steel structure and an analysis and design of the new post-tensioned concrete structural framing system. After this investigation, the systems were compared based on cost, schedule, and impact on foundations. The post-tensioned design is a good alternative based on cost and schedule, but its impact on foundations is unfavorable. If a caisson foundation system was implemented, it would drastically increase the cost of foundations by $3.5 million.

Although these comparisons show that the post-tensioned system is a good alternative, comparison based strictly on these factors is not enough. Architectural requirements and local market conditions tend to drive design more than the engineer. Taking these factors into consideration, the original composite steel framing is the favorable system. Wilmington, and most of Delaware, has remained dominated by steel design. Though composite steel has proved favorable for Gateway Plaza's requirements, the original objective of decreasing floor depth was achieved with the new design. Each floor had a savings of 7.25". This is an overall savings of 116", or 9.66', but not enough to add another floor level.

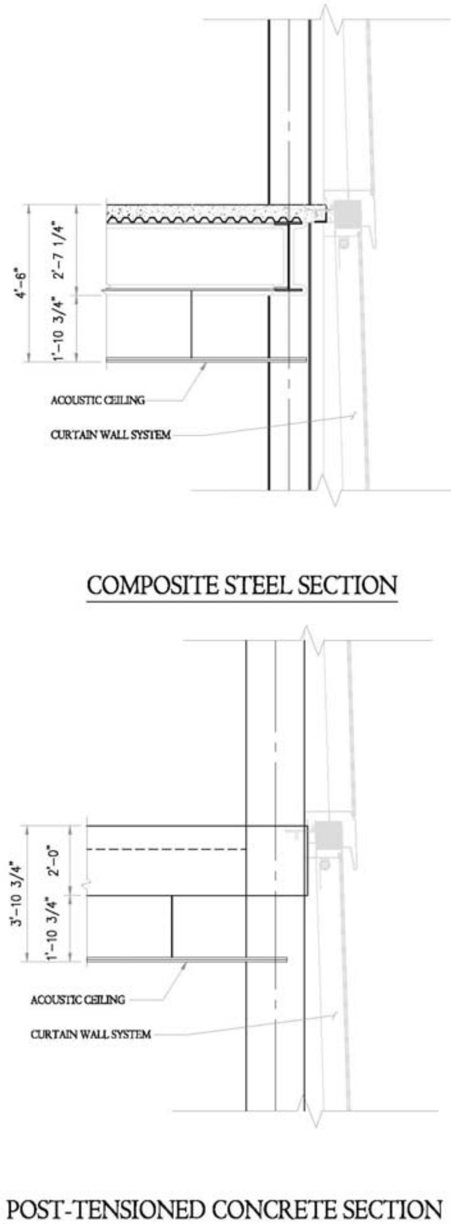


Figure 41-Floor section comparison.

Breadth studies also allowed a good deal of understanding to be gained into how the structural system affects the other systems in the building as well as the impact it has on construction.

Despite the final recommendation that the original system is the best design for Gateway Plaza, a great deal of knowledge was gained through extensive research and the original objective of decreasing floor depths was achieved.

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