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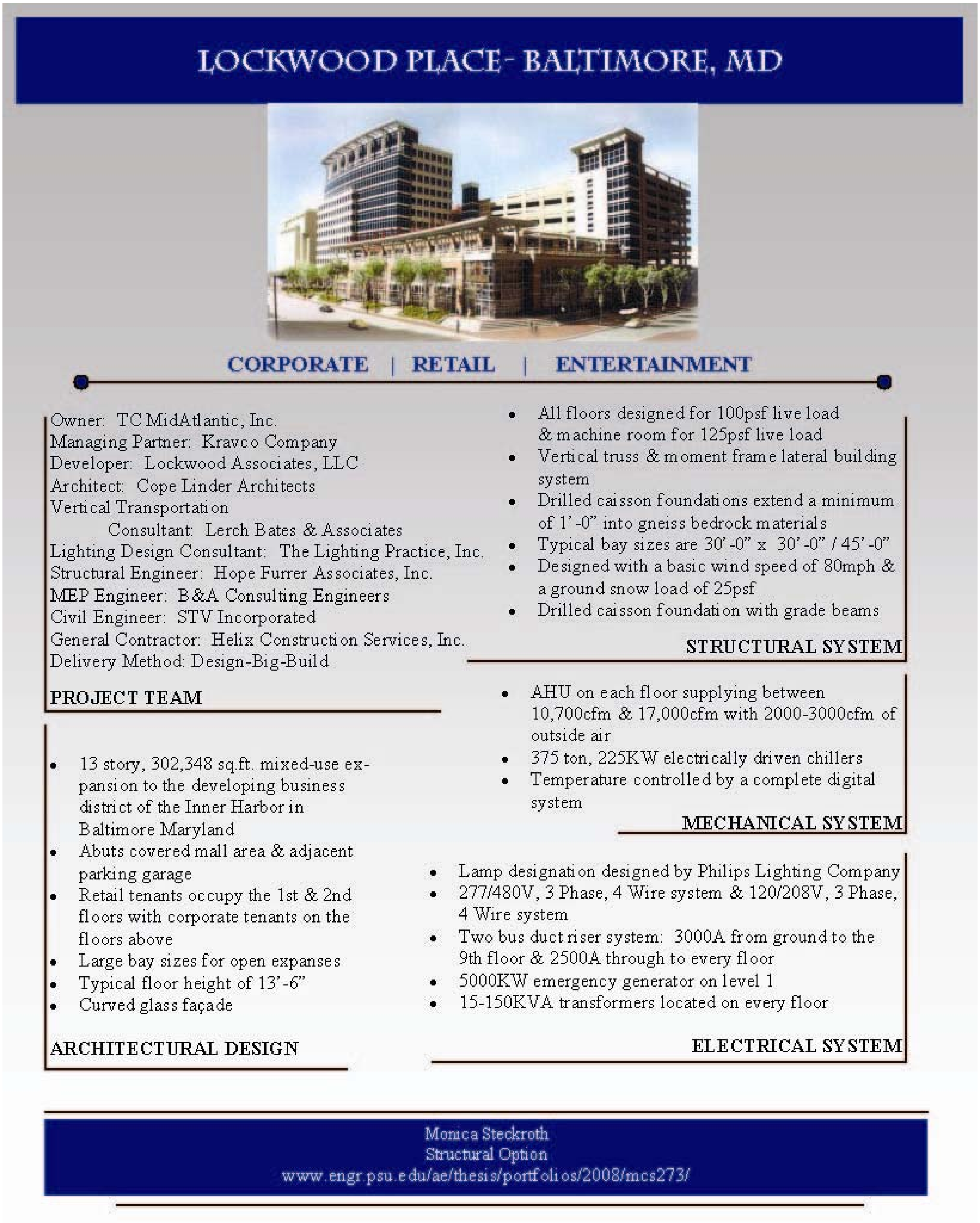
PSUAE

# LOCKWOOD PLACE
# BALTIMORE, MD

Monica Steckroth
Structural Option
Faculty Consultant: Dr. Linda Hanagan



# LOCKWOOD PLACE- BALTIMORE, MD

CORPORATE | RETAIL | ENTERTAINMENT

Owner: TC MidAtlantic, Inc.
Managing Partner: Kravco Company
Developer: Lockwood Associates, LLC
Architect: Cope Linder Architects
Vertical Transportation Consultant: Lerch Bates & Associates
Lighting Design Consultant: The Lighting Practice, Inc.
Structural Engineer: Hope Furrer Associates, Inc.
MEP Engineer: B&A Consulting Engineers
Civil Engineer: STV Incorporated
General Contractor: Helix Construction Services, Inc.
Delivery Method: Design-Big-Build

**PROJECT TEAM**

- 13 story, 302,348 sq.ft. mixed-use expansion to the developing business district of the Inner Harbor in Baltimore Maryland
- Abuts covered mall area & adjacent parking garage
- Retail tenants occupy the 1st & 2nd floors with corporate tenants on the floors above
- Large bay sizes for open expanses
- Typical floor height of 13'-6"
- Curved glass façade

**ARCHITECTURAL DESIGN**

- All floors designed for 100psf live load & machine room for 125psf live load
- Vertical truss & moment frame lateral building system
- Drilled caisson foundations extend a minimum of 1'-0" into gneiss bedrock materials
- Typical bay sizes are 30'-0" x 30'-0" / 45'-0"
- Designed with a basic wind speed of 80mph & a ground snow load of 25psf
- Drilled caisson foundation with grade beams

**STRUCTURAL SYSTEM**

- AHU on each floor supplying between 10,700cfm & 17,000cfm with 2000-3000cfm of outside air
- 375 ton, 225KW electrically driven chillers
- Temperature controlled by a complete digital system

**MECHANICAL SYSTEM**

- Lamp designation designed by Philips Lighting Company
- 277/480V, 3 Phase, 4 Wire system & 120/208V, 3 Phase, 4 Wire system
- Two bus duct riser system: 3000A from ground to the 9th floor & 2500A through to every floor
- 5000KW emergency generator on level 1
- 15-150KVA transformers located on every floor

**ELECTRICAL SYSTEM**

Monica Steckroth
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www.engr.psu.edu/ae/thesis/portfolios/2008/mcs273/

# 1. ACKNOWLEDGEMENTS

My Family

Dr. Linda Hanagan
The Pennsylvania State University, Faculty Advisor

Architectural Engineering Faculty
The Pennsylvania State University

Carol Hayek, PhD
SD Post-Tensioning

Hope Furrer
Hope Furrer Associates, Inc.

B&A Consulting Engineers

Milton Steel, Inc.

Jason Witterman
Penn State AE Mechanical Option

Todd Povell
Penn State AE Construction Option

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# 3. EXECUTIVE SUMMARY

Lockwood Place in Baltimore, Maryland is a thirteen story mixed-use development building utilized predominately for retail and corporate businesses. The existing building enclosure is made primarily of steel with a glass curtain wall façade. Directly adjacent to the building abuts a covered mall area and a parking garage. The parking garage connects to the second level of Lockwood Place through a corridor and lobby.

The goal of this report is to redesign and evaluate Lockwood Place as a post-tensioned concrete building and determine the viability of this solution. The effectiveness of the redesign is based on increased plenum depth for MEP systems, an increase in air duct size creating a quieter, energy efficient system, and reduction in cost and schedule for the building. These criteria were determined through a complete redesign of the building's structural system, resizing of mechanical air ducts and fan, and a cost and schedule analysis for both existing and proposed systems.

The building's structural steel system was completely replaced with concrete. The proposed floor was a 12" two-way post-tensioned floor. Moment frames and eccentric braces were replaced with five shear walls. Caisson sizes increased due to additional building weight. An increased depth of 18.25" plenum space was gained.

Mechanical air ducts were enlarged to utilize additional plenum space. With enlarged duct sizes, static pressure supply required by the fan decreased. A new typical fan was sized to supply 11.2horsepower, which is less than the 20horsepower required by the existing fan. The new fan also proves to provide more space in the mechanical room and lower installation costs due to the smaller size and reduction in weight of the fan unit itself.

Cost of the structural system was determined for each existing and proposed systems. The change from steel to concrete resulted in a 16% decrease in cost. A schedule was also determined for the existing and proposed systems. The proposed system resulted in an additional five weeks of construction time. This was expected due to the time required to form, pour, and cure concrete. Despite the additional construction time required, the proposed system was determined to be a viable solution to Lockwood's Place structural system.

# 4. INTRODUCTION

As an expansion to the corporate/entertainment district of Baltimore's Inner Harbor, the Lockwood Place Office Building is located directly across from the National Aquarium. The building has a curved glass, curtain wall façade and abuts a covered mall area and an adjacent parking garage. It is comprised of thirteen floors and over 300,000 square feet of floor space.

At ground level, a visitor is welcomed by a grand lobby entrance. At the second level, a visitor has direct access to the adjacent parking garage. At the third level tenants have the option to utilize two balcony spaces. Each floor is designed with large bay sizes, allowing for open floor plans. The spaces on the first two floors, occupied by retail tenants, rise to a combined height of 34 feet. The third through the twelfth floors are occupied by corporate tenants and each floor height is 13'-6". A penthouse is constructed on the thirteenth floor. The floor height is 18' and it sets back slightly from the rest of the building. Lockwood Place is designed to accommodate a range of tenants' needs, while providing a sleek exterior appearance with each story consisting of full height glass and large spans.

This document is the final report of the analysis and redesign of Lockwood Place. A structural depth is the main focus of the report. This depth involves the redesign of the building's structural system from existing steel to a completely concrete system. Breadth areas of mechanical systems and construction management have been studied to determine the benefits of the structural redesign.

All analysis and submittals prior to this report can be viewed at http://www.engr.psu.edu/ae/thesis/portfolios/2008/mcs273/.

# 5. BUILDING BACKGROUND

## 5.1 Gravity System

500 East Pratt Street has a typical superstructure floor framing system made of composite steel beams and girders. The slab is 3-1/4" light weight concrete topping on 3"x20gage galvanized metal deck. For composite beam action, ¾" diameter by 5-1/2" long headed shear studs are used, conforming to ASTM A108, Grades 1010 through 1020. Typical bay sizes are 30'-0" x 30'-0" and 45'-0" x 30'-0." Infill beams are spaced 10'-0" on center, framing into a typical girder size of W24x62. All steel conforms to ASTM A572, Grade 50, unless otherwise noted on the drawings. MEP systems are run through the structural framing system. Holes created in the beams and girders from the MEP systems are reinforced according to AISC Design Guide 2. A two hour fire rating is provided for all floor slabs, beams, girders, columns, roofs, and vertical trusses. The typical floor plan can be viewed in the diagram below. A typical bay size is highlighted by a red box.

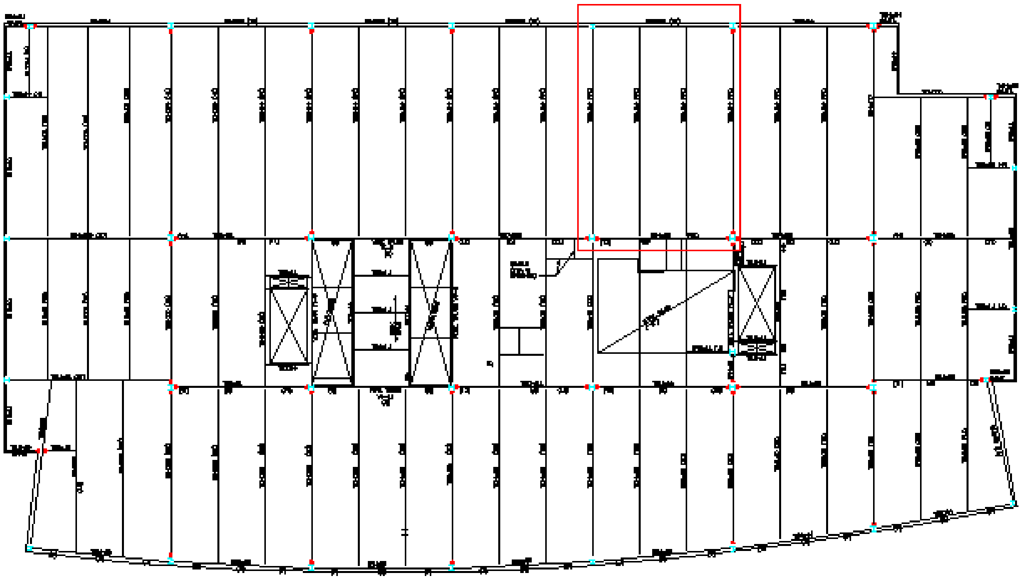


**Figure 5.1. Typical Floor Plan**

*Roof System*

At the penthouse level of Lockwood Place, the building steps back creating a high roof and a low roof. A third roof, the highest point of the building, is created by an extended machine room ceiling located at the penthouse level. The roof on the penthouse is sloped slightly down into the machine room wall. While the framing of the penthouse floor is consistent with the typical building superstructure system, infill beam sizes are reduced due to smaller bay widths. All three roof systems are 1-1/2"x20ga. Galvanized type 'B' metal deck. Infill beams are located at 6' on center. Beam sizes range from W10x12 to W24x76 depending on their location.

Exterior slabs that are located at level twelve are 4-1/2” normal weight concrete topping on 3”x20gage galvanized composite metal deck. The slabs are reinforced with 6x6-W2.9xW2.9 W.W.F. Waterproofing is required for all exterior slabs.

A screen wall is located on level twelve to disguise mechanical equipment. A canopy extends over a balcony on the twelfth floor. The canopy is also made of 1-1/2”x20gage galvanized type ‘B’ metal deck.

### 5.2 Lateral System

Lockwood Place’s lateral system is comprised of moment frames and eccentric braced frames. Moment frames run both east/west and north/south directions. Eccentric braced frames are located around the elevators/elevator lobby. Sizes of the braces range from W14x19 at the base of the building to W8x31 at the top of the building and are pinned connections. Lateral loads were distributed based on the rigidity of each frame. Columns that have eccentric braces framed into them are designed to be fixed to their supports at the base of the building. All other columns are designed to have pinned bases. The lateral system can be viewed in the Figure 5.2.1 and 5.2.2 shown below.

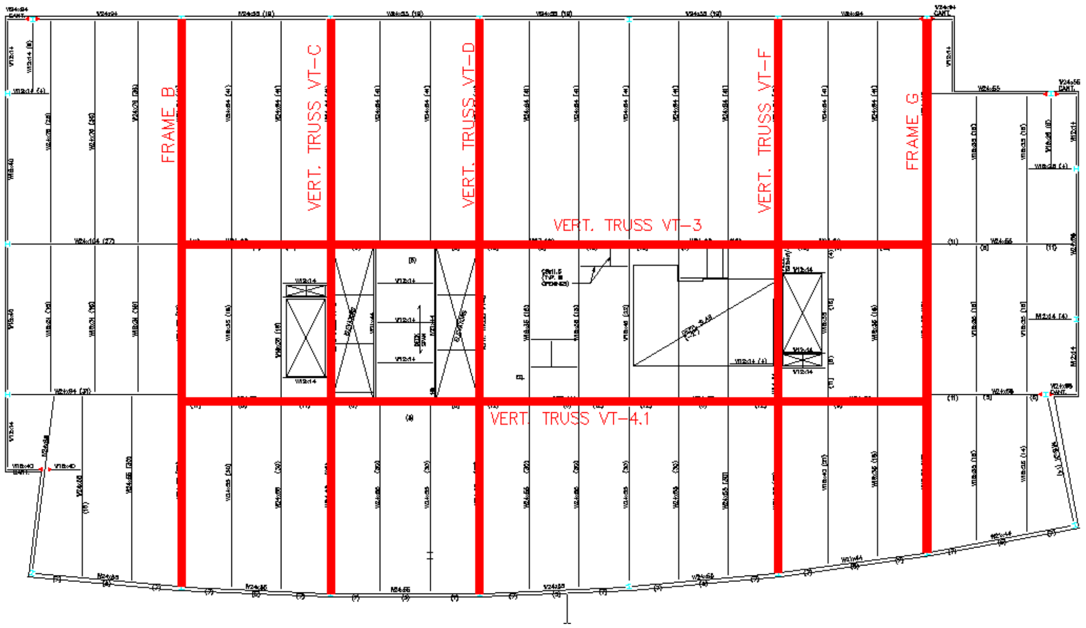
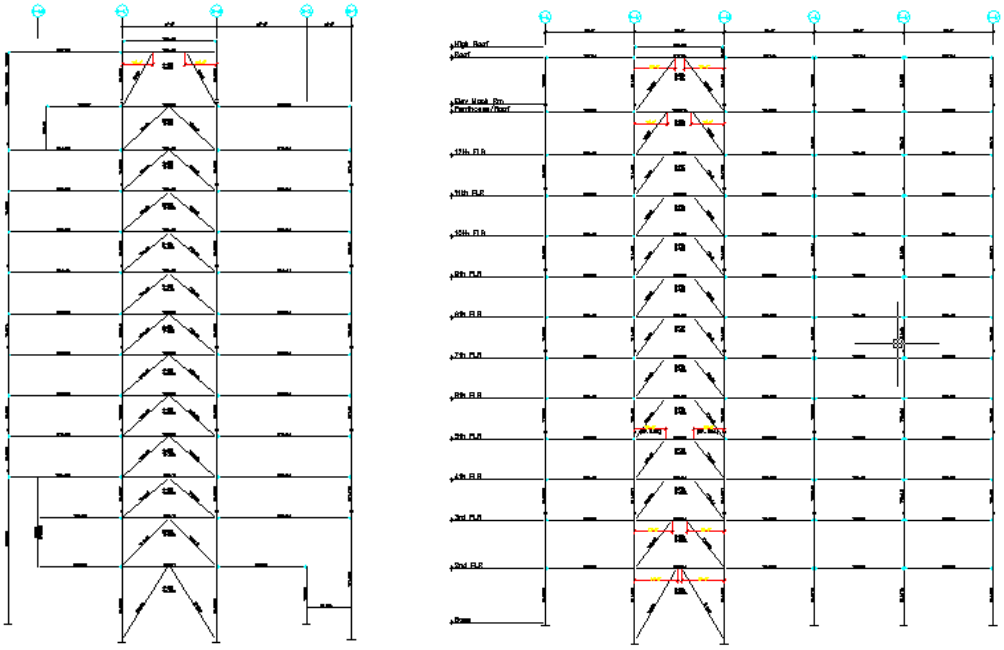


**Figure 5.2.1. Lateral System Plan**



VERTICAL TRUSS VT-D

VERTICAL TRUSS VT-4.1

**5.2.2. Lateral System Elevations**

### 5.3 Foundation

Being located along Baltimore's Inner Harbor, Lockwood Place's soils consist of existing man-made fill. The maximum soil bearing pressure for spread footings is 1000psf. To accommodate for this bearing capacity, the foundation system is made of drilled caissons. Caisson shaft diameters range from 2'-6" to 6'-0." Typically, they extend a minimum of 1'-0" into Gneiss bedrock and have a minimum concrete compressive stress of 4500psi.

Grade beams travel between pile caps and have a minimum concrete compressive strength of 4000psi. Each grade beam ranges in size from 18"x24" to 24"x42" and is reinforced with top and bottom bars.

### 5.4 Mechanical Air Distribution System

The existing air distribution system services each floor to meet tenant requirements. One air handling unit is placed at each level. Powered Induction Units take air from the ceiling plenum and distribute air to the occupied space through a duct system. Chilled water is supplied to the air handling units from a central refrigeration plant. Heating requirements are met by electrical resistance heating coils located integral with the powered induction units.

# 6. PROBLEM STATEMENT

The existing structural system accomplishes the goal of long spans and open spaces to allow for tenant flexibility. A composite steel structural system is the logical choice for the existing Lockwood Place building. Large bay sizes that allow for open floor plans and provide tenant flexibility are easily accomplished.

To accommodate high floor to ceiling height and small depth between floors, MEP systems run above the bottom of the structural beams and girders. Providing holes and necessary reinforcement through almost all beams and girders to allow space for MEP systems is costly and time consuming. The sizes of the existing steel members have been increased to accommodate vibration created in large spans and maintain enough capacity for the holes. Future change in the MEP systems is limited due to the necessity of holes in structural members.

Through the solution of a post-tensioned two-way flat slab, large floor to ceiling heights and a small structural sandwich between floors is achieved. The new floor system allows MEP systems to run under the structural floor and have flexibility for future changes. The lateral system is adjusted to accommodate the new concrete floor system. It is comprised of shear walls located around elevators/stairwells. To remain consistent with the new concrete system, columns are redesigned in concrete to resist gravity and lateral loads when applicable.

# 7. STRUCTURAL DEPTH

The structural depth of this report focuses on the complete redesign of Lockwood Place from the existing steel system to an entire concrete system. The floor is designed as a post-tensioned two-way flat slab with column capitals. Columns are redesigned in concrete to support gravity loads. Five cast-in-place concrete shear walls are introduced to resist 100% of lateral loads.

## 7.1 Codes & Referenced Standards

500 East Pratt Street was originally designed according to BOCA Building Code, 1996 Edition, referencing ASCE-7. ACI-318-02 was used as a guideline for the concrete portions of the building, along with the Allowable Stress Design (ASD) method according to AISC standards as a guideline for the structural steel portions the building.

The building's redesign utilizes the International Building Code (IBC 2006), referencing ASCE-7-05. ACI-318-05 was used for the design of all concrete components within the structure of the building and in accordance with the Load and Resistance Factor Design method.

*Load Combinations:*

1.4D
1.2D+1.6L+0.5(S or Lr)
1.2D+1.0E+L
1.2D+1.6W+L+0.5(S or Lr)
1.2D+1.6Lr+(L or 0.8W)
0.9D+1.6W
0.9D+1.0E

## 7.2 Design Loads

*Dead Load*

**DEAD LOAD (psf)**

| Location/Loading | Office | Lobby/ Corridor | Machine Room | Retail | 1st Floor Lobby | Balconies | Roof |
|---|---|---|---|---|---|---|---|
| Concrete Slab | 150 | 150 | 150 | 150 | 63 | 150 | 100 |
| Partitions | 5 | 5 | - | 5 | 5 | - | - |
| Pavers/ W.P. | - | - | - | - | - | 2 | 2 |
| M/E/C/L | 10 | 10 | 10 | 10 | 10 | 10 | 10 |
| Roofing | - | - | - | - | - | 2 | 2 |
| Insulation | - | - | - | - | - | 2 | 2 |
| **Total Dead Load** | 165 | 165 | 160 | 165 | 78 | 166 | 116 |

*Wall Dead Load*

Curtain Wall…………..25psf
8” CMU Wall………...41psf

*Live Load*

| LIVE LOAD (psf) | | |
|---|---|---|
| Location | Design Load | Minimum Required |
| Office | 100 | 50 for offices only |
| Lobby/Corridor | 100 | 100 first level, 80 above first level |
| Machine Room | 125 | 125 |
| Retail | 100 | 100 first level, 75 above first level |
| 1st Floor Lobby | 100 | 100 |
| Balconies | 100 | 100 exterior |
| Roof | 30 | 20 assuming no reduction |

*Wind Load Criteria*

| General Information | |
|---|---|
| Building Category | II |
| Importance Factor, I | 1.0 |
| Exposure Category | D |
| kd | 0.85 |
| Topographic Factor, kzt | 1.0 |
| V (mph) | 90 |
| Period (T) | 1.04 |
| Gust Effect Factor | 0.85 |
| Cp | 0.80 |
| Building Height, hn | 194 |
| x | 0.75 |
| frequency, n1 | 0.96 |
| North/South Length | 118.6 |
| East/West Length | 218.3 |
| Enclosure Classification | Fully Enclosed |

*Seismic Criteria*

| **General Information** | | |
|---|---|---|
| Occupancy Type | | II |
| Seismic Use Group | | I |
| Site Class | | D |
| Seismic Design Category | | B |
| Short Period Spectral Response | Ss | 0.170 |
| Spectral Response at 1 Second | S1 | 0.051 |
| Maximum Short Period Spectral Response | Sms | 0.272 |
| Maximum Spectral Response at 1 Second | Sm1 | 0.122 |
| Design Short Period Spectral Response | SDS | 0.181 |
| Design Spectral Response at 1 Second | SD1 | 0.082 |
| Response Modification Coefficient | R | 3 |
| Seismic Response Coefficient | Cs | 0.0155 |
| Effective Period | T | 1.767 |
| Height Above Grade | hn | 194 |
| Overstrength Factor | Ω | 3 |
| Deflection Amplification Factor | Cd | 3 |
| | | |
| Base Shear | | 948k |

## 7.3 Proposed Floor System

The feasibility of a post-tensioned floor system design relies heavily on the geometry of the building. Standard bay sizes are the ideal situation for post-tensioned design. The typical floor layout of Lockwood Place lends itself considerably to this type of design. Although the front face of the building is radial, the curvature is minimal allowing a fairly standard design.

Lockwood Place's typical existing floor system is comprised of 24" beams with a 6-1/4" light weight concrete slab. Mechanical equipment ran above the bottom of the structural steel through the web of the beams and girders. The new post-tensioned floor system aims to maintain the depth of the floor slab so as not to interfere with the existing mechanical equipment.

An initial floor thickness of 12" was determined by the ratio of L/45 with 45' spans. Placements of the banded tendons were considered with regard to elevator shaft and stairwell openings. It becomes logical to place banded tendons parallel to the long side of openings within the floor. Banded tendons run in the east/west direction, while evenly spaced tendons run in the north/south direction. With this arrangement, a minimal number of tendons require a splayed layout despite the curvature of the front façade. To accommodate the existing column layout, tendons along Line 3 were split in half at Line G and anchored above the respective columns along Line H. An alternative layout with banded tendons in the north/south direction was considered. The design would include wide beams 12" deep with an 8" floor slab. This design was not selected due to the large number of tendons that would require termination around openings and the number of bays that would require splayed tendons to accommodate the front curvature of the building.

A RAM Concept 2.0 model was developed in order to address irregularities in the typical floor plan. With RAM Concept being an analysis program verses a design program, a preliminary number of tendons and their equivalent effective tensile force was determined by hand calculations. These hand calculations can be found in Appendix A. ½" 270k wire strand tendons were selected with 1-1/4" cover on top and bottom. This created a maximum drape of 9.5". Cantilevered edges of the building were accommodated by adjusting the drape in the latitude and longitude directions. At the southwest corner of the building, an additional four strands were added and anchored into the slab to adjust for complicated geometry. The drape profiles are terminated at 6", the midpoint in the slab. A target of 60%-70% of load balancing was achieved in most bays with typical geometry. In bays where this target could not be achieved, tendon profiles provide as much load balancing as possible.

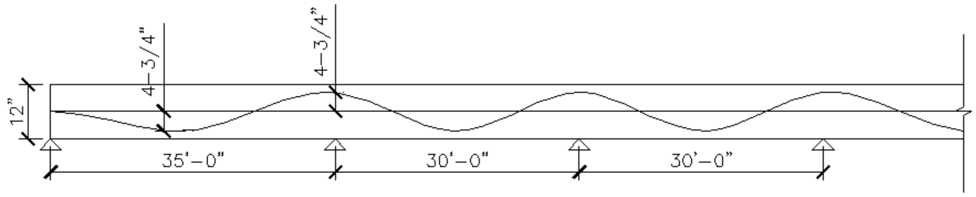


**Figure 7.3.1.** East/West Tendon Profile

With large exterior spans ranging from 40'-0" to 45-0", punching shear becomes a prominent failure possibility. Traditional drop panels were analyzed using hand calculations. The typical thickness of the drop panel, t/3, did not provide the shear capacity needed to support the large spans and heavy loads. Also, added thickness extending 1/6 of the span into the bay created an interference with the existing mechanical equipment. 18" thick column capitals were introduced with a radius of 4'-0" around the centerline of the column. These column capitals provide enough capacity to ensure punching shear failure will not occur. Punching shear was checked at the column and at the edge of the drop panels in the 12" slab. The limited extension of the shear cap into the bay eliminates any interference with the existing mechanical equipment. As expected, the column capitals in the RAM Concept model were consistent with hand calculations. Refer to Appendix A for shear cap hand calculations. Regular reinforcement required by ACI 318 was determined in the RAM Concept Model. Negative reinforcement for negative moment is specified on the proposed floor plan in Figure 7.3.5.

Ram Concept Model

In the RAM Concept Model, latitude and longitude design strips were generated by the computer and evaluated for consistency. Strips over the shear walls were eliminated to avoid redundancy. The number of banded tendons across each column line in the east/west direction is as follows: 1-14, 3-32, 4-24, 5-12. The effective prestress in the strands are 372K, 692K, 639K, and 320K respectively. The difference in the number of strands accommodates the varying tributary widths of each span. All stresses produced are within industry standard limits. In the north/south direction tendons are placed in groups of four evenly spaced at 4'-0" on center. The prestress force in these tendons is 32K/ft. Additional groups of four strands are placed in the 45' span in the far north bays of the building to generate the strength required. The additional prestress force in these tendons is 28K/ft. To view tendon layouts refer to Figure 7.3.2 and Figure 7.3.3. Prestress calculations can be found in Appendix A.

Where mechanical equipment interrupts the floor slab, tendon profiles were adjusted. Given the direction and size of the openings, only slight adjustments were necessary to be made to accommodate these openings. A ratio of 1:3 was maintained in locations where tendons were stretched diagonally for the purpose of openings. Few numbers of tendons that would not span around the elevator shafts and stairwells were terminated in the openings. As a whole, uniformity was desired for the north/south tendons to create redundancy and increased load redistribution characteristics. Splayed tendons were limited to three bays with the north/south distributed tendon layout.

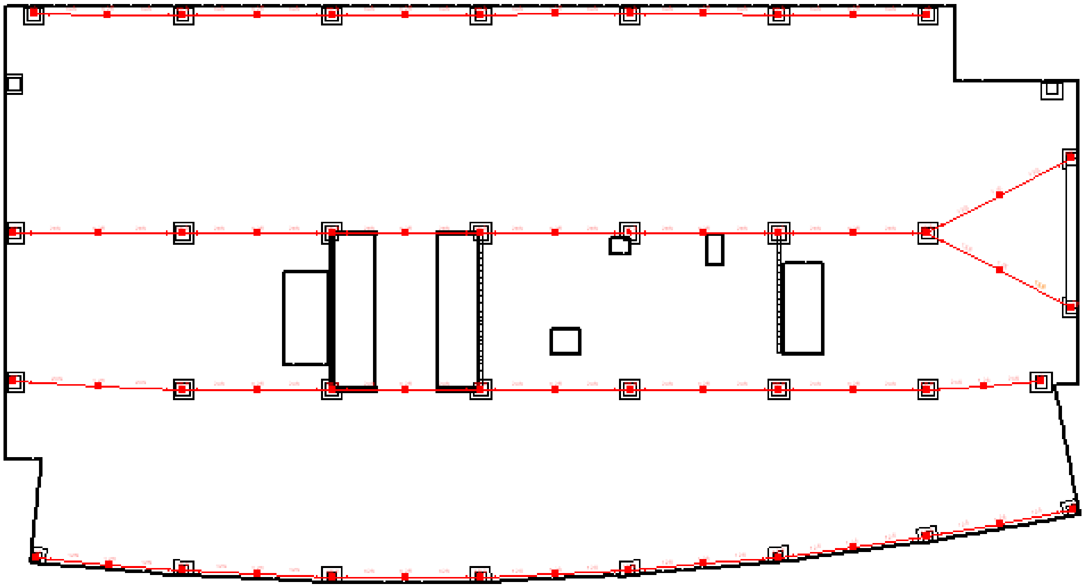


**Figure 7.3.2.** East/West Banded Tendon Layout

**Figure 7.3.3.** North/South Distributed Tendon Layout

It is necessary to consider deflections under full service load and the camber created in the slab in the absence of loading. The maximum camber deflection previous to loading is less than 1/2". The maximum deflection under full service loading is 1.40" and is equivalent to L/386. Camber in long-term loading is 0.36." As desired, the precompression plan is uniform. See Figure 7.3.4 for the full service long-term deflection diagram.

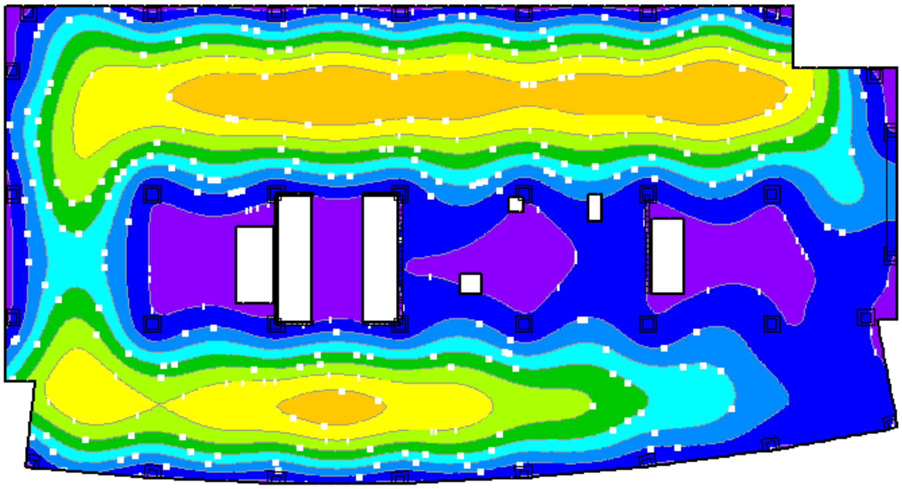


**Figure 7.3.4.** Long-Term Deflection Diagram

The final floor design provides a structural sandwich of 12." Whereas prior to redesign mechanical equipment was limited to sizes that fit within the previously existing structural steel; now the mechanical equipment can utilize a full 24" of plenum space. The final floor system design can be found in Figure 7.3.6. See Figure 7.3.5 for a comparison of the existing and new floor depths integrated with mechanical equipment. An additional 18-1/4” plenum space is provided by the new concrete system.

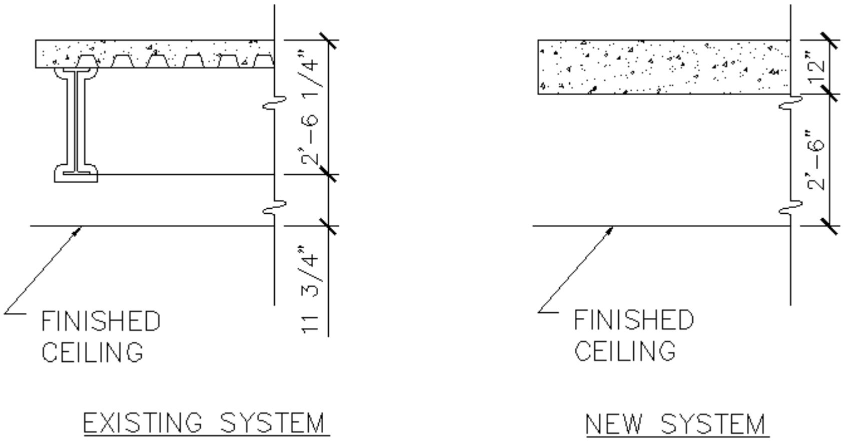


**Figure 7.3.5.** Structural Floor Depth Comparison.

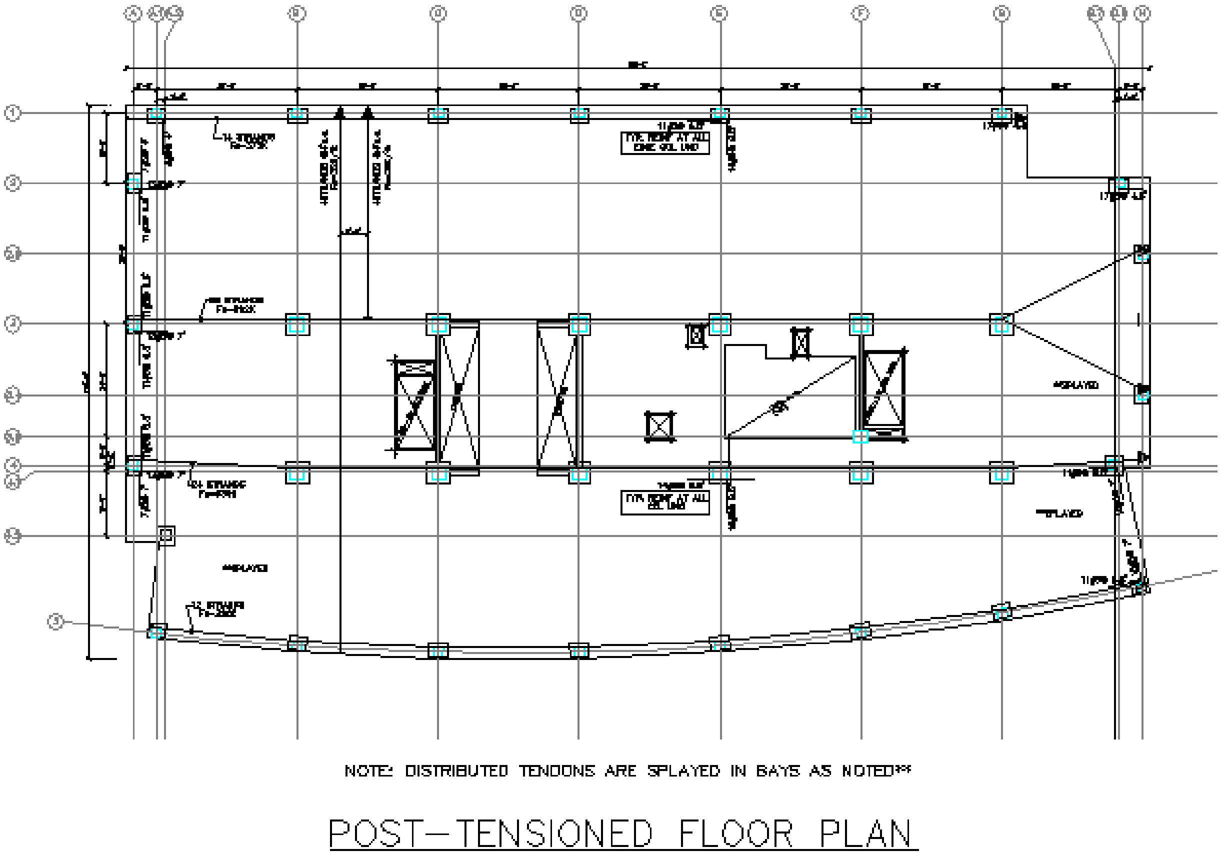


**Figure 7.3.6.** Typical Floor Plan

## 7.4 Column Redesign

Columns are designed based on the previously stated gravity loads in Section 7.2. Typical interior and exterior columns are designed based on the maximum accumulated loads within the building. For constructability purposes, only two different size columns are used. Reinforcement for each column was designed using PCA Column and was verified using simple hand calculations. Because all columns are designed to resist primarily gravity loads, the load combination of 1.2Dead +1.6Live controlled the design. The design for each typical column can be found in Table 7.4-1and Table 7.4-2. Figure 7.4.1 shows a typical cross section of an interior and exterior column at the base of the columns. Ties are spaced at 4" on center at the base of every column for 4'-0" in consideration of the post-tensioned floor shrinkage. Table 7.4-1 shows the change in reinforcement at two different levels of the building. Along with the change in reinforcement, concrete strength was reduced from 6000psi to 5000psi. PCA Column determined less than half the reinforcement was needed at level 8. Exactly half the reinforcement was used for constructability purposes. PCA Column results can be found in Appendix A.

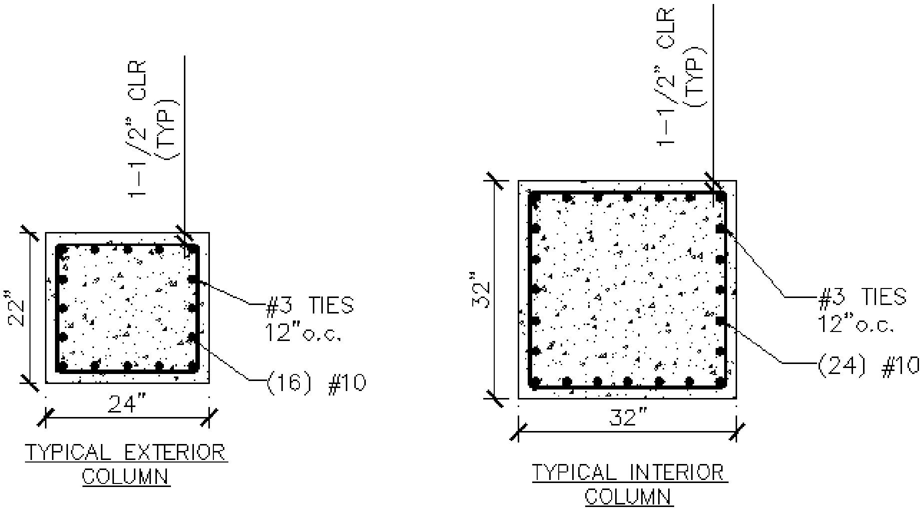


**Figure 7.4.1.** Typical Column Sections

| | Exterior Column | | | Interior Column | | | f'c |
|---|---|---|---|---|---|---|---|
| Level | Ties | Reinf. | Rho | Ties | Reinf. | rho | psi |
| 1 | #3@12" | (16) #10 | 3.85 | #3@12" | (24) #11 | 2.98 | 6000 |
| 8 | #3@12" | (8) #10 | 1.93 | #3@12" | (12) #11 | 1.49 | 5000 |

**Table 7.4-1** Column Design Reinforcement

When changing the basic structure of the building, it was discovered that certain geometrical constraints are much more feasible with steel construction verses concrete. At the third level, a 30'-0" hanging balcony is supported by tension hangers attached to the fourth level. To accommodate this geometrical constraint, four corbelled columns located at the front of the building, lining the lobby space were designed and detailed. A post-tensioned slab for a 30’-0” span balcony space is anchored into the end of the corbel. The corbelled column extends unbraced through the first two stories of the building. A transition is made from the 24” width of the columns above the third level to a 38” from the fourth to the third story. 3’-0” depth was provided at the top and bottom of the column. Four #9 reinforcement bars were used to resist applied moment at top of the column. These bars accommodate minimum spacing requirements when integrated with the column’s vertical reinforcement. An elevation and sections of the column detail can be viewed in Figure 7.4.2 and Figure 7.4.3.

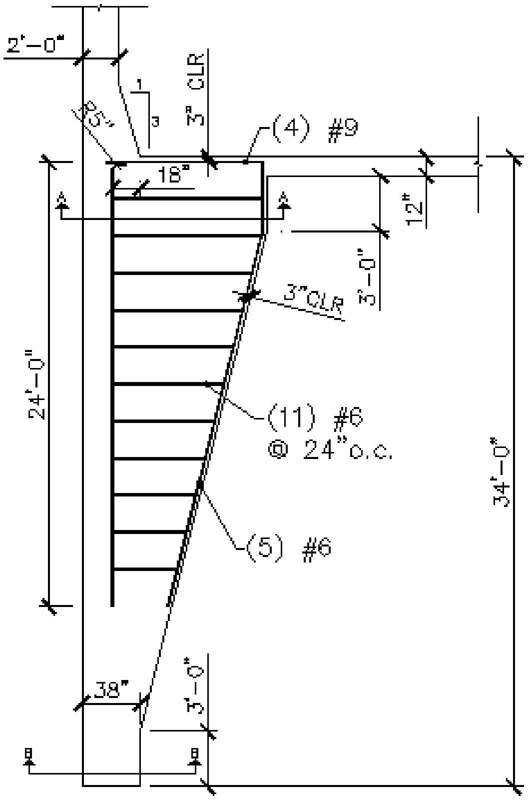


**Figure 7.4.2.** Corbelled Column Elevation

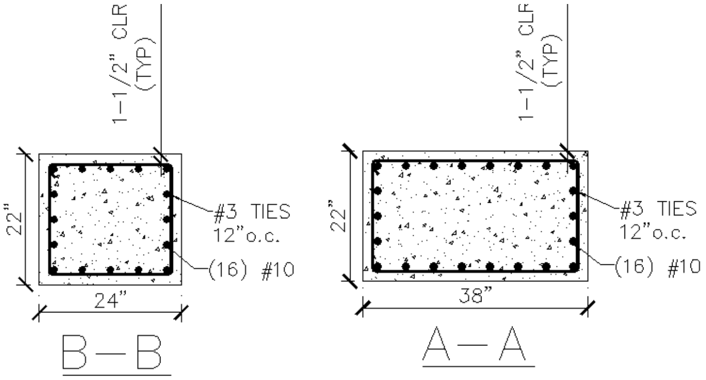


**Figure 7.4.3.** Corbelled Column Sections

The newly designed columns became an architectural feature of the building because of the exterior location. The front exterior perspective is slightly altered from the original design. The location of the corbelled columns can be found in Figure 7.4.4.

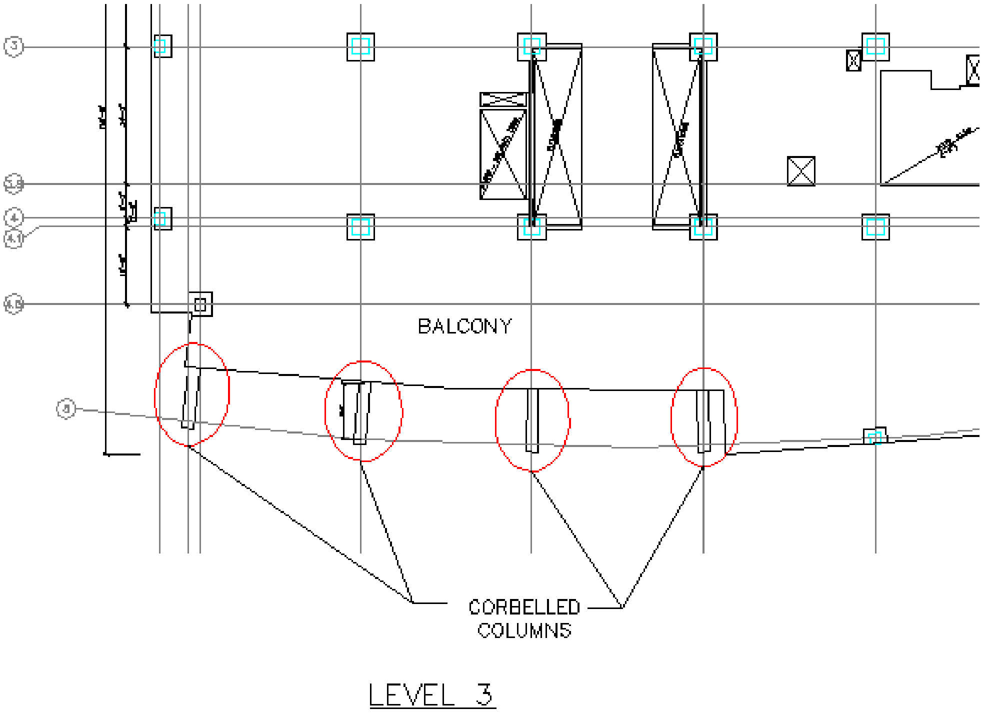


**Figure 7.4.4.** Location of Corbelled Columns

## 7.5 Proposed Lateral System

Lockwood Place's existing lateral system consists of steel eccentric braces and moment frames. Five shear walls were introduced in the new concrete structure. The shear walls are conveniently located at the building's elevator and stairwell cores. Shear walls around the elevator shaft form two C-Shaped walls. The locations of the walls are seen in Figure 7.5.1 below. The location of the shear walls creates a center of rigidity that is close to the center of mass, minimizing torsional effects. The load path remains the same as in the existing structural system. Lateral load is transferred through the rigid diaphragm to each shear wall. The shear walls resist load according to relative stiffness.

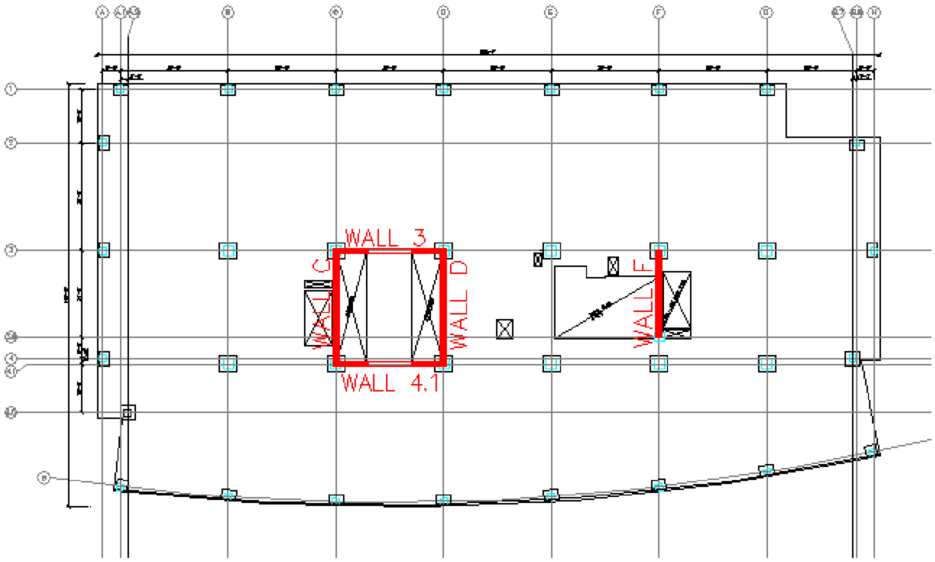


**Figure 7.5.1.** Shear Wall Location

Wind and seismic lateral loads were applied to the structure to determine the controlling forces. Wind loads applied to the building do not vary from the original wind loads calculated. The location and height of the building remain the same as in the existing building design. Due to the weight of the building more than doubling, seismic loads applied to the building increase significantly. Newly calculated seismic story shear forces are in Figure 7.5.2. For seismic calculations, refer to Appendix A.

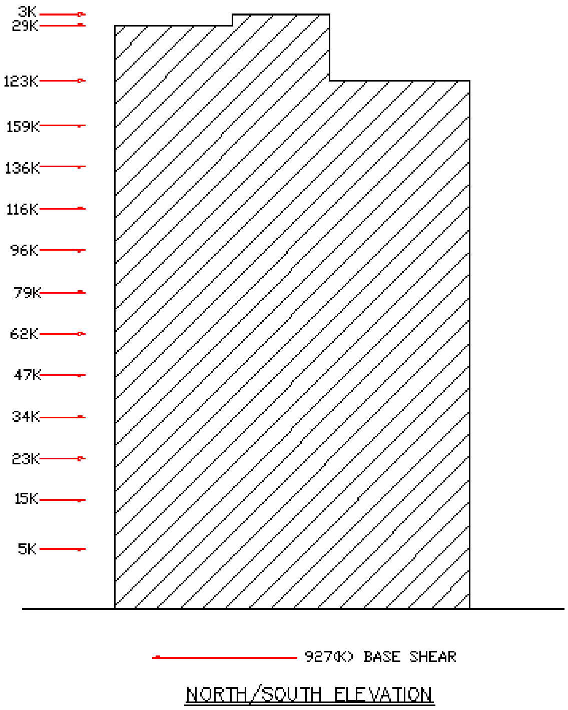


**Figure 7.5.2.** Seismic Redesign Story Forces

ETABS Model

To analyze the distribution of forces to the shear walls at each level for every load case, an ETABS model was developed. The model is a 3-Dimensional model that replicates the geometry of each individual floor. The geometry of the ETABS model can be viewed in Figure 7.5.3. Direct and torsional shears were considered by the ETABS model geometry. The center of rigidity was determined based on stiffness and location of each wall. The locations for the center of rigidity and center of mass can be found in Appendix A.

Direct wind forces were manually applied to each level's center of mass and also generated by the model itself for comparison purposes. The applied forces were within five percent of the generated wind forces in each direction. The generated wind forces were used to determine the controlling forces in each wall. Different ASCE7-05 required wind applications controlled in different walls. The controlling load cases according to ASCE7-05 are located in Table 7.5-1.

| Location | Load Case |
|---|---|
| Wall 3 | ASCE7-05, case 4 |
| Wall 4.1 | ASCE7-05, case 4 |
| Wall C | ASCE7-05, case 4 |
| Wall D | ASCE7-05, case 1 |
| Wall F | ASCE7-05, case 2 |

**Table 7.5-1** Wind controlling load cases for each wall

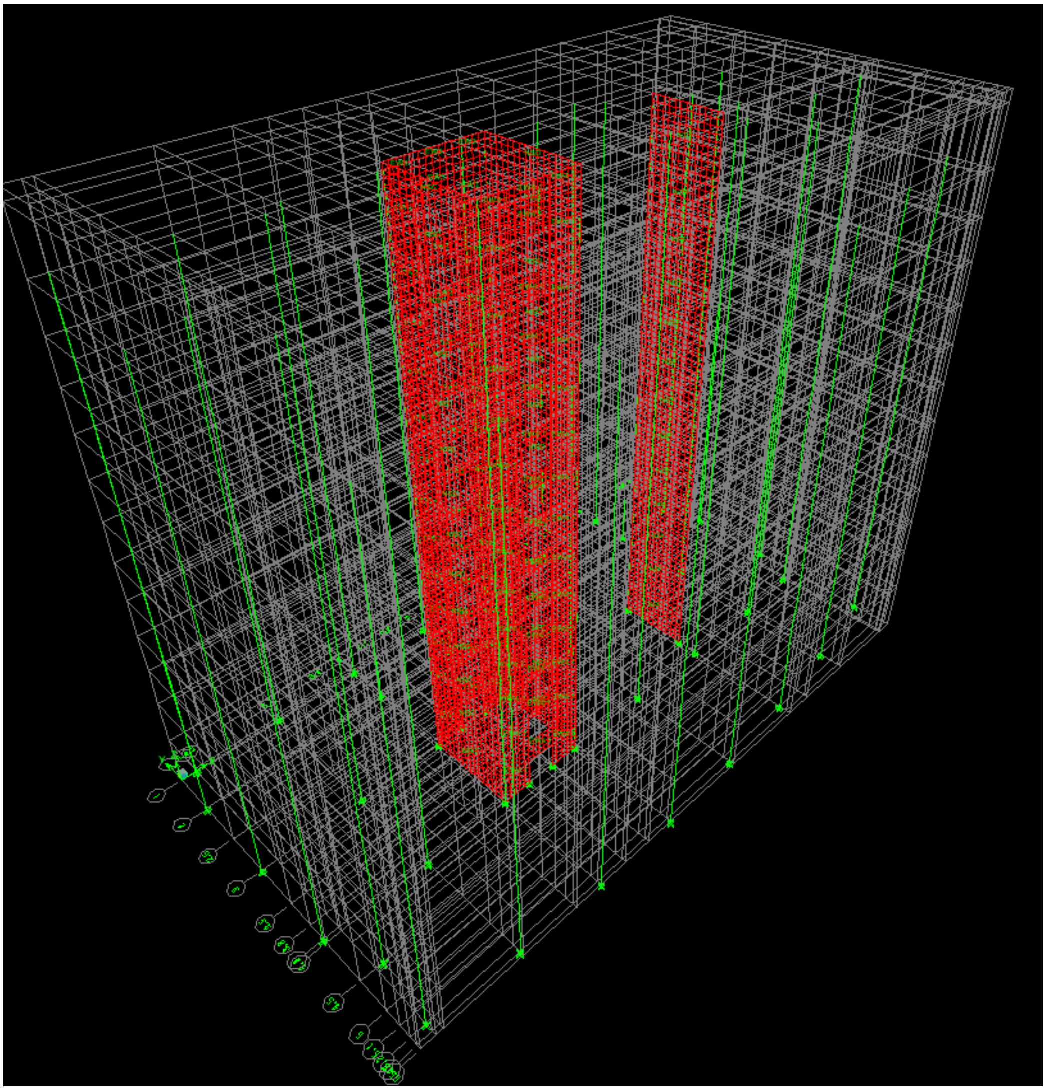


**Figure 7.5.3.** ETABS Model

Seismic loads were manually applied at each level and also generated by the ETABS model for comparison purposes. Building self-weight was determined from an applied mass at each level. The seismic building period was also calculated through ETABS using an applied mass at each level. The code value of 1.77seconds was used in calculating story shears at each level compared to the ETABS model period of 2.33seconds to remain conservative. The modal period was discovered to control in the east/west direction. Manually applied loads and accidental torsional moments were used when determining the controlling forces in each wall. When calculating accidental torsion, the amplification factor, Ax, was determined from drifts developed in the ETABS model. The locations of these drifts are located in Figure 7.5.4. Refer to Appendix A for calculations and a comparison of wind and seismic forces in each wall at each level.

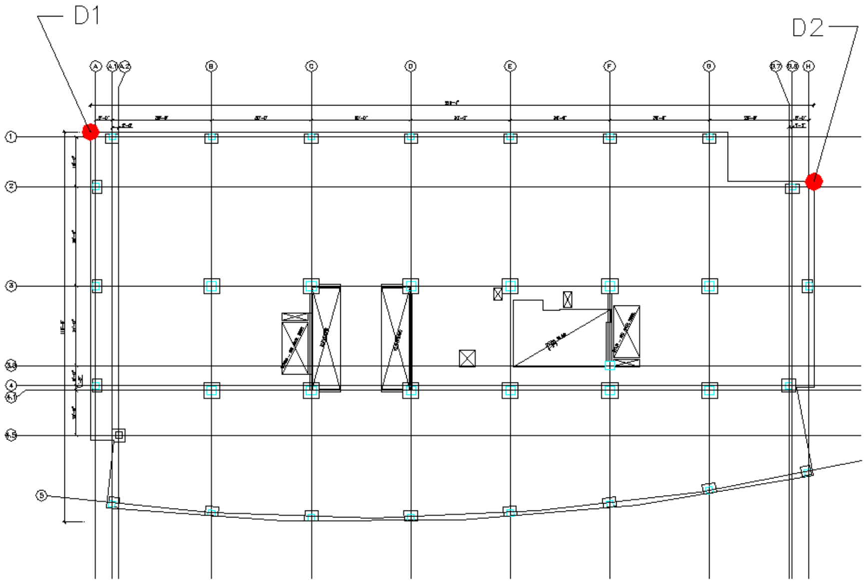


**Figure 7.5.4.** Location of Drift for Amplification Factor Calculation

Wall Design

For the design of all shear walls, load combinations of 0.9D+1.6W and 0.9D+1.0E were applied to the unfactored design lateral loads determined from the ETABS model. When applying these load cases, wind became the controlling lateral load. A total of three typical walls were designed. The factored lateral loads for which each wall was designed are found in Table 7.5-2.

| Location | Factored Design Load |
|---|---|
| Wall 3 | 861 |
| Wall 4.1 | 840 |
| Wall C | 698 |
| Wall D | 989 |
| Wall F | 586 |

**Table 7.5-2** Factored design wind load for shear wall design

To allow for access to the existing elevators, coupling beams were designed around the openings in Wall 3 and Wall 4.1. Each coupling beam was permitted to be designed as a regularly reinforced deep beam due to the geometry of the beam. Column designs from gravity loading were used as boundary elements for the shear walls. The design of each shear wall can be found in Figure 7.5.5, while the coupling beam design can be found in Figure 7.5.6. The original design of the east/west oriented shear walls required a 12” thick wall. After designing the required size and reinforcement of the coupling beam, the wall thickness was increased to 14”. When calculating overturning moment and uplift in the walls, only wall self-weight was considered as dead load to remain conservative. Columns are designed to carry 100% of the building’s gravity loads. All wall and coupling beam calculations can be found in Appendix A.

Wall 4.1 is designed identical to Wall 3 as Wall C is designed identical to Wall D. Concrete strength in the wall changes at level 8 from 6000psi to 5000psi to remain consistent with concrete strength of the adjacent columns. The strength change was determined to be acceptable to resist lateral forces at this level.

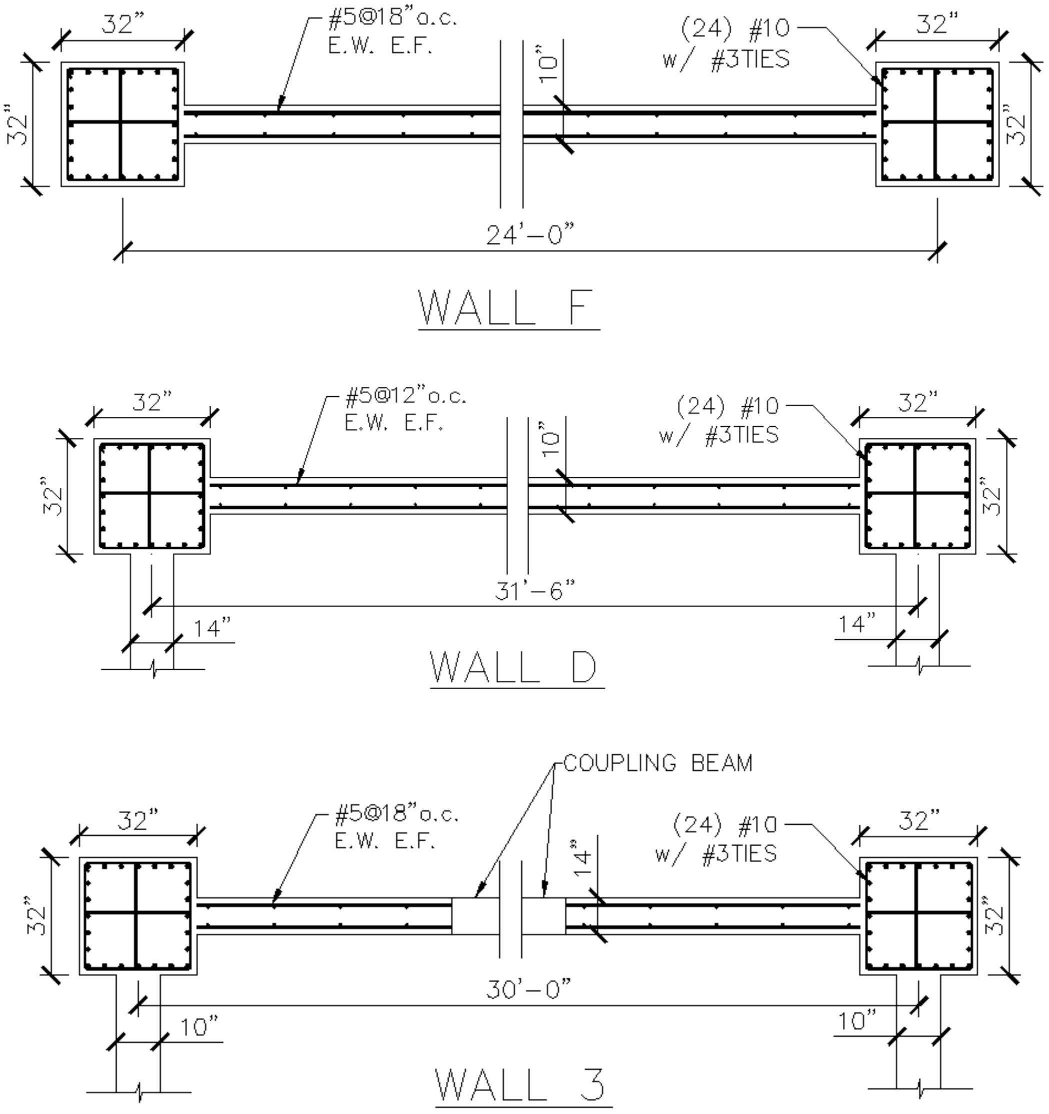


**Figure 7.5.5.** Shear Wall Designs

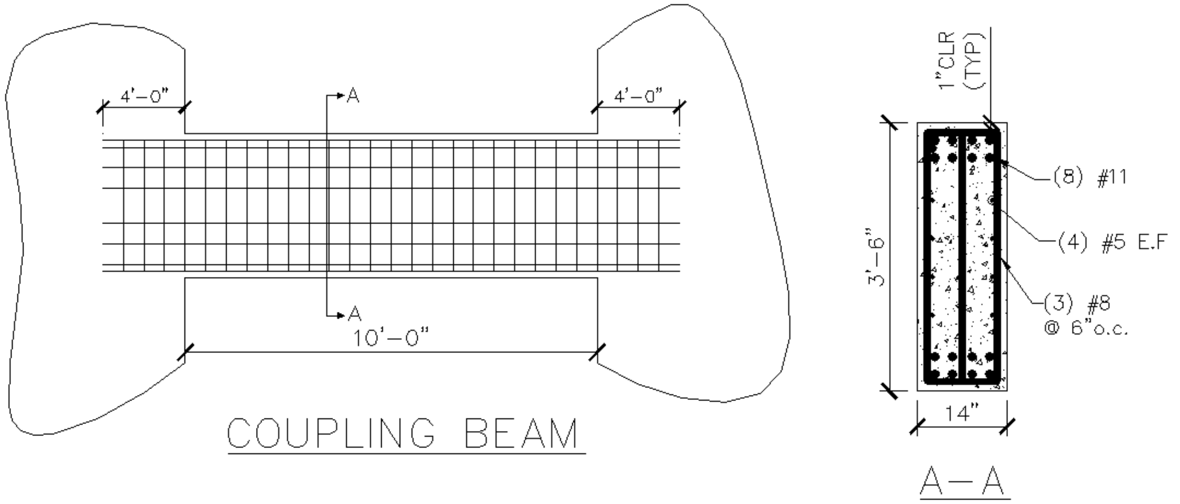


**Figure 7.5.6.** Coupling Beam Design

Total building drifts and story drifts were determined from the ETABS model and compared to the acceptable limit of H/400 (5.82") and ASCE7-05 seismic code. Drifts were examined from levels one through three to ensure that they did not surpass the allowance of the existing expansion joint located between Lockwood Place and its adjacent three story building. All drifts were deemed acceptable. Total building drifts are located in Table 7.5-3. Story drifts and story drift limits are located in Table 7.5-4.

| Maximum Drifts (in.) | | | | |
|---|---|---|---|---|
| Level | Seismic | | Wind | |
| | East/West | North/South | East/West | North/South |
| 2 | 0.05 | 0.03 | 0.03 | 0.04 |
| 3 | 0.14 | 0.09 | 0.08 | 0.11 |
| 4 | 0.25 | 0.17 | 0.13 | 0.18 |
| 5 | 0.38 | 0.25 | 0.20 | 0.27 |
| 6 | 0.53 | 0.35 | 0.27 | 0.38 |
| 7 | 0.67 | 0.46 | 0.35 | 0.48 |
| 8 | 0.85 | 0.58 | 0.42 | 0.56 |
| 9 | 1.01 | 0.70 | 0.49 | 0.72 |
| 10 | 1.17 | 0.82 | 0.56 | 0.84 |
| 11 | 1.32 | 0.85 | 0.63 | 0.96 |
| 12 | 1.46 | 1.07 | 0.70 | 1.08 |
| PH | 1.60 | 1.12 | 0.76 | 1.20 |
| LR | 1.72 | 1.40 | 0.82 | 1.42 |
| HR | 1.78 | 1.31 | 0.85 | 1.30 |

**Table 7.5-3** Total Building Drifts

| Seismic Story Drift Limits | Actual Story Drift |
|---|---|
| 4.32 | 0.60 |
| 3.84 | 1.08 |
| 3.24 | 1.32 |
| 3.24 | 1.56 |
| 3.24 | 1.80 |
| 3.24 | 1.68 |
| 3.24 | 2.16 |
| 3.24 | 1.92 |
| 3.24 | 1.92 |
| 3.24 | 1.80 |
| 3.24 | 1.68 |
| 3.48 | 1.68 |
| 4.32 | 1.44 |
| 1.44 | 0.72 |

**Table 7.5-4** Story drifts for ACSE7-05 seismic code

## 7.6 Impact on Foundations

The existing foundation design is comprised of drilled caissons that extend 1’-0” to 5’-0” into bedrock. Due to the switch in building construction from steel to concrete, the building’s weight significantly increased. The increase in building weight caused an increase in the shaft size of the drilled caissons and eliminated uplift in all columns for all load combinations. No caisson is required to take significant moment from the frame. Caissons that support the shear wall boundary elements are the largest in size due to the required resistance for overturning moment. The new shaft diameters and their loadings and capacities can be viewed in Table 7.6-1 below. The foundation plan can be found in Figure 7.6.1.

| Location | Existing Diameter | Allowable Load | New Diameter | New Allowable Load | Design Loads | | Elev. Top Varies | Elev. Bott. Varies |
|---|---|---|---|---|---|---|---|---|
| | | | | | Vmax | Uplift | | |
| B/5 | 30 | 720 | 48 | 1843 | 1645 | - | -4.33 | -82.00 |
| B/4.1 | 60 | 2879 | 60 | 2879 | 2790 | - | -4.33 | -80.00 |
| B/3 | 60 | 2879 | 60 | 2879 | 2790 | - | -3.00 | -77.00 |
| B/1 | 36 | 1037 | 48 | 1843 | 1650 | - | -6.33 | -78.00 |
| C/5 | 30 | 720 | 48 | 1843 | 1645 | - | -4.33 | -80.00 |
| C/4.1 | 50 | 2000 | 60 | 2879 | 2460 | 0 | -9.42 | -86.00 |
| C/3 | 50 | 2000 | 60 | 2879 | 2460 | 0 | -9.42 | -86.00 |
| C/1 | 36 | 1037 | 48 | 1843 | 1650 | - | -7.00 | -78.00 |
| D/5 | 30 | 720 | 48 | 1843 | 1645 | - | -4.33 | -77.00 |
| D/4.1 | 50 | 2000 | 60 | 2879 | 2460 | 0 | -9.42 | -86.00 |
| D/3 | 50 | 2000 | 60 | 2879 | 2460 | 0 | -9.42 | -86.00 |
| D/1 | 36 | 1037 | 48 | 1843 | 1650 | - | -7.00 | -79.00 |
| E/5 | 30 | 720 | 48 | 1843 | 1645 | - | -4.33 | -76.00 |
| E/4.1 | 40 | 1280 | 60 | 2879 | 2825 | - | -3.00 | -78.00 |
| E/3 | 40 | 1280 | 60 | 2879 | 2825 | - | -3.00 | -78.00 |
| E/1 | 36 | 1037 | 48 | 1843 | 1650 | - | -7.67 | -79.00 |
| F/5 | 30 | 720 | 48 | 1843 | 1675 | - | -4.33 | -74.00 |
| F/3.8 | 50 | 2000 | 60 | 2879 | 2825 | 0 | -5.75 | -74.00 |
| F/3 | 50 | 2000 | 60 | 2879 | 2825 | 0 | -4.75 | -74.00 |
| F/1 | 36 | 1037 | 48 | 1843 | 1650 | - | -7.67 | -79.00 |
| G/5 | 30 | 720 | 48 | 1843 | 1675 | - | -4.33 | -71.00 |
| G/4.1 | 36 | 1037 | 60 | 2879 | 2790 | - | -5.00 | -73.00 |
| G/3 | 40 | 1280 | 60 | 2879 | 2790 | - | -3.00 | -75.00 |

Note: Elevations are with respect to reference datum. Reference datum= Elevation 9.90’(finished first floor elevation.) Maximum uplift force from load combination: 0.9D+1.6W.

**Table 7.6-1** Caisson Design

Existing grade beams located under all shear walls were examined and determined to be acceptable to carry the required gravity load of the shear wall.

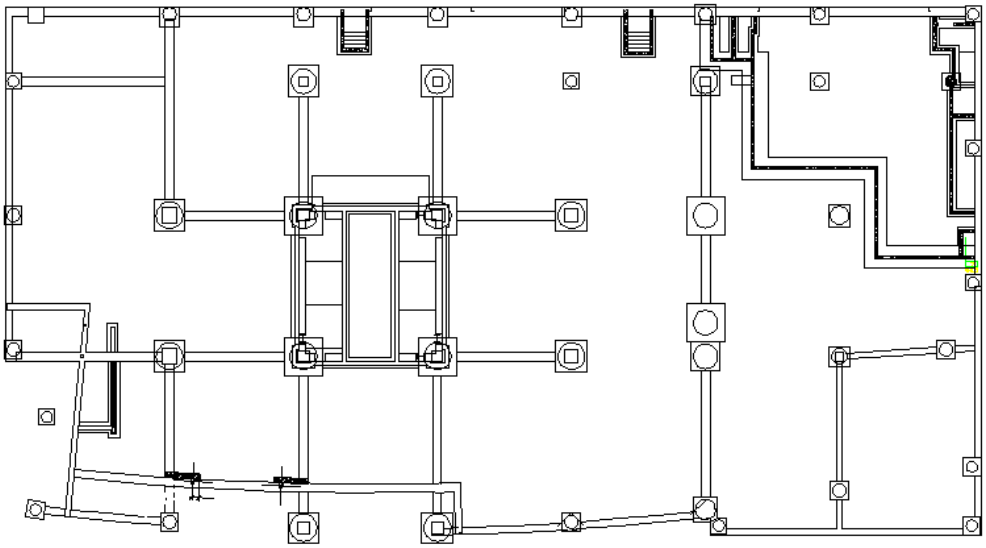


**Figure 7.6.1.** Foundation Plan

# 8. MECHANICAL RETROFIT

A thinner structural system between each floor level allows for an increase in plenum space available to MEP systems. It is possible to increase duct sizes with an increase in plenum space. Increasing duct sizes without a change in demand load will produce smaller velocities within the duct. Smaller velocities in the ducts require a smaller static pressure required by the fan. The goal of this analysis is to increase duct size and in turn decrease fan size and energy required by the fan. Additionally, with smaller air velocities traveling through the ducts, acoustical value is gained. The exact acoustical value would require further analysis and is not within the scope of this report.

The existing air distribution system supplies conditioned air to each level at 46 degrees through a medium pressure, medium velocity air distribution system to pressure independent series type fan powered induction units located throughout each floor. The heating requirements are met by electrical resistance heating coils located integral to the powered induction units, located in the ceiling return air plenum in the vicinity of various zones. An air handling unit is located at each floor level and supplies a maximum of 17000cfm on typical floors.

## 8.1 Powered Induction Units

Powered induction air unit fans mix supply air with intake air from the ceiling plenum and distribute it to the occupied space throughout a duct system. Perimeter coils are controlled in sequence with its respective powered induction unit's primary air valve, thereby eliminating the need for reheat. By delivering air to the powered induction units at a lower temperature, duct sizes are minimized. This allowed for the ducts to be fitted above the bottom of the structural steel in the existing system. The powered induction unit size is based on the demand load for the space. After examining these units, it was determined that they were efficiently sized. To replace these units, the entire system would require a change. To minimize effects on other aspects of the air handling system, duct sizes were exclusively examined.

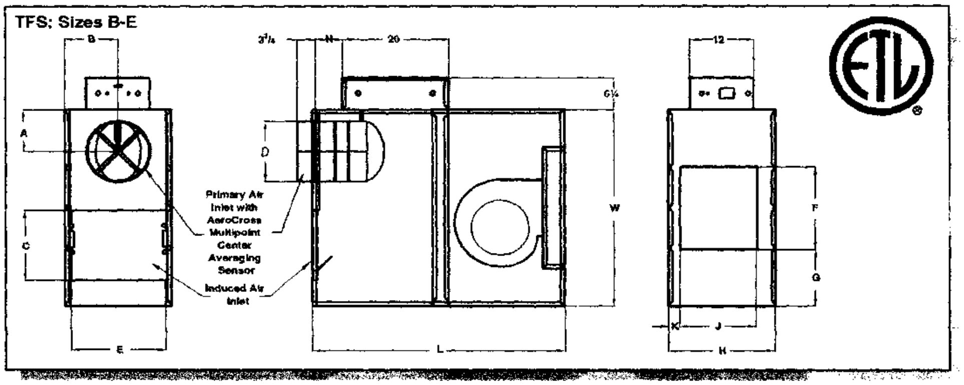


**Figure 8.1.** Powered Induction Unit Reference Diagram.
This diagram is taken from www.titus.com

## 8.2 Air Duct Design

The approach taken to analyze and redesign the duct system is in accordance with ASHRAE Fundamentals 2005. Static pressure losses were evaluated for the existing ducts. These calculations included diffusers, Powered Induction Units, duct runs and all fitting losses for the geometry of the air ducts located on the existing drawings. A value of 0.5 was assumed for pressure losses contributed by the existing fan. The static pressure required by the existing fan is 3.6" water pressure. The existing maximum air velocity in the ducts was found to be 2275ft/min. For all the pressure loss calculations refer to Appendix B.

Each duct was resized according to an Air Duct Calculator produced by TRANE based on the demand load for the duct. The new duct sizes and the critical path for static pressure can be viewed in Figure 8.2.1. Utilization of the additional plenum space can be viewed in Figure 8.2.2. Static pressure losses were determined as before in the existing system. The new static pressure required to size the fan was determined to be 3.03" water pressure. This static pressure is relatively lower compared to the existing system's static pressure requirements before fan losses of 4.14" water pressure. The maximum air velocity in the ducts is 1698ft/min. Pressure losses for the new duct sizes can be found in Appendix B.

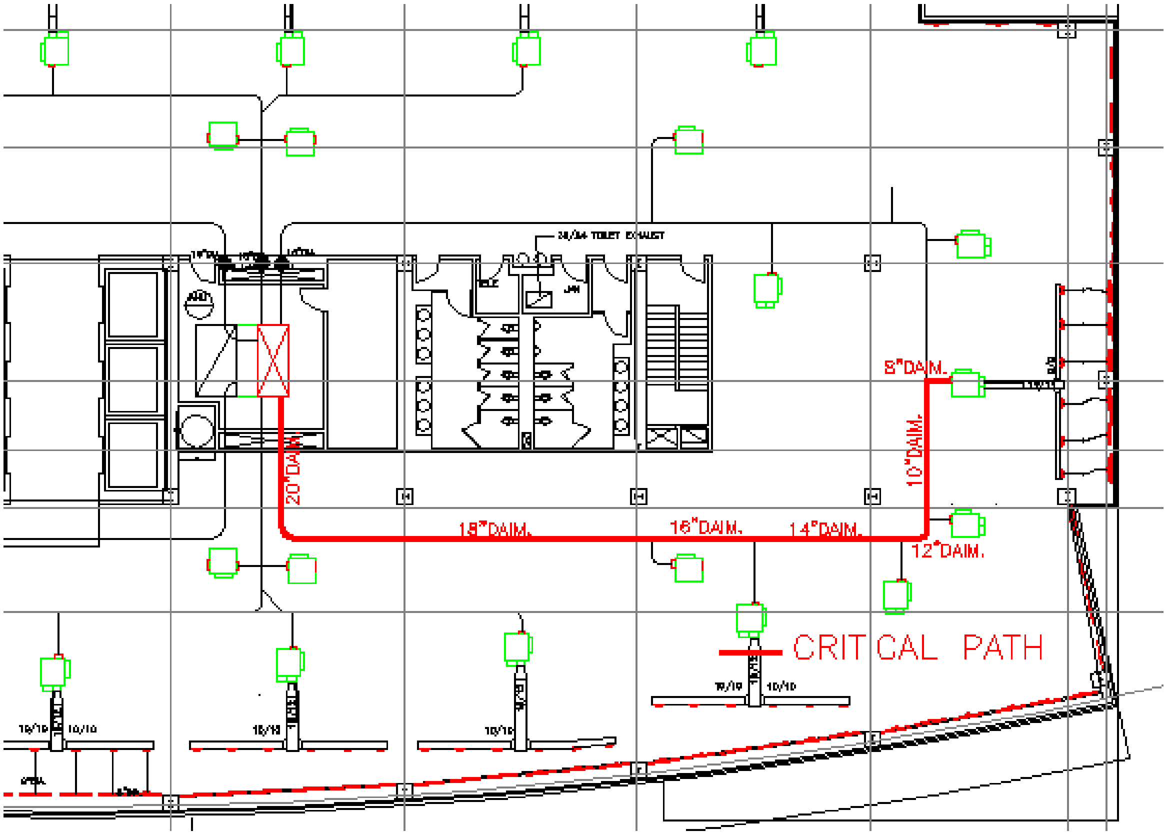


**Figure 8.2.1.** Proposed Duct Size Layout

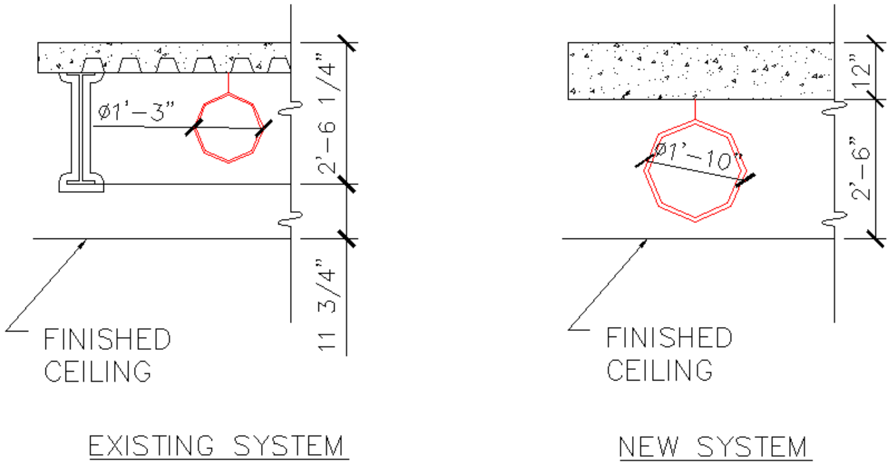


**Figure 8.2.2.** Comparison of existing (left) and proposed (right) utilization of plenum space

## 8.3 Fan Size

The fan size can be significantly reduced with a change in static water pressure before fan losses of 1.11" static water pressure. The existing fan at each typical floor level is as follows:

- 40", TRANE manufactured
- 20 Horsepower, 480V/3 Phase
- Force Flow Centrifugal Variable Frequency Drive, blow-thru
- Δ3.6" static water pressure required

To remain consistent with the manufacturer selected for the original design, TRANE fan products were researched. A new fan was selected according to the TRANE fan selection process. The fan selected is as follows:

- 40" TRANE manufactured
- 11.2 Horsepower, 480V/3Phase
- Type Q
- Δ2.83" static water pressure required (0.2" losses provided by the fan)

A reduction in horsepower between the fans from 20HP to 11.2HP results in life-cycle energy savings. By changing the type of fan, not only is horsepower reduced, but many other benefits are gained as well. The Type Q fan is suspension mounted compared to the blow-thru system that is floor mounted. By suspending the system, floor space becomes available for installation of pumps and other equipment and the size of the mechanical room can be reduced. Figure 8.3.1 below demonstrates the physical difference between the two fan types.

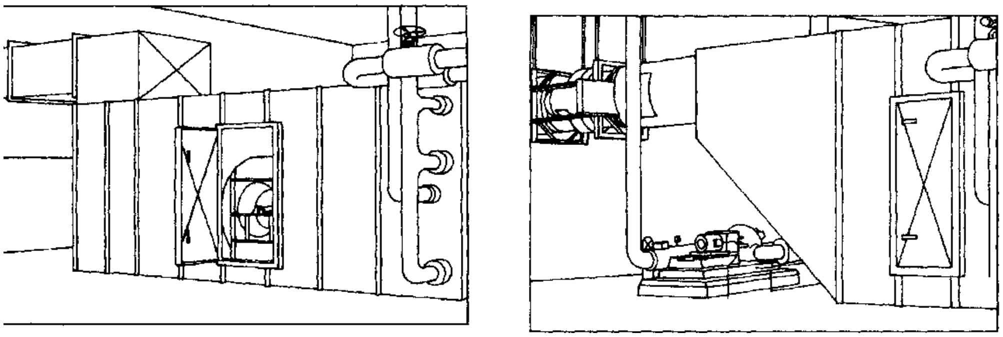


**Figure 8.3.1.** Typical blow-thru type fan (left) compared to Type Q fan (right). This photo is taken from www.trane.com.

Along with additional space for other equipment, the Trane Model Q has fewer components to install and has the advantage of a lesser weight. With a lighter weight, less manpower is required for rigging and setting the fan in place. The combined effect of lighter weight and fewer components results in direct dollar savings. To view fan selection data see Appendix B.

Overall, the increase in duct size creates a larger initial cost of the ducts. The cost will be offset by the acoustical value gained by the larger ducts, and the lower horsepower required to support the duct system.

# 9. COST AND SCHEDULE ANALYSIS

## 9.1 Cost

### 9.1.1 Existing Cost

An original cost estimate of the structure itself was not provided by the general contractor of Lockwood Place for this report. A cost estimation of this system was completed with the use of R.S. Means. The cost breakdown of the existing system can be found in Appendix C. Primary costs involved with the building are as follows:

1. Structural steel
2. Super-structure concrete
3. Spray on fireproofing
4. Additional punched hole detailing in structural steel

### 9.1.2 Proposed Cost

The proposed structural concrete system was estimated on a relative basis to the structural steel system. The primary costs that vary from the original system included in this estimation are as follows:

1. Super-structure concrete (including forms and placing)
2. Additional foundation concrete
3. Regular and post-tensioned reinforcement

Take-off and estimation tables of this system can be found in Appendix C. **The average total savings between the two systems is 16%.** When examining the savings it is important to consider that a post-tensioned building is less common than a steel building in the Baltimore area. Contractors local to the area, who are less familiar with post-tensioned construction, may add additional charges for construction.

## 9.2 Schedule

Schedules were isolated to the structural system. Only the structural system's timelines vary. All other aspects remain unchanged.

### 9.2.1 Existing Schedule

The existing steel construction schedule was not provided by the general contractor for this report. A schedule was created for the building based on construction start and finish dates, and size and geometry of the building. Total building construction began June 2003 and ended September 2004, for a total construction period of fifteen months.

Typical durations for activities listed in the schedule are equivalent to industry standards and calculated through RS Means crew daily outputs. Steel procurement time most likely took 20 weeks. The building was sequenced into three parts. Each segment includes three sequential floors and three typical bays. A diagram of the sequence zones can be found in Figure 9.2.1.1.

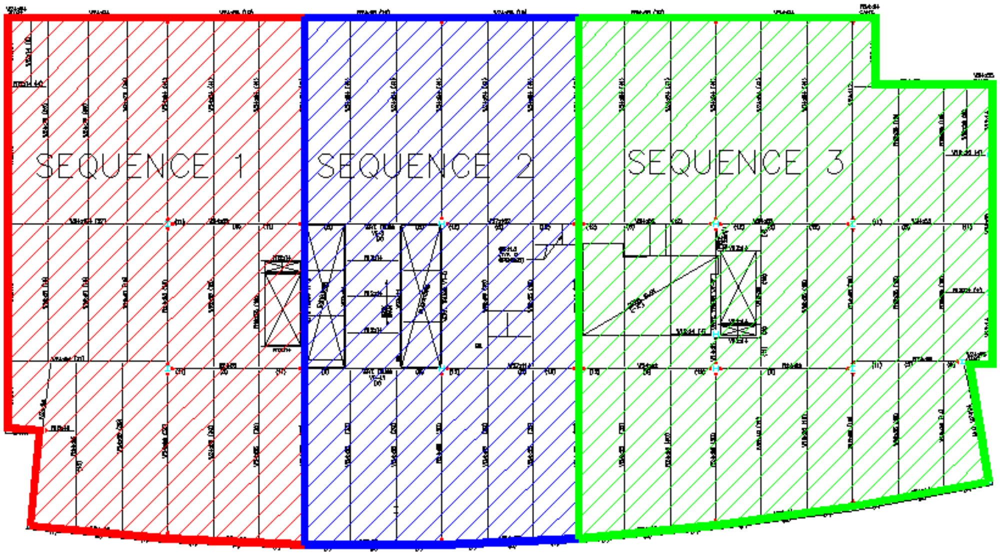


**Figure 9.2.1.1.** Steel Sequence Breakdown

Steel is erected first, followed by placement of the deck, shear studs, and welding. Finally the concrete is placed and cured. From procurement to placing and curing of the final slab, total structural construction time is estimated at 35 weeks. The breakdown is listed below:

1. Steel Erection……………………………………7days/per sequence
2. Deck, Shear Studs, Welding……………...............7days/per sequence
3. Slab Placement and Curing………………………..1day/per sequence
4. Shop Drawing/ Detailing…………………………..44days
5. Steel Procurement…………………………………14 weeks

The construction of the structure itself is estimated at 60% of the total construction time. The steel construction schedule can be found in Appendix C.

### 9.2.2 Proposed Schedule

A concrete construction timeline was developed with the same approach as the steel construction. Material quantities of the proposed concrete system were divided by crew daily outputs taken from RS Means. A total of 35 weeks steel construction time will be compared to the proposed concrete structural schedule. It is reasonable to include shop drawing/ detailing and procurement time in this estimation.

Activities involved in constructing each floor involve concrete formed, reinforced pour, placing and tensioning strands, curing and stripping. Tensioning the floor strands is estimated to occur 2-3 days after each concrete pour. Each typical floor is estimated to take three pours total. These activities are shown with their projected erection times below:

1. Concrete Formed, Reinforced Pour……….10days
2. Place and Tension Strands………………….4days
3. Cure Concrete and Strip Formwork……….5days
4. Shop Drawing/ Detailing………………….40days

The proposed schedule can be viewed in Appendix C. The concrete structural system takes a total of 40 weeks to construct. A comparison of the two can be found in Figure 9.2.2.1. The steel and concrete times are similar when procurement time is considered. Detailing and procurement time required of steel is lengthy, but physical construction time is relatively short. Concrete takes longer to construct, but less time to detail and no procurement time. Additional time required in the concrete system may be due to the large quantity of concrete and post-tensioned strands needed to accommodate large bay sizes. Despite five weeks additional construction time, concrete is determined to be a reasonable solution in terms of schedule.

| | Existing | Proposed |
|---|---|---|
| Start Date | 06/02/03 | 06/02/03 |
| Finish | 03/21/04 | 04/29/04 |
| Total Time | 35 weeks | 40 weeks |

**Figure 9.2.2.1** Total Schedule Comparison

# 10. ANALYSIS & CONCLUSION

The purpose of this report is to design and analyze an alternate structural system to allow more flexibility for the mechanical air distribution system. The current system provides punched holes in the structural steel of each typical floor system to permit room for the mechanical duct system. These holes can be costly and allow for only one specific size and placement of the air ducts. The proposed system was a 12" two-way flat slab, post-tensioned floor system. Overall, Lockwood Place as a post-tensioned concrete structure is a success.

*Structural Redesign*

The 12" flat slab structural floor system provides an 18.25" open plenum space for mechanical air duct systems. This system was reported to have a 1.40" maximum long-term deflection and balance and an average dead load of 60-70%.

Five shear walls replaced moment frames and eccentric braces. The shear wall locations replace each location of eccentrically braced frames. Despite added building weight due to the large amount of concrete, wind remained the controlling lateral force. A maximum lateral deflection was analyzed to be 1.78" with a deflection less than the building expansion joint at the third level. A coupling beam was designed for the east/west walls to allow for openings to the lobby elevators.

Additional building weight caused an increase in caisson sizes at the foundation. Although the size of the caissons increased due to gravity, building uplift was completely eliminated.

*Mechanical Retrofit*

With increased plenum availability, air duct sizes were increased. The increase in air duct size reduced static pressure supply for the fan. A reduced air velocity in the ducts from 2275ft/min to 1698ft/min due to the larger sizes improves acoustical value. The added cost of larger size air ducts is assumed to be offset by increased acoustical value and less energy required by the fan.

The fan at each typical floor level was resized for the reduction in static pressure required. A new TRANE Type Q model was selected. This model has lower installation costs and is suspended from the ceiling, allowing more space for piping and equipment.

*Cost and Schedule Analysis*

A change from a structural steel system to a structural concrete system left way for a cost and schedule analysis. Cost and schedule were developed for both the existing and proposed systems. Although the proposed system provided a 16% cost reduction, construction time extended five weeks beyond the existing system. The proposed solution was determined to be viable.

# APPENDIX A

# STRUCTURAL CALCULATIONS

## 11.1 Post-Tensioned Floor Design

Hand Calculations:

Typical Floor Layout Hand Check Calcs

Follows PCA Desigin Aid

- Loads:

Framing Dead Load = selfweight
Superimposed DL = 10psf ! partitions/MEP
Live Load = 100psf
2hr fire rating

- Materials:

Concrete - normal weight 150pcf

$f'_c = 5000 psi$

$f'_{ci} = 3000 psi$

Rebar: $f_y = 60,000 psi$

PT: Unbonded Tendons

1/2 ϕ, 7-wire strand, $A = 0.153$

$f_{pu} = 270 Ksi$

Estimated Prestress Losses = 15Ksi

$f_{se} = 0.7(270) - 15 = 174 ksi$

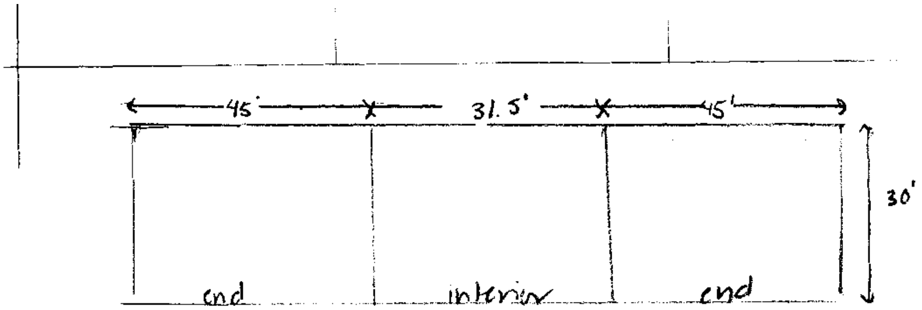
$P_{eff} = A \cdot f_{se} = 0.153(174) = 26.6$ kip/tendon

- Determine preliminary slab thickness:

$h = (\frac{30+45}{2})(12)/45 = 10''$

- Loading:

DL = 10(150) = 125 psf
SIDL = 10psf
LL = 100psf
→ LL Reductions
$A_T = 30(45) = 1350$
$K_{LL} = 1$
LL = 0.83(100) = 83psf



North-South Interior Frame

- Equivalent Frame Method ACI 13.7
- Total bay width = 30'
- Ignore column stiffness in equations for simplicity of hand calculations
- No pattern Loading required $\frac{LL}{DL} < 3/4$ (ACI 13.7.6)

  $83/135 = 0.61 < 0.75$

- Calculate Section Properties:

  $A = bh = 30(12)(10) = 3600 \text{ in}^2$ 4320

  $S = 30(12)(10)^2/6 = 6000 \text{ in}^3$

- Set Design Parameters

Allowable Stress: Class U

- At time of jacking
  - $f'_{ci} = 3000$ psi
  - compression $= 0.6 f'_{ci} = 0.6(3000) = 1800$ psi
  - tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi
- At service loads
  - $f'_{ci} = 5000$ psi
  - compression $= 0.6 f'_c = 0.6(5000) = 2250$ psi
  - tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

Average precompression limits

$P/A = 125$ psi min

300 psi max

Target Load balances

60% - 80% of DL for slabs

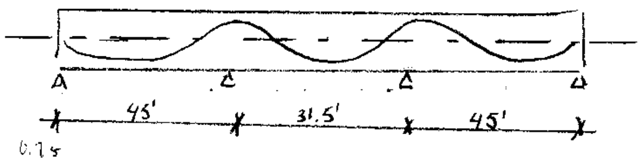
use 75% = 0.75(125) = 93.75 psf

- Cover Requirements (2hr fire rating, carbonate aggregates)

Restrained slabs: 3/4" bottom
Unrestrained slabs: 1 1/2" bottom
3/4" top

- Tendon profile:

| | | |
|---|---|---|
| exterior support | 5" | *measure from bottom of slab |
| Interior support - top | 8.75" | |
| interior span - bottom | 1.25" | |
| end span - bottom | 1.25" | |



$a_{INT} = 8.75 - 1.25 = 7.5"$

$a_{ext} = (8.75 + 1.25)/2 - 1.25 = 3.75"$

- Prestress force required to balance 75% of DL
⇒ end span will govern b/c longer span & smaller drape

$w_b = 0.75 w_L = 0.75(125)(30) = 2.813 \text{ k/ft}$

- force in tendons to counteract the load in end of bay:

$P = w_b l^2 / 8 a_{end} = \dfrac{2.813(45)^2}{8(3.75/12)} = 2278.53^k$

- Check compression allowance

$\# \text{tendons} = 2278.53 / 26.6^k/\text{tendon} = 85.66 \approx 85 \text{ tendons}$

$P_{actual} = 85(26.6) = 2261^k$

$w_p = 2261/2279(2.813) = 2.791 \text{ k/ft}$

$P_{act}/A = 2261(1000)/3600 = 628 \text{ psi} > 125$

412 psi $< 300$

NO GOOD

- If banded in East/West Direction

$A = 45(12)(10) = 5400$

$S = 9000$

$w_p = 0.75(125)(45) = 4.219$

$P = \frac{4.219(30)^2}{8(3.75/12)} = 1518.84^k$ (end) $\quad P_{INT} = \frac{4.219(30)^2}{8(7.5/12)} = 759.42$

#tendons = 1518.84/26.6 = (57 tendons) (28 tendons)

$P_{act} = 57(26.6) = 1516.2^k \quad P_{act} = 744.8^k$

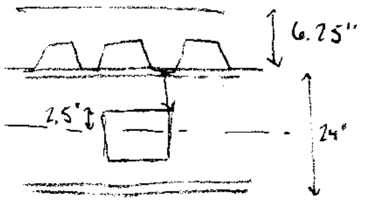
$w_b = 1516.2/1519\,(4.219) = 4.211 \quad w_b = 4.138\ ^k/ft$

$P_{act}/A = 1516(1000)/5400 = 280.8$ psi $\quad >125$

$<300$

138 psi OK

- Try 12" slab



6.25 + 9.5 = 15.75 > 12"

∴ still achieve goal

$13 \times 13 \Rightarrow 13/2 = 6.5$

$12 - 2.5 = 9.5"$

$A = 30(12)(12) =$

$S = 30(12)(12)^2/6 =$

$w_p = (0.75)(125)(30) = 2\,813\ ^k/ft$

Tendon Profile:

| | |
|---|---|
| ext support | 6" |
| interior support top | 16.75' |
| interior span - bot | 1.25' |
| end span - bot | 1.25" |

$a_{INT} = 9.5"$

$a_{ext} = 4.75"$

$P = \frac{2.813(45)^2}{8(4.75/12)} = 1798.84^k$

#tendons: 1798.84 /26.6 = 67.6 ≐ 67 tendons

$P_{act}$ = 67(26.6) = 1782.2

$w_p$ = (1782.2/1798.84)(2.813) = 2.787 k/ft

$P_{act}/A$ = $\frac{1782.2(1000)}{4320}$ = 412.5 >125 <300 <u>NO GOOD</u>

∴ Band Tendons East-West Direction
make column adjustment

Try 1: 57 tendons end, 28 tendons interior

Try 2: 55 tendons, different tendon profile

Try tendon profile:

end support = 5"
interior support = 8.75"
interior span = 3"
end bottom = 1"

$a_{INT}$ = 5.75"
$a_{ext}$ = 3.875"

A = 5400
S = 9000
$w_b$ = 4.219 k/ft

$P = \frac{4.219(30)^2}{8(3.875/12)} = 1469.8$

# tendons = 1469.8 / 26.6 = 55

$P_{act}$ = 1463 k $\qquad w_b$ = 4.110 k/ft

$P_{act}/A = \frac{1463(1000)}{5400} = 271$ > 125, < 300 ∴ <u>ok to proceed</u>

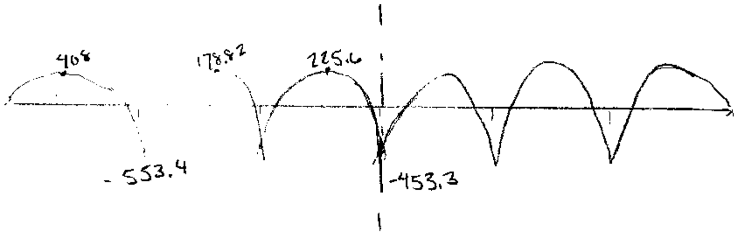
- Check amount to be balanced:

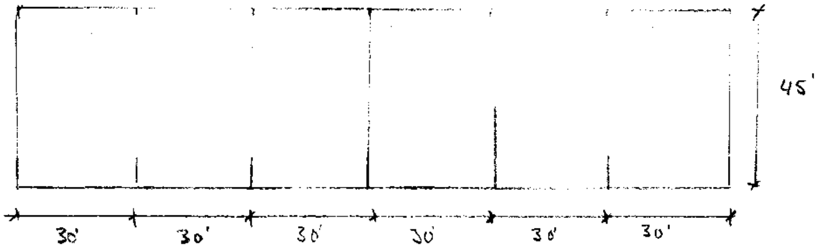
$w_p = 1463(8)(5.75/12)/(30)^2 = 6.231$ k/ft

$w_{DL} = 125(45)/(1000) = 5.625$ → 1.11 > 100% ∴ [illegible]

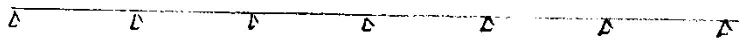
$P_{eff} = 1463^k$

$w_b = \frac{4(5.625) + 2(6.231)}{6} = 5.827$ k/ft





East-West Direction



- Check amount of load to be balanced

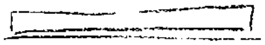
$w_b = 1516(8)(7.5/12)/(30)^2 = 8.42^{k}/ft \rightarrow 4.138^{k}/ft$

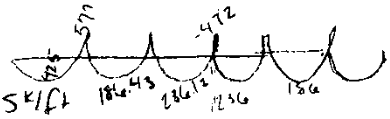
$w_{DL} = 125(45)/1000) = 5.625 \qquad 73.6\%$

$P_{eff} = 1516^{k}, 745^{k}$

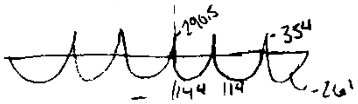
- Check slab stresses

→ $w_{DL} = (135)(45)/1000 = 6.075^{k}/ft$

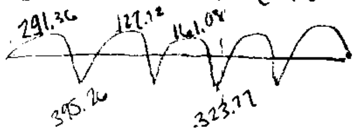




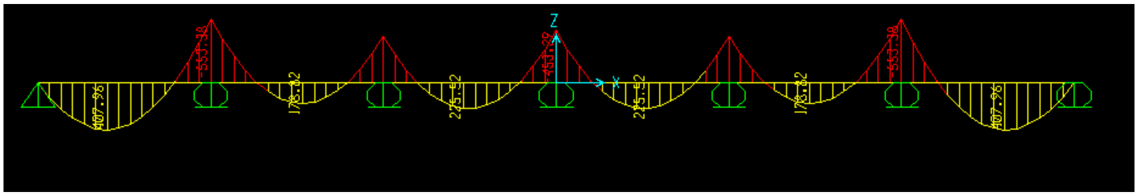
→ $w_{LL} = 83(45)/1000 = 3.735^{k}/ft$



→ Total balancing moments
$w_b = 4.16^{2}$ ft (average of le bays)



SAP Secondary Moments:



## Stage 1: Stresses Immediately after jacking

$f_{top} = (-M_{DL} + M_{bal}) / S - P/A$
$f_{bot} = (M_{DL} - M_{bal}) - P/A$

- Interior Span

$f_{top}$ $(-186.4 + 127.12)(12)(1000)/9000 - 138$ psi $= -217^{c} < 1800$
$(-236.12 + 161.08)(12)(1000)/9000 - 138$ psi $= -159.36^{c} < 1800$ OK

$f_{bot} = (186.4 - 127)(12)(1000)/9000 - 138 = -58.8^{c} < 1800$
$(236.1 - 161.1)(12)(1000)/9000 - 138 = -38 < 1800$ OK

- End Span

$f_{top}$ $(-425 + 291.36)(12)(1000)/9000 - 280.8 =$ c
$-317.32^{c} < 1800$ psi OK

$f_{bot}$ $(425 - 291.36)(12)(1000)/9000) - 280.8$ c
$-102.6^{c} < 1800$ OK

- Support Stresses

$f_{top} = (577 - 395.26)(12)(1000)/9000) - 280.8 = -37.48^{c} < 1800$
$(472 - 323.8)(12)(1000)/9000) - 138 = 59.6^{T} < 164$ OK

c $f_{bot} = (-575 + 395)(12)(1000)/9000 - 280.8 = -520.8 < 1800$
$(-472 + 324)(12)(1000)/9000 - 138 = -335.3$ OK

## Stage 2: Stresses @ service Load

- Interior Span

c $f_{top} = (-186.4 - 114 + 127)(12)(1000)/9000 - 138 = -369.2^{c} < 2250$
$(-236.12 - 144 + 161.1)(12)(1000)/(9000) - 138 = -431$ OK

T $f_{bot} = (186.4 + 114 - 127)(12)(1000)/9000 - 138 = 93.2^{T} < 424$ OK
$(+236.12 + 144 - 161.1)(12)(1000)/9000 - 138 = 114.03^{T}$

- End Span

c $f_{top} = (-425 - 261 + 291)(12)(1000)/9000 - 280.8 = -807 < 2250$

T $f_{bot} = (425 + 261 - 291)(12)(1000)/9000 - 280.8 = 245.9 < 424$

OK

6

Try 2:

Stage 1

- Interior Span

$f_{top} = (-186.4 + 178.8)(12)(1000)/9000 - 271 = -281.1$ $< 1800$
$(-236.12 + 226)(12)(1000)/9000 - 271 = -284.5$

$f_{bot} = (+186.4 - 179)(12)(1000)/9000 - 271 = -261.1$ $< 1800$
$(+236.12 - 226)(12)(1000)/9000 - 271 = -257.5$

- End Span

$f_{top} = (-425 + 408)(12)(1000)/9000 - 271 = -294$

$< 1800$

$f_{bot} = (+425 - 408)(12)(1000)/9000 - 271 = -248$

- Support Stresses

$f_{top} = (577 - 553)(12)(1000)/9000 - 271 = -239$
$(472 - 453.3)(12)(1000)/9000 - 271 = -246.1$

$< 1800$

$f_{bot} = (-577 + 553)(12)(1000)/9000 - 271 = -303$
$(-472 + 453.3)(12)(1000)/9000 - 271 = -296$

Stage 2

- Interior Span

$f_{top} = (-186.4 - 114 + 179)(12)(1000)/9000 - 271 = -433$ $< 2250$
$(-236.12 - 144 + 226)(12)(1000)/9000 - 271 = -476.5$

$f_{bot} = (186.4 + 114 - 179)(12)(1000)/9000 - 271 = -109.1$ $< 2250$
$(236.12 + 144 - 226)(12)(1000)/9000 - 271 = -65.5$

- End Span

$f_{top} = (-425 - 261 + 408)(12)(1000)/9000 - 271 = -641.7$ $< 2250$

$f_{bot} = (425 + 261 - 408)(12)(1000)/9000 - 271 = 99.7$ $< 424$

- Support Stresses

$f_{top} = (577 + 354 - 553)(12)(1000)/9000 - 271 = 206.3$ $< 424$
$(472 + 290.5 - 453.3)(12)(1000)/9000 - 271 = -110.9$ $< 2250$

$f_{bot} = (-577 - 354 + 553)(12)(1000)/9000 - 271 = -775$ $< 2250$
$(-472 - 290.5 + 453.3)(12)(1000)/9000 - 271 = -683$

$\therefore$ all stresses are $\underline{\underline{OK}}$

## Support Stresses

T $f_{top} = (577+354-395.26)(12)(1000)/9000 - 280.8 = 431 > 424$
$(472+290.5-324)(12)(1000)/9000 - 138 = 447$

Not ok

C $f_{bot} = (-577-354+395.26)(12)/9 - 280.8 = -302 < 2250$
$(-472-290.5+324)(12)/9 - 138 = -723 < 2250$

ok

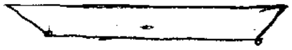
## Ultimate Stress

Try 1: $M_1 = P \cdot e$

End:
$e = 0$ @ exterior
$e = 3.75$ @ interior support

$M_1 = 1516(3.75)/12) = 473.75$ ft-k

$M_{sec} = M_{bal} - M_1$



$395.26 -$

$M_u = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$

midspan: $1.2(425) + 1.6(261) + 1.0(81.23) = 1008.83^{'k}$

support $= 1.2(-577) + 1.6(-354) + 1.0(162.46) = -1096.34$

Interior:
$e = 3.75$
$M_1 = 745(3.75/12) = 232.8$
$M_{sec} = 161 - 232.8$

Try 2:
End:
$1463(3.875/12) = 472.4$
$M_{sec} = 553.4 - 472.4 = 81^{'k}$
$M_u = 1.2(425) + 1.6(261) + 1.0(40.5) = 968.1^{'k}$
$1.2(-578) + 1.6(-354) + 1.0(81) = 1178^{'k}$

Engineer's Computation Pad

· Moment Capacity Check

Try 2:
End:

$$\phi M_n = 0.9(A)(f_{ps})(d - a/2)/12$$

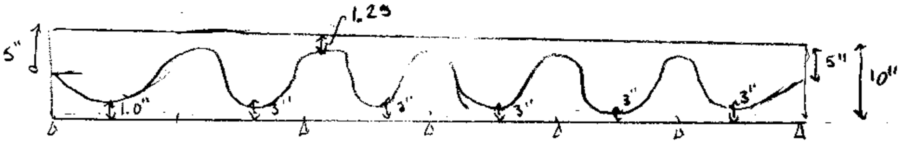
$$0.9(.153)(55)(251024)(8.75 - .942/2)/12 = \underline{1343}^{\prime K}$$

$$a = \frac{(0.153)(55)(251024)}{\underset{f'_c}{.85(5000)}\underset{b}{(45)(12)}} = 0.942 \qquad > 968.1^{\prime K}$$

$$f_{ps} = 270,000\left(1 - \frac{0.4}{0.8} \cdot \frac{.00178(270,000)}{5000}\right) = 257024$$

$$\rho_p = \frac{0.153(55)}{45(12)(8.75)} = 0.00178$$

Design:



55 tendons

* Complete Design in RAMconcept
These calculations are for trial only

Code Requirements:

$V_c = \left( 0.6\sqrt{f'_c} + 700 \frac{V_u d_p}{M_u} \right) b_w d$ (11.4)

(18.9) Min bonded Reinforcement

$A_s = 0.004 A_{ct}$

$A_s = \frac{N_c}{0.5 f_y}$ } positive 1/3 ln centered

$A_s = 0.00075 A_{cf}$ } negative spacing <12" within 1.5h of column faces
-extend 1/6 ln beyond support @ all faces

(18.8) $\phi M_n > 1.2 M_{cr}$

- bends = ≤ 1/12 verified
- state deflections (+) & (-)
- make completed design page:
  - prestressed layout
  - bonded reinforcement layout
  - column capital section
  - code required stresses
  - stresses in slab actual
  - shear forces -> allowed vs. actual
  - moments
  - effective prestressed forces in tendons
  - balanced load - % ?
  - anchorage zone details
  - overlay mechanical drawing

## Floor Shear Design

Punching shear: interior

-assume column cap = 15" $d = 12.5"$

$b_o = 4(32 + 12.5) = 178"$

$V_c = 4\sqrt{5000}(178)(12.5)/1000 = 629.32^k$

$\phi V_c = (629.32^k)(.75) = 472^k$

$V_u = \left[(38.25)(30) - \frac{(44.5)(44.5)}{144}\right](.358) = 406.2^k$

338K vs 519K @ column cap edge

$w_u = ((12/12)(150) + 15)1.2 + 1.6(100) = 358 psf$

OK

Punching Shear: Exterior

$b_o = 2(28 + 12.5/2) + (28 + 12.5) = 109"$

$V_c = 4\sqrt{5000}(12.5)(109) = 385.4$

$\phi V_c = 289^k$

$V_u = \left[(45/2)(30) - \frac{34.25 + 40.5}{144}\right](.358) = 238.2^k$

OK

- For transfer of unbalanced moment to shear resistance check

$0.75\phi V_c > V_n$

-if column cap = 18" $d = 15.5$

interior: $(\phi V_c)0.75 = 468^k$
$V_u = 408^k$

exterior: $(\phi V_c)0.75 = 283.6^k$
$V_u = 237.8^k$

- assume unbalanced moment resisted by flexural capacity

-column capital diameter = 10% to 25% average span

$\frac{30+45}{2}$ = 7.5'

(11.4) Shear Check provided by prestressed members

$$V_c = \left(0.6\sqrt{f'_c} + 700\,\frac{V_u d_p}{M_u}\right) b_w d$$

$d = 0.8(h) = 9.6"$

$12-1.25 = 10.75"$

$$= \left[0.6\sqrt{5000} + 700\,\frac{(\quad)(\quad)}{(\quad)}\right](\quad)(10.75)$$

Determine of Final Design Prestress Forces:

1 $w_p = 0.75(150)(45/2) = 2.53^{k}/ft$

2 $w_p = 0.75(150)(38.25) = 4.30^{k}/ft$

3 $w_p = 0.75(150)(34.75) = 3.91^{k}/ft$

4 $w_p = 0.75(150)(38/2) = 2.14^{k}/ft$

goal

$P = w_p (L^2 / 8a$

2 $\frac{2.75(35)^2}{8(7.125/12)} = 709^k$ (actual)

1 $\frac{1.98(30)^2}{8(7.125/12)} = 375^k$

3 $\frac{2.62(35)^2}{8(7.125/12)} = 676^k$

4 $\frac{1.73(30)^2}{8(7.125/12)} = 328^k$

For design

1 $14(26.6) = 372^k$

2 $26(26.6) = 692^k$

3 $24(26.6) = 639^k$

4 $12(26.6) = 320^k$

Longitudinal: $9(4) = 36$
$8(4) = 32$

$P = 36(26.6) = 957.6 / 30' = 32^{k}/ft$

$P = 32(26.6) = 851.2 / 30 = 28.4^{k}/ft$

## 11.2 Column Design

Load Summary:

| | Total Factored Load (k) |
|---|---|
| Interior Column | |
| B-3 | 3600 |
| E-3 | 3550 |
| E-4.1 | 3400 |
| Exterior Column | |
| E-5 | 1960 |
| F-5 | 1965 |
| A-3 | 1920 |

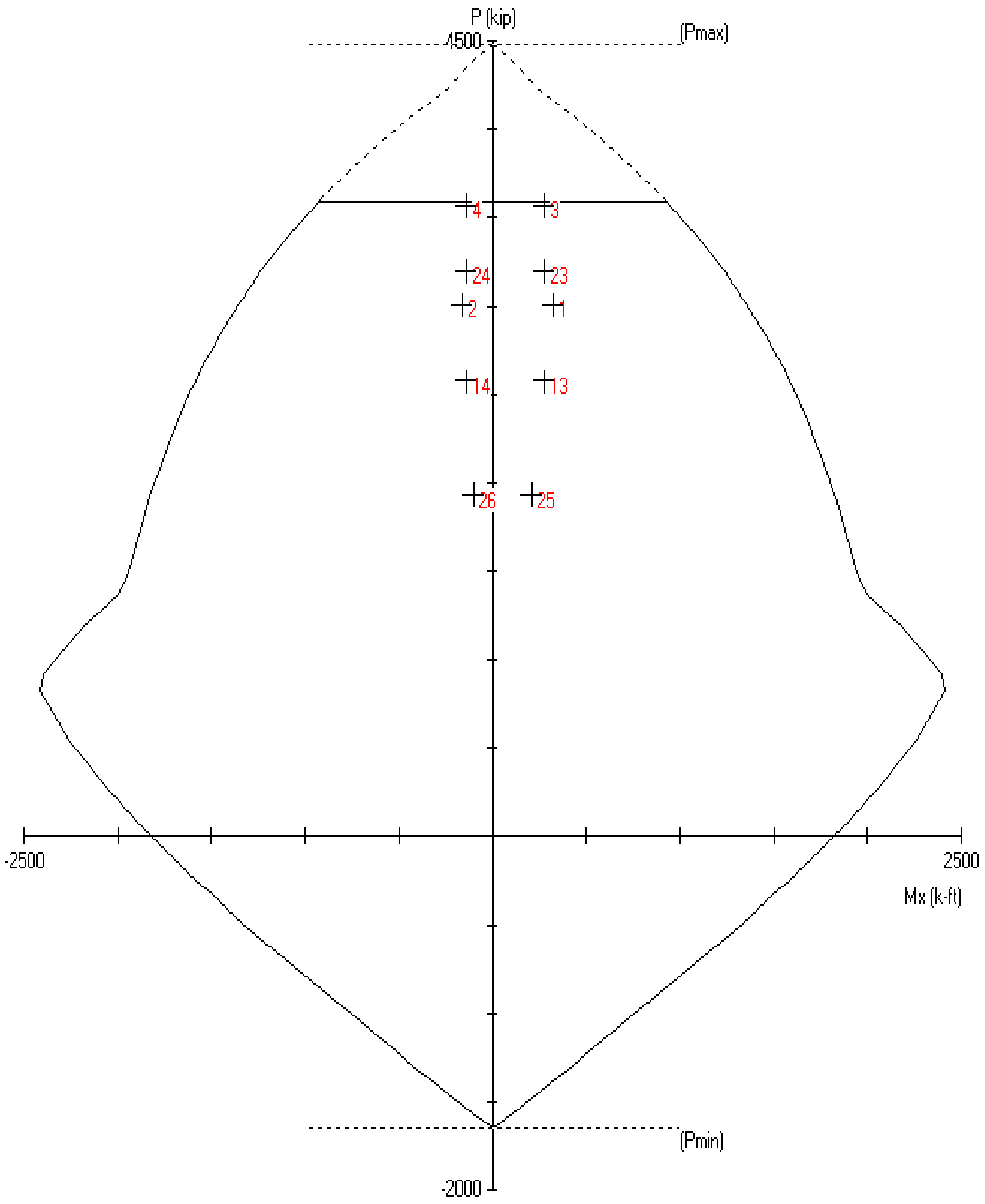
*omitted columns around large openings
and significantly smaller tributary areas.
*controlling gravity load combination: 1.2D + 1.6L

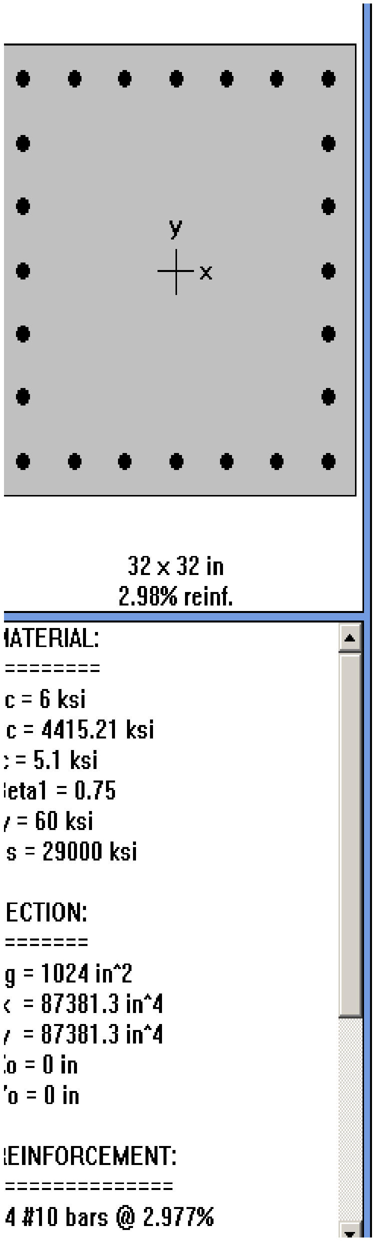
Please request to view column load breakdown spreadsheets.

Moment Distribution- Determination of moment in columns:

| Joint | a | | | b | | | | c | | | | d | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| member | ah | ao | ab | ba | bi | bp | bc | cb | cg | cq | cd | dc | dk | dr | dc |
| FEM | 0.127 | 0.143 | 0.73 | 0.422 | 0.073 | 0.083 | 0.422 | 0.422 | 0.073 | 0.083 | 0.422 | 0.422 | 0.073 | 0.083 | 0.422 |
| DF | 0 | 0 | -322 | 322 | 0 | 0 | -322 | 322 | 0 | 0 | -322 | 322 | 0 | 0 | -322.00 |
| D1 | 40.89 | 46.05 | 235.06 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| C1 | 0.00 | 0.00 | 0.00 | 117.53 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| D2 | 0.00 | 0.00 | 0.00 | -49.60 | -8.58 | -9.75 | -49.60 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| C2 | 0.00 | 0.00 | -24.80 | 0.00 | 0.00 | 0.00 | 0.00 | -24.80 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| D3 | 3.15 | 3.55 | 18.10 | 0.00 | 0.00 | 0.00 | 0.00 | 10.47 | 1.81 | 2.06 | 10.47 | 0.00 | 0.00 | 0.00 | 0.00 |
| C3 | 0.00 | 0.00 | 0.00 | 9.05 | 0.00 | 0.00 | 5.23 | 0.00 | 0.00 | 0.00 | 0.00 | 5.23 | 0.00 | 0.00 | 0.00 |
| D4 | 0.00 | 0.00 | 0.00 | -6.03 | -1.04 | -1.19 | -6.03 | 0.00 | 0.00 | 0.00 | 0.00 | -2.21 | -0.38 | -0.43 | -2.21 |
| C4 | 0.00 | 0.00 | -3.01 | 0.00 | 0.00 | 0.00 | 0.00 | -3.01 | 0.00 | 0.00 | -1.10 | 0.00 | 0.00 | 0.00 | 0.00 |
| D5 | 0.38 | 0.43 | 2.20 | 0.00 | 0.00 | 0.00 | 0.00 | 1.74 | 0.30 | 0.34 | 1.74 | 0.00 | 0.00 | 0.00 | 0.00 |
| Moments (K-in) per unit strip | 44.43 | 50.17 | | | -9.62 | -10.94 | | | 2.11 | 2.40 | | | -0.38 | -0.43 | |
| Mom (k-in) | | 48 | | | 10 | | | | 2.3 | | | | 0 | | |
| Mom (K-ft) | 153 | | | | 31.88 | | | | 7.33 | | | | 0 | | |

PCA Interior Column (Leve1 1- Level 8):





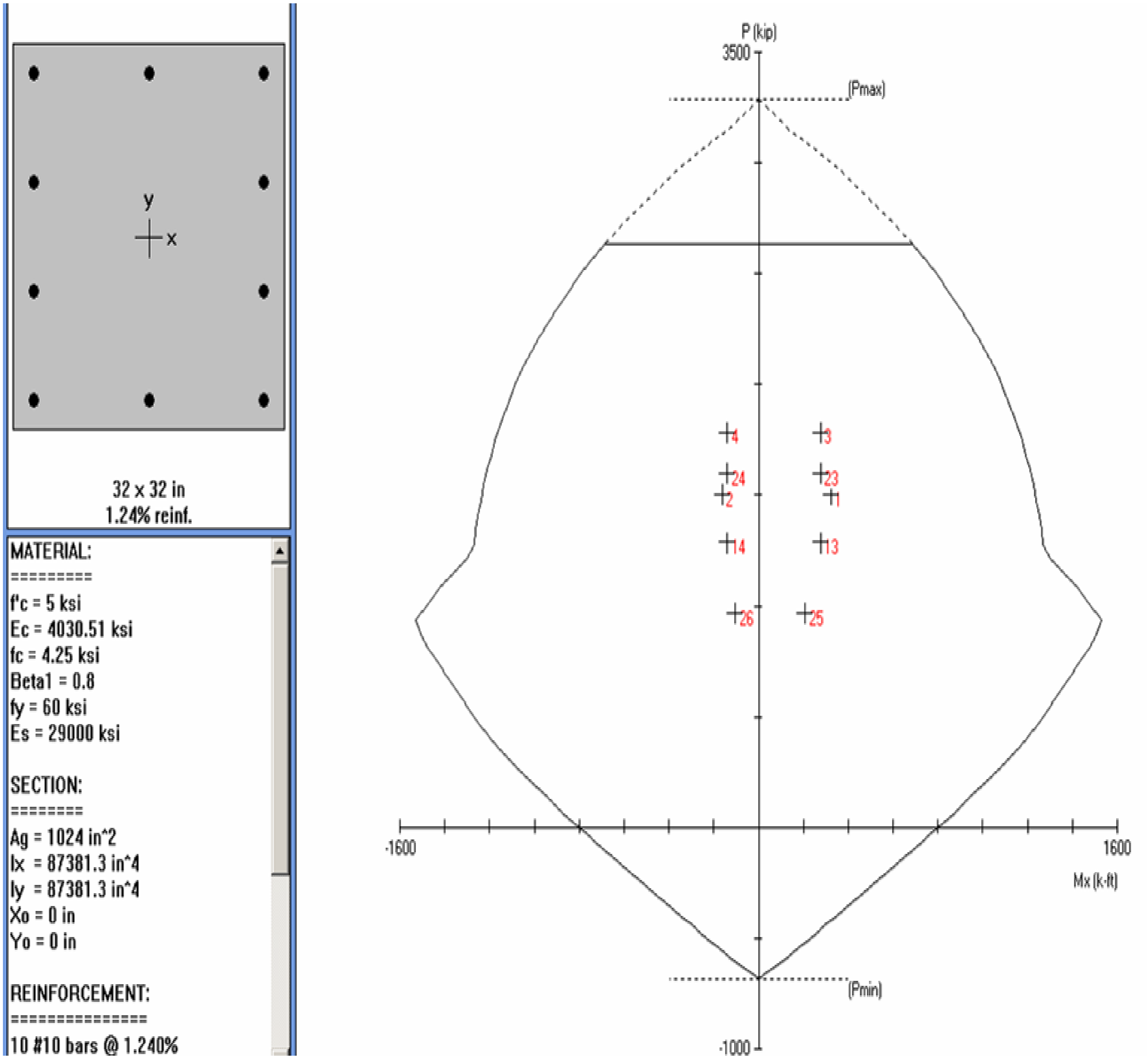
32 x 32 in
2.98% reinf.

```
ATERIAL:
========
c = 6 ksi
c = 4415.21 ksi
 = 5.1 ksi
eta1 = 0.75
 = 60 ksi
s = 29000 ksi

ECTION:
=======
g = 1024 in^2
  = 87381.3 in^4
  = 87381.3 in^4
o = 0 in
o = 0 in

EINFORCEMENT:
==============
4 #10 bars @ 2.977%
```

PCA Column Interior Column (Level 9-Roof):



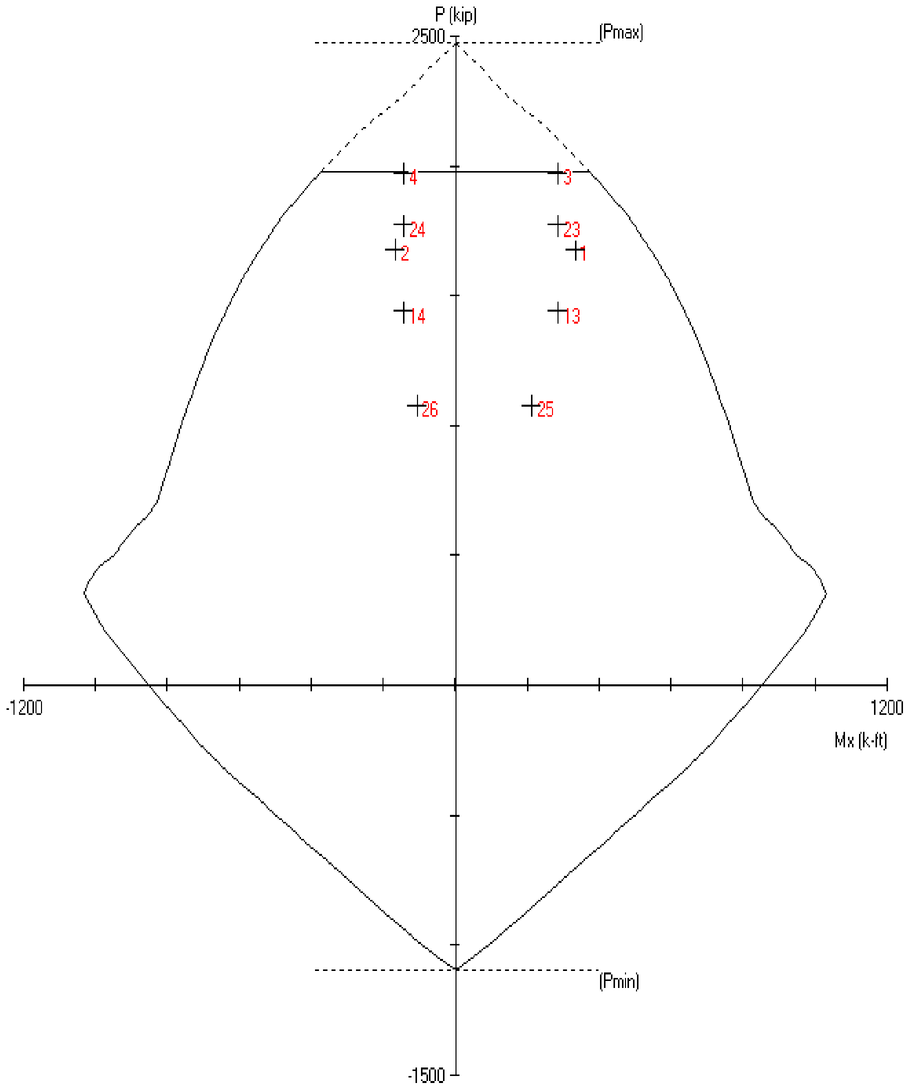
32 x 32 in
1.24% reinf.

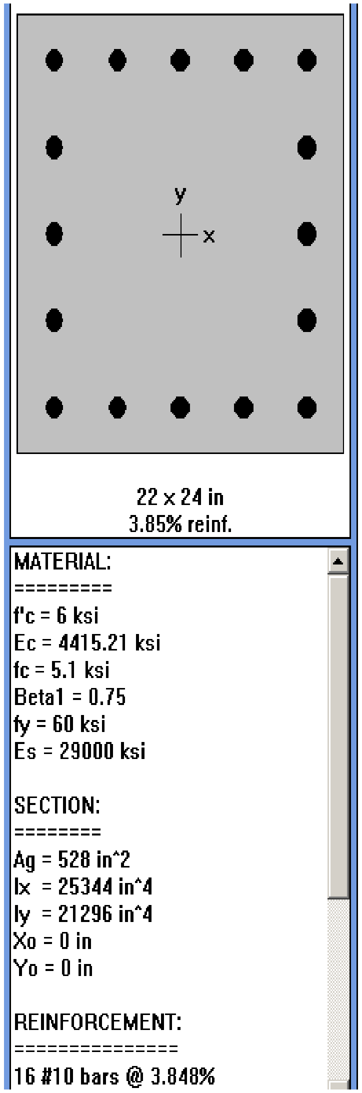
MATERIAL:
=========
f'c = 5 ksi
Ec = 4030.51 ksi
fc = 4.25 ksi
Beta1 = 0.8
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 1024 in^2
Ix = 87381.3 in^4
Iy = 87381.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
10 #10 bars @ 1.240%

PCA Column Exterior Column (Level 1-8):





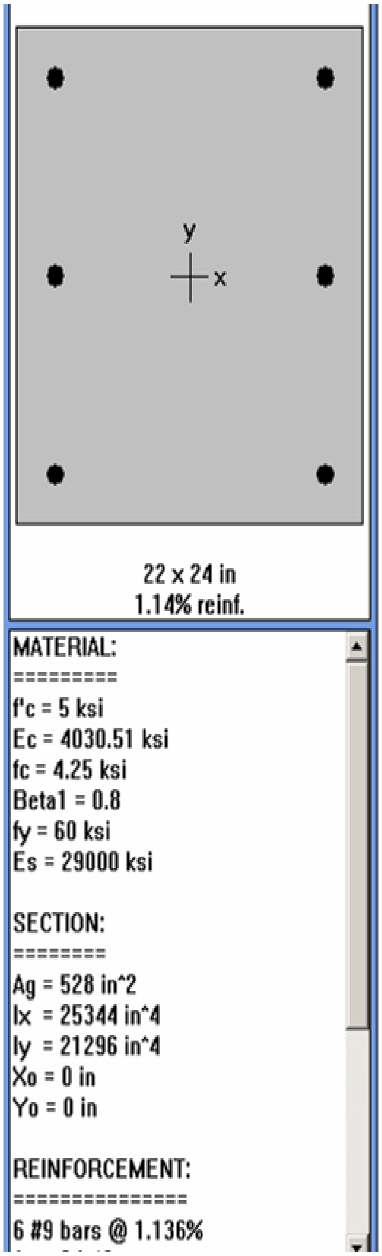
22 x 24 in
3.85% reinf.

MATERIAL:
=========
f'c = 6 ksi
Ec = 4415.21 ksi
fc = 5.1 ksi
Beta1 = 0.75
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 528 in^2
Ix = 25344 in^4
Iy = 21296 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
16 #10 bars @ 3.848%

PCA Exterior Column (Level 9-Roof):

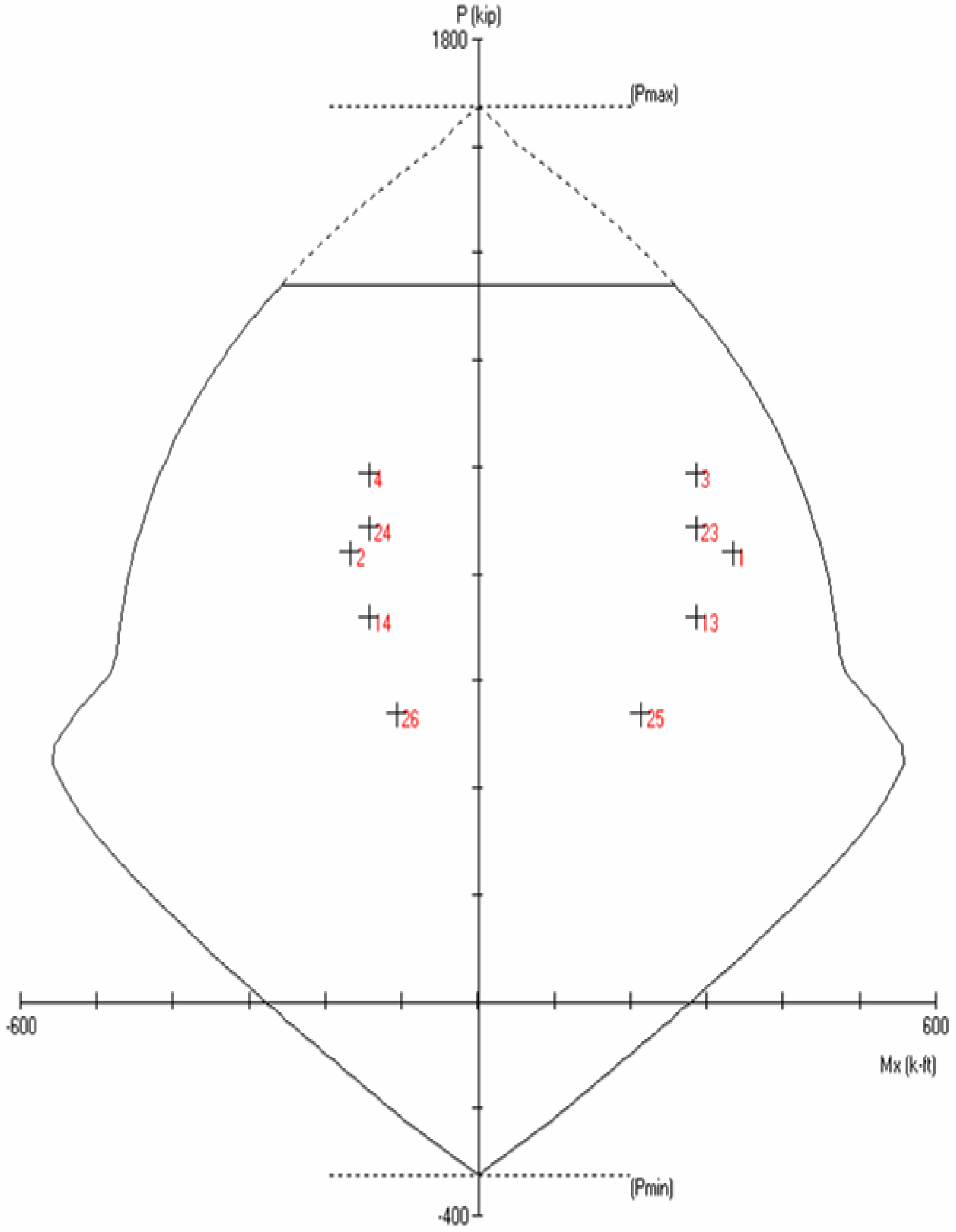


22 x 24 in
1.14% reinf.

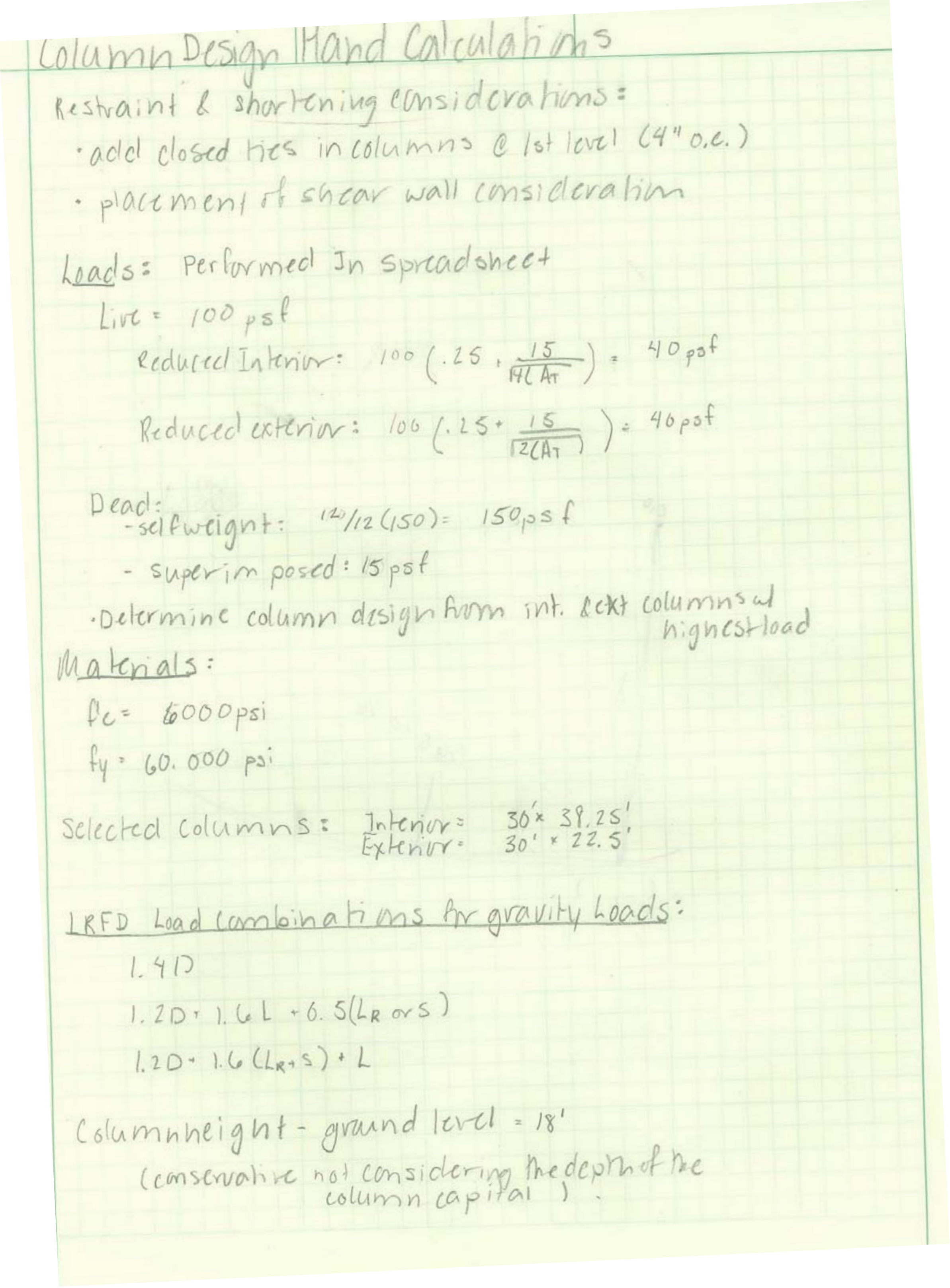
MATERIAL:
=========
f'c = 5 ksi
Ec = 4030.51 ksi
fc = 4.25 ksi
Beta1 = 0.8
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 528 in^2
Ix = 25344 in^4
Iy = 21296 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
6 #9 bars @ 1.136%



Hand Calculations:



# Column Design Hand Calculations

Restraint & shortening considerations:

- add closed ties in columns @ 1st level (4" o.c.)
- placement of shear wall consideration

Loads: Performed In spreadsheet

Live = 100 psf

Reduced Interior: $100\left(.25 + \frac{15}{\sqrt{4(A_T}}\right) = 40$ psf

Reduced exterior: $100\left(.25 + \frac{15}{\sqrt{2(A_T)}}\right) = 40$ psf

Dead:

- selfweight: $^{(12)}/12(150) = 150$ psf
- superimposed: 15 psf

- Determine column design from int. & ext columns w/ highest load

Materials:

$f'c = 6000$ psi

$f_y = 60.000$ psi

Selected columns: Interior = 30' × 38.25'
Exterior: 30' × 22.5'

LRFD Load combinations for gravity loads:

1.4D

$1.2D + 1.6L + 0.5(L_R \text{ or } S)$

$1.2D + 1.6(L_R + S) + L$

Columnheight - ground level = 18'
(conservative not considering the depth of the column capital).

ACI 318-05 Code Limitations

- check second order analysis
  w/ material nonlinearity and cracking
  in ETABS model

  cracking using $0.5 - 0.7\ I_g$

- unsupported column length

$$\frac{K\ell}{r} = \frac{1(18)(12)}{.3(32'')} = \begin{matrix} 22.5^{(int)} \\ 27.7\ (ext) \end{matrix} \leq 34 - 12(.5) = 28$$

↳ rectangular

*ignore slenderness effects

K = 1.0 assume nonsway column
shearwalls to resist 100% of lateral load
only moment from floor system

Check: $Q = \frac{\Sigma P_u \Delta_o}{V_{us}\ \ell_c} \leq 0.5$

Check: $M_c = \delta_{ns} M_2$

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{0.75 P_c}} \qquad P_c = \frac{\pi^2 EI}{(K\ell_u)^2}$$

$$EI = \frac{0.2 E_c I_g + E_s I_{se}}{1 + \beta d} =$$

$$M_2 = P_u(0.6 + 0.3h)$$

↳ inches

Columns of two-way prestressed slabs - shear 11.12.2.2

$$V_c = (\beta_P \sqrt{f'_c} + 0.3 f_{pc}) b_o d + V_p$$

· Determine Mu from gravity moments

$K_{col}$ below $= \frac{4EI}{L-2t} = \frac{4E_c(38081.3)}{216-2(10)} = 777.17$

$I_c = \frac{26(26)^3}{12} = 38081.3$

$K_{col}$ above $= 885.61$

$K_t = \frac{9E_c C}{\ell_2(1-c_2/\ell_2)^3} = \frac{9(6567)E_c}{38.25(12)(1-26/38.25\cdot12)^3} = 153.38 E_c$

$C = (1-0.63(10/26)(10^3(26)/3) = 6567$

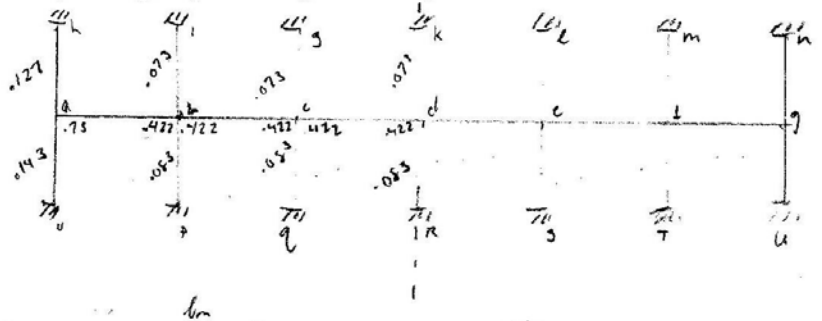
·Equivalent column stiffness:

$\frac{1}{K_{eq}} = \frac{1}{(777.17)^{1}} \frac{1}{2(153.38)} + \frac{1}{885.81}$ $K_{eq} = 176.2 E_c$

· Slab stiffness: $I_s = \frac{(38.25)(12)(10)^3}{12} = 38250$

$K_s = \frac{4E_c I_s}{\ell_n - c_1/2} = \frac{4(38250)(E_c)}{334-26/2} = 476.6 E_c$

$\ell_n = (30(12) - 26) = 334$



$M_u = \frac{358\overset{\ell_n}{(27.8)^2}}{12} \cdot \frac{12}{1000} = 248.9^{k\cdot in}$

·Fixed-End Moments

$w_u = 1.2(165) + 1.6(100) = 358$

* Equivalent Frame Method to determine slab stiffness

·examined in East/West Direction b/c of banded direction

Distribution Factors:

$D_F = \frac{K_s}{\Sigma K} = \frac{476.6}{476.6(2) + 176.2} = \begin{matrix} 0.422 & \text{ext} \\ 0.730 & \text{int} \end{matrix} \Big\} \text{ slab}$

column → $0.27\,(.47) = 0.127\ ;\ .143$
$0.156\,(.47) = 0.073\ ;\ 0.083$

Moment Distribution: spreadsheat

$\text{JNT} \begin{cases} M_{u\ TOP} = 86^{'k} \\ M_{u\ BOT} = 43^{'k} \end{cases}$

$\text{ext.} \begin{cases} M_{TOP} = 160^{'k} \\ M_{BOT} = 80^{'k} \end{cases}$ Column A-3

$\frac{50.17}{12}(38.25) = 160^{'k}\ ,\ 80^{'k}$

$\frac{9.62}{12}(38.25) = 30.7^{'k}\ ,\ 15.33^{'k}$

$\frac{.38}{12}(38.25) = 1.21^{'k} / 2 = 0.6^{'k}$

* verified w/ moment distribution in 13.6.9.2

ACI 11.4.1 - columns assumed to be nonsway
if column end moments do not
increase by more than 5% of first order moments

verify: $Q = \frac{\Sigma P_u \Delta_o}{V_{us} l_c} < 5\%$

* gravity axial loads determined in spreadsheet

Moment in column check: 13.6.9.2

Live: N-S: $M_o = \dfrac{0.1(30)(45-30/12)^2}{8} = 677.3^{\prime k}$

$ext. = 0.3(M_o) = 0.3(677.3) = 203.2^{\prime k}$

- w/dead
- factored

$int.: M_u = 0.07[(q_{Du} + 0.5q_{Lu})\ell_2\ell_n^2 - q_{Du}'\ell_2'(\ell_n')^2]$

$= 0.07[(1.2(.165)+0.5(1.6)(.1))(30)(45-30/12)^2 - 1.2(.165)(30)(38.25-30/12$

$= 523.07^{\prime k}$

$.53(523.07) = 277.23^{\prime k}$

E-W: $M_o = \dfrac{0.1(38.25)(30-30/12)^2}{8} = 361.58^{\prime k}$

$ext. = 0.3M_o = 0.3(361.58) = 108.47^{\prime k}$

- w/dead
- factored

$int. = 0.07[(1.2(.165)+.5(1.6)(.1))(38.25)(30-30/12)^2 - 1.2(.165)(38.25)(30-30/12)^2]$

$= 162.4^{\prime k}$

Dead: N-S $M_o = \dfrac{0.165(30)(45-30/12)^2}{8} = 1117.6$

$ext. = 0.3(1117.6) = 335.28^{\prime k}$

E-W $M_o = \dfrac{0.165(38.25)(30-30/12)^2}{8} = 596.6$

$ext. = 0.3(596.6) = 178.98^{\prime k}$

Total:

→ N/S ext = $203.2(.53) + 335.28(.53) = 285.4^{\prime k}$ (E-5)

→ E/W ext = $\underset{M_L}{108.47}(.53) + \underset{M_D}{178.98}(.53) = 152.3^{\prime k}$

$153^{\prime k}$ found from EFM (assume A-3)

*loads should be considered already factored

int: $162.43^{\prime k}(.53) = 86.1^{\prime k}$ (assume E-3)

$32^{\prime k}$ found from EFM

# Design Verification of PCA column

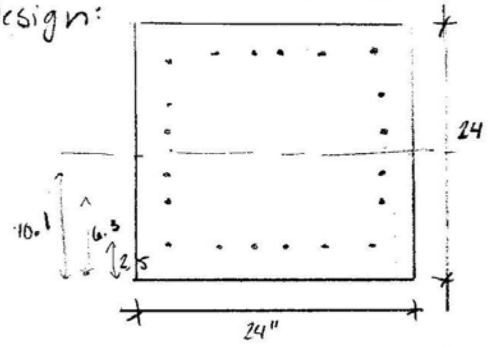
→Exterior column: E- 5

$P_u = 1930^k$ Load Combination: 1.2D+ 1.6+ 0.5S controlling

$M_u = 285.41^{\prime k}$

(N/S moment controls)

Design:



(20) #10

Clear cover: 1.5"

$b = 11(1.27) + 2(.375) + 2(1.5)$

$17.72 < 24"$ OK

spaced @ 3.8"

· Pure Axial:

$P_0 = 0.85(5)(24 \times 24 - 20(1.27)) + 1.27(20)(60) = 3864.05^k$

$\phi P_n = \underline{2505.13^k}$

· Pure Bending:

$f_{s1} = \frac{0.003}{c}(c-2.5)(29000)$

$f_{s2} = \frac{0.003}{c}(c-6.3)(29000)$

$f_{s3} = \frac{0.003}{c}(c-10.1)(29000)$ yields $= -60$

$f_{s4} = f_{s5} = f_{s6} = -60$

$\Sigma F = 0 = 0.85(5)(24)(.85)(c) + 6f_{s1} + 2f_{s2} + 2f_{s3} + 2f_{s4} + 2f_{s5} + 6f_{s6}$

$0 = 86.7c + \frac{6(.003)(29000)}{c}(c-25) + \frac{2(.003)(29000)(c-6.3)}{c} + \frac{2(.003)(29000)(c-10.1)}{c}$

$+ 12(-60)$

$= 86.7c + 522 - 1305/c + 174 + 174 - 1096.2/c - 1757.4/c - 600$

$86.7c - \frac{4158.6}{c} + 270 = 0$

$86.7c^2 + 270c - 4158.6 = 0 \qquad c = 5.54$

corrected: $86.7c + 522 - 1305/c + 174 - 1096.2/c - 720 = 0$

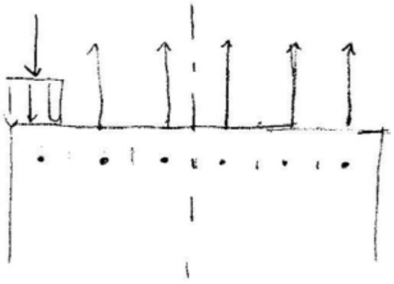
$86.7c^2 - 24c - 2401.2 = 0$

$c = 5.403$

verify assumption:

$f_{s1} = +46.7$, $f_{s2} = -14.5$ } $< 60$ ksi OK

$\varepsilon_{s3} = -.0026$, $\varepsilon_{s4} = -.0042$, $\varepsilon_{s5} = -.0068$, $\varepsilon_{s6} = -.0089$ } $< -0.002$ OK



$$M_o = 0.85(5)(6.85)(24)(5.4)\left(12 - \frac{0.85(5.4)}{2}\right) + 6(46.7)(12-2.5)$$
$$+ 2(-14.5)(12-6.3) + 2(-60)(12-10.1) + 2(-60)(12-13.9)$$
$$+ 2(-60)(12-17.7) + 6(-60)(12-21.5) = 1005^{K}(.9)$$

$\phi M_n = 909^{\prime K}$

<u>Code</u> -

slenderness -

$$\frac{Kl}{r} = \frac{1(18)(12)}{.3(24)} = 30 \leq 34 + 12(.5) = 40$$

** <u>slenderness need not be considered</u>

$A_{ST} = 0.01(24)(24) = 5.76 \text{ in}^2$ $\quad 1.27(20) = 25.4 \text{ in}^2$

$0.08(24)(24) = 46.08 \text{ in}^2$

<u>OK</u>

stiffness reduction factor = 0.75

cracked section: I = 0.7 $\quad I = 1/12(24)(24)^3 = 27648 \text{ in}^4$

$$P_c = \frac{\pi^2(EI)}{(30)^2} = \frac{3.14^2(4030.5)(27648)}{(31.18 \cdot 12)^2} = 7848^{K}$$

$M_2 = 1930(.6 + 0.3(18)(12)) = 75037.4^{\prime\prime K}$

* 24 x 24 w/ (20) #10
#3 ties

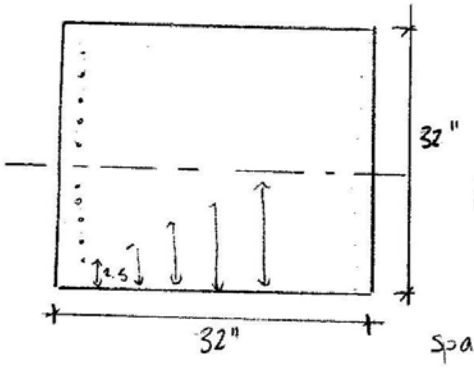
Adequate Column size

→ Interior Column: E-3

$P_u = 3520^k$ Load Combination $1.2D + 1.6L + 0.5S$ controlling

$M_u = 277.23'^k$

(N/S Moment controls)



(36) #10 bars w/ #3 ties

$b_{min} = 1.5(2) + .375(2) + 10(1.27) + 9(1.27) = 27.88" < 32'$ OK

spaced 3" o.c.

- Pure Axial:

$P_o = 0.85(5)(32 \times 32 - 36(1.27)) + 1.27(36)(60) = 6900$

$\phi P_n = 0.65(6374)\ 4485.6^K$

- Pure Bending:

$f_{s1} = \frac{0.003}{c}(c - 2.5)(29000)$

$f_{s2} = \frac{0.003}{c}(c - 5.5)(29000)$

$f_{s3} = \frac{0.003}{c}(c - 8.5)(29000)$

$f_{s4} = \frac{0.003}{c}(c - 11.5)(29000$

$f_{s5} = f_{s10} = -60$ (yield)

$$\Sigma F = 0 = 0.85(5)(32)(0.85)c + \frac{10(0.003)(29000)(c-2.5)}{c} + \frac{2(0.003(29000)(c-5.5)}{c}) + \frac{0.003(2)(29000)(c-8.5)}{c} + \frac{2(0.003)(29000)(c-11.5)}{c} + 22(-60)$$

$$= 115.6c + 870 - 2175/c + 174(2) - 1320 - 957/c - 1479/c$$

$$= 115.6c^2 - 102c - 4611 = 0 \qquad c = 6.77$$

· verify assumption:

$f_{s1} = 54.9$
$f_{s2} = -16.4$  } $< 60\text{ksi}$ ok
$f_{s3} = -22.2$
$\varepsilon_{s4} = -0.0021$  } $< -0.002$ ok
$\varepsilon_{s5} = \varepsilon_{s10}$

$$M_o = 0.85(5)(0.85)(32)(6.77(16 - \tfrac{0.85(6.74)}{2}) + 10(46.7)(16-2.5)$$

$$+ 2(-16.4)(16-5.5) + 2(-22.2)(16-8.5) + 2(-60)(16-11.5) = 15357$$

$$+ 2(-60)(16-14.5) + 2(-60)(16-17.5) + 2(-60)(16-20.5) = 15897.1k$$

$$+ 2(-60)(16-23.5) + 2(-60)(16-26.5) + 10(-60)(16-29.5) = 26157$$

$$\phi M_n = 0.9(2180) = \underline{\underline{1962}}^{1k}$$

· Code Check:

$A_{st} = 0.01(32)(32) = 10.24\text{ in}^2 \qquad 1.27(36) = 45.72\text{in}^2$ ok
$0.08(32)(32) = 81.92\text{ in}^2$

· slenderness:

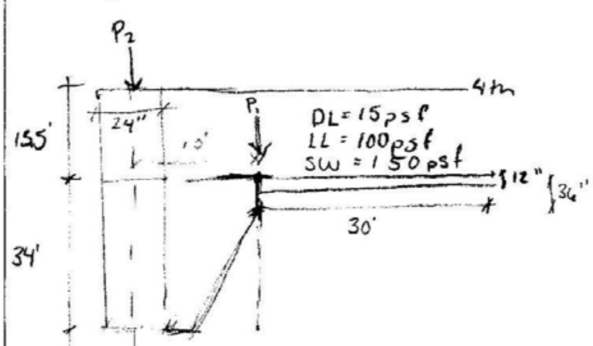
$$\frac{kl}{r} = \frac{1.0(18)(12)}{.3(32)} = 22.5 < 34 + 12(.5) = 40$$

* slenderness need not be considered

$I = 87381.3\text{in}^4$
$E = 4030.51\text{ ksi}$

$\boxed{32 \times 32 \text{ w/ } (36)\ \#10,\ \#3 \text{ ties} \qquad \text{Adequate Column size}}$

# Design of Blade Column (4)

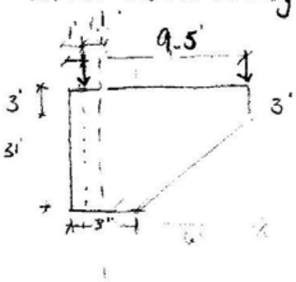


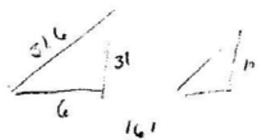
Load Combination:
$1.2D + 1.6L + 0.5S$

$P_2 = 1960 - 157 - 215 = 1588^k$

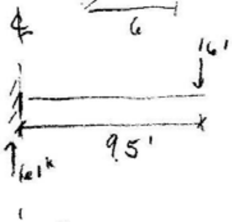
$P_1 = (165\overset{Tw}{(30)}(15))1.2$
$+ 1.6[(100)(30)(15)] / 1000$
$= 161.1^k \approx 161^k$

- unbraced length = 34'



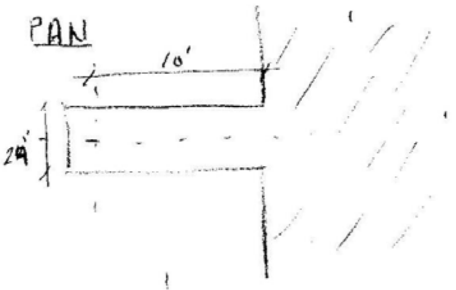


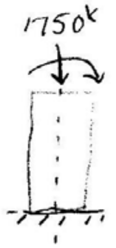
$\frac{31}{6} = \frac{16}{x}$



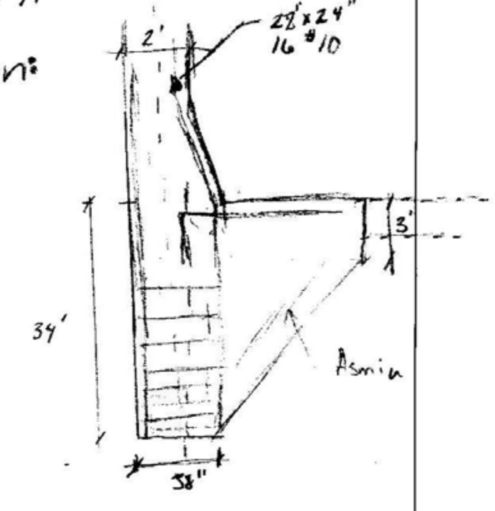
$M_1 = 1530^{1k}$

PAN

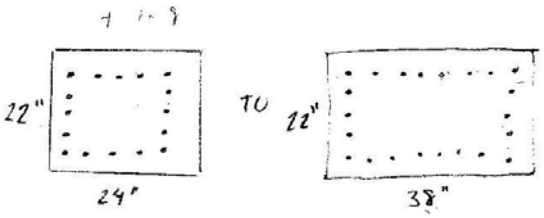




PCA Column:



$M_2 = 1588(.5) = 794^{1k}$



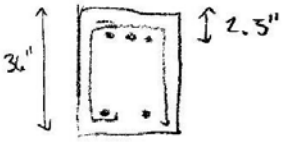
$\frac{4}{24}$ $\frac{8}{38}$

spacing = 24 - 1.5(2) - 2(.375) - 5(1.27) = 13.9/4 = 3.475"

spacing = 38 - 1.5(2) - 2(.375) - 8(1.27) = 24.09/7 = 3.44"

- Cantilever:

M = 161(8.5') = 1368.5'k

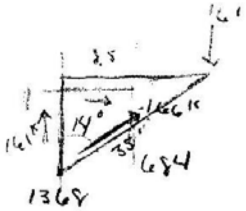


$A_s = \frac{M_u}{4d} = \frac{1368.5}{4(33.5)} =$ 10.21

$\phi M_n = A_s F_y (.9)(d - a/2)$

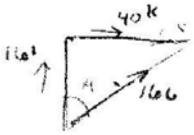
$1368.5 = A_s (60)(.9)(33.5 - \ \ /2)$

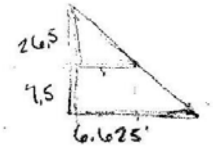
$a = \frac{A_s f_y}{0.85 f'_c b} =$



4 # 9 (to fit):

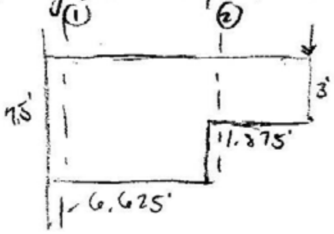
$4 = \frac{1368.5}{4(d)}$ d = 85.53 = 7.13'





$\frac{26.5}{x} = \frac{34}{8.5}$

use geometry for design:



$M_1 = 1368.5'^k$

$a = \frac{4(60)}{.85(6)(22)} =$ 2.14

$\phi M_n = 0.9(4)(60)(87.5 - 2.14/2) = 1556$ $\underline{\underline{\text{OK}}}$

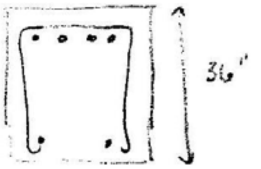
$M_2 = 161(1.875) = 302'^k$

$a = 2.14$

$\phi M_n = 0.9(4)(60)(33.5 - 2.14/2) = 583.7'^k$

$\underline{\underline{\text{OK}}}$

①



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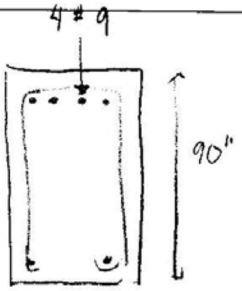


#3 stirrups

- shear

$V_c = 2\sqrt{6000}\,(33.5)(22)/1000 = 114.2^k$

$V_c = 2\sqrt{6000}\,(87.5)(22)/1000 = 298.2^k$

$\phi V_n = 42.8^k$
$111.8^k$

- $V_s = V_u/\phi - V_c = 161/.75 - 114.2 = \underline{101.5}$

$161.75 - 298.2 = \underline{-83.5}$

- $V_s = 8\sqrt{f'c}\,(b_w)(d)$

$8\sqrt{6000}\,(22)(33.5) = 456.7^k$

$V_s \leq 228.35$

$s_{max} = \min \begin{matrix} d/2 \\ 24 \end{matrix} = 33.5/2 = 16.75 \neq 16" \;\&\; 24"$

- determine shear reinforcement:

$$A_{v\,min} = \max \begin{cases} 0.75\sqrt{6000}\,(22)(\overset{24}{16})/60{,}000 = 0.34\text{in}^2 \;\cancel{\phantom{x}}\; 0.511\text{in}^2 \\ 50(22)(\underset{24}{16})/60000 = 0.29\text{in}^2 \;\cancel{\phantom{x}}\; 0.44\text{in}^2 \end{cases}$$

$\Rightarrow 0.2(2) = 0.40\text{in}^2$ use #4 stirrups

· design shear reinforcement

$V_s = A_v f_{yt} d / s$

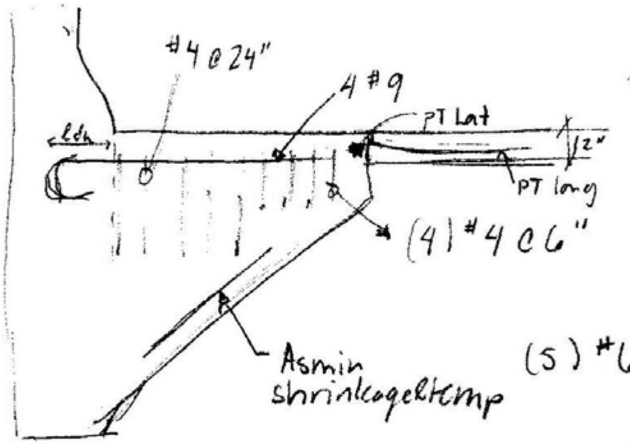
$s = \frac{A_v f_{yt} d}{V_s} = \frac{0.44(60)(33.5)}{114.2} = 7.75 \rightarrow @\ 6''$

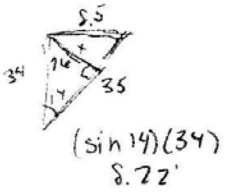
$\underset{?}{0.22(60)(87.5)}$

$\boxed{\#4\ @\ 6''\ \text{for}\ 2'\ ;\ \#4\ @\ 24''\ \text{for}\ 6.5'}$

$\phi V_n = 0.75\left[114.2 + \frac{0.4(60)(33.5)}{6}\right] = 186.15^k > 161^k$

$0.75\left[298.2 + \frac{0.4(60)(87.5)}{24}\right] = 289.3^k \ \underline{\underline{OK}}$ $> 161^k \ \underline{\underline{OK}}$





(sin 14)(34)
8.22'

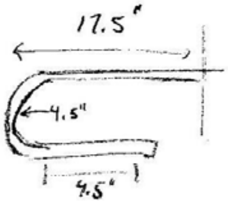
$\rho = \frac{A_s}{bd} = 0.0018$ $\qquad A_{s,min} = 0.0018(22)(\overset{\text{avg assume } 4'}{48}) \quad 1.9 in^2$

use (5) #6 $A_s = 2.2 in^2$

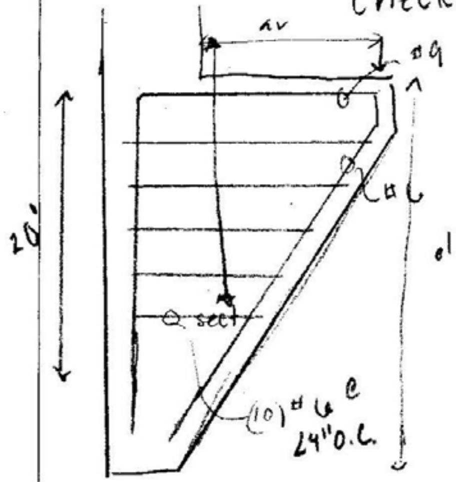
$l_d = \frac{0.02(\beta)(\lambda)(f_y)(d_b)}{\sqrt{f'_c}} = \frac{0.02(1.0)(1.0)(60,000)(1.128)}{\sqrt{6000}} = 17.5''$

ext = $4d_b \approx 4.5''$

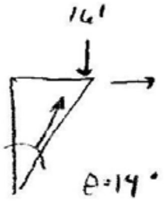
r = 4.5"



Check: 11.9. Brackets & corbels



$\frac{a_v}{d} = \frac{8.5}{34} < 1$



tan14 = x/161
x = 40^k < 161^k
cos14 = 161/h
h = 166^k

- sect to resist: $161^k = V = 161^k$ OK
  $161^k(8.5') + 40^k(h-d) = M = 1375^k$ OK
  $N_{uc} = T = 40^k > 0.2(161) = 32.2^k$

- shear-friction reinforcement $A_{vf}$
  $166^k = A_{vf}(60)(1.4\sin 14 + \cos 14)$
  $A_{vf} = 2.11\ in^2$
  $A_{ct} = 5(.44) = 220\ in^2$ OK

- $V_n = \begin{matrix} 0.2 f'_c b_w d \\ 800 b_w d \end{matrix} \Big|_{min} = \begin{matrix} 0.75(2)(6000)(22)(406) = 8039^k \\ 0.75(800)(22)(406) = 5359^k \leftarrow \text{controls} \end{matrix}$

- $A_n(\phi)(f_y) \geq 40^k = 10(4)(.75)(60)\ 180^k > 40^k$ OK

  $A_{sc} = A_f + A_n$
  $2/3\ A_{vf}/3 + A_n$

- $A_h \geq (A_{sc} - A_n)(.5)$ distribute uniformly over 2/3 d
  $A_h = 4(1.0) = 4(.5) = 2 in^2$ → space @ 24" o.c.

- $\frac{A_{sc}}{bd} = \frac{4}{22(34)(12)} = 0.0004 \geq 0.04\left(\frac{6}{60}\right) = 0.004$ X
  10 #6 = 4.4 in² OK

## 11.3 Shear Wall Design

Building Geometry:

| Center of Mass | | |
|---|---|---|
| Level | X (ft.) | Y (ft.) |
| 2 | 114 | 55 |
| 3 | 105 | 57 |
| 4 | 105 | 57 |
| 5 | 105 | 57 |
| 6 | 104 | 57 |
| 7 | 104 | 57 |
| 8 | 104 | 57 |
| 9 | 104 | 57 |
| 10 | 104 | 57 |
| 11 | 104 | 57 |
| 12 | 104 | 57 |
| PH | 104 | 53 |
| LR | 112 | 81 |
| HR | 95 | 61 |

*from ETABS model

| Center of Rigidity | | |
|---|---|---|
| Level | X (ft.) | Y (ft.) |
| 2 | 101 | 60.75 |
| 3 | 101 | 60.75 |
| 4 | 101 | 60.75 |
| 5 | 101 | 60.75 |
| 6 | 101 | 60.75 |
| 7 | 101 | 60.75 |
| 8 | 101 | 60.75 |
| 9 | 101 | 60.75 |
| 10 | 101 | 60.75 |
| 11 | 101 | 60.75 |
| 12 | 101 | 60.75 |
| PH | 101 | 60.75 |
| LR | 101 | 60.75 |
| HR | 101 | 60.75 |

*from hand calculations

| Relative Stiffness | |
|---|---|
| W3 | 0.50 |
| W4.1 | 0.50 |
| WC | 0.36 |
| WD | 0.36 |
| WF | 0.28 |

Wind Loads:

| Floor | Height Above Ground(ft.) | Floor Height (ft.) | Forces (k) | | Story Shears | |
|---|---|---|---|---|---|---|
| | | | North/South | East/West | North/South | East/West |
| 1 | 0 | 18 | 64.23 | 28.54 | 1611.98 | 736.04 |
| 2 | 18 | 16 | 125.15 | 55.94 | 1547.75 | 707.50 |
| 3 | 34 | 13.5 | 113.48 | 51.13 | 1422.60 | 651.56 |
| 4 | 47.5 | 13.5 | 106.73 | 48.33 | 1309.12 | 600.43 |
| 5 | 61 | 13.5 | 109.27 | 49.68 | 1202.39 | 552.10 |
| 6 | 74.5 | 13.5 | 110.65 | 50.41 | 1093.12 | 502.43 |
| 7 | 88 | 13.5 | 112.96 | 51.64 | 982.47 | 452.01 |
| 8 | 101.5 | 13.5 | 114.81 | 52.62 | 869.51 | 400.38 |
| 9 | 115 | 13.5 | 115.73 | 53.11 | 754.69 | 347.75 |
| 10 | 128.5 | 13.5 | 117.35 | 53.97 | 638.96 | 294.64 |
| 11 | 142 | 13.5 | 118.04 | 54.34 | 521.61 | 240.67 |
| 12 | 155.5 | 14.5 | 123.90 | 57.14 | 403.57 | 186.33 |
| Penthouse | 170 | 18 | 145.42 | 67.18 | 279.66 | 129.19 |
| Low Roof | 188 | 6 | 107.39 | 49.61 | 134.24 | 62.01 |
| High Roof | 194 | | 26.85 | 12.40 | 26.85 | 12.40 |

Seismic Building Weight:

| Location | Area | Load (psf) | Weight (kip) | Mass |
|---|---|---|---|---|
| Level 2 | | | | |
| Retail | 22002 | 165 | 3630.3 | 112.7 |
| Lobby | 2000 | 165 | 330.0 | 10.2 |
| Curtain Wall | 10800 | 25 | 270.0 | 8.4 |
| Columns & Capitals | 4082 | 150 | 612.3 | 19.0 |
| Shear Wall | 2159 | 150 | 323.9 | 10.1 |
| Masonry Wall | 1800 | 62 | 111.6 | 3.5 |
| **Floor Total** | **-** | **-** | **5278.1** | **163.9** |
| Level 3 | | | | |
| Office | 24923 | 165 | 4112.3 | 127.7 |
| Curtain Wall | 9576 | 25 | 239.4 | 7.4 |
| Masonry Wall | 1592 | 62 | 98.7 | 3.1 |
| Balcony | 2266 | 166 | 376.2 | 11.7 |
| Columns & Capitals | 3379 | 150 | 506.9 | 15.7 |
| Shear Wall | 1873 | 150 | 281.0 | 8.7 |
| **Floor Total** | **-** | **-** | **5614.4** | **174.4** |
| Level 4-11 | | | | |
| Office | 24486 | 165 | 32321.5 | 1003.8 |
| Curtain Wall | 8600 | 25 | 1720.0 | 53.4 |
| Columns & Capitals | 3115 | 150 | 3738.0 | 116.1 |
| Shear Wall | 1714 | 150 | 2056.8 | 63.9 |
| **Floor Total** | **-** | **-** | **39836.3** | **1237.2** |
| Level 12 | | | | |
| Office | 21600 | 165 | 3564.0 | 110.7 |
| Curtain Wall | 8812 | 25 | 220.3 | 6.8 |
| Columns & Capitals | 3221 | 150 | 483.2 | 15.0 |
| Shear Wall | 1778 | 150 | 266.7 | 8.3 |
| Balcony | 2886 | 166 | 479.1 | 14.9 |
| **Floor Total** | **-** | **-** | **5013.2** | **155.7** |
| Penthouse | | | | |
| Office | 12800 | 165 | 2112.0 | 65.6 |
| Balcony | 733 | 166 | 121.7 | 3.8 |
| Curtain Wall | 9054 | 25 | 226.4 | 7.0 |
| Roof | 8800 | 14 | 123.2 | 3.8 |
| Columns & Capitals | 3106 | 150 | 465.9 | 14.5 |
| Shear Wall | 2064 | 150 | 309.6 | 9.6 |
| **Floor Total** | **-** | **-** | **3358.7** | **104.3** |
| Low Roof | | | | |
| Surface | 12800 | 100 | 1280.0 | 39.8 |
| Columns & Capitals | 2098 | 150 | 314.7 | 9.8 |
| Shear Wall | 1143 | 150 | 171.5 | 5.3 |
| **Floor Total** | **-** | **-** | **1766.2** | **54.8** |
| High Roof | | | | |
| Surface | 2688 | 100 | 268.8 | 8.3 |
| Columns & Capitals | 171 | 150 | 25.7 | 0.8 |
| **Floor Total** | **-** | **-** | **294.5** | **9.1** |
| **TOTAL BUILDING WEIGHT** | | | **61161.3** | **1899.4** |

Seismic Forces for ETABS East/West:

| Level | h | h | w | w*h$^k$ | Cvx | fi | Vi | By | %5 By | Ax | Mz |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | ft | kips | | | kips | kips | ft | ft | | k-ft |
| High Roof | 6 | 194 | 63 | 344832.8 | 0.003 | 3 | 3 | 77 | 3.85 | 1.0 | 11 |
| Low Roof | 18 | 188 | 665 | 3457765.8 | 0.031 | 29 | 29 | 218.33 | 10.9165 | 1.0 | 314 |
| PH | 14.5 | 170 | 3360 | 14821522.4 | 0.133 | 123 | 123 | 218.33 | 10.9165 | 1.0 | 1345 |
| 12 | 13.5 | 155.5 | 5013 | 19115478.2 | 0.171 | 159 | 159 | 218.33 | 10.9165 | 1.0 | 1735 |
| 11 | 13.5 | 142 | 4980 | 16370747.0 | 0.147 | 136 | 136 | 218.33 | 10.9165 | 1.0 | 1486 |
| 10 | 13.5 | 128.5 | 4980 | 13905194.0 | 0.125 | 116 | 116 | 218.33 | 10.9165 | 1.0 | 1262 |
| 9 | 13.5 | 115 | 4980 | 11598708.0 | 0.104 | 96 | 96 | 218.33 | 10.9165 | 1.0 | 1053 |
| 8 | 13.5 | 101.5 | 4980 | 9457903.0 | 0.085 | 79 | 175 | 218.33 | 10.9165 | 1.0 | 858 |
| 7 | 13.5 | 88 | 4980 | 7490555.5 | 0.067 | 62 | 237 | 218.33 | 10.9165 | 1.0 | 680 |
| 6 | 13.5 | 74.5 | 4980 | 5706012.2 | 0.051 | 47 | 285 | 218.33 | 10.9165 | 1.0 | 518 |
| 5 | 13.5 | 61 | 4980 | 4115838.2 | 0.037 | 34 | 319 | 218.33 | 10.9165 | 1.0 | 374 |
| 4 | 13.5 | 47.5 | 4980 | 2734933.1 | 0.025 | 23 | 342 | 218.33 | 10.9165 | 1.0 | 248 |
| 3 | 16 | 34 | 5614 | 1785290.8 | 0.016 | 15 | 357 | 218.33 | 10.9165 | 1.0 | 162 |
| 2 | 18 | 18 | 5278 | 593724.1 | 0.005 | 5 | 362 | 218.33 | 10.9165 | 1.0 | 54 |
| Sum | 194 | | | 111498505.0 | | 927 | | | | | |

Seismic Forces for ETABS North/South:

| Level | h | h | w | $w*h^k$ | Cvx | fi | Vi | By | %5 By | Ax | Mz |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | ft | kips | | | kips | kips | ft | ft | | k-ft |
| High Roof | 6 | 194 | 63 | 344832.8 | 0.003 | 3 | 3 | 31.5 | 1.575 | 1.0 | 5 |
| Low Roof | 18 | 188 | 665 | 3457765.8 | 0.031 | 29 | 29 | 60.5 | 3.025 | 1.0 | 87 |
| PH | 14.5 | 170 | 3360 | 14821522.4 | 0.133 | 123 | 123 | 118.67 | 5.9335 | 1.0 | 731 |
| 12 | 13.5 | 155.5 | 5013 | 19115478.2 | 0.171 | 159 | 159 | 118.67 | 5.9335 | 1.0 | 943 |
| 11 | 13.5 | 142 | 4980 | 16370747.0 | 0.147 | 136 | 136 | 118.67 | 5.9335 | 1.0 | 808 |
| 10 | 13.5 | 128.5 | 4980 | 13905194.0 | 0.125 | 116 | 116 | 118.67 | 5.9335 | 1.0 | 686 |
| 9 | 13.5 | 115 | 4980 | 11598708.0 | 0.104 | 96 | 96 | 118.67 | 5.9335 | 1.0 | 572 |
| 8 | 13.5 | 101.5 | 4980 | 9457903.0 | 0.085 | 79 | 175 | 118.67 | 5.9335 | 1.0 | 467 |
| 7 | 13.5 | 88 | 4980 | 7490555.5 | 0.067 | 62 | 237 | 118.67 | 5.9335 | 1.0 | 370 |
| 6 | 13.5 | 74.5 | 4980 | 5706012.2 | 0.051 | 47 | 285 | 118.67 | 5.9335 | 1.0 | 281 |
| 5 | 13.5 | 61 | 4980 | 4115838.2 | 0.037 | 34 | 319 | 118.67 | 5.9335 | 1.0 | 203 |
| 4 | 13.5 | 47.5 | 4980 | 2734933.1 | 0.025 | 23 | 342 | 118.67 | 5.9335 | 1.0 | 135 |
| 3 | 16 | 34 | 5614 | 1785290.8 | 0.016 | 15 | 357 | 118.67 | 5.9335 | 1.0 | 88 |
| 2 | 18 | 18 | 5278 | 593724.1 | 0.005 | 5 | 362 | 118.67 | 5.9335 | 1.0 | 29 |
| Sum | 194 | | | 111498505.0 | | | | | | | |

Wall Unfactored Shear Forces:

**Controlling Story Shear Forces(kip)**

| Level | Seismic | | | | | Wind | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Wall 3 | Wall 4.1 | Wall C | Wall D | Wall F | Wall 3 | Wall 4.1 | Wall C | Wall D | Wall F |
| 2 | 569 | 522 | 220 | 358 | 174 | 450 | 441 | 432 | 559 | 274 |
| 3 | 624 | 560 | 207 | 428 | 187 | 538 | 525 | 436 | 618 | 366 |
| 4 | 563 | 509 | 172 | 508 | 163 | 450 | 440 | 422 | 624 | 299 |
| 5 | 593 | 533 | 149 | 526 | 145 | 479 | 468 | 393 | 604 | 242 |
| 6 | 546 | 488 | 134 | 524 | 129 | 440 | 429 | 360 | 567 | 197 |
| 7 | 548 | 484 | 122 | 506 | 111 | 459 | 448 | 324 | 548 | 154 |
| 8 | 475 | 418 | 109 | 477 | 96 | 395 | 385 | 287 | 470 | 121 |
| 9 | 447 | 389 | 110 | 458 | 93 | 390 | 381 | 249 | 417 | 88 |
| 10 | 356 | 308 | 96 | 417 | 77 | 317 | 310 | 211 | 364 | 60 |
| 11 | 295 | 250 | 68 | 360 | 51 | 295 | 289 | 174 | 342 | 29 |
| 12 | 191 | 155 | 14 | 295 | 20 | 240 | 238 | 137 | 305 | 19 |
| PH | 126 | 75 | 1 | 224 | -28 | 316 | 305 | 95 | 274 | -49 |
| LR | 30 | 2 | 8 | 122 | -64 | 223 | 251 | 43 | 194 | -94 |
| HR | 20 | 7 | 22 | 31 | 0 | 140 | 117 | | | - |

*shear reversals are max values from differing load cases

## Wall Factored Shear Forces:

| Load Combinations: |
|---|
| 0.9D+1.6W |
| 0.9D+1.0E |

| Factored Controlling Story Shear Forces (kip) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Level | Seismic | | | | | Wind | | | | |
| | Wall 3 | Wall 4.1 | Wall C | Wall D | Wall F | Wall 3 | Wall 4.1 | Wall C | Wall D | Wall F |
| 2 | 569 | 522 | 220 | 358 | 174 | 720 | 706 | 691 | 894 | 438 |
| 3 | 624 | 560 | 207 | 428 | 187 | 861 | 840 | 698 | 989 | 586 |
| 4 | 563 | 509 | 172 | 508 | 163 | 720 | 704 | 675 | 998 | 478 |
| 5 | 593 | 533 | 149 | 526 | 145 | 766 | 749 | 629 | 966 | 387 |
| 6 | 546 | 488 | 134 | 524 | 129 | 704 | 686 | 576 | 907 | 315 |
| 7 | 548 | 484 | 122 | 506 | 111 | 734 | 717 | 518 | 877 | 246 |
| 8 | 475 | 418 | 109 | 477 | 96 | 632 | 616 | 459 | 752 | 194 |
| 9 | 447 | 389 | 110 | 458 | 93 | 624 | 610 | 398 | 667 | 141 |
| 10 | 356 | 308 | 96 | 417 | 77 | 507 | 496 | 338 | 582 | 96 |
| 11 | 295 | 250 | 68 | 360 | 51 | 472 | 462 | 278 | 547 | 46 |
| 12 | 191 | 155 | 14 | 295 | 20 | 384 | 381 | 219 | 488 | 30 |
| PH | 126 | 75 | 1 | 224 | -28 | 506 | 488 | 152 | 438 | -78 |
| LR | 30 | 2 | 8 | 122 | -64 | 357 | 402 | 69 | 310 | -150 |
| HR | 20 | 7 | 22 | 31 | | 224 | 187 | | | - |

## Wall Overturning Moment:

| Wind Overturning Moments | | | |
|---|---|---|---|
| Height (ft) | Moment | | |
| | Wall F | Wall D | Wall 3 |
| 18 | 67 | 87 | 48 |
| 34 | 820 | 1054 | 335 |
| 48 | 1754 | 2255 | 1185 |
| 61 | 1917 | 2465 | 1390 |
| 75 | 2280 | 2931 | 1651 |
| 88 | 2661 | 3421 | 1930 |
| 102 | 3034 | 3901 | 2201 |
| 115 | 3394 | 4363 | 2461 |
| 129 | 3737 | 4805 | 2711 |
| 142 | 4061 | 5221 | 2946 |
| 156 | 4359 | 5604 | 3161 |
| 170 | 4649 | 5977 | 3371 |
| 188 | 5424 | 6974 | 3934 |
| 194 | 6129 | 7880 | 4445 |
| Total(ft-k) | 44285.97 | 56939 | 31768 |

| Seismic Overturning Moments | | | |
|---|---|---|---|
| Height (ft) | Moment | | |
| | Wall F | Wall D | Wall 3 |
| 18 | 15 | 19 | 27 |
| 34 | 276 | 355 | 493 |
| 48 | 1636 | 2103 | 2921 |
| 61 | 2716 | 3492 | 4850 |
| 75 | 2837 | 3648 | 5066 |
| 88 | 2858 | 3675 | 5104 |
| 102 | 2728 | 3508 | 4872 |
| 115 | 2544 | 3271 | 4543 |
| 129 | 2231 | 2868 | 3984 |
| 142 | 1869 | 2403 | 3337 |
| 156 | 1480 | 1903 | 2644 |
| 170 | 1095 | 1408 | 1955 |
| 188 | 790 | 1015 | 1410 |
| 194 | 272 | 349 | 485 |
| Total(ft-k) | 23074 | 30016 | 41689 |

**Factored Overturning Moments

Wall Design Loads:

| Dead Load for Walls (kip) | |
|---|---|
| W3 | 678 |
| W4.1 | 678 |
| WC | 764 |
| WD | 764 |
| WF | 564 |

| Factored Dead Load for Walls (kip) | | |
|---|---|---|
| | Seismic | Wind |
| W3 | 635 | 610 |
| W4.1 | 635 | 610 |
| WC | 715 | 688 |
| WD | 715 | 688 |
| WF | 528 | 508 |

Wall Hand Calculations:

## WALL DESIGN

- Determine wind & EQ seperate for forces w/ factor of 1.0
- apply load combinations: controlling for lateral resistance

  $0.9D + 1.6W$

  $0.9D + 1.0E \approx (0.9 - 0.2S_{DS})D + 1.0E_h$

- gravity = self weight of wall
  - walls around openings → do not include LL & SDL
- determine forces from each direction in each wall → design for max loading in each plane

Earthquake: add E & ET to find max

$E_h$: W3 = 620
W4.1 = 540
WC = 220
WD = 428
WF = 187

Dead weight of walls:

$W3 = (194')(30')(12/12)(150) - 13(10)(10)(12/12)(150) = 678^k$
$W4.1 = 678^k$

$WC = 194(31.5)(10/12)(150) = 763.9^k$
$WD = 763.9^k$

$WF = 188(24)(10/12)(150) = 564^k$

Wind: Determine controlling forces

Wall 3 - wind direction 0° (wind 10)
ASCE 7-05, case 4
$V_{max} =$ 638k

Wall 4.1 - wind direction 90°
ASCE 7-05, case 4
Vmax = 525k

Wall C - wind direction 90° (wind 12)
ASCE 7-05, case 4
Vmax = 436k

Wall D - wind direction 90°
ASCE 7-05 case 1
Vmax = 618k

Wall F - wind Direction 90°
ASCE 7-05 case 2 (wind 5)
Vmax = 366k

Design: Wall 3
Wall F
Wall D

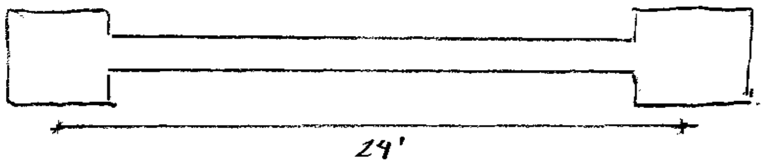
2 layers w/ 2" of outside of wall
smax = 18"

3

## Design of Wall F

- Wind: $P_u = 508^k$ $\quad V_{u\,max} = 366(1.6) = 585.6^k$

$M_u = 44,286'^k$



Boundary Elements:

$P_{uBE} = M_u/z = 44,286/24' = 1845.25 + 508/2 = 2099.25$

$A_g = 24(10/12) = 20\ ft^2$

$I = \frac{10/12(24)^3}{12} = 960\ ft^4$

$\frac{P_u}{A_g} = \frac{508^k}{20} = 25.4 + \frac{M_u(w/2)}{I_g} = \frac{44286(24/2)}{960} = 554$

$25.4 + 554 = 579\ ksf$

$= 4.02\ ksi$

$0.2(6) = 1.2\ ksi <$

∴ need boundary elements

- Determine long & transverse reinf ACI 12.7.2.2

$2A_{cv}\sqrt{f'_c} = 2(10(24)(12))\sqrt{6000}/1000 = 446.7^k$

$586^k > 446.7^k$

∴ need two curtain of reinforcement

$\rho_t = \frac{A_t}{A_{cv}} \quad \rho_\ell = \frac{A_\ell}{A_{cv}}$ estimate $\rho_\ell, \rho_t \geq 0.0025$

$A_{cv} = 12(10) = 120\ in^2/ft$

$A_{s\ell} = 0.0025(120) = 0.30\ in^2$ → assume #5

$s_{req'd} = \frac{.62}{s} = \frac{0.30}{12} \quad s = 18''$

- Nominal shear capacity

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{l_w} = \frac{194}{24} = 8.1 \qquad \alpha_c = 2$

$A_{cv} = 10(24)(12) = 2880$

$\rho = \frac{0.62}{18(10)} = 0.0034$

$V_n = 2880[2\sqrt{6000} + 0.0034(60,000)]/1000 = 1034$

$\phi V_n = 0.6(1034) = 620.2^k$

$620.2^k > 586^k \qquad \underline{\underline{ok}}$

- check 3E capacity

$A_{ST} = 24\ \#10 = 24(1.27) = 30.48\ in^2$

$\rho_{st} = \frac{30.48}{32(32)} = 0.0298$

$\rho_{max} = 0.06 \qquad \rho_{min} = 0.01$

$\phi P_n = 0.8(\overset{2}{\cancel{06}})[0.85(6)(32 \cdot 32 - 30.48) + 60(30.48)]$

$= 4413^k > 2099^k \qquad \underline{\underline{ok}}$

w/ 0.9 D from floor loading

$.9(2200) + 2099 = 4079^k \qquad \underline{\underline{ok}}$

- level to change $f'_c$ to 5000:

$\phi V_n = 0.6[2880(2\sqrt{5000} + 0.0034(60.000)]/1000] = 596.9^k$

* can change anywhere
-keep consistent w/ columns

- confinement reinforcement

$s_{max} = \begin{cases} 1/4(32) = 8'' \\ \text{min} \quad 4'' \leftarrow \text{governs} \quad (6d_b) \end{cases}$

req'd area

$$A_s \geq \begin{cases} 0.09\, s b_s \dfrac{f'_c}{f_{yh}} \\ 0.3\, s b_c \left(\dfrac{A_g}{A_{ch}} - 1\right) \dfrac{f'_c}{f_y} \end{cases}$$

$A_{ch} = (32 - 3)(32-3) = 841\ in^2$

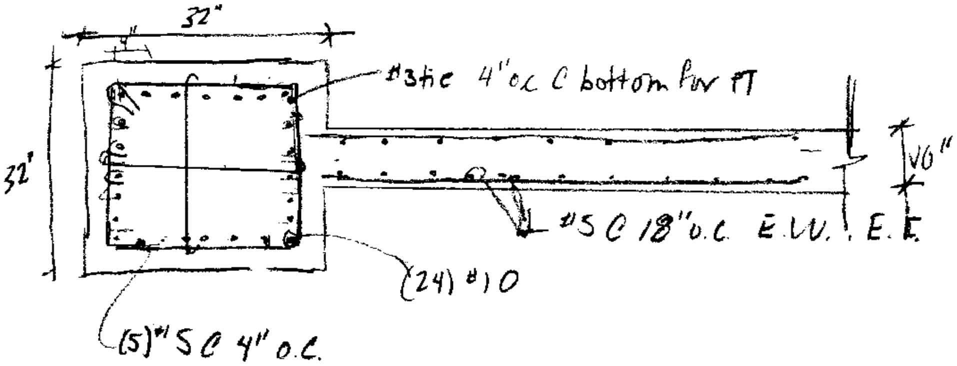
$= 0.09(4)(29)(4/60) = 1.044$ ← governs

$0.3(4)(29)\left(\dfrac{1024}{841} - 1\right)\left(\dfrac{6}{60}\right) = 0.76$

$A_{s\,prov} = 7(.31) = 2.17\ in^2 > 1.04\ in^2$ ok

*assume #5 @ 4" o.c.

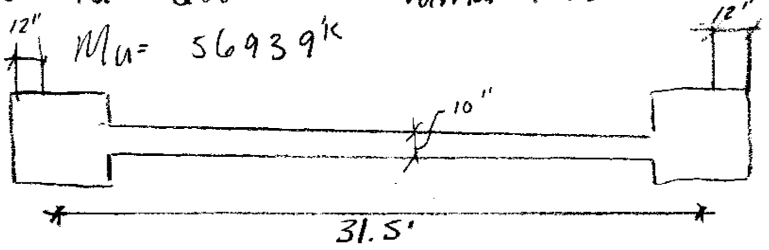
Design:



## Design of Wall D.

Wind - $P_u = 688$ $V_{u\,max} = 1.6(618) = 989^k$

$M_u = 56939^k$



$P_{u\,BE} = 688/2 + 56939 / 31.5 = 2151.6^k$

$A_g = 31.5(10/12) = 26.25\ ft^2$

$I = \frac{10/12\,(31.5)^3}{12} = 2171$

$\frac{P_u}{A_g} + \frac{M_u x}{I} = \frac{688}{26.25} + \frac{56939^k(15.75)}{2171} = 4393$

$= 3.05\ ksi > 1.2\ ksi$

$\therefore$ need boundary elements

- determine long. & transverse reinf.

$2A_{cv}\sqrt{f'_c} = 2(31.5(10)(12)\sqrt{6000}/1000) = 585.6^k$

$V_{u\,max} = 989^k > 585.6^k$

$\therefore$ use 2 curtains reinforcement

estimate $\rho_\ell, \rho_t \geq 0.0025$

$A_{cv} = 12(10) = 120\ in^2/ft$

$A_{st} = 0.0025(120) = 0.30$ assume #5

s req'd $= \frac{0.62}{s} = \frac{0.30}{12}$ $s = 18"$

- Nominal Shear Capacity

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$h_w / l_w = \frac{194}{31.5} = 6.16$ $\alpha_c = 2$

$A_{cv} = 10(12)(31.5) = 3780$

$\rho = \frac{0.62}{18(10)} = 0.0034$

$V_n = 3780[2\sqrt{6000} + 0.0034(60,000)]/1000 = 1356.7^k$

$\phi V_n = 0.6(1357) = 814^k < 989^k$

∴ increase reinf. spacing to 12"

$V_n = 3780[2\sqrt{6000} + 0.0052(60,000)]/1000 = 1765^k$

$\phi V_n = 0.6(1765) = 1059^k > 989^k$ ok

- Check BE Capacity

$A_{st} = 24 \ \#10 = 24(1.27) = 30.48$

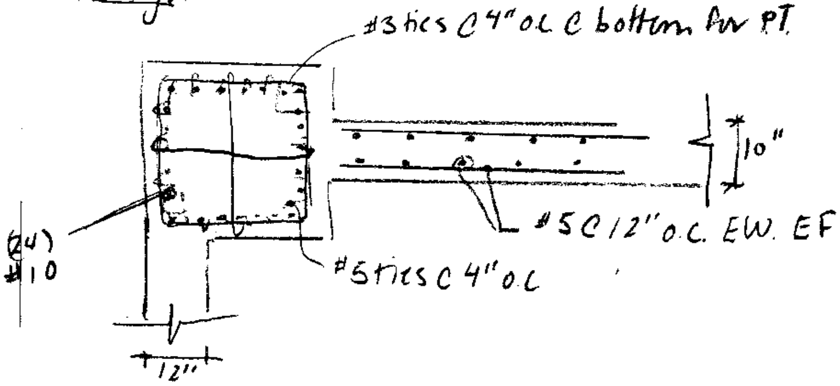
$\rho_{st} = \frac{30.48}{32(32)} = 0.0298 \begin{matrix} > 0.1 \\ < 0.6 \end{matrix}$

$\phi P_n = 4413^k$ (from before)

$0.9(2200) + 2151 = 4131^k < 4413^k$ ok

- Confinement reinforcement → as before
- no need for C shaped wall BE to take load

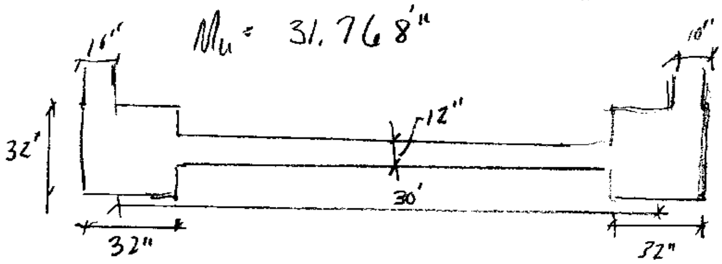
Design:



## Design of Wall 3

- Wind: $P_u = 610^k$ $\quad V_{u\,max} = (538^k)(1.6) = 860.8^k$

$M_u = 31{,}768'^k$



- $P_{uBE} = 610/2 + 31768/30 = 1364^k$

$A_g = 30(12/12) = 30\ ft^2$

$I = \dfrac{12/12\,(30)^3}{12} = 2250\ ft^4$

$\dfrac{P_u}{A_g} + \dfrac{Mc}{I} = \dfrac{610}{30} + \dfrac{31768(15)}{2250} = 232.12 = 1.6ksi > 1.2ksi$

$\therefore$ need boundary elements

- Determine long. & transverse reinf.

$2A_{cv}\sqrt{f'c} = 2(30(12)(12)\sqrt{6000})/1000 = 669^k$

$V_{u\,max} = 860.8^k > 669^k$

$\therefore$ use 2 curtains of reinforcement

- estimate $\rho_\ell, \rho_t \geq 0.0025$

$A_{cv} = 12(12) = 144\ in^2/ft$

$A_{st} = 0.0025(144) = 0.36\ in^2$ $\quad$ assume #5

$S_{req'd} = \dfrac{0.62}{S} = \dfrac{0.30}{12} = 24.5 \approx 18''$ max

Nominal Shear Capacity

$V_n = A_{cv}((\alpha_c \sqrt{f'c}) + \rho_t f_t)$

$\frac{h_w}{l_w} = \frac{194}{30} = 6.5 \Rightarrow \alpha_c = 2$

$A_{cv} = 12(12)(30) = \overset{5040\ (14")}{4320}$

$\rho = \frac{0.62}{18(12)} = \underset{.0025}{0.00287}$

Check piers:

$V_n \leq 8(4320)\sqrt{6000} = 2677^k$

$V_n = \underset{5040}{4320}[2\sqrt{6000} + 0.00287(60,000)]/1000 = 1413$

$\phi V_n = 0.6(\overset{1537}{1413}) = 847.9^k < \overset{922.07}{860.8^k}$

↳ 18" OK w/ 14" wall

$\therefore$ increase spacing to 16" o.c.

$\phi V_n = 0.6[4320(2\sqrt{6000} + 0.00323(60,000)/1000] = 904^k$

$909^k > 861^k$ $\underline{\underline{OK}}$

- Check BE Capacity

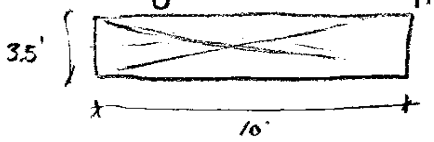
$A_{st} = 24\ \#10 = 24(1.27) = 30.48$

$\rho_{st} = \frac{30.48}{32(32} = 0.0298 \begin{matrix} > 0.1 \\ < 0.6 \end{matrix}$

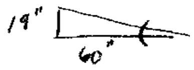
$0.9(2200) + 1364 = 3344^k$ $\underline{\underline{OK}}$

- Confinement reinforcement → as before
- Coupling beam: (typ)

$35/2 - 2"$

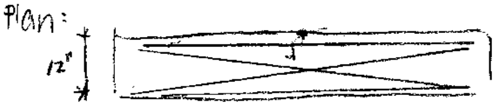


$10/3.5 = 2.86$



$\alpha = \tan^{-1}(^{70}/_{120})$
$18.4°$

Plan:



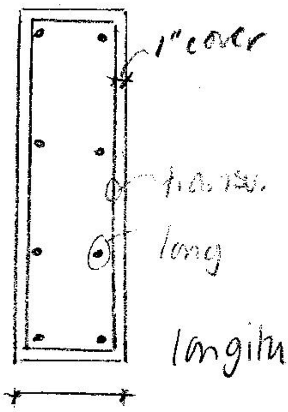
$d = 3.5 - 7"(2)/12 - .65" = 38"$
$d' = 2"$

$T_u = C_u = \phi A_s f_y$

$V_u = 2T_u \sin\alpha$
$= 2\phi A_s f_y \sin\alpha$

$M_u = \phi A_s f_y \cos\alpha (h - 2d')$

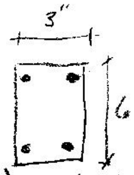
$V_n = 2 A_{vd} f_y \sin\alpha \leq 10\sqrt{f'_c}\, \overset{b_w d}{A_{cw}}$ $V_u = 861^k$



$b_w/2 = 12/2 = 6"$

$b_w/5 = 2.4 \approx 3"$

min: cage size



$A_{vd} = \underset{4\#5}{4(.31)} = 1.24 in^2$

·transverse conforms to 21.4.4

$V_n = 2\underbrace{(4)(.31)}_{4\#5}(60)\sin(17.6) = 45^k$

longitudinal & transverse conforms to 11.8.4 & 11.8.5

·diagonal bars developed for tension into the wall

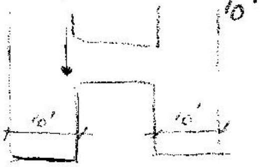
trans = $A_v$ 0.0025 $b_w s$ $s = \min \left| \begin{matrix} d/5 = 8" \\ 12" \end{matrix} \right.$

long = $A_{vh}$ = 0.0015 $b_w s_2$ $s_2 = \min \left| \begin{matrix} d/5 = 8" \\ 12" \end{matrix} \right.$

$.0025(12)(12) = 0.36 in^2$

* keep constant w/ wall #5 @ 8" o.c. E.W.

$A_{cw} = 12(3.5)(12) = 504 in^2$



$10(\sqrt{6000})(504)/1000 = 390.4^k \Leftarrow$

$\phi V_n = [2(10)(1.27)(60)(\sin 18.4)]0.85 = 409^k$

$V_{uB} = \dfrac{2(\overset{M}{1068})(1.6)}{10} = 342^k < 390.4^k$ OK

∴ Try 10 #10 diagonal

$b = 12 - 2(1) - .625(4) = 7.5" - 3" = 4.5"/2 \approx 2"$

$h = 42 - 2 - 0.625(4) = 37.5" - 9(1.27)(2) = 14.64$

Coupling beam → reg reinf.

$A_{s\,min} = \frac{200 b_w d}{f_y} \cdot \frac{200(12)(40)}{60} = 1.6 in^2$

Design as deep beam → 10.7, 11.8

$V_n \le 10\sqrt{f'_c}\, b_w d = 390.4^k$

$V_u = 861$ axial
$V_{uB} = 342^k$
$M_{uB} = 1709^{1k}$

$\perp \rho = 0.0025 b_w S$ $\quad s_{min} = \left| \begin{array}{l} d/5 = 8'' \\ 12 \end{array} \right.$

$\parallel \rho = 0.0015 b_w s_2$ $\quad s_2 = \left| \begin{array}{l} d/5 = 8'' \\ 2 \end{array} \right.$

$\phi M_n = 0.9(12.48)(60{,}000)(40 - {}^{12.24}/2)(1/12)(1/1000) = 1903^{1k} > 1709^{1k}$

$a = \frac{60(12.48)}{.85(6)(12)} = 12.24$ $\quad$ Try 8#11

$\phi V_n = \frac{0.85(3)(\overset{A}{.79})(60{,}000)(\overset{ld}{48})}{(\underset{s}{6})(1000)} = 967^k$

use (3)#8 @ 6" o.C.

$ld = \frac{60(1)(1.41)}{20\sqrt{6000}} = .55$

39"

use $\underline{48''}$

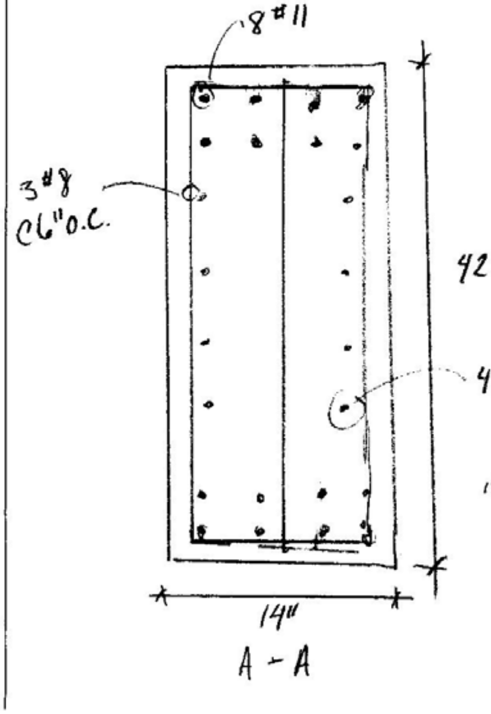
hook $12d_b = 12(1.41) = 18''$

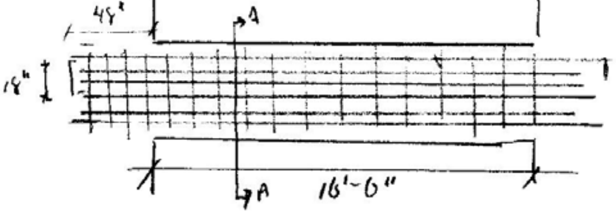
$b = 12 - 2 - 1(3) - 4(1.41) <$ too small

$h = 42 - 2 - 2(1) - 4(1.41) - 4 = 28.36/8''$

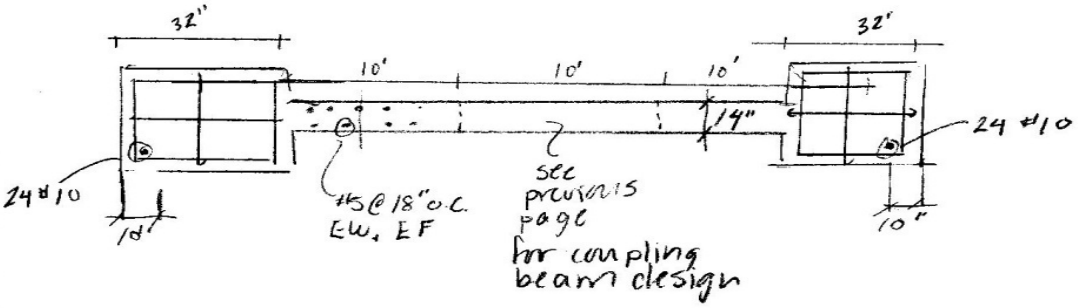
$= 4\#5$

* increase wall to 14" to allow for coupling beam reinforcement





Design:


32"
32"
10'
10'
10'
14"
24 #10
24 #10
10"
10"
#5@18" o.c.
EW, EF
see previous page
for coupling beam design

Seismic Hand Calculations:

## Seismic Calculations

$S_s = 0.170g$ $F_a = 1.6$ $S_{DS} = 0.181g$
$S_1 = 0.051g$ $F_v = 2.4$ $S_{D1} = 0.082g$

Site Class D (Soil Properties unknown)

Seismic Design Category B

$T_{min} = (1.04)(1.7) = 1.767$

$T_a = C_t h_n^x = 0.02(194)^{0.75} = 1.04$ (other structure)

* seismic design category permits equivalent lateral force procedure

Base Shear: $V = C_s W$

$$C_s = \min \begin{cases} 0.181/(3/1.1) = 0.06 \\ 0.082/(1.767 \cdot 3) = 0.0155 \\ 0.082(6)/(1.767^2 \cdot 3) = 0.053 \end{cases}$$

\+ $C_s = 0.01$ min required by code

$W = 55188^k$ (from spreadsheet)

$V = .0155(55188) = 855.4^k$

· Torsional amplification factor

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$$

X DIRECTION: $\delta_{max} = 1.60 + 0.032 = 1.632$
$\delta_{avg} = 1.60$

$$A_x = \left(\frac{1.632}{1.2(1.6)}\right)^2 = 0.722 = \boxed{1.0}$$

Y DIRECTION: $\delta_{max} = 1.19 + 0.075 = 1.265$
$\delta_{avg} = 1.19$

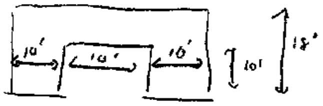
$$A_y = \left(\frac{1.265}{1.2(1.19)}\right)^2 = 0.785 \approx \boxed{1.0}$$

- redundancy factor
  - shear walls X & Y DIRECTIONS
  - check height to length ratios:

Y: $18/30$ $< 1.0$ $\rho = 1.0$
$18/26$

X: shearwall: $\frac{18}{31.5}$

wallpier: $\frac{10'}{10'} = 1$ $\rho = 1.0$



## Building Weight Columns & Capitals

2) 23(18+16)/2 (28)(28)/144 = 2129

+ [4'(2+1.167) - 28(28)/144]23 = 166

+ 12(18+16)/2 (32)(32)/144) = 1451

+ [4'(4') - 32(32)/144]12 = 107

+ ((18/2)(28(28)/144)5 = 245

4082 $ft^3$

3) height = (16+13.5/2) = 14.75

TOTAL = 3379 $ft^3$

TYP) height = 13.5'

TOTAL = 3115 $ft^3$

12) height = (14.5+13.5)/2 = 14'

TOTAL = 3221 $ft^3$

PH) 16.25(11)(28)(28)/144

7.25(12)(28)(28)/144

12(16.25)(32)(32)/(144)

TOTAL = 3106 $ft^3$

L ROOF) 9(11)(28)(28)/144 → 9 DC

4(9)(32)(32)/144 → 4 DC

8(12)(32)(32)/144

TOTAL = 2098 $ft^3$

HROOF) 8(3)(32)(32)/144

TOTAL = 171 $ft^3$

TOTAL Shear WALL LENGTH = 127'

## Mass / unit area

2) Area = 24000 = 0.00683 m/A
mass = 164

3) Area = 27200 0.006397 m/A
mass = 174

TYP) AREA = 24500 0.00633 m/A
mass = 155

12) Area = 24500 0.00637 m/A
mass = 156

PH) Area = 22333 0.00466 m/A
mass = 104

LR) Area = 12800 0.00428 m/A
mass = 54.8

HR) Area = 2688 0.00339 m/A
mass = 9.1

## 11.4. Foundation Analysis

Hand Calculations:

FOUNDATION

Check Loads:

- compressive max load = $1.2D + 1.6L + 0.5S$
- max uplif from $0.9D + 1.6W$

Uplift Forces: F3 & F3.8 WF

$M_u = \frac{44286}{24} = 1845.25^k$

$P_u = 0.9(2205) + 0.9(508/2) = 2213.1$

$V_{uplift} = +1845.25^k + 2231.1 = 385.85$ (c)

No uplift

Uplift Forces: C3 & C4.1 WC

$M_u = \frac{56939}{31.5} = 1808^k$ (T)

$P_u = 0.9(1918) + 0.9(688) = 2345$ (c)

$V_{uplift} = 0^k$ ($537^k$(c))

Uplift Forces: C3 & D3 W3

$M_u = \frac{31768}{30} = 1059^k$ (T)

$P_u = (310)(.9) + (1918)(.9) = 2005.2^k$

$Uplift = 0^k$ ($946.2^k$) (c)

Bott Bearing capacity = A · ksf
shaft weight = A(L)(150psf)/1000
net soil capacity = Total - shaft weight
concrete capacity = (0.25)(4.5)(A)

$Q_{all}$ = 160 ksf *assume F.S. = 3
Skin Friction = 0 ksf
$f'_c$ = 4ksi

# APPENDIX B

# MECHANICAL CALCULATIONS

Existing Duct System Pressure Loss Calculation Table:

| Number | Section | CFM | Size | Velocity | Velocity Pressure | FL Coeff. | P/L | Length | Delta P |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Diffuser | 127 | 34 | - | - | - | - | - | 0.10 |
| 2 | Duct | 127 | 5x10 | 366 | - | - | 0.01 | 3.16 | 0.00 |
| 4 | Duct | 381 | 10x10 | 549 | - | - | 0.04 | 14.22 | 0.00 |
| 5 | Tee | 762 | - | 549 | 0.02 | 0.08 | - | - | 0.00 |
| 6 | Duct | 762 | 16x12 | 572 | - | - | 0.04 | 12.64 | 0.01 |
| 7 | P9 | - | - | - | - | - | 0.40 | - | 0.40 |
| 8 | Duct | 590 | 8 | 1691 | - | - | 0.55 | 3.16 | 0.02 |
| 9 | Tee | 590 | - | 1691 | 0.18 | 0.87 | - | - | 0.16 |
| 10 | Duct | 590 | 10 | 1082 | - | - | 0.18 | 19 | 0.03 |
| 11 | Radius | 1030 | 12 | 1312 | 0.11 | 0.34 | - | - | 0.04 |
| 12 | Tee | 1670 | - | 1312 | 0.11 | 1.18 | - | - | 0.13 |
| 13 | Duct | 1670 | 13 | 1813 | - | - | 0.25 | 19 | 0.05 |
| 14 | Tee | 2370 | - | 1813 | 0.20 | 0.85 | - | - | 0.17 |
| 15 | Duct | 2370 | 14 | 2218 | - | - | 0.40 | 11.85 | 0.05 |
| 16 | Tee | 2790 | - | 2218 | 0.31 | 5.17 | - | - | 1.59 |
| 17 | Duct | 2790 | 15 | 2275 | - | - | 0.40 | 47.4 | 0.19 |
| 18 | Radius | 2790 | 15 | 2275 | 0.32 |  | - | - | 0.00 |
| 19 | Duct | 2790 | 15 | 2275 | - | 5.17 | 0.40 | 8 | 0.03 |
| 20 | Tee | 3340 | - | 2275 | 0.32 |  | - | - | 0.00 |
| 21 | Duct | 3340 | 15 | 2723 | - | - | 0.51 | 12.64 | 0.06 |
| 22 | Radius | 3340 | 15 | 2723 | 0.46 | 0.33 | - | - | 0.15 |
| 23 | Duct | 3340 | 15 | 2723 | - | - | 0.51 | 5.53 | 0.03 |
| 24 | 90 Elbow | 3340 | - | 2723 | 0.46 | 0.25 |  | - | 0.12 |
| 25 | Inlet | 3340 | - | 2723 | 0.46 |  |  |  | 0.00 |
| **26** | **AHU** | **17000** | **-** | **2723** | **-** | **-** |  | **-** | **0.82** |
| **Total** |  |  |  |  |  |  |  |  | **4.14** |

Proposed Duct System Pressure Loss Calculation Table:

| Number | Section | CFM | Size | Velocity | Velocity Pressure | FL Coeff. | P/L | Length | Delta P |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Diffuser | 127 | 34 | - | - | - | - | - | 0.10 |
| 2 | Duct | 127 | 5x10 | 366 | - | - | 0.01 | 3.16 | 0.00 |
| 4 | Duct | 381 | 10x10 | 549 | 0.02 | - | 0.04 | 14.22 | 0.00 |
| 5 | Tee | 762 | - | 549 | 0.02 | 0.08 | - | - | 0.00 |
| 6 | Duct | 762 | 16x12 | 572 | 0.02 | - | 0.04 | 12.64 | 0.01 |
| 7 | P9 | - | - | - | - | - | 0.40 | - | 0.40 |
| 8 | Duct | 590 | 8 | 1691 | - | - | 0.55 | 3.16 | 0.02 |
| 9 | Tee | 590 | - | 1691 | 0.18 | 0.87 | - | - | 0.16 |
| 10 | Duct | 590 | 10 | 1082 | - | - | 0.18 | 19 | 0.03 |
| 11 | Radius | 1030 | 12 | 1312 | 0.11 | 0.34 | - | - | 0.04 |
| 12 | Tee | 1670 | - | 1312 | 0.11 | 0.94 | - | - | 0.10 |
| 13 | Duct | 1670 | 14 | 1563 | - | - | 0.20 | 19 | 0.04 |
| 14 | Tee | 2370 | - | 1563 | 0.15 | 0.80 | - | - | 0.12 |
| 15 | Duct | 2370 | 16 | 1698 | - | - | 0.20 | 11.85 | 0.02 |
| 16 | Tee | 2790 | - | 1698 | 0.18 | 2.73 | - | - | 0.49 |
| 17 | Duct | 2790 | 18 | 1580 | - | - | 0.18 | 47.4 | 0.09 |
| 18 | Radius | 2790 | 18 | 1580 | 0.16 | 0.32 | - | - | 0.05 |
| 19 | Duct | 2790 | 18 | 1580 | - | - | 0.18 | 8 | 0.01 |
| 20 | Tee | 3340 | - | 1580 | 0.16 | 2.73 | - | - | 0.42 |
| 21 | Duct | 3340 | 20 | 1532 | - | - | 0.14 | 12.64 | 0.02 |
| 22 | Radius | 3340 | 20 | 1532 | 0.15 | 0.32 | - | - | 0.05 |
| 23 | Duct | 3340 | 20 | 1532 | - | - | 0.14 | 5.53 | 0.01 |
| 24 | 90 Elbow | 3340 | - | 1532 | 0.15 | 0.24 | - | - | 0.04 |
| 25 | Inlet | 3340 | - | 1532 | 0.15 |  | - | - |  |
| 26 | AHU | 17000 | - | 1532 | - | - | - | - | 0.82 |
| **Total** |  |  |  |  |  |  |  |  | **3.03** |

Fitting Loss Summary:

| Fitting # | Type | ASHRAE | New Loss Coeff. | Existing Loss Coeff. |
|---|---|---|---|---|
| 5 | Tee,Branch | | | |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.08** | **0.08** |
| 9 | Tee,Branch | SD5-9 | 0.79 | 0.79 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.87** | **0.87** |
| 11 | Elbow | CD3-5 | 0.26 | 0.26 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.34** | **0.34** |
| 12 | Tee,Branch | SD5-9 | 0.86 | 1.1 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.94** | **1.18** |
| 14 | Tee,Branch | SD5-9 | 0.72 | 0.77 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.8** | **0.85** |
| 16 | Tee,Branch | SD5-9 | 2.65 | 5.09 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **2.73** | **5.17** |
| 18 | Elbow | CD3-5 | 0.24 | 0.25 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.32** | **0.33** |
| 20 | Tee,Branch | SD5-9 | 2.65 | 5.09 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **2.73** | **5.17** |
| 22 | Elbow | CD3-5 | 0.24 | 0.25 |
| | Damper | CR9-1 | 0.08 | 0.08 |
| | Sum | | **0.32** | **0.33** |

Fitting Loss Calculations:

Fitting Parameters (New)

5 $A_s =$ $A_s/A_c =$

$A_c =$ $A_b/A_c =$

$A_b =$ $Q_b/Q_c =$

9\. $A_s = 10\phi = 0.55$ $A_s/A_c = 1.0$

$A_c = 10\phi = 0.55$ $A_b/A_c = 0.64$

$A_b = 0.35$ $Q_b/Q_c = 0.9$ $\boxed{C_b = 0.19}$

11\. $r/D = 1.5$ $\boxed{C_o = 0.26}$

pleated

$\theta = 90°$

$D = 12"$

12 $A_s = 14\phi = 1.07$ $A_s/A_c = 0.73$

$A_c = 12\phi = 0.785$ $A_b/A_c = 0.33$

$A_b = 0.35$ $Q_b/Q_c = 0.43$ $\boxed{C_b = 0.86}$

14\. $A_s = 14\phi = 1.07$ $A_b/A_c = 0.25$

$A_c = 16\phi = 1.40$ $Q_b/Q_c = 0.38$

$A_b = 0.35$ $C_b = \boxed{0.12}$

16\. $A_s = 16\phi = 1.40$ $A_b/A_c = 0.20$

$A_c = 18\phi = 1.77$ $Q_b/Q_c = 0.15$ $\boxed{C_b = 2.65}$

$A_b = 0.35$

26. $A_b = 0.35$ $A_b/A_c = 0.20$

$A_s = 18\phi = 1.77$ $Q_b/Q_c = 0.15$

$A_c = 18\phi = 1.77$ $\boxed{C_b = 2.65}$

<u>Fitting Parameters (Existing)</u>

12. $A_b = 0.35$ $A_s/A_c = 0.38$

$A_c = 13\phi = 0.92$ $Q_b/Q_c = 0.43$ $\boxed{C_b = 1.10}$

14. $A_b = 0.35$ $A_b/A_c = 0.33$

$A_c = 14\phi \rightarrow 1.07$ $Q_b/Q_c = 0.38$ $\boxed{C_b = 0.77}$

16. $A_b = 0.35$ $A_b/A_c = 0.28$

$A_c = 15\phi = 1.23$ $Q_b/Q_c = 0.15$ $\boxed{C_b = 5.09}$

20 $A_b = 0.35$ $A_b/A_c = 0.28$

$A_c = 15\phi$ $Q_b/Q_c = 0.15$ $\boxed{C_b = 5.09}$

<u>Total static Pressures</u>

Existing: $4.14 - \overset{\text{assumed}}{\underline{0.5}} = 3.64''$

New System: $3.03 - \underline{0.2} = 2.83''$

Fan outlet velocity pressure: 0.5
0.2

Friction Loss Selection Table:

35.8 2005 ASHRAE Handbook—Fundamentals

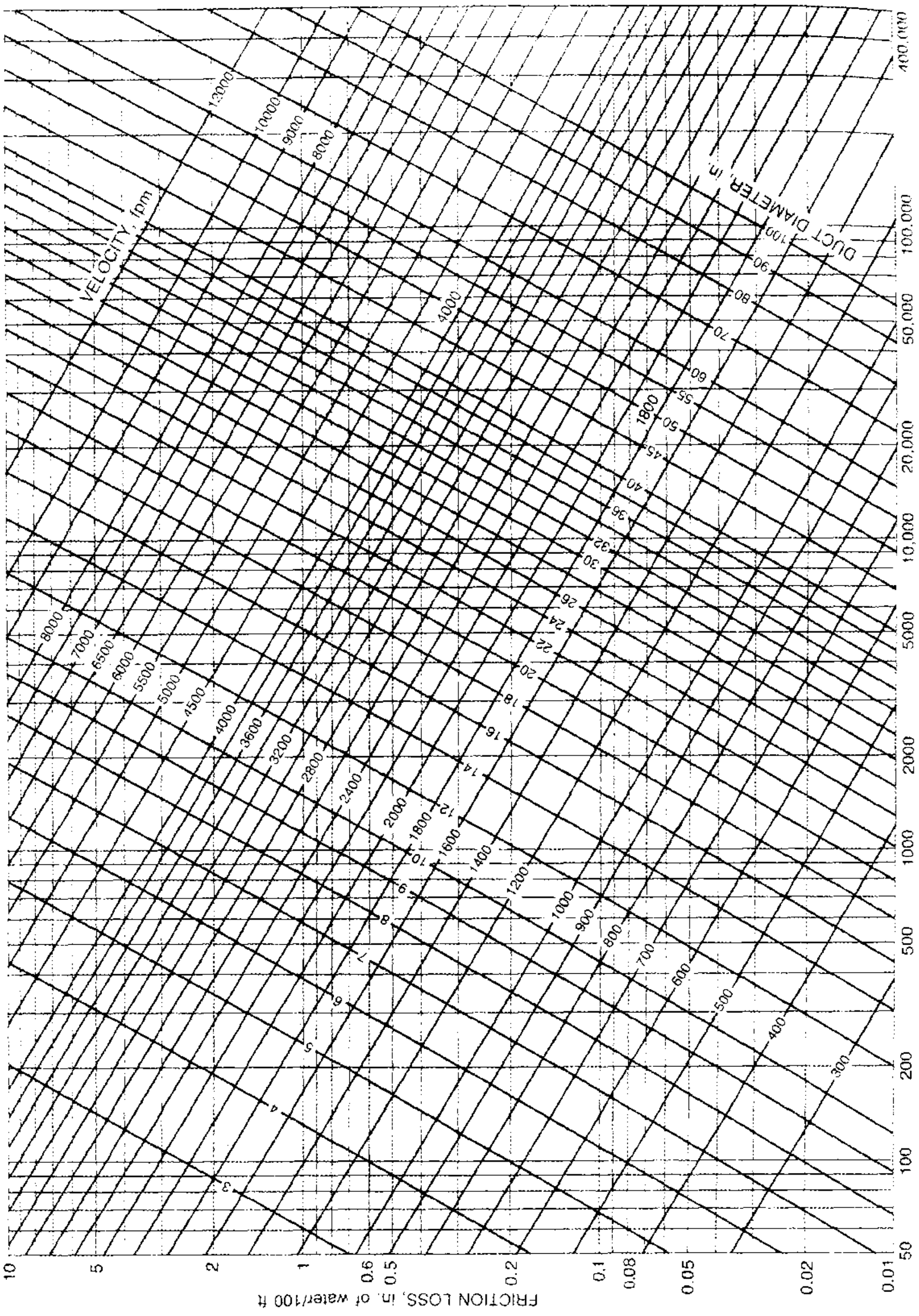


Fig. 9 Friction Chart for Round Duct (ρ = 0.075 lbm/ft³ and ε = 0.0003 ft)

# APPENDIX C

# COST & SCHEDULE ANALYSIS

## Existing Building Cost:

| Description | Crew | Daily Output | Labor Hours | Unit of Meas. | Quantity | Unit Mat Cost | Mat Cost | Unit Labor Cost | Labor Cost | Unit Equip/Sub Cost | Item Cost |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **FLOOR ASSEMBLY** | | | | | | | | | | | |
| Floor 3 thru 12 | - | - | - | SF | 24500 | 13.95 | 341775 | 6.1 | 149450 | - | $4,912,250.00 |
| Floor 2 | - | - | - | SF | 22000 | 13.95 | 306900 | 6.1 | 134200 | - | $441,100.00 |
| PH | - | - | - | SF | 13533 | 13.95 | 188785.4 | 6.1 | 82551 | - | $271,336.65 |
| Roof Decking | E-4 | 4170 | 0.008 | SF | 24500 | 1.51 | 36995 | 0.33 | 8085 | 735 | $45,815.00 |
| **COLUMNS** | | | | | | | | | | | |
| W14 X176 | E-2 | 912 | 0.061 | LF | 940 | 213 | 200220 | 2.57 | 2415.8 | 1616.8 | $204,252.60 |
| W14x120 | E-2 | 960 | 0.058 | LF | 1032 | 145 | 149640 | 2.44 | 2518.1 | 1682.16 | $153,840.24 |
| W14X74 | E-2 | 984 | 0.057 | LF | 2050 | 89.5 | 183475 | 2.38 | 4879 | 3259.5 | $191,613.50 |
| W12x120 | E-2 | 960 | 0.058 | LF | 176 | 145 | 25520 | 2.44 | 429.44 | 286.88 | $26,236.32 |
| W12X87 | E-2 | 984 | 0.057 | LF | 482 | 105 | 50610 | 2.38 | 1147.2 | 766.38 | $52,523.54 |
| W12x50 | E-2 | 1032 | 0.054 | LF | 735 | 60.5 | 44467.5 | 2.27 | 1668.5 | 1117.2 | $47,253.15 |
| W10x68 | E-2 | 984 | 0.057 | LF | 200 | 82.5 | 16500 | 2.38 | 476 | 338 | $17,314.00 |
| W10x45 | E-2 | 1032 | 0.054 | LF | 670 | 54.5 | 36515 | 2.27 | 1520.9 | 1018.4 | $39,054.30 |
| **BRACES** | | | | | | | | | | | |
| W14x74 | E-2 | 984 | 0.057 | LF | 147.44 | 89.5 | 13195.88 | 2.38 | 350.91 | 234.43 | $13,781.22 |
| W12X87 | E-2 | 984 | 0.057 | LF | 161.6 | 105 | 16968 | 2.38 | 384.61 | 256.944 | $17,609.55 |
| W12X50 | E-2 | 1032 | 0.054 | LF | 121.2 | 60.5 | 7332.6 | 2.27 | 275.12 | 184.224 | $7,791.95 |
| W10X68 | E-2 | 984 | 0.057 | LF | 202 | 82.5 | 16665 | 2.38 | 480.76 | 341.38 | $17,487.14 |
| W10X45 | E-2 | 1032 | 0.054 | LF | 606 | 54.5 | 33027 | 2.27 | 1375.6 | 921.12 | $35,323.74 |
| W8x48 | E-3 | 1032 | 0.054 | LF | 444.4 | 58 | 25775.2 | 2.27 | 1008.8 | 675.488 | $27,459.48 |
| W8x31 | E-2 | 1080 | 0.052 | LF | 888.8 | 37.5 | 33330 | 2.17 | 1928.7 | 1288.76 | $36,547.46 |
| **PUNCHED HOLES** | | | | | | | | | | | |
| **Unreinforced** | | | | hole | 203 | 60 | 12180 | | | | 146160 |
| **Reinforced** | | | | hole | 2 | 170 | 340 | | | | 4080 |

*Does not include added cost of moment connections

| | |
|---|---|
| **TOTAL** | **$6,708,829.83** |

Proposed Building Cost:

| Description | Crew | Daily Output | Labor Hours | Unit of Meas. | Quantity |
|---|---|---|---|---|---|
| **FLOOR ASSEMBLY** | | | | | |
| Floor 3 thru 12 | | | | | |
| Concrete/Placement | C-20 | 180 | 0.356 | CY | 855.8 |
| Post-Tensioning | C-4 | 1475 | 0.022 | LB | 27070 |
| Formwork | C-2 | 560 | 0.086 | SFCA | 24500 |
| Mild Steel Reinf. | 4 Rodm | 2.9 | 11.034 | TON | 7.859 |
| Total | | | | | |
| Floor 2 | | | | | |
| Concrete/Placement | C-20 | 180 | 0.356 | CY | 768 |
| Post-Tensioning | C-4 | 1475 | 0.022 | LB | 24363 |
| Formwork | C-2 | 560 | 0.086 | SFCA | 22050 |
| Mild Steel Reinf. | 4 Rodm | 2.9 | 11.034 | TON | 7.07 |
| Penthouse | | | | | |
| Concrete/Placement | C-20 | 180 | 0.356 | CY | 473 |
| Post-Tensioning | C-4 | 1475 | 0.022 | LB | 14889 |
| Formwork | C-2 | 560 | 0.086 | SFCA | 13475 |
| Mild Steel Reinf. | 4 Rodm | 2.9 | 11.034 | TON | 4.174 |
| Roof | | | | | |
| Concrete/Placement | C-20 | 180 | 0.356 | CY | 713.2 |
| Post-Tensioning | C-4 | 1475 | 0.022 | LB | 27070 |
| Formwork | C-2 | 560 | 0.086 | SFCA | 24500 |
| Mild Steel Reinf. | 4 Rodm | 2.9 | 11.034 | TON | 7.859 |
| **COLUMN** | | | | | |
| Exterior | C-14A | 17.71 | 11.293 | CY | 563 |
| Interior | C-14A | 23.32 | 8.576 | CY | 613 |
| **SHEAR WALLS** | | | | | |
| Concrete | | | | CY | 968 |
| Placing | C-6 | 100 | 0.48 | CY | 968 |
| Formwork | C-2 | 395 | 0.122 | SFCA | 57026 |
| Reinforcement | 4 Rodm | 3 | 10.667 | TON | 32.714 |
| **FOUNDATION (ADDITIONAL)** | | | | | |
| 4'-0" to 5'-0" | - | - | - | Each | 6 |
| 3'-0" to 4'-0" | - | - | - | Each | 7 |
| 2'-6" to 4'-0" | - | - | - | Each | 5 |

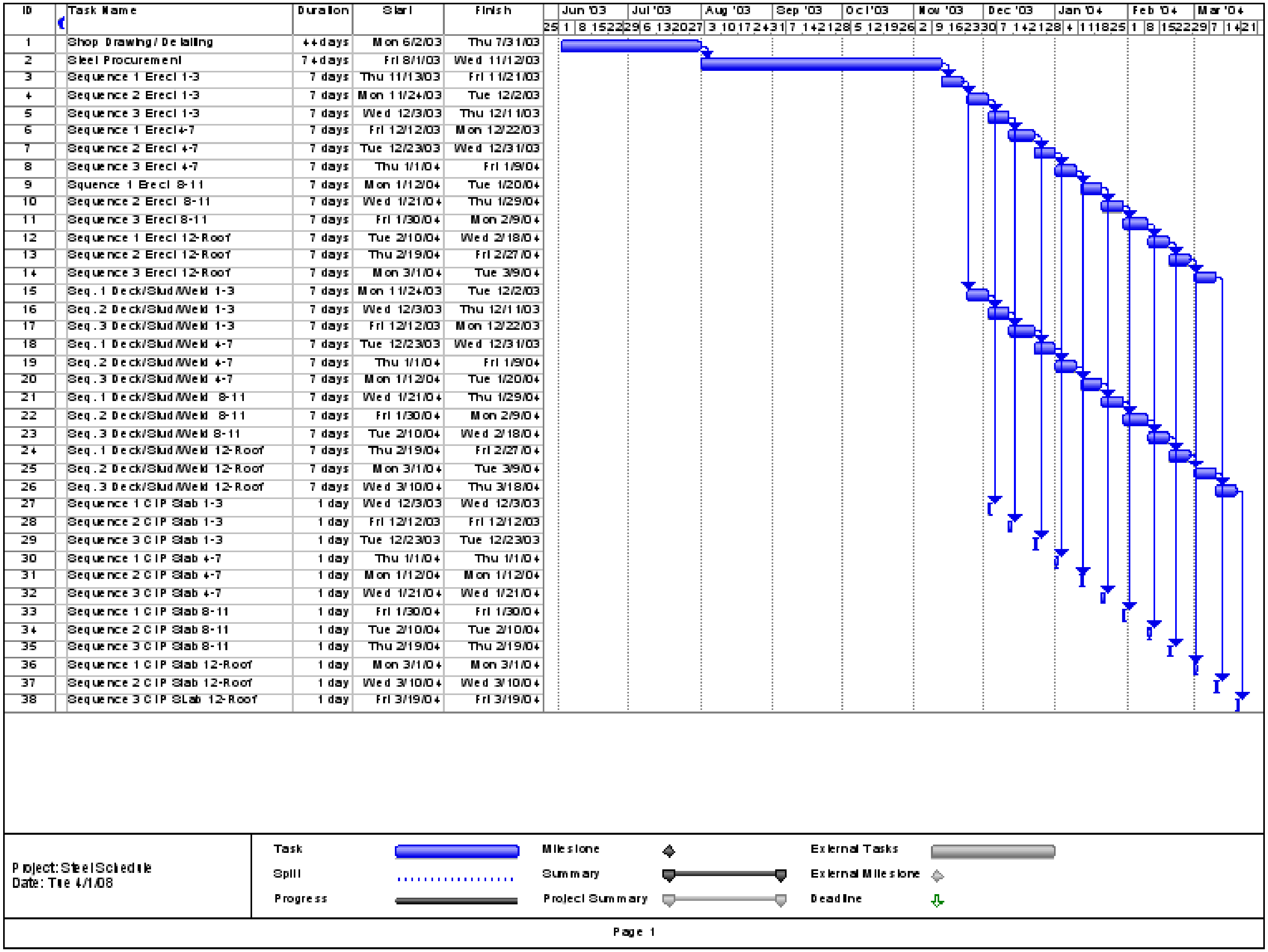
*Roof assumes 10" slab

*Reinforcement increased by 10% in walls to consider ties and coupling beams

*Slab on grade and overhead costs not considered b/c same values between both systems

| Unit Mat Cost | Mat Cost | Unit Labor Cost | Labor Cost | Unit Equip/Sub Cost | Item Cost |
|---|---|---|---|---|---|
| | | | | | |
| | | | | | |
| 109 | 93282.2 | 50 | 42790 | 3697.1 | $139,769.26 |
| 1.95 | 52786.5 | 0.69 | 18678.3 | - | $71,464.80 |
| 1.42 | 34790 | 3.18 | 77910 | - | $112,700.00 |
| 990 | 7780.41 | 475 | 3733.03 | - | $11,513.44 |
| | | | | | $3,354,474.91 |
| | | | | | |
| 109 | 83712 | 50 | 38400 | 3317.8 | $125,429.76 |
| 1.95 | 47507.9 | 0.69 | 16810.5 | - | $64,318.32 |
| 1.42 | 31311 | 3.18 | 70119 | - | $101,430.00 |
| 990 | 6999.3 | 475 | 3358.25 | - | $10,357.55 |
| | | | | | |
| 109 | 51557 | 50 | 23650 | 2043.4 | $77,250.36 |
| 1.95 | 29033.6 | 0.69 | 10273.4 | - | $39,306.96 |
| 1.42 | 19134.5 | 3.18 | 42850.5 | - | $61,985.00 |
| 990 | 4132.26 | 475 | 1982.65 | - | $6,114.91 |
| | | | | | |
| 109 | 77738.8 | 50 | 35660 | 3081 | $116,479.82 |
| 1.95 | 52786.5 | 0.69 | 18678.3 | - | $71,464.80 |
| 1.42 | 34790 | 3.18 | 77910 | - | $112,700.00 |
| 990 | 7780.41 | 475 | 3733.03 | - | $11,513.44 |
| | | | | | |
| 410 | 230830 | 435 | 244905 | 23928 | 499662.5 |
| 360 | 220680 | 330 | 202290 | 19923 | 442892.5 |
| | | | | | |
| 124 | 120032 | - | - | | 120032 |
| - | - | 15.2 | 14713.6 | 474.32 | 15187.92 |
| 0.77 | 43910 | 4.51 | 257187 | - | 301097.28 |
| 890 | 29115.5 | 460 | 15048.4 | - | 44163.9 |
| | | | | | |
| 8775 | 52650 | 4550 | 27300 | - | 79950 |
| 2950 | 20650 | 1725 | 12075 | - | 32725 |
| 6125 | 30625 | 6700 | 33500 | - | 64125 |
| | | | | **TOTAL** | **$5,752,661.93** |

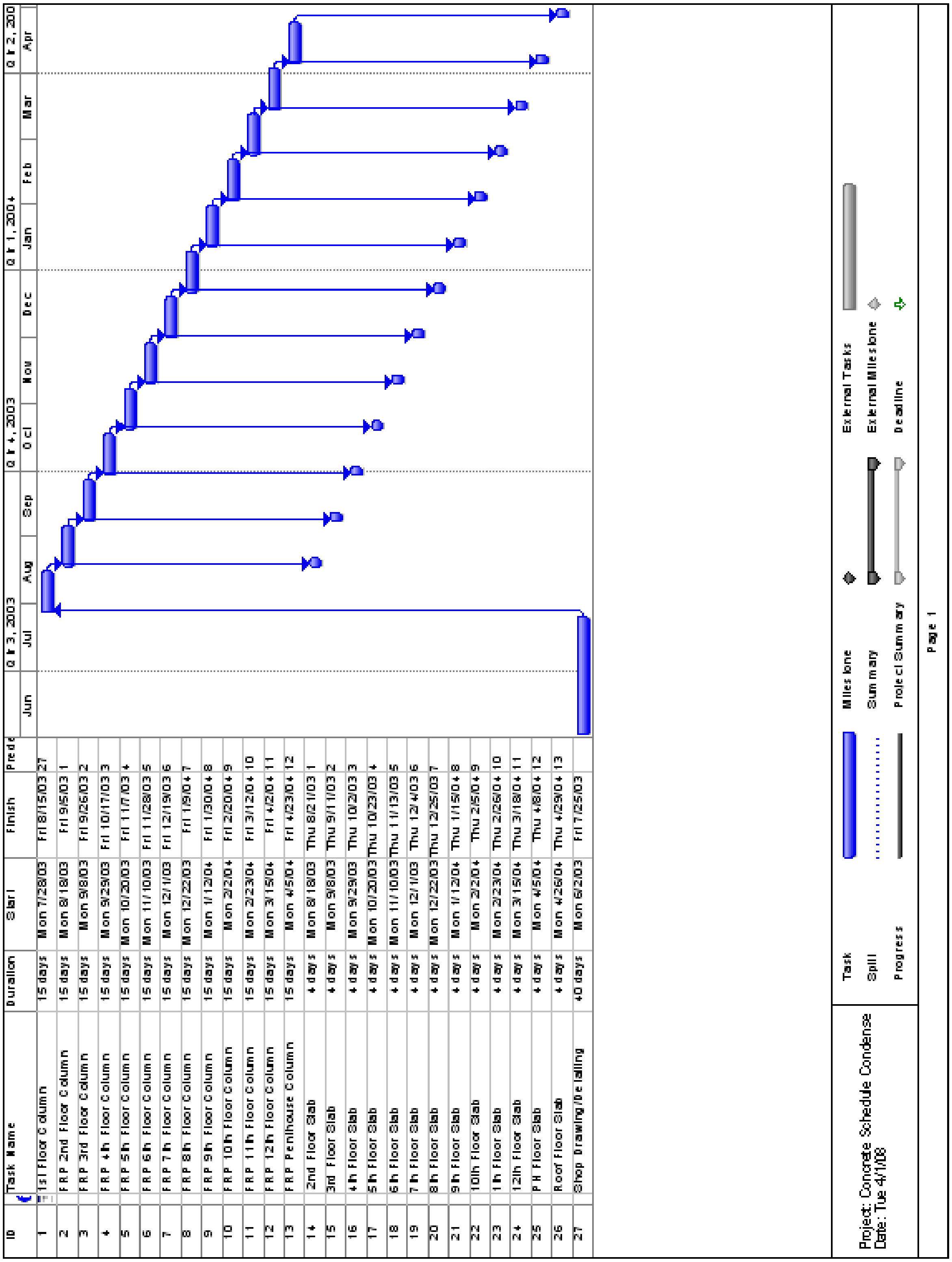
Existing Building Schedule:



| ID | Task Name | Duration | Start | Finish |
|---|---|---|---|---|
| 1 | Shop Drawing/ Detailing | 44 days | Mon 6/2/03 | Thu 7/31/03 |
| 2 | Steel Procurement | 74 days | Fri 8/1/03 | Wed 11/12/03 |
| 3 | Sequence 1 Erect 1-3 | 7 days | Thu 11/13/03 | Fri 11/21/03 |
| 4 | Sequence 2 Erect 1-3 | 7 days | Mon 11/24/03 | Tue 12/2/03 |
| 5 | Sequence 3 Erect 1-3 | 7 days | Wed 12/3/03 | Thu 12/11/03 |
| 6 | Sequence 1 Erect 4-7 | 7 days | Fri 12/12/03 | Mon 12/22/03 |
| 7 | Sequence 2 Erect 4-7 | 7 days | Tue 12/23/03 | Wed 12/31/03 |
| 8 | Sequence 3 Erect 4-7 | 7 days | Thu 1/1/04 | Fri 1/9/04 |
| 9 | Squence 1 Erect 8-11 | 7 days | Mon 1/12/04 | Tue 1/20/04 |
| 10 | Sequence 2 Erect 8-11 | 7 days | Wed 1/21/04 | Thu 1/29/04 |
| 11 | Sequence 3 Erect 8-11 | 7 days | Fri 1/30/04 | Mon 2/9/04 |
| 12 | Sequence 1 Erect 12-Roof | 7 days | Tue 2/10/04 | Wed 2/18/04 |
| 13 | Sequence 2 Erect 12-Roof | 7 days | Thu 2/19/04 | Fri 2/27/04 |
| 14 | Sequence 3 Erect 12-Roof | 7 days | Mon 3/1/04 | Tue 3/9/04 |
| 15 | Seq.1 Deck/Stud/Weld 1-3 | 7 days | Mon 11/24/03 | Tue 12/2/03 |
| 16 | Seq.2 Deck/Stud/Weld 1-3 | 7 days | Wed 12/3/03 | Thu 12/11/03 |
| 17 | Seq.3 Deck/Stud/Weld 1-3 | 7 days | Fri 12/12/03 | Mon 12/22/03 |
| 18 | Seq.1 Deck/Stud/Weld 4-7 | 7 days | Tue 12/23/03 | Wed 12/31/03 |
| 19 | Seq.2 Deck/Stud/Weld 4-7 | 7 days | Thu 1/1/04 | Fri 1/9/04 |
| 20 | Seq.3 Deck/Stud/Weld 4-7 | 7 days | Mon 1/12/04 | Tue 1/20/04 |
| 21 | Seq.1 Deck/Stud/Weld 8-11 | 7 days | Wed 1/21/04 | Thu 1/29/04 |
| 22 | Seq.2 Deck/Stud/Weld 8-11 | 7 days | Fri 1/30/04 | Mon 2/9/04 |
| 23 | Seq.3 Deck/Stud/Weld 8-11 | 7 days | Tue 2/10/04 | Wed 2/18/04 |
| 24 | Seq.1 Deck/Stud/Weld 12-Roof | 7 days | Thu 2/19/04 | Fri 2/27/04 |
| 25 | Seq.2 Deck/Stud/Weld 12-Roof | 7 days | Mon 3/1/04 | Tue 3/9/04 |
| 26 | Seq.3 Deck/Stud/Weld 12-Roof | 7 days | Wed 3/10/04 | Thu 3/18/04 |
| 27 | Sequence 1 CIP Slab 1-3 | 1 day | Wed 12/3/03 | Wed 12/3/03 |
| 28 | Sequence 2 CIP Slab 1-3 | 1 day | Fri 12/12/03 | Fri 12/12/03 |
| 29 | Sequence 3 CIP Slab 1-3 | 1 day | Tue 12/23/03 | Tue 12/23/03 |
| 30 | Sequence 1 CIP Slab 4-7 | 1 day | Thu 1/1/04 | Thu 1/1/04 |
| 31 | Sequence 2 CIP Slab 4-7 | 1 day | Mon 1/12/04 | Mon 1/12/04 |
| 32 | Sequence 3 CIP Slab 4-7 | 1 day | Wed 1/21/04 | Wed 1/21/04 |
| 33 | Sequence 1 CIP Slab 8-11 | 1 day | Fri 1/30/04 | Fri 1/30/04 |
| 34 | Sequence 2 CIP Slab 8-11 | 1 day | Tue 2/10/04 | Tue 2/10/04 |
| 35 | Sequence 3 CIP Slab 8-11 | 1 day | Thu 2/19/04 | Thu 2/19/04 |
| 36 | Sequence 1 CIP Slab 12-Roof | 1 day | Mon 3/1/04 | Mon 3/1/04 |
| 37 | Sequence 2 CIP Slab 12-Roof | 1 day | Wed 3/10/04 | Wed 3/10/04 |
| 38 | Sequence 3 CIP SLab 12-Roof | 1 day | Fri 3/19/04 | Fri 3/19/04 |

Project: Steel Schedule
Date: Tue 4/1/08

Proposed Building Schedule:



| ID | Task Name | Duration | Start | Finish | Prede |
|---|---|---|---|---|---|
| 1 | 1st Floor Column | 15 days | Mon 7/28/03 | Fri 8/15/03 | 27 |
| 2 | FRP 2nd Floor Column | 15 days | Mon 8/18/03 | Fri 9/5/03 | 1 |
| 3 | FRP 3rd Floor Column | 15 days | Mon 9/8/03 | Fri 9/26/03 | 2 |
| 4 | FRP 4th Floor Column | 15 days | Mon 9/29/03 | Fri 10/17/03 | 3 |
| 5 | FRP 5th Floor Column | 15 days | Mon 10/20/03 | Fri 11/7/03 | 4 |
| 6 | FRP 6th Floor Column | 15 days | Mon 11/10/03 | Fri 11/28/03 | 5 |
| 7 | FRP 7th Floor Column | 15 days | Mon 12/1/03 | Fri 12/19/03 | 6 |
| 8 | FRP 8th Floor Column | 15 days | Mon 12/22/03 | Fri 1/9/04 | 7 |
| 9 | FRP 9th Floor Column | 15 days | Mon 1/12/04 | Fri 1/30/04 | 8 |
| 10 | FRP 10th Floor Column | 15 days | Mon 2/2/04 | Fri 2/20/04 | 9 |
| 11 | FRP 11th Floor Column | 15 days | Mon 2/23/04 | Fri 3/12/04 | 10 |
| 12 | FRP 12th Floor Column | 15 days | Mon 3/15/04 | Fri 4/2/04 | 11 |
| 13 | FRP Penthouse Column | 15 days | Mon 4/5/04 | Fri 4/23/04 | 12 |
| 14 | 2nd Floor Slab | 4 days | Mon 8/18/03 | Thu 8/21/03 | 1 |
| 15 | 3rd Floor Slab | 4 days | Mon 9/8/03 | Thu 9/11/03 | 2 |
| 16 | 4th Floor Slab | 4 days | Mon 9/29/03 | Thu 10/2/03 | 3 |
| 17 | 5th Floor Slab | 4 days | Mon 10/20/03 | Thu 10/23/03 | 4 |
| 18 | 6th Floor Slab | 4 days | Mon 11/10/03 | Thu 11/13/03 | 5 |
| 19 | 7th Floor Slab | 4 days | Mon 12/1/03 | Thu 12/4/03 | 6 |
| 20 | 8th Floor Slab | 4 days | Mon 12/22/03 | Thu 12/25/03 | 7 |
| 21 | 9th Floor Slab | 4 days | Mon 1/12/04 | Thu 1/15/04 | 8 |
| 22 | 10th Floor Slab | 4 days | Mon 2/2/04 | Thu 2/5/04 | 9 |
| 23 | 1th Floor Slab | 4 days | Mon 2/23/04 | Thu 2/26/04 | 10 |
| 24 | 12th Floor Slab | 4 days | Mon 3/15/04 | Thu 3/18/04 | 11 |
| 25 | PH Floor Slab | 4 days | Mon 4/5/04 | Thu 4/8/04 | 12 |
| 26 | Roof Floor Slab | 4 days | Mon 4/26/04 | Thu 4/29/04 | 13 |
| 27 | Shop Drawing/Detailing | 40 days | Mon 6/2/03 | Fri 7/25/03 | |

Project: Concrete Schedule Condense
Date: Tue 4/1/08

*This is a condensed version.  Full schedule is available upon request.