Final Report

MICA Gateway Residence

Scott R. Molongoski

Structural Option ~ Heather Sustersic ~ April 3, 2013

MICA GATEWAY RESIDENCE

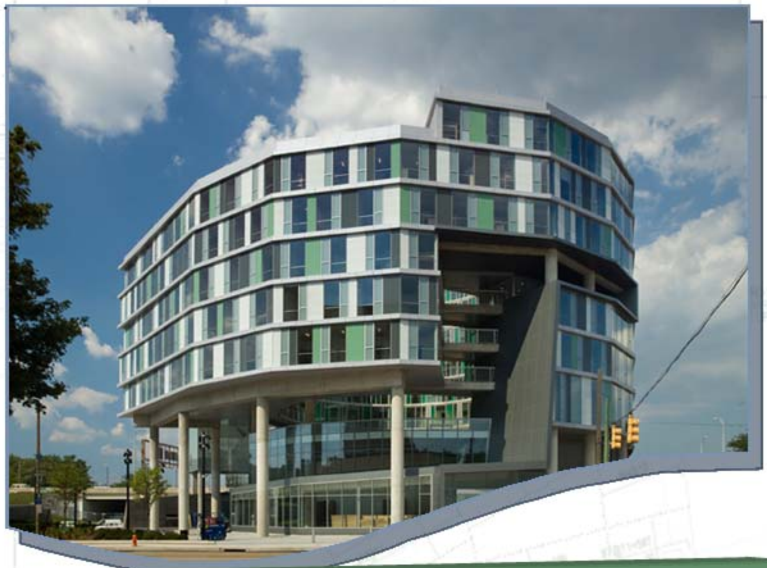
~ BALTIMORE, MARYLAND ~

~ SCOTT R MOLONGOSKI ~

~ STRUCTURAL ~

~ CPEP SITE ~

http://www.engr.psu.edu/ae/thesis/portfolios/2013/srm5167/index.html



ARCHITECTURE:

- Signature building of MICA campus
- 9 stories and a mechanical penthouse
- 64 dormitory rooms, studio space, galleries, conference room, café, “black box” theater
- Full 360 degree façade with white, gray, and mint green glazing
- Central courtyard begins on the third floor

STRUCTURE:

- Primarily concrete superstructure
- Two-way flat plate slab floor systems
- Long span beams 48x48 support the enclosed courtyard & “black-box” theater ceiling
- Slender columns nearly 40’ in height
- Ordinary concrete shear walls provide lateral support

MECHANICAL:

- Variable Air Volume system
- 4 air handling units with 9600-14500 cfm supply
- 2 natural gas cast iron boilers with min. output of 1632 mbh
- 2 200 ton screw type chillers

GENERAL INFORMATION:

BUILDING SIZE: 108,000 sqft

BUILDING HEIGHT: 122 ft tall

CONSTRUCTED: Aug.2006– Aug.2008

DELIVERY: Design-Bid-Build

COST: $30 Million

PROJECT TEAM:

OWNER: Maryland Institute College of Art

ARCHITECT/ENGINEER: RTKL

GC/CM: Whiting Turner

CIVIL: KCW Engineering

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Acknowledgements:

I would personally like to thank the Maryland Institute College of Art for allowing me to use their Gateway Residence hall as my senior thesis project. I would also like to thank RTKL Associates of Baltimore, MD for their assistance in helping me complete the project. I would specifically like to thank Bob Knight, Peter Malmquist, and David Dymond.

I especially want to thank Adam Reed of Penn State Facilities Engineering Institute for helping me obtain the Gateway building as a thesis project.

In addition I would like to thank the following Penn State Architectural Engineering professors:

Professor Heather Sustersic

Professor Kevin Parfitt

Professor Bob Holland

And the entire AE faculty and staff

Executive Summary:

This Final Thesis Report is the culmination of a yearlong study of the MICA Gateway Residence. The Gateway is a residence hall at the Maryland Institute College of Arts designed to be a cornerstone of their campus in downtown Baltimore, Maryland. Gateway is 122' tall, with 9 stories and a mechanical penthouse and has a useable floor area of 108,000 square feet. The Gateway is primarily circular in plan with a rectangular tower on the side that faces a highway. The circle, or drum component of the building encloses an open-air courtyard that actually begins on the third floor of the structure. This plaza is located directly above a large "black-box" multipurpose room capable of multiple arrangements to fit a variety of functions. Beyond the multipurpose assembly room, Gateway features 64 student apartments, several art galleries and studios, and a café. The Gateway was designed by RTKL Associates and KCW Engineering Technologies and was built by Whiting Turner.

The proposal for this thesis was an investigation of the necessary structural design changes required to design the Gateway as a museum rather than as a residence hall. The live loads for a museum are considerably higher than those for a residence hall. This fact led to changes in the gravity system of the structure, specifically an increase in strength of the concrete floor slabs. The increased live and dead loads, along with architectural changes that increase the floor-to-floor height of the structure, necessitate new designs for the columns, including new sizes, reinforcement, and slenderness checks.

Changes to the gravity system and the overall number of floors required an in-depth look at the Gateway lateral system. Under the new loading conditions and gravity structure it was important to check if the shear walls could resist the new lateral loads and to redesign them if they could not. Changes in the overall superstructure of the building also required that the foundation of the building be assessed to handle the new loading conditions.

In addition to the structural depth of this thesis, two breadth topics were also studied. An architectural breadth was chosen due to the many changes required to make the Gateway adequate for a museum. Changes to the floor plan, circulation of people, elevations, and façade were all analyzed. Sustainability was chosen as the second breadth due to the Gateway's lack of sustainable features. A green roof was designed for the building as an additional architectural component as well as a sustainable measure to reach LEED certification. Several LEED credits were also deemed feasible to achieve in the Gateway, thus accumulating enough credits for the building to become LEED certified.

Building Introduction:

The Gateway residence hall at the Maryland Institute College of Arts was designed to be a cornerstone of their campus in downtown Baltimore, Maryland. Gateway is 122' tall, with 9 stories and a mechanical penthouse and has a useable floor area of 108,000 square feet. The building is located on a constricted site near the intersection of several major roads and Interstate 83. Due to its visibility from all directions, the building has a full 360 degree façade. Gateway is primarily circular in plan with a rectangular tower on the side that faces the highway. The circle, or drum component of the building encloses an open-air courtyard that actually begins on the third floor of the structure. This plaza is located directly above a large "black-box" multipurpose room capable of multiple arrangements to fit a variety of functions. This unique condition was explored in Technical Report I. Beyond the multipurpose assembly room, Gateway features 64 student apartments, several art galleries and studios, and a café.

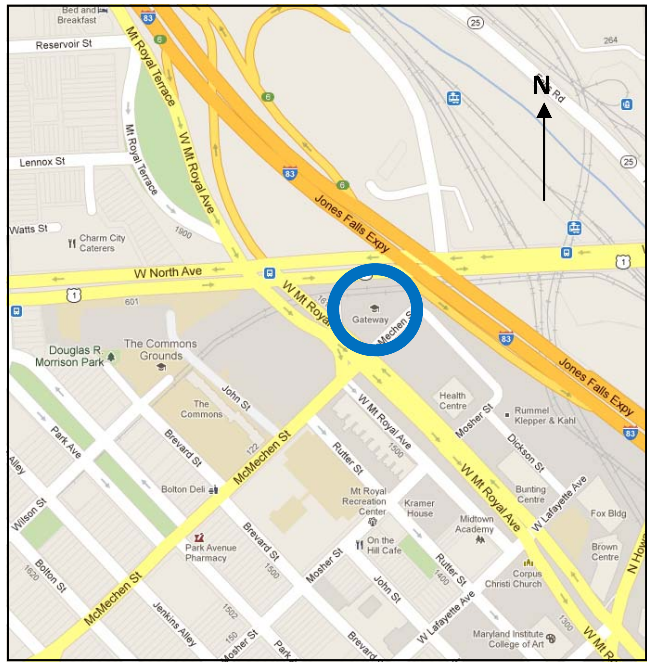

Figure 1: Gateway location in Baltimore. Courtesy of Google

RTKL Associates Inc. were the architects and engineers on the project, with KCW Engineering Technologies as the civil engineer, and Whiting Turner as the general contractor. The project was delivered with the design-bid-build method for an approximate cost of $30 million. The initial design began in 2005, with construction starting in August 2006 and concluding in August 2008. The building was designed using the Baltimore City Code, which at the time was in accordance with IBC 2000. Due to its various functions, the building has the occupancy types R-2, A-3, and B.

The building structure is primarily concrete, consisting of two-way flat plate slabs, beams, and columns. There are a few steel framed sections of the building, including the entrance vestibule and lobby. Being a prominent building, Gateway has a full 360 degree façade made almost entirely of glass curtain wall panels. The façade has clear, fritted, and frosted glass panels of white, gray, and mint green. Besides the glass curtain wall the superstructure is exposed in a number of places, such as the vertical cuts through the building and the 40' columns holding up a section of the fourth floor. The edge of each concrete floor slab is also exposed.

Structural Overview:

The Mica Gateway Residence is a predominately concrete structure with some steel members in certain places. Due to the unique circular shape of the building, the designers developed a radial grid with columns located by their X and Y coordinates in the four quadrants of the Cartesian coordinate system. The zero-zero point of the grid is located in the exact center of the courtyard. Thus a column located in the lower left of the plan will have a negative X and Y coordinate while a column in the upper right will have a positive X and Y coordinate. This was done to avoid an unreasonable amount of column lines clustered together at odd intervals.

Foundation:

The geotechnical report was prepared by D.W. Kozera, Inc. They submitted the geotechnical report on February 23, 2005. In their report they found that the site had very dense soil and soft rock, earning a site soil classification of C.

The foundation of the MICA Gateway features drilled caissons that bear directly on bedrock and have a safe bearing capacity of 10 ksf. All columns that start at ground level start at the top of a drilled caisson. Caissons are also located directly under the shear walls and the walls that support the load from the long span beams over the “black box” theater. All caissons are between 3’-0” and 4’-6” in diameter.

Where exterior walls meet the foundation, strip footings are incorporated and are a minimum of 30” below the finished grade. For the steel framed entrance vestibule and lobby, steel columns are supported by spread footings with a minimum safe bearing capacity of 1.5 ksf.

Gravity System:

The gravity load system for the Gateway features numerous two-way flat plate slabs as well as several one-way slabs and two-way slabs with drop panels. Below Level 4, there are several one way slabs of 7” thickness that span the areas below the courtyard. They work in conjunction with concrete beams that span very irregular areas. On Level 3, the courtyard spans over the “black-box” theater, to give a column free space for intended use. As such, 48”x48” beams were designed to span over the almost 60’ of the theater and accommodate the large dead and live loads from the plaza and planters in the courtyard above. These beams have (16)#10 bottom reinforcing bars to resist the large moments produced by the load.

On Level 4 there is an area featuring one-way slabs and beams. This area is supported by large exterior columns that rise nearly 40’ from grade to the bottom of the slab. Here a transfer beam runs between columns so as to support new columns that rise to support the upper floors. Beams are also used extensively to support the exterior walkways that connect the various parts of the drum.

The rest of Level 4 and all floors above have 8” two-way flat plate slabs between radial column lines as shown in Figure 2 to the right. The dotted lines represent the boundaries between the column and middle strips.

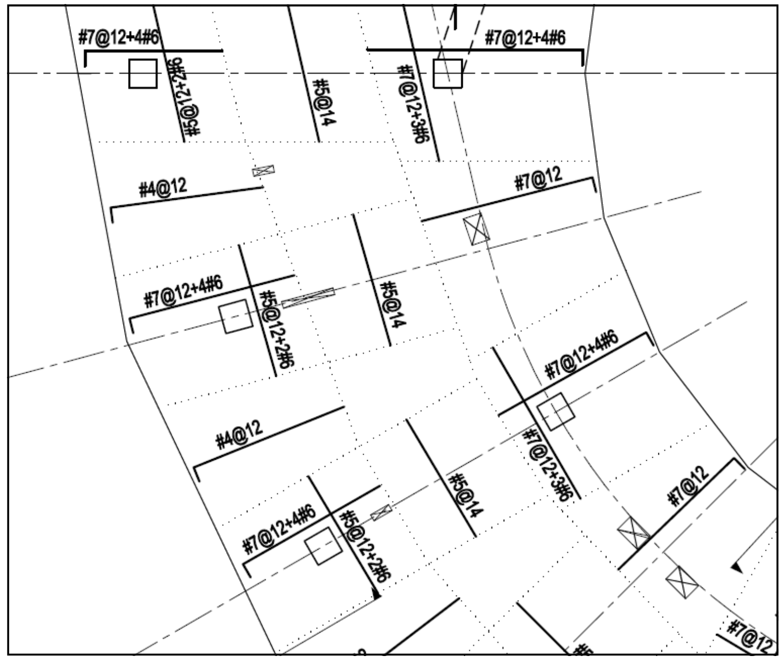

Figure 2: Typical two-way flat plate slab. Courtesy of RTKL

Other unique floor framing conditions include a section of the slab on each floor that frames into a column with a drop panel. This area is located in the northeast quadrant of the plans centered around column 7. The only uses of steel framing in this building are over the entrance and lobby, using mainly W10x15, W10x12, and HSS8x3x3/16.

The slabs and beams of the Gateway are all supported by concrete columns that form two concentric circular lines around the drum of the building. In most interior areas and on the upper floors these columns are rectangular, with sizes ranging from 12x12 to 24x24. In other places where the columns are on the exterior of the building, such as the 40’ slender columns that support Level 4, the columns are circular with sizes ranging from 24” diameter to 36” diameter.

The roof system of the Gateway is no different from a normal floor. One-way slabs frame into beams that transfer load to the columns. The main difference is the smaller slab thicknesses, between 6”-7” due to the smaller loads on the roof areas.

The following pages detail the original dead, live, and snow loads used in the building design.

Dead Loads:

In the General Notes (S001) the designers provided a loading schedule of superimposed dead loads which varied by location. That schedule lists each component of the dead load separately, but the following table lists only the total superimposed dead load for each building space. Concrete slab, column, beam, etc. self weights are not included in this table.

Area	**Dead Load (psf)**
Residences	9
Circulation Ring	10
Storage Rooms	9
Roof	13
Level 3 Planters	258*
Planters on Multi Use Room Space Roof	283†
Level 3 Plaza	38‡
Mechanical Rooms	9
Multi Use Room Space Roof	67§
Offices	9
Gallery Roof	17
Level 2 Balcony	37

* Takes into account a 240 psf saturated soil load. Only applies to structure supporting planters that are not above the multi-use performance space.

† Takes into account a 240 psf saturated soil load and the multi-use performance space roof ceiling components (steel grid, lighting, etc.). Only applies to structure supporting planters above the multi-use performance space.

‡ Takes into account pavers of the plaza not above the multi-use performance space.

§ Takes into account pavers of the plaza above the multi-use performance space.

Live Loads:

The Generals Notes also provided a table of live load values for the various areas of the building. Partitions are included in the live load for the residence and office areas. Oddly no live load was given for the floor of the multi-use performance room space on the loading schedule. Therefore a 100 psf live load for dance halls and ballrooms will be assumed, as per IBC 2006.

Area	Dead Load (psf)
Residences	60
Circulation Ring	100*
Storage Rooms	125*
Roof	30*
Level 3 Planters	240
Planters on Multi Use Room Space Roof	40
Level 3 Plaza	100*
Mechanical Rooms	150*
Multi Use Room Space Roof	100*
Offices	70
Gallery Roof	30*
Level 2 Balcony	100*
Multi-Use Performance Space	100 (per IBC 2006)

* Indicates that live load reduction was not allowed.

Snow Load:

Based on ASCE 7-05, which assumes a ground snow load of 25 psf, the roof snow load was calculated at 20 psf. This was checked against ASCE 7-10 and no change in snow load requirements between the two codes was noted.

Lateral Systems:

The lateral system of the Gateway features two concrete shear wall groups located near the stair and elevator cores, one in the tower and the other in the drum. Due to the low seismic risk of the region, it was assumed that the lateral system was ordinary concrete shear walls. Each of the eight shear walls extend from the ground to the highest point in their respective part of the building; 122' in the tower and 103' in the drum. The exception is shear wall 8 which goes up to Level 6. The walls are all 12" thick and from 10' to 25' long. The shear walls are highlighted in red in Figure 3 below.

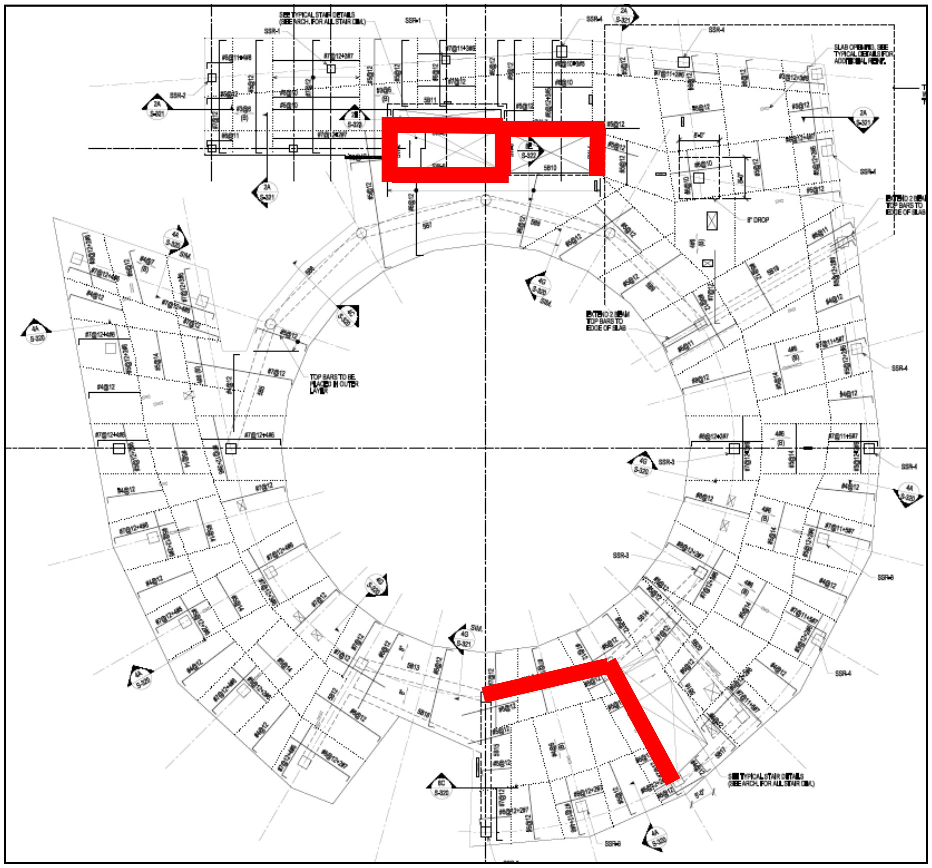

Figure 3: Shear wall locations. Courtesy of RTKL.

The lateral load path is as follows: wind pressure bears on the building cladding, which is supported by the edge slab. From here the slab transfers the load into shear walls either directly or through beams. The shear walls then direct the load into the foundation. Shear walls prevent unwanted torsion and large displacements of the building from occurring in the event of an earthquake or a severe storm with high winds.

Thesis Challenge:

The Maryland Institute College of Arts is a prestigious visual art school known for innovative curriculum and a well-equipped campus. Though there are a number of art galleries located in many buildings on the campus, there is no one building dedicated as an art museum. As such the MICA Gateway Residence will be designed as if it was an art museum. Changing the building's use creates a number of design challenges to be explored with the current structure.

Currently the live load for the residence floors of the Gateway is 40 psf, which is too low for the circulation space live load of a museum. The floor to floor height of the structure is 10 feet between floors 4-9, which would be too low a space for a museum gallery. Also, the current all glass façade of the building would make it very difficult to control the amount of light entering the gallery spaces, an important factor in museum design.

There are also a variety of architectural concerns that need to be addressed when changing the buildings use. This aspect of the redesign will be covered in the Breadth Topic Section. The current Gateway Residence was not designed with any regard to sustainability. Lacking any sustainable measures is not only bad for the environment but also for MICA's reputation and yearly expenses. The sustainable problem will be covered more in depth in the Breadth Topic Section.

Proposed Solution:

The entire Gateway structure above level 2 will be redesigned to accommodate a live load of 100psf, the live load for a circulation space in a museum. The slabs will still be designed as flat plates but will likely experience changes in thickness and reinforcement. As a museum is unlikely to have 10 foot floor to floor heights, the distance between the floors will be extended to 15 feet and floors 8, 9, and 10 will be taken out of the building to maintain the same overall building height, as shown in Figure 4 below. The increased floor to floor height will necessitate a redesign of the gravity columns throughout the building.

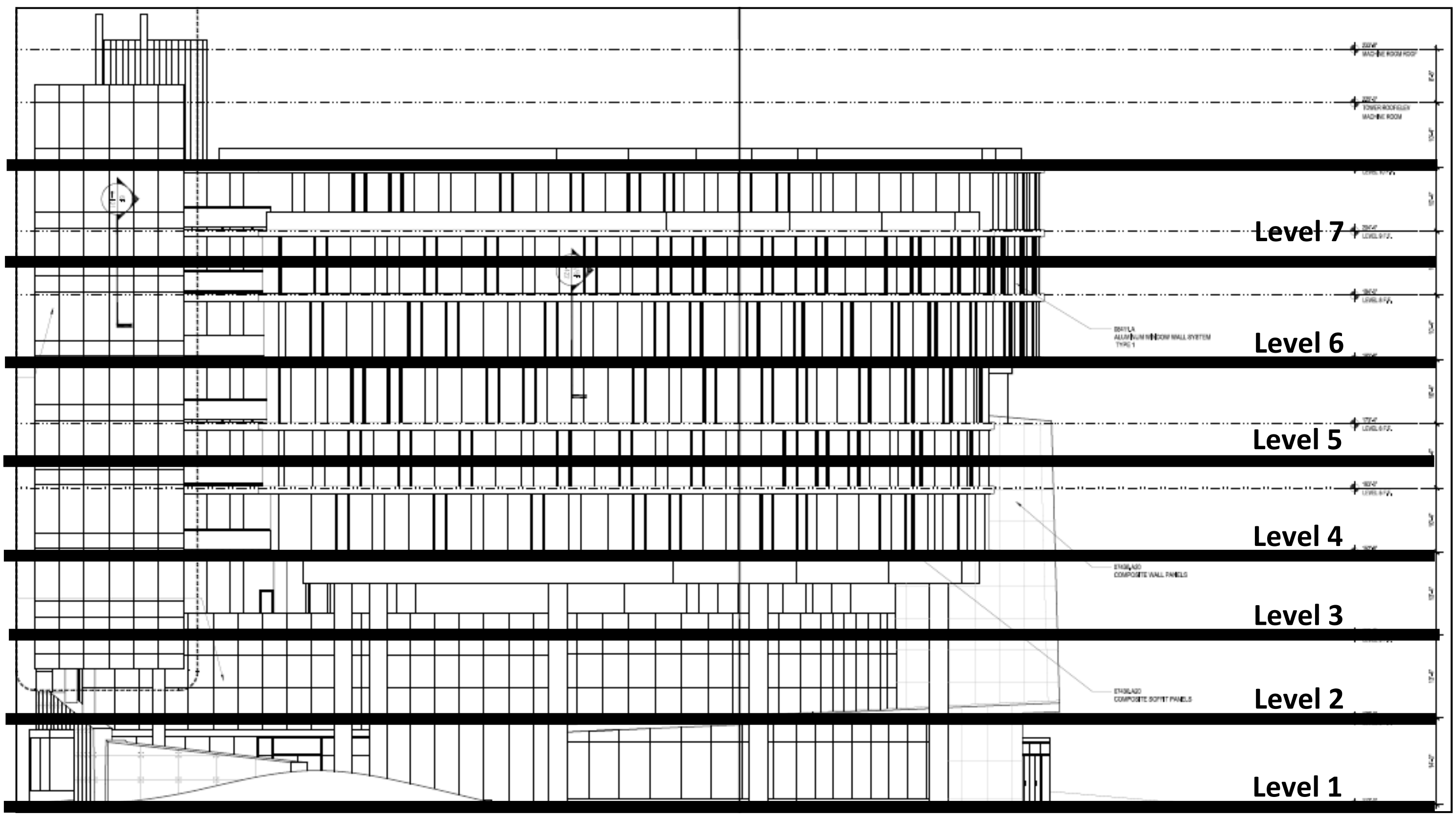

Figure 4: New level locations imposed on a building elevation. Courtesy of RTKL.

Changing the gravity system of the building will have direct impacts on the Gateway lateral system. Taking out several floors will decrease the overall building weight, but the increased floor to floor height will change the story shear at each level. The building will need to be re-analyzed for wind and seismic forces and the lateral shear walls will likely need to be redesigned. The foundations of the building will also need to be considered to see if any changes are necessary.

Solution Methods:

The design of the gravity system will follow ACI 318-11, Chapter 13, using the equivalent frame method. The equivalent frame method must be used due to the irregular geometry of the Gateway floor slabs. Typical flat plate slabs, beams, and columns within the Gateway structure will be redesigned through hand calculations. The foundations will also be checked by hand calculation to determine if any changes are necessary.

New lateral loading based on new story forces will be determined similar to the procedure used in Technical Report 1. ASCE 7-10 will be used to determine the new wind and seismic story forces. An ETABS model will then be used to analyze the new structure. This procedure will parallel Technical Report 3 and will check the drift, overturning moments, and torsion on the building. The shear walls will also be spot checked by hand to determine their adequacy in the new structural system.

Design Codes:

The MICA Gateway Museum will be designed in compliance with the following:

- Baltimore City Code in accordance with IBC 2000
- ASCE 7-10– Minimum Design Loads for Buildings and Other Structures
- ACI 318-11– General Design of Reinforced Concrete
- AISC 14th Edition– Specifications for Structural Steel Buildings
- AWS D1.1– Structural Welding Code– Steel
- ACI 530-05– masonry structures

Building Materials:

MICA Gateway was designed and constructed using the following materials as specified on the General Notes Sheet S001. The new design will use the same materials:

- 3500 psi Concrete*– used in spread footings, drilled caissons, and slab on grade
- 4000 psi Concrete*– used in walls, piers, grade beams, columns, slabs, and beams
- ASTM A615, Grade 60– deformed bars
- ASTM A185– welded wire fabric
- ASTM A992– W and WT shapes
- ASTM A36– channels and angles
- ASTM A500, Grade B– rectangular and square HSS, and round HSS
- ASTM A53, Grade B– steel pipe
- ASTM A36 2, Grade 50– steel plates
- ASTM A325 or A490– high strength bolts
- ASTM F1554, Grade 36– anchor bolts
- ASTM A307– standard fasteners
- ASTM A653, Quality SS, Grade 33– metal roof deck
- ASTM C476– grout
- ASTM C270, Type S– mortar
- 1500 psi Masonry– used in masonry walls

*Normal weight concrete shall have a maximum dry unit weight of 150 pcf

Breadth Topics:

An architectural breadth will be studied due to the changes in floor plan and floor to floor heights associated with changing the building's use. All of the apartments in the building will be taken out and replaced with museum galleries. The circulation of people through the museum will also be an important aspect of the architectural breadth. The last part of this breadth will be a redesign of the building façade to more appropriately fit a museum. The amount of glass will be diminished to limit the amount of direct sunlight penetrating the building into the museum spaces. This will be done in an aesthetically pleasing way so as to maintain the architectural significance of the Gateway. A Revit model will be created to accurately develop floor plans, elevations, and renderings of the new interior spaces and exterior ascetics.

A sustainability breadth will also be studied due to the lack of sustainable features on the existing building. The United States Green Building Council's LEED program will be the basis for the sustainability breadth, with the goal of making the Gateway at least LEED Certified. Based on the LEED scorecard, shown in Figure 5 on the following page, a total of 40 points are needed for LEED Certification. For this breadth the points will be drawn heavily from the Sustainable Sites, Water Efficiency, and Materials and Resources sections. Due to time constraints the Indoor Environmental Quality and Energy and Atmosphere sections will be neglected, though in an actual building design they would be considered. The end result of this breadth will include a detailed design of a green roof system as well as highlight other credits needed in order to gain LEED certification.

MAE Requirements:

Two MAE classes will be used to meet the requirements of the integrated degree. The knowledge gained in AE 597A– Computer Modeling of Buildings will be incorporated into the thesis project with the construction and analysis of an ETABS model. The knowledge from AE 597A will be essential in accurately building the model in ETABS and interpreting the results of the analysis. AE 542– Building Enclosure Science and Design will also be incorporated into the redesign of the Gateway. This course studies the design and analysis of building facades which will be relevant to both the architecture and sustainability breadths.

LEED for New Construction and Major Renovations (v2009)

SUSTAINABLE SITES		POSSIBLE: 26
SSp1	Construction activity pollution prevention	REQUIRED
SSc1	Site selection	1
SSc2	Development density and community connectivity	5
SSc3	Brownfield redevelopment	1
SSc4.1	Alternative transportation - public transportation access	6
SSc4.2	Alternative transportation - bicycle storage and changing rooms	1
SSc4.3	Alternative transportation - low-emitting and fuel-efficient vehicles	3
SSc4.4	Alternative transportation - parking capacity	2
SSc5.1	Site development - protect or restore habitat	1
SSc5.2	Site development - maximize open space	1
SSc6.1	Stormwater design - quantity control	1
SSc6.2	Stormwater design - quality control	1
SSc7.1	Heat island effect - nonroof	1
SSc7.2	Heat island effect - roof	1
SSc8	Light pollution reduction	1

WATER EFFICIENCY		POSSIBLE: 10
WEp1	Water use reduction	REQUIRED
WEc1	Water efficient landscaping	4
WEc2	Innovative wastewater technologies	2
WEc3	Water use reduction	4

ENERGY & ATMOSPHERE		POSSIBLE: 35
EAp1	Fundamental commissioning of building energy systems	REQUIRED
EAp2	Minimum energy performance	REQUIRED
EAp3	Fundamental refrigerant Mgmt	REQUIRED
EAc1	Optimize energy performance	19
EAc2	On-site renewable energy	7
EAc3	Enhanced commissioning	2
EAc4	Enhanced refrigerant Mgmt	2
EAc5	Measurement and verification	3
EAc6	Green power	2

MATERIAL & RESOURCES		POSSIBLE: 14
MRp1	Storage and collection of recyclables	REQUIRED
MRc1.1	Building reuse - maintain existing walls, floors and roof	3
MRc1.2	Building reuse - maintain interior nonstructural elements	1
MRc2	Construction waste Mgmt	2
MRc3	Materials reuse	2
MRc4	Recycled content	2

MATERIAL & RESOURCES		CONTINUED
MRc5	Regional materials	2
MRc6	Rapidly renewable materials	1
MRc7	Certified wood	1

INDOOR ENVIRONMENTAL QUALITY		POSSIBLE: 15
EQp1	Minimum IAQ performance	REQUIRED
EQp2	Environmental Tobacco Smoke (ETS) control	REQUIRED
EQc1	Outdoor air delivery monitoring	1
EQc2	Increased ventilation	1
EQc3.1	Construction IAQ Mgmt plan - during construction	1
EQc3.2	Construction IAQ Mgmt plan - before occupancy	1
EQc4.1	Low-emitting materials - adhesives and sealants	1
EQc4.2	Low-emitting materials - paints and coatings	1
EQc4.3	Low-emitting materials - flooring systems	1
EQc4.4	Low-emitting materials - composite wood and agrifiber products	1
EQc5	Indoor chemical and pollutant source control	1
EQc6.1	Controllability of systems - lighting	1
EQc6.2	Controllability of systems - thermal comfort	1
EQc7.1	Thermal comfort - design	1
EQc7.2	Thermal comfort - verification	1
EQc8.1	Daylight and views - daylight	1
EQc8.2	Daylight and views - views	1

INNOVATION		POSSIBLE: 6
IDc1	Innovation in design	5
IDc2	LEED Accredited Professional	1

REGIONAL PRIORITY		POSSIBLE: 4
RPc1	Regional priority	4

TOTAL		110

40-49 Points	50-59 Points	60-79 Points	80+ Points
CERTIFIED	SILVER	GOLD	PLATINUM

Figure 5: LEED Scorecard for new construction. Courtesy of USGBC.

Structural Design:

Gravity System Design:

Flat Plate Slab Design:

The new design for the MICA Gateway Museum must support live loads of 100 psf as per IBC 2006. The selection of this live load was based on the assumption that a museum is just a series of long corridors that must support the weight of large crowds. The weight of the art on display was also considered in choosing 100 psf. Partition load was also included in the design to provide for the division of gallery space based on program needs. The partition live load was kept at 20 psf as used in the original building design.

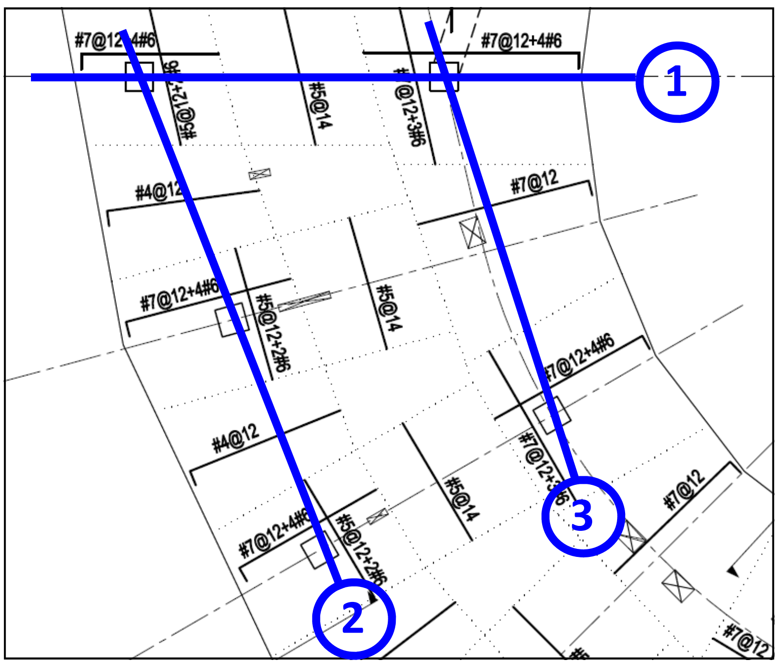

Figure 6: Lines of equivalent frame analysis.

For the new slab design two-way flat plate slabs will still be used because of the cost and construction time savings in form work, and the maximum floor-to-floor height. Due to the irregular geometry of the Gateway floor system, an equivalent frame method will be employed to determine slab reinforcement. Three different directions of the slab will be analyzed as detailed in Figure 6. Besides determining rebar in the column and middle strips of each line, punching shear and long term deflections were also checked. All calculation were done by hand and can be found in Appendix A. The roof slab was redesigned as a one-way slab with beams to accommodate the new green roof. That design is discussed more in the Sustainability breadth.

Initially the slab was checked to see if the original slab thickness of 8" was adequate to support the increased loading. The analysis determined that the slab experienced excessive deflections under the new loading and the slab thickness was increased to 12." A slab thickness of 12" was deemed adequate for each equivalent frame analysis. For reinforcement, #6 bars were used at spacings of 6," 12," and 18" depending on the bending moments. A #5@12" bottom mat was also used for shrinkage and temperature reinforcement. The punching shear check also determined that #4@3" should be used around the columns. Figure 7 on the following page details the new slab reinforcement design as compared to the original slab reinforcement design in Figure 8.

Figure 7: New Gateway slab reinforcement.

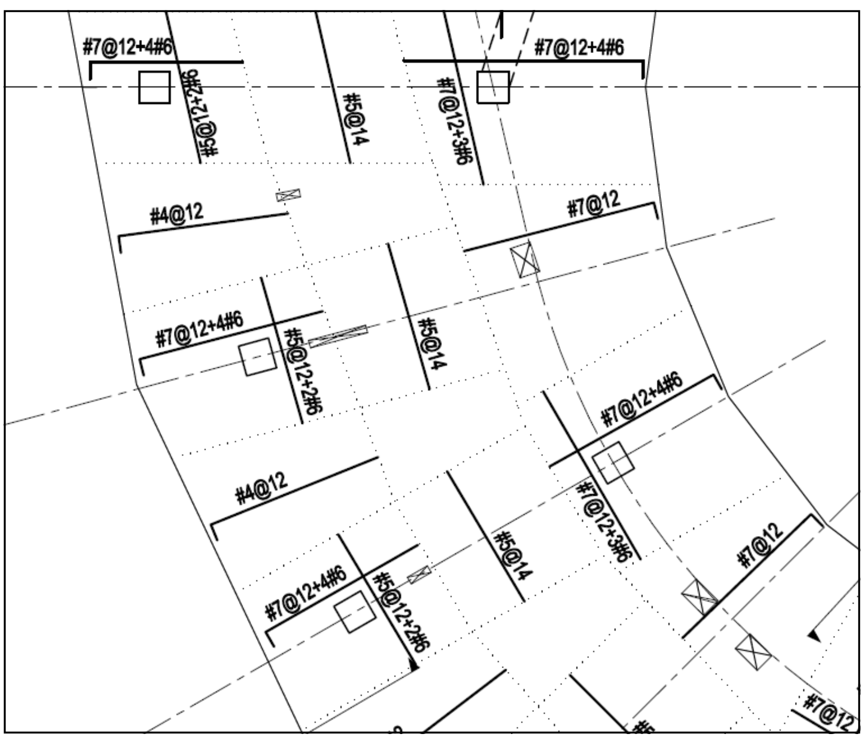

Figure 8: Original Gateway reinforcement. Courtesy RTKL.

The development lengths for the reinforcement in the slab will follow ACI 318-11 Figure 13.3.8 as shown below in Figure 9. All bottom bars will be continuous throughout the slab. The top reinforcement in the column strips will extend 6' beyond the support in frame line 1, 5' beyond in frame line 2, and 7' beyond in frame line 3. The bottom bars terminate at the edge of slab with 3" cover in a 90^o hook with a length of 6." The top bars terminate at the edge of slab with 3" cover in a 90^o hook with development length of 9.8." Development length calculations are found in Appendix A.

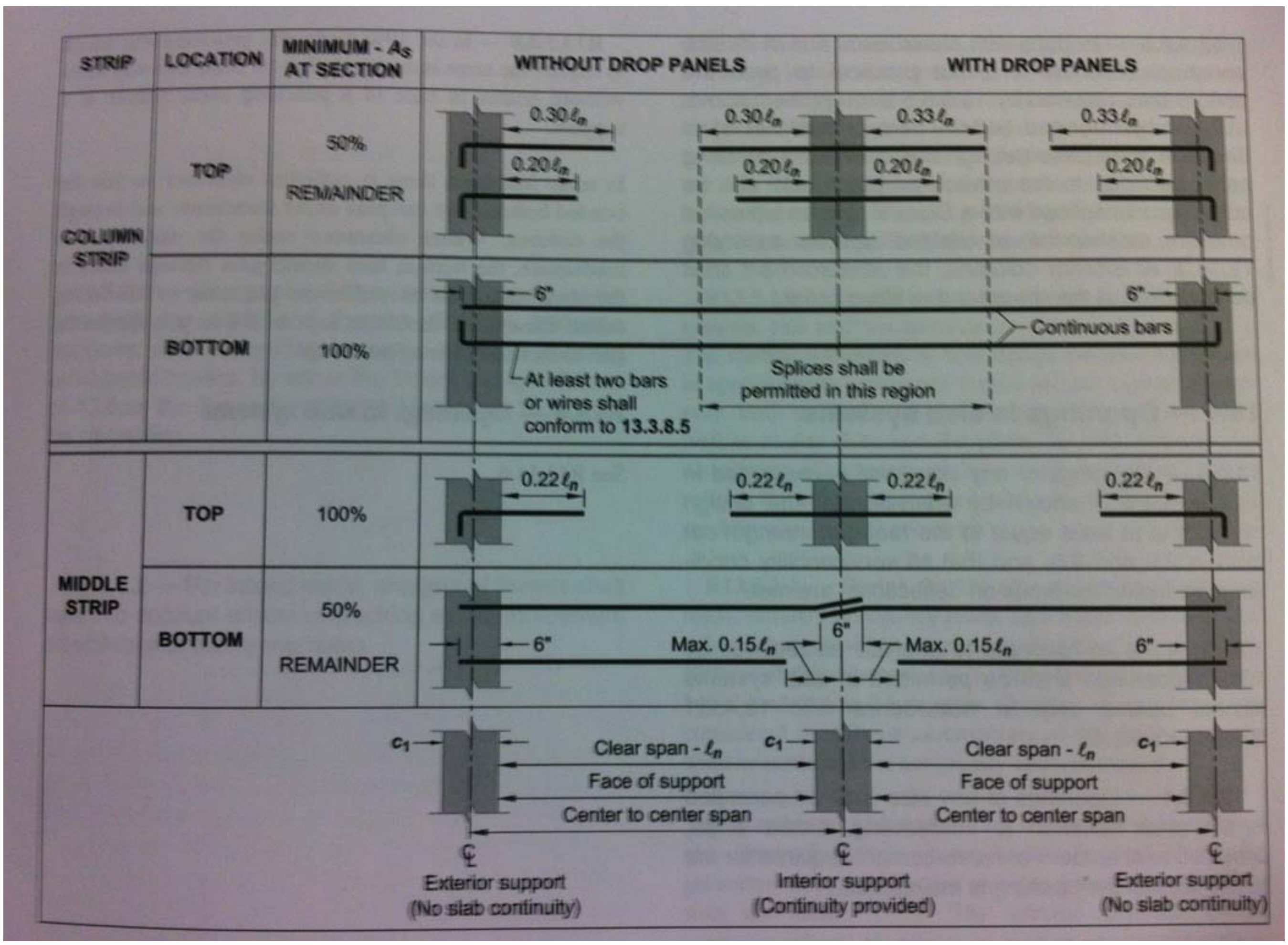

Figure 9: Figure 13.3.8 detailing slab rebar development length. Courtesy ACI 318-11.

The new design for the Gateway would have eliminated 2 levels from the building but would have increased the amount of concrete used in each slab by 33%. The resulting changes would have increased the amount of concrete used in the slab construction by 27% even though the overall floor area of the building decreases. A quantity increase of this magnitude would have made the Gateway more expensive to build as a museum than as a residence hall.

Column Design:

Due to the increased live load and self weight of the slabs the gravity columns of the Gateway must be redesigned. Column 16 was chosen for analysis with the higher loads. Column 16 is significant due to its tall height between grade and Level 4. In this area Column 16 is unbraced for 41.' SpColumn was used to analyze the column in two separate parts. The first part was the 41' tall section from grade to Level 4 and the second part was the 15' tall section between Level 4 and Level 5. The loads were determined by hand calculations and the moments were taken from the equivalent frame analysis for the slab.

For the spColumn analysis, the original column diameter was increased from 36" to 42" as a trial size. Above Level 4 the size was increased from 24"x24" to 30"x30." The final design found that for the 41' tall section a 42" column with 11#10 bars was sufficient to support the gravity loads. For the 15' tall section a 30"x30" column with 8#10 bars was found to be sufficient. The 30"x30" column size was continued for all floors above Level 4 to facilitate faster construction by reusing the same formwork. Figure's 10 and 11 show the round, 41' tall section and the square 15' tall section reinforcement respectively.

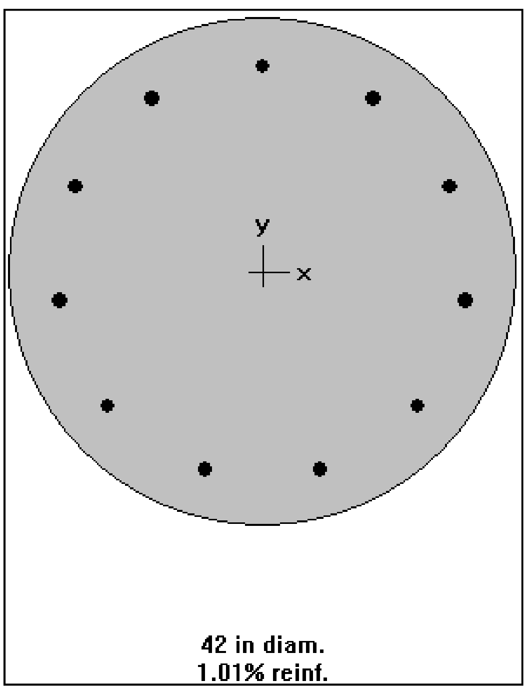

Figure 10: 42" column with (11)#10

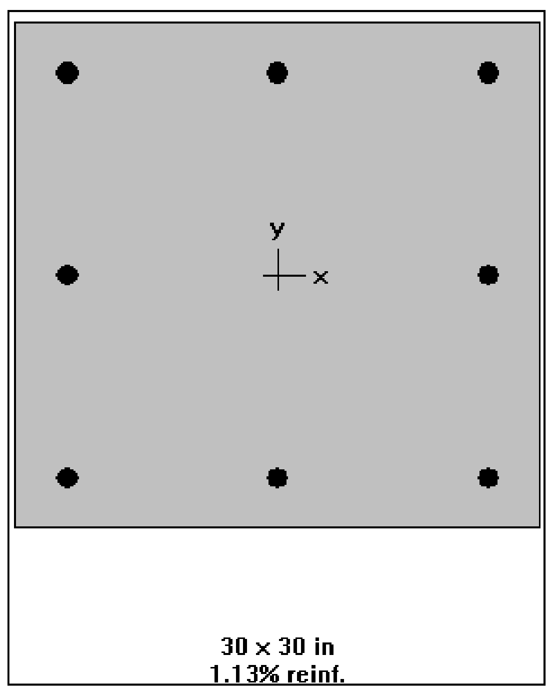

Figure 11: 30"x30" column with (8)#10

Column 16 is typical of all the columns in the building with the exception of the 41' tall section. Eight other columns in the structure also have a slender 41' tall section. These columns were all changed from 36" diameter columns to 42" diameter columns. The rebar was also changed from 11#9 bars to 11#10 bars. This change represents a 26% increase in concrete used and a 21% increase in steel used.

The 15' tall section of Column 16 is typical for all columns on every level of the structure. All other columns were changed to the new design from 24"x24" to 30"x30" sections, representing a 36% increase in concrete used. The reinforcement used increased by 38% by changing from 8#8 bars to 8#10 bars. Based on these findings it can be assumed that the higher material quantities would have made the construction of the Gateway as a museum more expensive.

After developing an interaction diagram through spColumn the slenderness effects of the column was checked per ACI 318-11 section 10.10. The analysis found that for the 15' tall sections slenderness effects could be neglected. Slenderness effects could not be neglected for the 41' tall section. For this case it was determined that the column was nonsway. The factored moment amplified for the effects of member curvature was found to be 1193 ft-kip and the factored axial load was found to be 717 kip. The resulting moment and axial load are plotted on the interaction diagram in Figure 12. The point is located within the interaction curve and therefore the column is adequate to carry the loads.

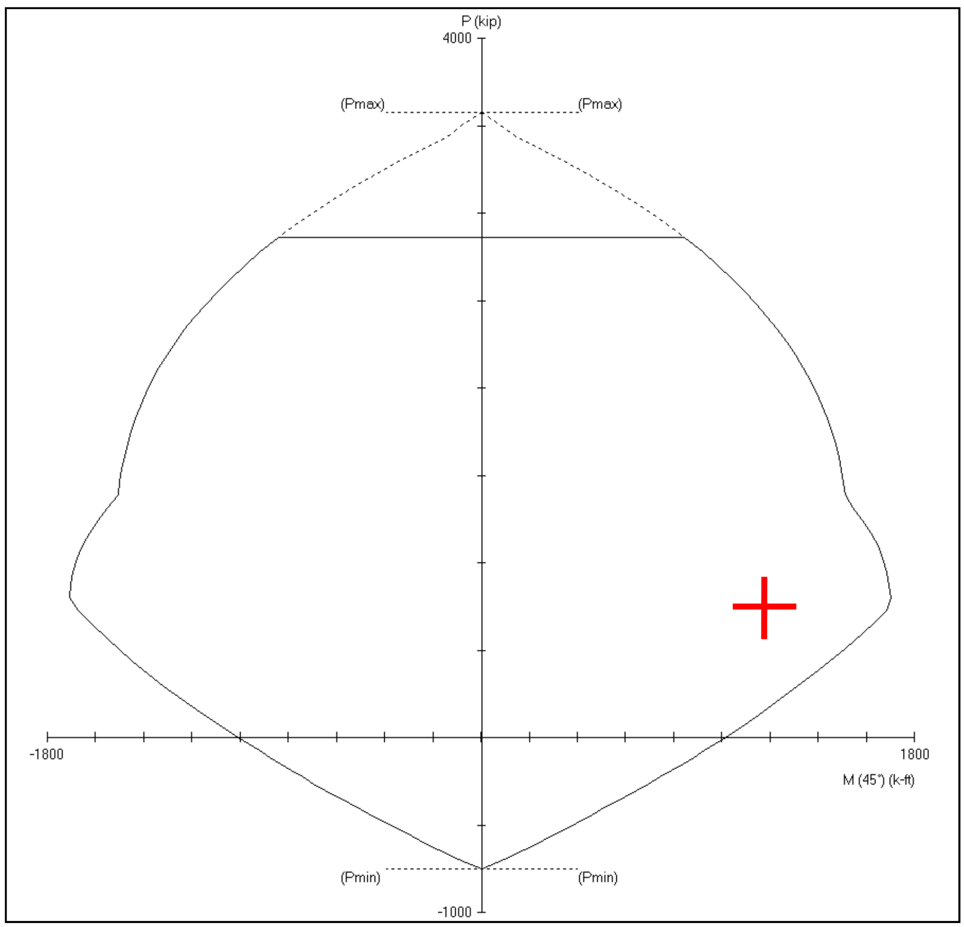

Figure 12: 41' tall column interaction diagram

Lateral System Design:

An ETABS model was used to determine the adequacy of the Gateway's lateral system under the new design considerations. The new model included the increased floor to floor height between Level 4 and the roof. Two levels were eliminated in order to keep the Gateway at the same height as the original design. All slabs and columns were changed to the new sizes determined in the gravity analysis.

For the analysis the shear walls were modeled as shell elements with a maximum mesh size of 24" while the floor slabs were not meshed so as to simplify the analysis. Bending modifiers were applied to all members as well so as to account for concrete cracking. Rigid diaphragms were modeled at each floor to determine the story drift of each level. The shear walls were restrained with fixed connections at the base while all other members were pinned at the base to ensure that the shear walls took all of the lateral loads. Figure 13 shows the finished ETABS model for the new Gateway design.

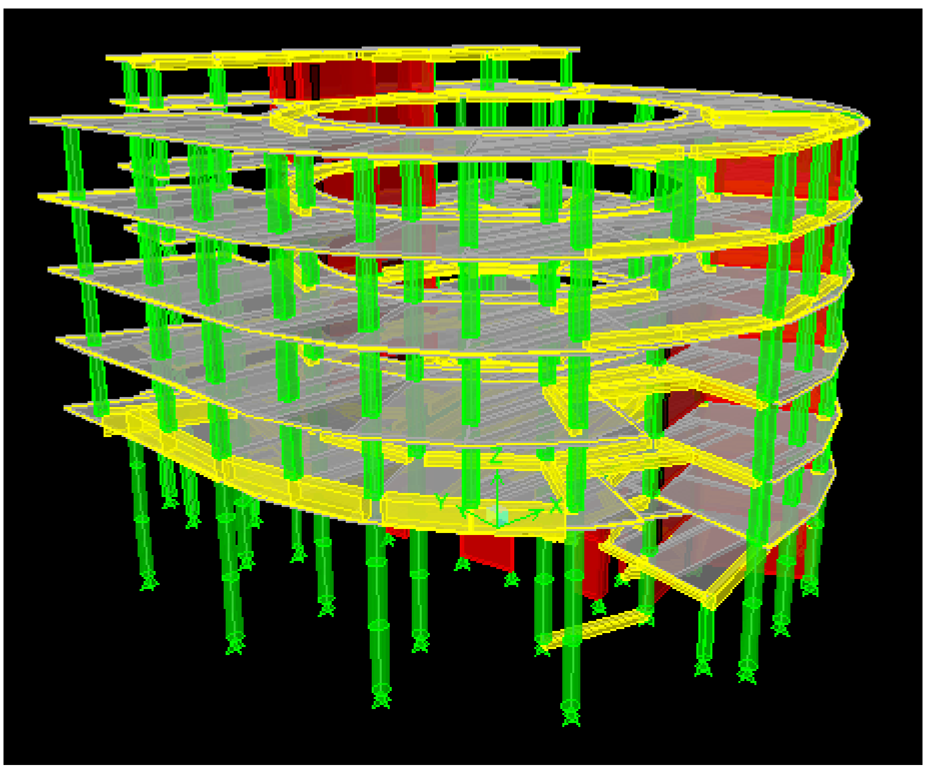

Figure 13: Complete Gateway ETABS model

The analysis checked the Gateway under several load conditions as per ASCE 7-10. Wind load cases were defined under Chapter 6, Method 2 of ASCE 7-10 and seismic load cases were determined from Chapter 11 and 12 of ASCE 7-10. Appendix B includes the wind and seismic load calculations that were entered as user defined loads and coefficients in ETABS.

Two separate analysis were performed for the new design. The first analysis found that the shear walls in the Y direction were unable to resist the moments created by seismic forces. This led to shear walls 2,3, and 4 being increased in length by 4 feet. Figure 14 shows which shear walls were lengthened. The 4' addition on the walls did not interfere with any architectural elements as the space behind the elevator core will now be used for museum administration and storage. A second analysis was then performed to determine the new shear forces in each shear wall. The increase in length of the walls allowed them to have an appropriate amount of reinforcement to withstand the overturning moments. Detailed calculations and shear wall tables are contained in Appendix B.

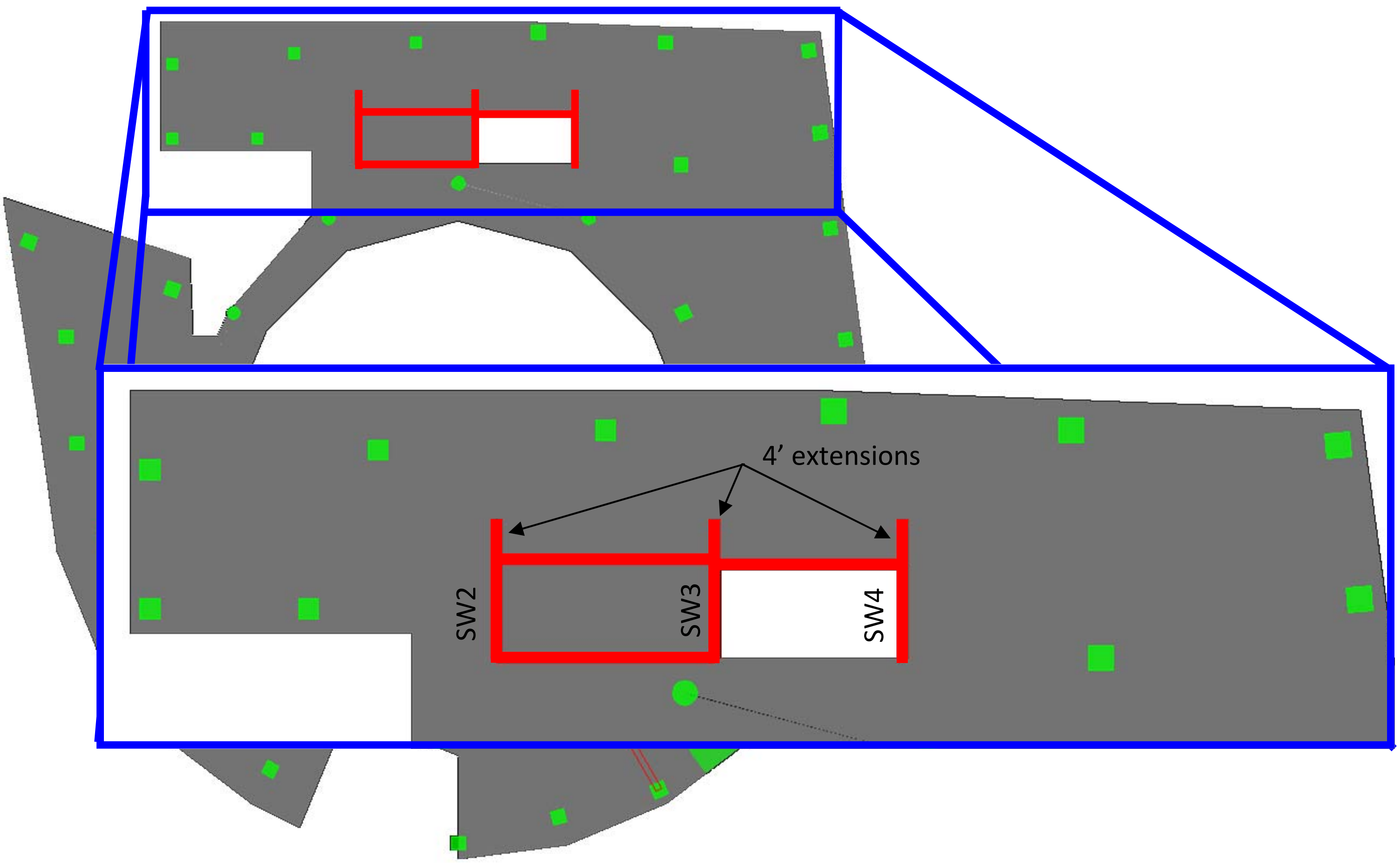

Figure 14: Location of modified shear walls

Based on the results of the ETABS analysis, the controlling load case was seismic in the Y or N-S direction minus an eccentricity in the X or E-W direction. The ETABS model for the new design calculated the story drift and displacement for each level. Figure 15 details the center of mass displacement and the story drift at each level for the controlling load case. The allowable drift limit comes from ASCE 7-10 and permits a story drift of 0.015 x story height. The story drifts for the new design were well below the allowable drift limit.

Story Drift and Displacement (Seismic in the Y Dir. With -X ecc.)				
Story	Story height (ft)	Displacement (in)	Story Drift (in)	Allowable Drift (in)
Tower Roof	113	0.9814	0.0892	1.695
Roof	103	0.8922	0.1363	1.545
7	88	0.7559	0.142	1.32
6	72	0.6139	0.1392	1.08
5	56	0.4747	0.113	0.84
4	41	0.3617	0.1255	0.615
3	27	0.2362	0.0968	0.405
2	14	0.1394	0.1394	0.21

Figure 15: Center of mass displacement and story drift caused by seismic loading

The overturning moment of the building as a result of the controlling load case is computed in Figure 16.

Overturning Moment			
Story	Height (ft)	Story Force (k)	**Moment (k-ft)**
Tower Roof	113	40.9	**4622**
Roof	103	207.2	**21342**
7	88	193	**16984**
6	72	156.9	**11297**
5	56	119.2	**6675**
4	41	90.7	**3719**
3	27	80.3	**2168**
2	14	27.2	**381**
		Total=	**67187**

Figure 16: Overturning moment caused by seismic loading

Torsional effects on the lateral system of the building were also considered. Torsional moment develops when there is an eccentricity between the center of mass of a diaphragm and the center of rigidity. This eccentricity applies an additional shear force on the lateral resisting elements on top of the direct shear into those elements. Based on the seismic design requirements from ASCE 7-10, torsional irregularities apply to Seismic Design Category B as per Table 12.3-1. That table details that any buildings that fall in SDC B with a torsional irregularity must be analyzed using a 3D representation. This required analysis with ETABS and as such the torsional shear was distributed into each shear wall.

The shear forces that were distributed to each shear wall necessitated a re-design of the reinforcement in each wall. The reinforcement was redesign for the end zones to resist flexure as well as the vertical and horizontal shear reinforcement. Shear walls 2,3,4,5,6, and 7 all feature a 36" end zone with (12) #11's and #4@18" in two layers for vertical shear reinforcement. For horizontal shear reinforcement, #5@18" in two layers were used. The rebar layout for the shear walls is detailed in Figure 16. Shear walls 1 and 8 are built integrally with columns and thus use the column reinforcement as end zones. The reinforcement for those columns was calculated and can be viewed in Appendix C.

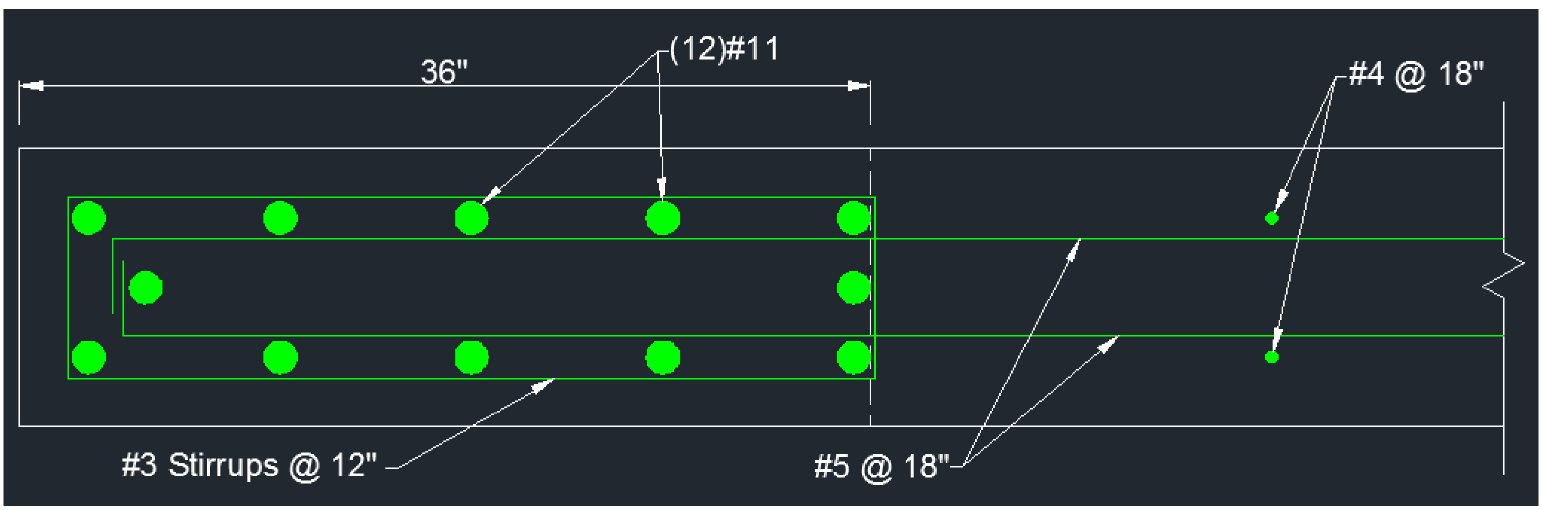

Figure 18: New shear wall reinforcement detail

Foundation Design:

The changes in the Gateway gravity system and lateral system necessitated a check of the foundation system. The caissons under the shear walls were designed to resist the overturning moment of the building as determined through the ETABS analysis. The CRSI 2008 handbook was used to size the caisson shaft and bell diameters to meet the load requirements. Based on a 10ksf bearing strength, the bell diameter was found and that in turn was used to size the appropriate shaft diameter. Once the shaft diameter was determined the chart shown in Figure 18 was used to determine the vertical reinforcement and ties within the caisson shaft. Figure 19 shows the loads, shaft and bell size, and reinforcement of each caisson under the shear walls. Calculations can be found in Appendix D.

Reinforcement Details (Percentage of Reinforcement ≈ 0.005) for Unlined Drilled Piers Designed as Plain Concrete

DRILLED PIER—REINFORCEMENT

Shaft Diameter (ft-in.)	Number, Sizes, Spacing		Total Weight (lbs) per Pier*		
d	Vertical	Ties**	Vertical	Ties**	Total
1′-6″	6-#5	#3 @ 10″	63	20	83
2′-0″	6-#6	#3 @ 12″	90	23	113
2′-6″	6-#7	#3 @ 14″	123	26	149
3′-0″	7-#8	#3 @ 16″	187	28	215
3′-6″	7-#9	#3 @ 18″	250	28	278
4′-0″	7-#10	#4 @ 18″	361	69	430
4′-6″	9-#10	#4 @ 18″	523	87	610
5′-0″	9-#11	#4 @ 18″	717	107	824
5′-6″	11-#11	#4 @ 18″	964	129	1093
6′-0″	13-#11	#4 @ 18″	1243	153	1397
6′-6″	16-#11	#4 @ 18″	1658	180	1838
7′-0″	18-#11	#4 @ 18″	2008	208	2217

* Total weights for minimum embedment. See Fig. 13-16.
** Begin first tie 2 inches below top.

Figure 18: Reinforcement details of caissons. Courtesy CRSI 2008 Handbook.

	Caisson Design						
	M_{over} (k-ft)	P_{axial} (k)	Total Load (k)	Shaft Dia. (ft)	Bell Dia. (ft)	Vertical Reinf.	Ties
SW 1	34424	504	1399	13.5	4.5	9- #10	#4@ 18"
SW 2	11467	246	942	11	4	7- #10	#4@ 18"
SW 3	11467	246	942	11	4	7- #10	#4@ 18"
SW 4	9829	246	825	10.5	3.5	7- #9	#3@ 18"
SW 5	20697	390	1095	12	4	7- #10	#4@ 18"
SW 6	20697	390	1095	12	4	7- #10	#4@ 18"
SW 7	16093	390	927	11	4	7- #10	#4@ 18"
SW 8	9700	270	523	8.5	3	7- #8	#3@ 16"

Figure 19: Caisson design for Gateway museum.

Conclusion:

The structural depth proved that designing the Gateway as a museum would be significantly different from the current residence hall design. The increase in slab depth and column diameters would have increased the overall cost of the structure. The increase in floor to floor height would have also required a stronger lateral system to resist seismic forces. Based on this analysis it is unlikely that conversion from a residence to a museum would ever be possible due to the necessary structural strengthening. Despite the differences in sizes and depths between the residence and the museum, the overall system types of the Gateway would be adequate for both uses. This depth proved that two-way flat plate slabs, ordinary reinforced concrete shear walls, and drilled caissons would be appropriate in both building uses.

Architectural Breadth:

Redesigning the Gateway from a residence hall into a museum required a number of significant architectural changes. A Revit model was used to accurately model the new changes in the building structure and architecture. Several important aspects of museum design that were considered include:

- Large viewing spaces for art
- Open gallery areas with the potential for segmentation
- Circulation of people to avoid dead ends, turnarounds, missed exhibits, etc.
- Control of exterior sunlight to permit appropriate viewing light
- Rest areas

Levels 3-7 would be repurposed as museum galleries while Levels 1 and 2 would remain the same. Spaces appropriate to the function of a museum such as the café, multipurpose room, offices, and mechanical spaces would remain the same in the new building. In order to accommodate the large spaces for museum galleries, the interior partitions of the separate apartments were removed. Removing all partitions created a single continuous gallery on every floor as shown in Figure 20 on the following page. The space located behind the secondary stairs and elevator core would be repurposed as museum storage and maintenance areas.

Circulation of people is also a very important aspect of museum design. To go from floor to floor people would primarily use the main stairs located near the main entrance. This stair tower is adjacent to the outer curtain wall and would afford excellent views of the MICA campus and downtown Baltimore. Entering onto each gallery floor people would have the option of moving clockwise or counter-clockwise along the circular gallery. Restrooms and rest areas would be adjacent to the main stair tower. Following the gallery around the complete circle would lead back to the main stair tower thus ensuring that a visitor would see all the art on the floor. Halfway along the gallery the second stairs and elevators would be located to provide an alternate route of travel and handicap access to the gallery floors. The red arrows on Figure 20 denote the circulation of people through the gallery.

All gallery levels were also increased from 10’ floor-to-floor height to 15’ floor-to-floor height to provide a larger viewing area for the art. Art would be displayed along the interior and exterior walls of the building. The columns of the structure are actually set in from the building exterior and thus help to divide the galleries into a series of alcoves where a single piece of art can be viewed. Figure 21 shows a rendering of a typical gallery in the Gateway.

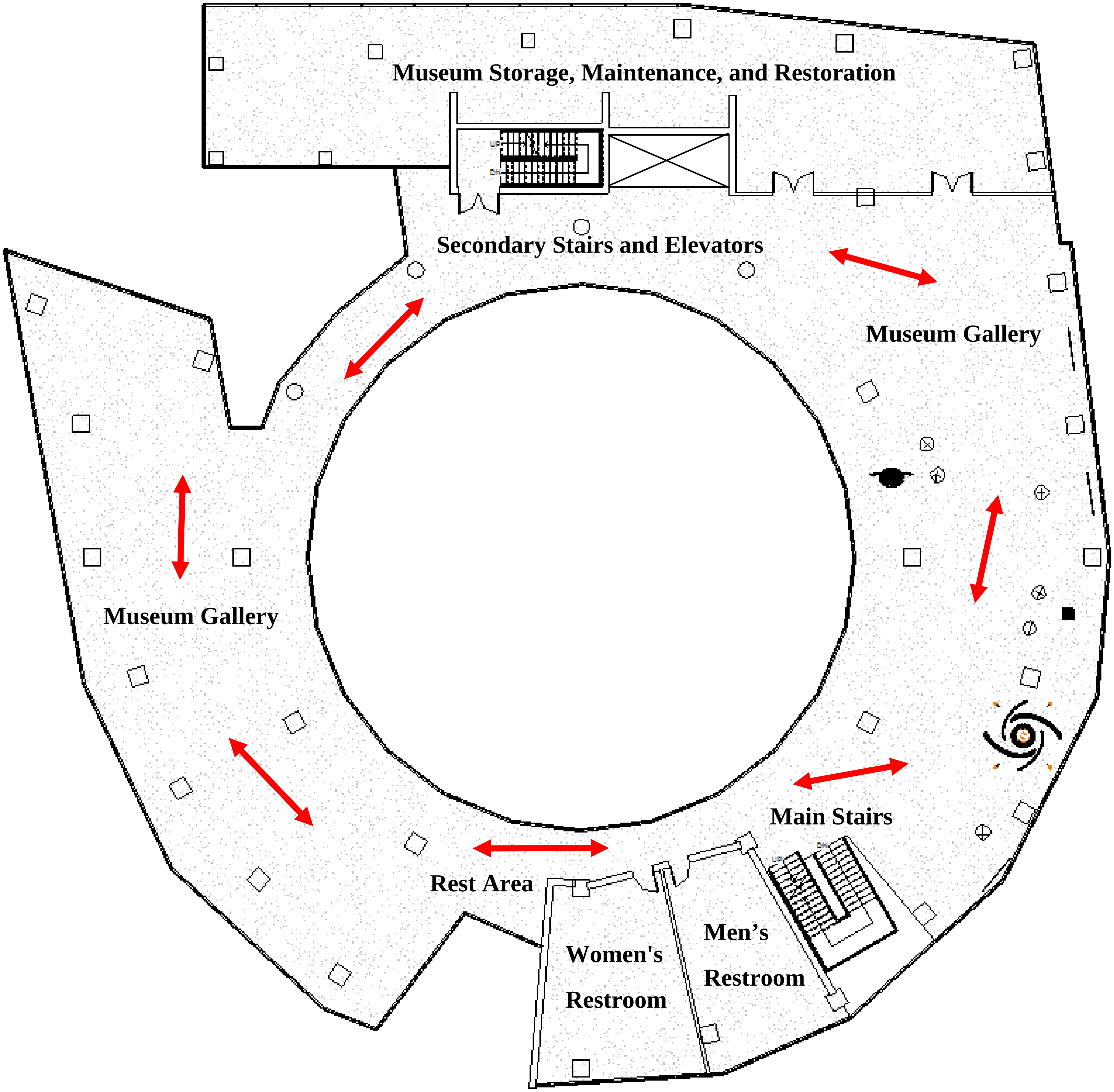

Figure 20: A typical floor plan for the Gateway Museum. Red arrows show the circulation paths for visitors.

Figure 21: View of a typical gallery. Columns divide the space into alcoves to view single pieces of art. Notice also the limited amount of sunlight that enters the gallery due to the changes in façade glazing.

Changing the façade of the Gateway was another important architectural design issue. In a museum for art it is necessary to control the amount of light in a space so as to provide optimal viewing conditions. The original design of the Gateway features an curtain wall façade made of several different types of glass and solid panels in a variety of patterns. The amount of clear and translucent glass on the southern faces of the building allow too much sunlight into the building for art to be viewed properly. To solve this problem all clear glass panels on the southern side of the building was replaced by green and grey glass with high reflectance to match the current colors of the building. The number of solid panels was also increased. Though not studied in this breadth, new lighting would have to be designed in the gallery space to provide the optimal viewing conditions for the art. Figure 21 shows the lighting conditions of a typical gallery. Figure 22 shows the exterior view of the old façade versus the new façade.

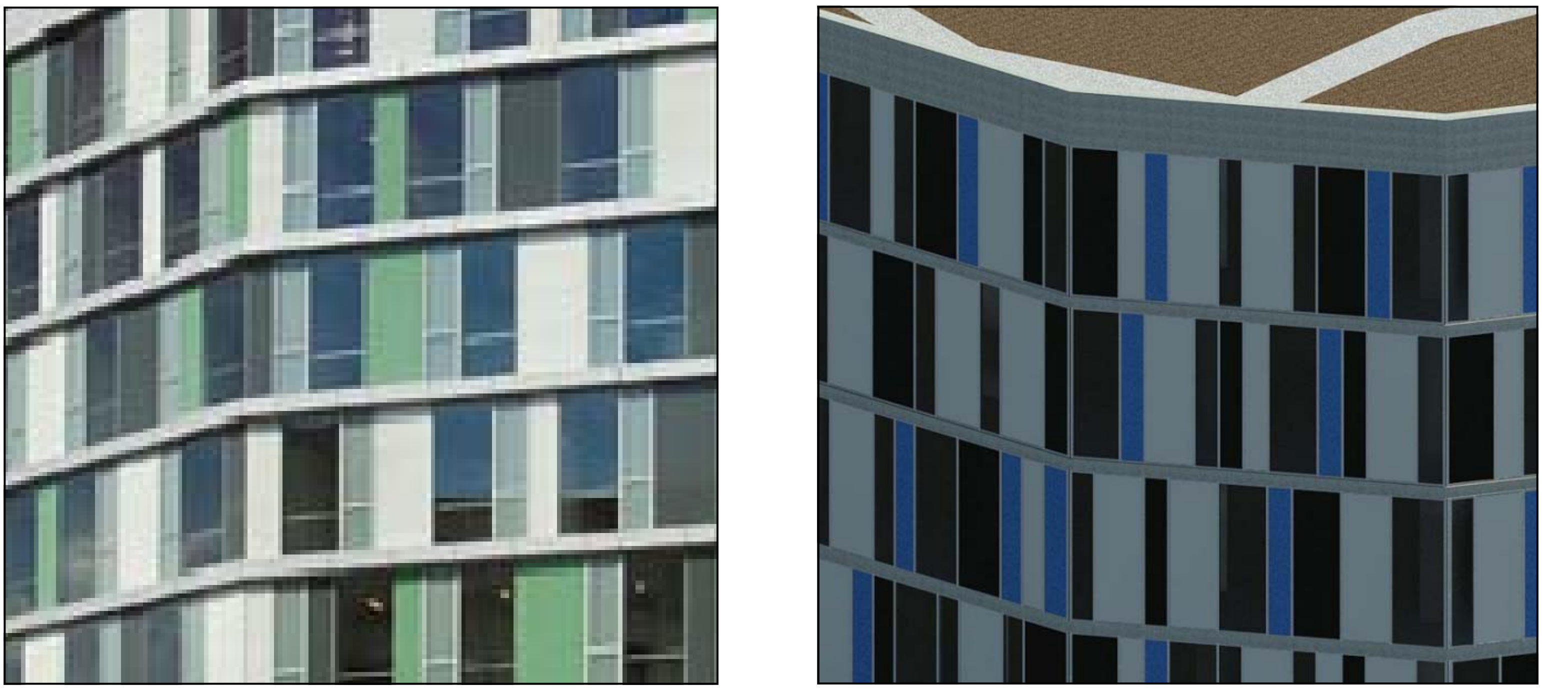

Figure 22: The original Gateway façade at left. Courtesy RTKL. The new Gateway façade at right with darker reflective panels.

Sustainability Breadth:

Sustainability was not a principle design consideration when the Gateway was built. As a signature building on a prestigious college campus, it was deemed that additional sustainability measures should be implemented to bring the building up to LEED certification. A total of 40 LEED points are needed for certification. This breadth analyzes sustainable options from the Sustainable Sites, Water Efficiency, and Materials and Resources sections of the LEED program. In addition to the individual credits from the LEED scorecard, an intensive green roof was also designed. This green roof will be utilized as a sculpture garden for modern art.

Study of the minimum LEED requirements found that the despite accumulating enough points for certification, the Gateway could not become certified. The building cannot be certified because the gross building area is greater than 2% of the gross site area as outlined by credit MPR7 of the LEED program. While the Gateway could not actually be certified on its current site, 40 LEED credits were deemed attainable from the three studied sections of the LEED scorecard. The following is a list of those credits.

- Site Selection: 1 credit

  Building was not developed on prime farmland, previously undeveloped land, land identified as a habitat for threatened or endangered species, land within 100 feet of wet lands, or land that was public parkland.

- Development density and community connection: 5 credits

  Building is located on a previously developed site, within 1/2 mile of a residential area, within 1/2 mile of at least 10 basic services, and has pedestrian access between building and services.

- Alternative transportation– public transportation access: 6 credits

  Building is within 1/2 mile of a rail station and within 1/4 mile of a bus stop.

- Alternative transportation– parking capacity: 2 credit

  Building plan provides no new parking for the site.

- Site development– protect or restore habitat: 1 credit

  The green roof on the building, along with the vegetation in the courtyard restores more than 20% of the total site area with native or adapted vegetation.

- Site development– maximize open space: 1 credit

  Building program provides vegetated open space over 20% of the site area.

- Storm water design– quantity control: 1 credit

  By providing water drainage from roof to a storage tank, rainwater can be captured to limit the amount of water entering the storm drains.

- Heat island effect– roof: 1 credit

  Green roof covers at least 50% of the roof area

- Water efficient landscaping: 4 credits

  Reduce potable water consumption by 50% due to plant species and use of captured rainwater.

- Innovative wastewater technologies: 2 credits

  Use water conserving fixtures and captured rainwater for use in toilets.

- Water use reduction: 4 credits

  Reduce water usage by 40% with low flow toilets, and faucets under the current base line water usage.

- Construction waste management: 2 credits

  Have 75% of nonhazardous construction waste be recycled.

- Materials reuse: 2 credits

  Use salvages, refurbished or reused materials which constitute 10% of the total project cost. Materials reused could include concrete, rebar, and glass.

- Recycled content: 2 credits

  Use materials with recycled content over 20% based on cost.

- Regional materials: 2 credits

  Use materials manufactured within 500 miles of the site. Minimum of 20% based on cost

- Regional priority: 3 credits

  Gaining credits for restoring habitat, stormwater quantity control and innovative wastewater technologies earns 3 credits for Baltimore region.

- Light pollution reduction: 1 credit

  Limit the amount of nighttime light pollution by 50%.

Green Roof Design:

A green roof will be installed on the Gateway as a sculpture garden for the display of modern art. The green roof will be intensive and feature plants that are native or adapted to the region. The plants used will be hardy to withstand disease and cold winters. The courtyard gardens were used as the model to accurately calculate the loads the green roof will induce.

An intensive green roof can provided a number of benefits to a building. The addition of a green roof helped the Gateway achieve a number of credits for LEED certification. Several of the benefits are listed below:

- Storm water retention
- Slows down roof material degradation
- Insulates building interior
- Replaces lost green space
- Reduces CO_2 emissions

A green roof consists of several layers needed for plant grow, insulation, and protection of the structure. Below the plants is a lightweight growing medium or substrate. Inorganic substrates are preferred because they can resist erosion. This green roof was designed to have a growing medium layer of 24," consistent with the courtyard planters. A filter fabric is located below the substrate. This fabric is a very thin layer that prevents fine particles of the substrate from clogging up the drainage system. A drainage medium is located beneath the filter fabric and is usually a highly permeable granular material. This layer collects the excess water the plants do not use and then direct the water into the drainage pipes. Following the drainage layer is a root protection layer aimed at keeping the plant roots from damaging the insulation and waterproofing membranes. Extruded polystyrene of polyisocyanurate are commonly used as insulation in a green roof and EPDM rubber is common for the roof waterproofing material. Figure 23 on the following page shows the layering details for the new green roof

Finally the roof structure is located below the waterproofing. The roof slab was redesigned to accommodate the high loads. Due to the unusually high loads from the green roof and the live load, it was deemed that a one-way slab with beams was more economical than a thicker two-way flat-plate slab. The resulting design features a 12" slab with 30"x24" beams spanning along the interior and exterior column rings. The calculations can be seen in Appendix A.

For plant irrigation, rainwater will be collected and piped to a storage tank located in the museum storage area on the seventh floor. This rainwater will then be used to water the plants during dry weather. A drip irrigation system will be employed in the Gateway. A drip system is a network of small flexible emitter pipes with holes in them that water the plants. They use less water than other systems and are less expensive as well.

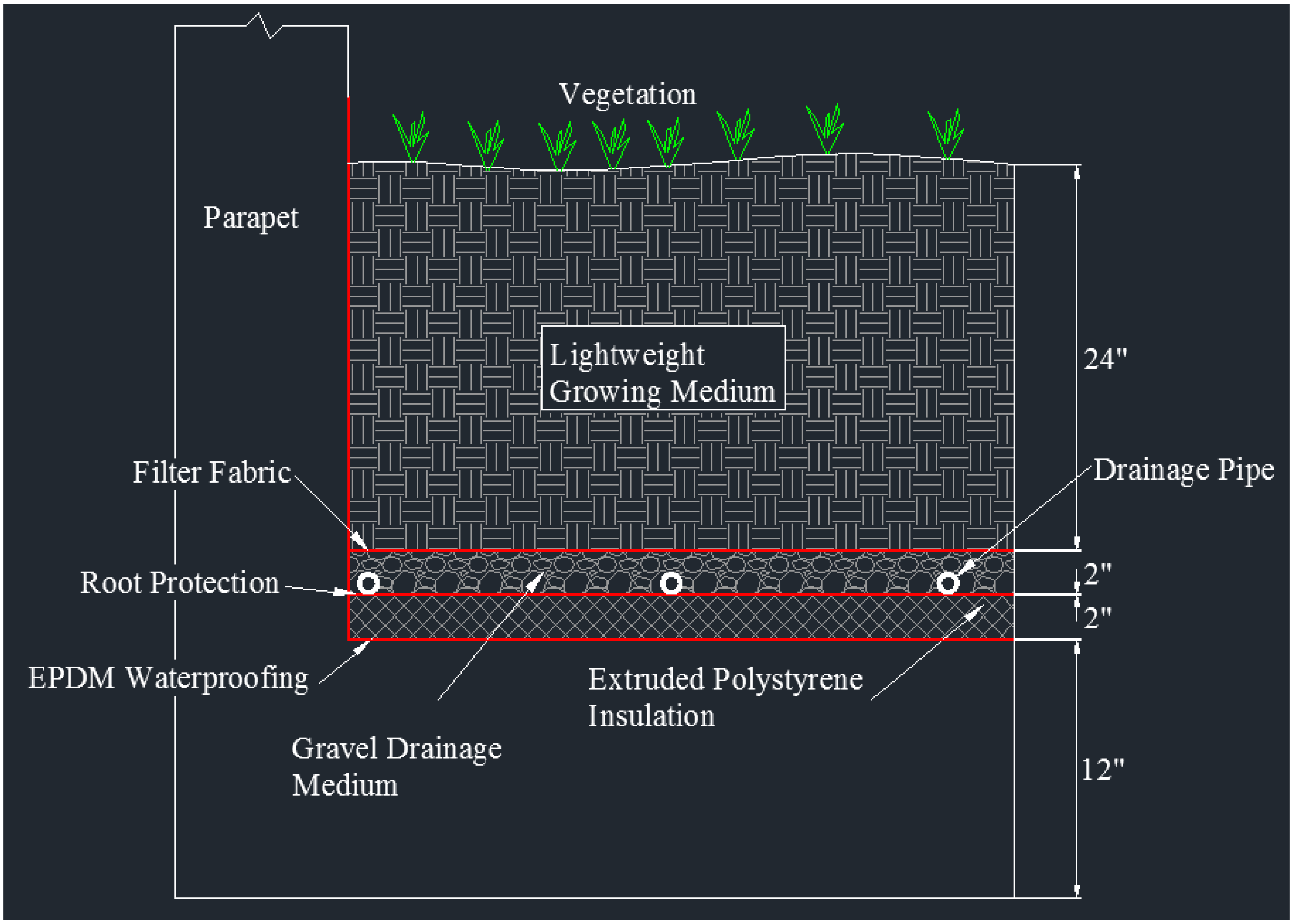

Figure 23: Green roof layer detail.

Graduate Course Integration:

The knowledge learned in AE 597A-Computer Modeling of Buildings was used in this thesis to build and analysis a structural model of the building using ETABS. This model was used to find building deflections, shear, and moments under a variety of loading conditions. AE 542– Building Enclosure Science and Design was also incorporated into this thesis in the two breadth studies. Knowledge of facades was used to design the low light emitting façade for the museum galleries and green roof details.

Final Summary:

The Gateway was adequately designed to fit museum usage, thus meeting the challenge proposed for this thesis. A lengthy design process analyzed and made changes to the building slabs, columns, and lateral system. The end result was that the structure for a museum would have utilized the same systems but would have required stronger capacities to deal with the increased loads on the building. This thesis also considered changes to the building architecture and the inclusion of sustainable measures.

The structural depth analyzed the new loading conditions on the gravity system and then looked at how the changes in the gravity system and architecture impacted the building lateral system. For the Gateway to be a museum the slabs would be have to be increased in depth by 4” and all the columns would have to increase in diameter by 6 or more inches. These additions to the size of the structural members would have made the building more expensive to build as a museum. The increased size and increased floor-to-floor height of several floors also led to changes in the Gateway lateral system. After constructing and analyzing an ETABS model it was found that three concrete shear walls had to be increased in length to resist flexure and all shear walls required an increase in the amount of end zone reinforcement.

The architecture breadth determined appropriate floor plans, circulation patterns, and façade glazing for a museum. Large gallery spaces were designed on every level that allowed people to naturally walk around the building’s circular plan. The columns of the building also had the benefit of breaking the large gallery up into smaller alcoves where art can be viewed. The southern façade of the building was also changed to limit the amount of light entering into the gallery. More solid panels and reflective glass was used in problem areas of the galleries.

The sustainability breadth considered potential credits for LEED certification and looked into the design of a green roof to function as a sculpture garden. Forty credits from the Sustainable Sites, Water Efficiency, and Materials and Resources sections of the scorecard were deemed attainable. The green roof was successfully designed and added to the credit score and overall sustainability of the building.

This thesis reinforced many concepts and procedures related to concrete design and computer modeling. Designing slabs, columns, and shear walls promoted greater familiarity with ACI 318-11. Modeling the structure in ETABS likewise increased understanding of the input parameters and output data of that computer program.

Appendices:

Appendix A: Gravity System Calculations

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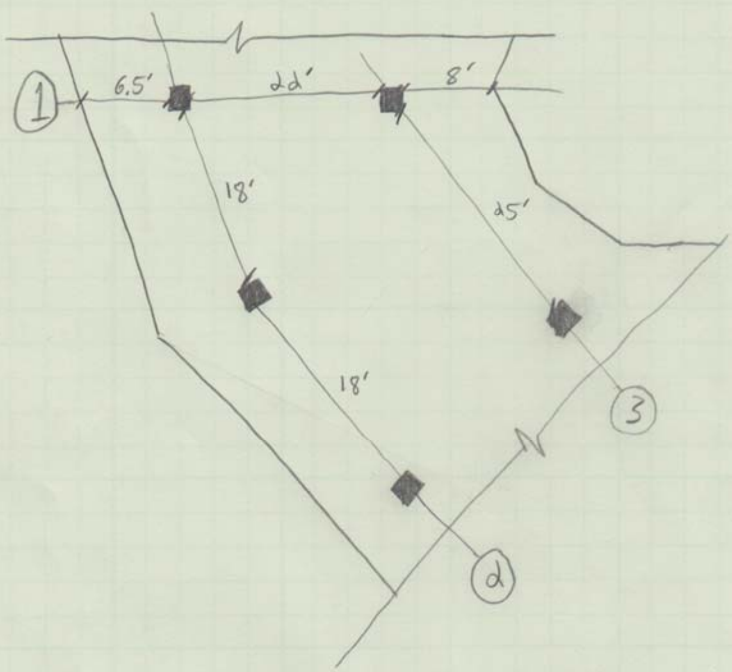
Museum Live Load = 100 psf per IBC 2006
-can be assumed that all of the museum is one long corridor, The weight of the art must also be considered.

Superimposed Dead Load = 9 psf

Use Equivalent Frame Method:



Analyze along lines 1, 2, and 3 to determine appropriate reinforcement

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Check ACI 318-11 for minimum slab thickness:

- Table 9.5(c)

$\ell_n = 20'$, w/out drop panels, exterior panel, w/out edge beams

$f_y = 60$ ksi

$\ell_n/30 = 20\cdot 12/30 = 8''$

Use 8" flat plate slab

Slab-Beam Stiffness:

$$I_1 = \frac{\ell_2 h^3}{12} = \frac{(18\cdot 12)(8)^3}{12} = 9216 \text{ in}^4$$

$$I_2 = \frac{I_1}{(1-c_2/\ell_2)^2} = \frac{9216}{(1-24/18\cdot 12)^2} = 10368 \text{ in}^4$$

$$K_{sb} = \frac{k E I_1}{\ell_1}$$ → From Table A-14 of Wight & MacGregor 2009

$c_2/\ell_2 = 0.111$, $c_1/\ell_1 = 0.091$ → by bi-linear interpolation → $k = 4.183$

$$K_{sb} = \frac{4.183(9216)E}{22\cdot 12} = 146E$$

COF by bi-linear interpolation → COF = 0.513

Column Stiffness:

$$C = \Sigma\left(1-0.63\left(\frac{x}{y}\right)\right)\left(\frac{x^3y}{3}\right) = (1-0.63)(8/24)\left(\frac{8^3\cdot 24}{3}\right) = 3236 \text{ in}^4$$

-assuming all columns are 24"x24"

$$K_t = \Sigma\frac{9EC}{\ell_2(1-c_2/\ell_2)^3} = 2\left[\frac{9\cdot 3236\cdot E}{18\cdot 12(1-24/18\cdot 12)^3}\right] \longrightarrow K_t = 384E$$

$\ell_c = 15'\cdot 12 = 180''$ $\quad \ell_c/\ell_u = 1.05 \quad t_a/t_b = 1$

$\ell_u = 180'' - 8'' = 172''$

From Table A-17 of Wight & MacGregor 2009

$k = 4.52$

$$K_c = \Sigma\frac{kEI}{\ell_c} = 2\left[\frac{4.52\left(\frac{24\cdot 24^3}{12}\right)E}{180}\right] \longrightarrow K_c = 1389E$$

$$\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t} = \frac{1}{1389E} + \frac{1}{384E} \longrightarrow K_{ec} = 301E$$

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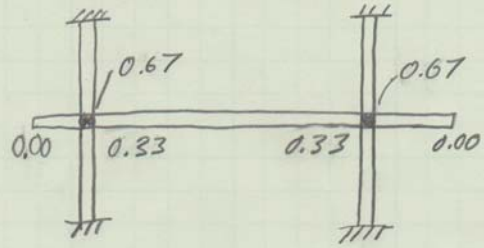
$$DF_{col} = \frac{301}{146+301} = 0.67$$

$$1-0.67 = 0.33$$



FEM:

$$w = 1.2\left(\tfrac{8}{12}\cdot 150 + 9\right) + 1.6(100+20) = 323 \text{ psf}$$

$$M = 0.084\, w\, \ell_2 \ell_1^2 = 0.084(0.323)(18)(22)^2$$

$$M = 236 \text{ ft-k}$$

Cantilever FEM:

- assume curtain wall = 100 plf

Left:

$$M = 1.2(18\cdot 0.1)(6.5') + 1.2(0.109\cdot 18\cdot 6.5)(3.25') + 1.6(0.12\cdot 18\cdot 6.5)(3.25)$$

$$M = 137 \text{ ft-k}$$

Right:

$$M = 1.2(18\cdot 0.1)(8') + 1.2(0.109\cdot 18\cdot 8)(4') + 1.6(0.12\cdot 18\cdot 8)(4')$$

$$M = 204 \text{ ft-k}$$

Moment Distribution:

Joint	C0-16			C0-17		
DF	0	0.67	0.33	0.33	0.67	0
	Cant.	Col.	Slab	Slab	Col.	Cant.
COF			0.513			
FEM	-137		236	-236		-204
BAL		-158	-78	78	158	
CO			40	-40		
BAL		-27	-13	13	27	
CO			6.7	-6.7		
BAL		-4.5	-2.2	2.2	4.5	
CO			1.1	-1.1		
BAL		-0.7	-0.4	0.4	0.7	
Total	-137	-190	190	-190	190	-204

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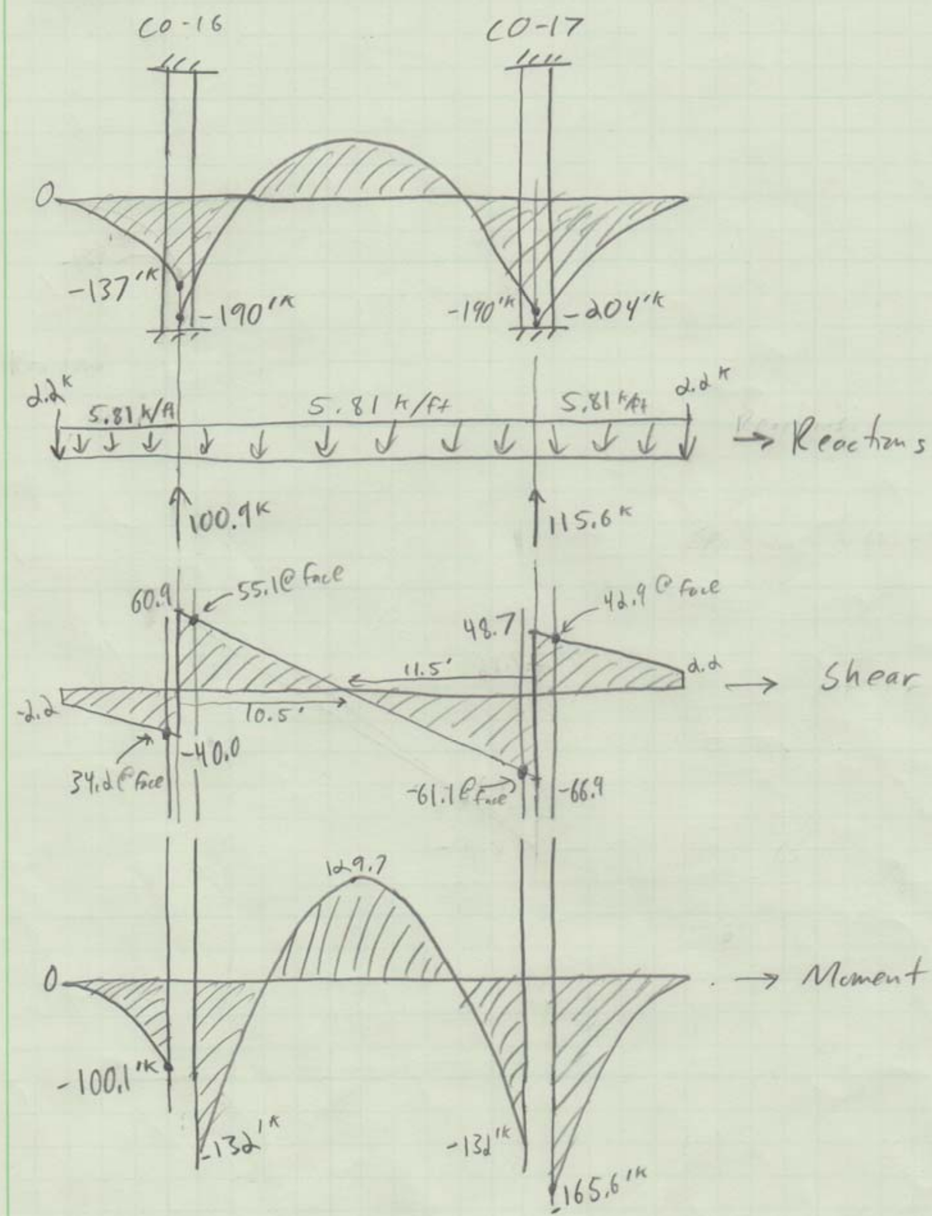
Moment Diagram:



Moment Distribution per DDM: ACI 318-11

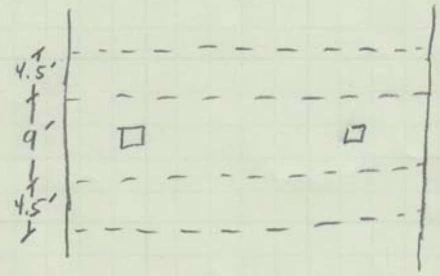
Left Cantilever:

$M^-_{u_{col}} = 0.75 \cdot 100.1 = 75$ ft·k

$M^-_{u_{mid}} = 0.25 \cdot 100.1 = 25$ ft·k

Right Cantilever:

- No edge beam



$M^-_{u_{col}} = 0.75 \cdot 165.5 = 124$ ft·k

$M^-_{u_{mid}} = 0.25 \cdot 165.5 = 41.4$ ft·k

Column Span:

Interior Negative Moment:

$M = -132$ ft·k

- No edge beam ∴

$M^-_{u_{col}} = 0.75 \cdot 132 = 99$ ft·k

$M^-_{u_{mid}} = 0.25 \cdot 132 = 33$ ft·k

Positive Moment:

$\alpha_{f1} \ell_1 \ell_2 = 0$ ∴

$M^+_{o_{col}} = 0.6 \cdot 129.7 = 77.8$ ft·k

$M^+_{o_{mid}} = 0.4 \cdot 129.7 = 51.9$ ft·k

Reinforcement:

Left Cantilever Column Strip:

$M^-_u = -75$ ft·k

$\rightarrow d = 8'' - \frac{3}{4}'' - 0.875 - \frac{0.875}{2} = 5.9375$

$A_{s_{reqd}} = \frac{M_u}{\phi f_y jd} = \frac{75.1 \cdot 12000}{(0.9)(60000)(0.95)(5.9375)}$

$A_{s_{reqd}} = 2.95$ in²

$A_{s_{min}} = 0.0018\, b\, h = 0.0018(9 \cdot 12)(8) = 1.56$ in² < 2.95 in² ✓

Use #7@ 12" → 0.6 in² $\cdot 9' = 5.4$ in² > 2.95 in² ✓

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Reinforcement:

Interior Negative Moment Column Strip:

$M = -99$ ft-k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y j d} = \frac{99 \cdot 12000}{(0.9)(60000)(0.95)(5.9375)}$

$A_{s_{req'd}} = 3.9 \text{ in}^2 > 1.56 \text{ in}^2$ ✓

Use #7 @ 12" $\rightarrow$ $0.6 \text{ in}^2 \cdot 9' = 5.4 \text{ in}^2 > 3.9 \text{ in}^2$ ✓

Positive Moment Column Strip:

$M = 77.8$ ft-k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y j d} = \frac{77.8 \cdot 12000}{(0.9)(60000)(0.95)(5.9375)}$

$A_{s_{req'd}} = 3.07 \text{ in}^2 > 1.56 \text{ in}^2$

Use #7 @ 16" $\rightarrow$ $0.6 \text{ in}^2 \cdot 9 \cdot 0.75 = 4.05 \text{ in}^2 > 3.07 \text{ in}^2$

Right Cantilever Column Strip:

$M = -124$ ft-k

$A_{s_{req'd}} = \frac{M_u}{\phi f_y j d} = \frac{124 \cdot 12000}{(0.9)(60000)(0.95)(5.9375)}$

$A_{s_{req'd}} = 4.89 \text{ in}^2$

Use #7 @ 12" $= 0.6 \text{ in}^2 \cdot 9 = 5.4 \text{ in}^2 > 4.89 \text{ in}^2$

Positive Moment Middle Strip:

$M = 51.9$ ft-k

$A_{s_{req'd}} = \frac{M_u}{0.9 f_y j d} = \frac{51.9 \cdot 12000}{(0.9)(60000)(0.95)(5.9375)}$

$A_{s_{req'd}} = 2.04 \text{ in}^2$

Use #7 @ 16" $= 4.05 \text{ in}^2 > 2.04 \text{ in}^2$ ✓

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Punching shear:

$w = 323$ psf

$d = 5.9375''/2 = 2.969'' \cdot 2 + 24 = 29.938''$

$V_u = 0.323\left(17.5 \cdot 18 - \left(\frac{29.938}{12}\right)^2\right)$

$V_u = 99.7$ k

$\phi V_n = \phi 6\sqrt{f'_c}\, b_o d = 0.75(6)\sqrt{4000}\,(29.938 \cdot 4)(5.9375)$

$\phi V_n = 202$ k > 99.6 k

$V_c = 2\lambda\sqrt{f'_c}\, b_o d = 2\sqrt{4000}\,(119.75)(5.9375)(0.75)$

$\phi V_c = 67.5$ k

$A_v = \frac{(V_u - \phi V_c)s}{\phi f_y d} = \frac{(99.7 - 67.5)(2)}{(0.75)(60)(5.9375)}$

$A_v = 0.24$ in^2

Use #5 @ 2" to distance: $\rightarrow$ may be too large

$V_u \le \phi 2\lambda\sqrt{f'_c}\, b_o d$

$99700 \le 0.75 \cdot 2\sqrt{4000} \cdot 4(24 + a\sqrt{2}) \cdot 5.9375$

$a = 14.3'' \rightarrow \underline{16''}$

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Deflection Check:

$w = 323$ psf

Dead Load $= 1.0(100+9) = 109$ psf

Service Load $= 1.0(109+120) = 229$ psf

Const. Load $= 2.0(100) = 200$ psf

$M_a = 229/323 = 0.709$

Neg. Moment $= 0.709 \cdot 132 = 94$ ft·k

Pos. Moment $= 0.709 \cdot 77.8 = 55$ ft·k

$I_g = \frac{(9\cdot12)(8)^3}{12} = 4608$ in^4

$M_{cr} = \frac{f_r I_g}{y_t} = \frac{403 \cdot 4608}{4}/12000 = 39$ ft·k

$n = \frac{E_s}{E_c} = \frac{29 \cdot 10^6}{57000\sqrt{4000}} = 8.1$

$n\rho = 0.0648$

$\rho = \frac{5.4}{9\cdot12\cdot5.9375} = 0.008$

$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$

$k = 0.301$

Neg $I_{cr} = \frac{b k^3 d^3}{3} + nA_s(d-kd)^2$

$= \frac{(9\cdot12)(0.301\cdot5.9375)^3}{3} + 8.1(5.4)(5.9375 - 0.301\cdot5.9375)^2$

Neg. $I_{cr} = 955$ in^4

Neg $I_e = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3 = 955 + (4608 - 955)\left(\frac{39}{94}\right)^3$

Neg $I_e = 1216$ in^4

Positive Moment:

$\rho = \frac{4.05}{108\cdot5.9375} = 0.006$

$n\rho = 0.0486$

$k = 0.267$

Pos. $I_{cr} = \frac{(108)(0.267\cdot5.9375)^3}{3} + 8.1(4.05)(5.9375 - 0.267\cdot5.9375)^2$

Pos. $I_{cr} = 771$ in^4

Pos. $I_e = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3 = 771 + (4608 - 771)(39/55)^3$

Pos $I_e = 2139$ in^4

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Deflection check:

$(I_e)_{avg} = 0.7 I_{em} + 0.15(I_{e1} + I_{e2})$

$(I_e)_{avg} = 0.7(2139) + 0.15(1216 + 1216)$

$(I_e)_{avg} = 1862 \text{ in}^4$

$w_D = 0.109 \cdot 18 \cdot 0.6 = 1.18 \text{ k/ft}$

$w_L = 0.120 \cdot 18 \cdot 0.6 = 1.30 \text{ k/ft}$

$\Delta_{D\,max} = 0.0026 \dfrac{w_D \ell^4}{EI} = 0.0026 \dfrac{(1.18)(22)^4(1728)}{(3600)(1862)} = 0.185 \text{ in.}$

$\Delta_{L\,max} = 0.0048 \dfrac{w_L \ell^4}{EI} = 0.0048 \dfrac{(1.30)(22)^4(1728)}{(3600)(1862)} = 0.377 \text{ in.}$

Long-Term:

$\lambda = 2.0$

$\Delta_{cp+sh} = \lambda(\Delta_i)_{sus} = 2.0(0.5 \cdot 0.377 + 0.185) = 0.747 \text{ in}$

$\Delta_{long\text{-}term} = 0.747 + 0.377 = 1.124 \text{ in} > \ell/480 = 0.55'' \quad \times$

No good → increase slab thickness (12")

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Exterior Column Ring:

Use 12" slab

Slab-Beam Stiffness:

$$I_1 = \frac{(22\cdot 12)(12)^3}{12} = 38016 \text{ in}^4$$

$$K_{sb} = \frac{kEI_1}{\ell_1} \qquad c_2/\ell_2 = 0.091, \quad c_1/\ell_1 = 0.111 \rightarrow k = 4.183, \quad COF = 0.513$$

$$K_{sb} = \frac{(4.183)(38016)E}{18\cdot 12} = 736E$$

Column Stiffness:

$$C = \Sigma\left(1-0.63\right)\left(12/24\right)\left(\frac{12^3\cdot 24}{3}\right) = 9469 \text{ in}^4$$

$$K_t = \Sigma \frac{9EC}{\ell_2(1-c_2/\ell_2)^3} = 2\left[\frac{9\cdot 9469\cdot E}{22\cdot 12(1-24/22\cdot 12)^3}\right] = 859E$$

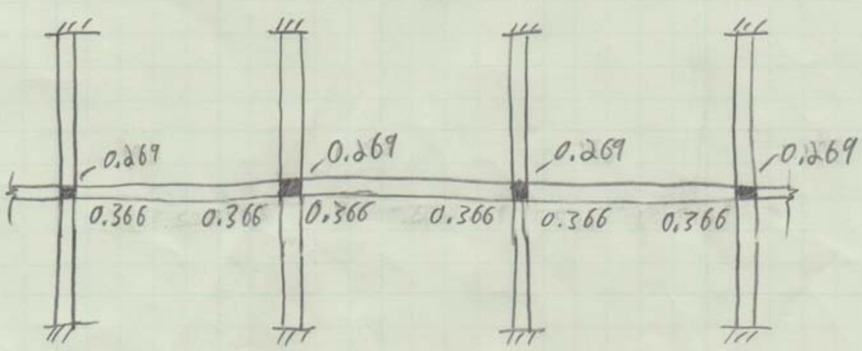
$$k = 4.75$$

$$K_c = \Sigma \frac{kEI}{\ell_c} = 2\left[\frac{4.75\left(\frac{24\cdot 24^3}{12}\right)E}{180}\right] = 1459E$$

$$\frac{1}{K_{ec}} = \frac{1}{K_c} + \frac{1}{K_t} = \frac{1}{1459E} + \frac{1}{859E} \longrightarrow K_{ec} = 541E$$

$$DF_{col} = \frac{541}{541+736\cdot 2} = 0.269$$

$$DF_{slab} = \frac{736}{541+736\cdot 2} = 0.366$$



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FEM:

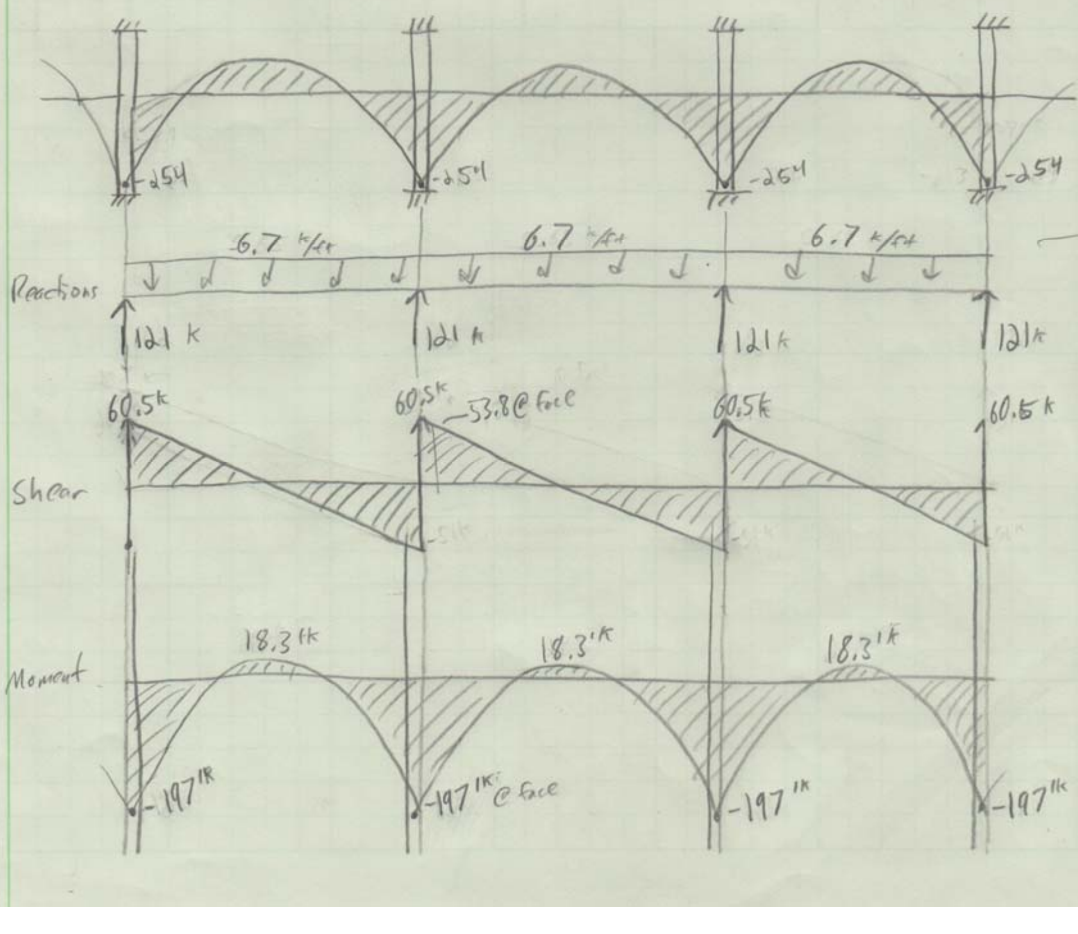
$w = 323\ psf$

$M = 0.084\, w\, l_2 l_1^2 = 0.084(0.323)(22)(18)^2$

$M = 229\ ft\text{-}k$

Moment Distribution:

Joint	CO-16		CO-67			CO-22			CO-66	
DF	0.269	0.366	0.366	0.269	0.366	0.366	0.269	0.366	0.366	0.269
	Col.	Slab	Slab	Col	Slab	Slab	Col	Slab	Slab	Col
COF		0.513			0.513			0.513		
FEM		229	-229		229	-229		229	-229	
BAL	-61.6	-83.8							83.8	61.6
CO			-43					43		
BAL			15.7	11.6	15.7	-15.7	-11.6	-15.7		
CO		8.1			-8.1	8.1			-8.1	
BAL	-2.2	-3.0	3.0	2.2	3.0	-3.0	-2.2	-3.0	3.0	2.2
CO		1.5	-1.5		-1.5	1.5		1.5	-1.5	
BAL	-0.4	-0.5	1.1	0.8	1.1	-1.1	-0.8	-1.1	0.5	0.4
Total	-62.4	151.3	-254	14.6	239.2	-239.2	-14.6	254	-151.3	62.4



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Moment Distribution per DDM:

Exterior Negative Moment:

$\alpha_{f_1} = 0 \quad \therefore$

$M^-_{u_{col}} = 1.00 \cdot -197 = -197 \text{ ft}\cdot\text{k}$

Positive Moment:

$\alpha_{f_1} = 0 \quad \therefore$

$M^+_{u_{col}} = 0.6 \cdot 18 = 10.8 \text{ ft}\cdot\text{k}$

$M^+_{u_{mid}} = 0.4 \cdot 18 = 7.2 \text{ ft}\cdot\text{k}$

Reinforcement:

Exterior Neg. Moment Column Strip:

$d = 12'' - 0.75'' - 0.625/2 = 10.94''$

$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{197 \cdot 12000}{(0.9)(60000)(0.95)(10.94)}$

$A_{s_{req}} = 4.21 \text{ in}^2$

$A_{s_{min}} = 0.0018(11 \cdot 12)(12) = 2.85 \text{ in}^2 < 4.21 \text{ in}^2$ ✓

Use $\#6 @ 12'' = 0.44 \cdot 11 = 4.84 \text{ in}^2 > 4.21 \text{ in}^2$ ✓

Positive Moment Column Strip:

$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{9.6 \cdot 12000}{(0.9)(60000)(0.95)(10.94)} = 0.34 \text{ in}^2 < 2.85 \text{ in}^2$

Use $\#6 @ 18'' = 3.23 \text{ in}^2 > 2.85 \text{ in}^2$ ✓

~~Interior Neg. Moment Middle Strip:~~

$\cancel{A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{42 \cdot 12000}{(0.9)(60000)(0.95)(10.94)} = 0.9 \text{ in}^2 < 2.85 \text{ in}^2}$

$\cancel{\text{Use } \#6 @ 18'' = 3.23 \text{ in}^2 > 2.85 \text{ in}^2}$ ✓

Positive Moment Middle Strip:

$A_{s_{min}} = 2.85 \text{ in}^2$

Use $\#6 @ 18'' = 3.23 \text{ in}^2 > 2.85 \text{ in}^2$ ✓

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Punching Shear Check:

$w = 383$ psf

$d = 10.94/2 = 5.5" \cdot 2 = 11 + 24 = 35$ in

$V_u = 0.383(17.5 \cdot 18 - (35/12)^2) = 117^k$

$\phi V_n = \phi 6\sqrt{f'c}\, b_o d = 0.75(6)\sqrt{4000}(35 \cdot 4)(10.94)$

$\phi V_n = 436\ k > 117\ k$

$\phi V_c = \phi 2 \sqrt{f'c}\, b_o d = 2\sqrt{4000}(140)(10.94)(0.75)$

$\phi V_c = 145\ k$

No shear reinforcement necessary

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Deflection Check:

$w = 383$ psf

Dead Load = 159 psf

Service Load = 229 psf

Const. Load = 200 psf

$M_a = 229/383 = 0.598$, $M_{cr} = \frac{f_r \cdot I_g}{y} = \frac{403 \cdot 19008}{6} = 106$ ft-k

Neg. Moment = $0.598 \cdot 197 = 118$ ft-k

Pos. Moment = $0.598 \cdot 10.8 = 6.5$ ft-k

$n = 8.1$

Neg. Moment:

$\rho = \frac{4.84}{11 \cdot 12 \cdot 10.94} = 0.003$

$\rho n = 0.0243$

$k = \sqrt{2(0.0243) + (0.0243)^2} - 0.0243$

$k = 0.197$

Neg. $I_{cr} = \frac{bk^3d^3}{3} + nA_s(d - kd)^2$

$= \frac{(11 \cdot 12)(0.197 \cdot 10.94)^3}{3} + (8.1)(4.84)(10.94 - 0.197 \cdot 10.94)^2$

Neg $I_{cr} = 3465$ in^4

Neg $I_e = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3 = 3465 + (19008 - 3465)(106/118)^3$

Neg $I_e = 14732$ in^4

Positive Moment:

$M_a = 106/9.6 = 11 > 1.0$ $\therefore$ Pos $I_e = 19008$ in^4

$(I_e)_{avg} = 0.7(19008) + 0.15(14732 + 14732) = 17725$ in^4

$w_D = 0.159 \cdot 17.5 \cdot 0.6 = 1.67$ k/ft

$w_L = 0.12 \cdot 17.5 \cdot 0.6 = 1.26$ k/ft

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Deflection Check:

$$\Delta_{D_{max}} = 0.0026 \frac{(1.67)(18)^4(1728)}{(3600)(17725)} = 0.012 \text{ in}$$

$$\Delta_{\ell_{max}} = 0.0048 \frac{(1.26)(18)^4(1728)}{(3600)(17725)} = 0.017 \text{ in}$$

Long-Term:

$$\lambda = 2.0$$

$$\Delta_{cp+sh} = \lambda(\Delta_i)_{sus} = 2.0(0.5 \cdot 0.017 \text{ in} + 0.012) = 0.041 \text{ in}$$

$$\Delta_{long\text{-}term} = 0.041 + 0.017 = 0.058 \text{ in} < \ell/480 = 0.45 \text{ in} \checkmark$$

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Interior Column Ring:

use 12" slab

Slab-Beam Stiffness:

$I_1 = \frac{(22 \cdot 12)(12)^3}{12} = 38016 \text{ in}^4$

$c_2/\ell_2 = 0.091 \rightarrow k = 4.134$

$c_1/\ell_1 = 0.08 \quad COF = 0.510$

$k_{sb} = \frac{kEI_1}{\ell_1} = \frac{(4.134)(38016)E}{25 \cdot 12} = 524\ E$

Column Stiffness:

$C = 9469 \text{ in}^4$

$k_t = \sum \frac{9EC}{\ell_2(1 - c_2/\ell_2)^3} = 2\left[\frac{9(9469)E}{22 \cdot 12(1 - 24/22 \cdot 12)^3}\right] = 859\ E$

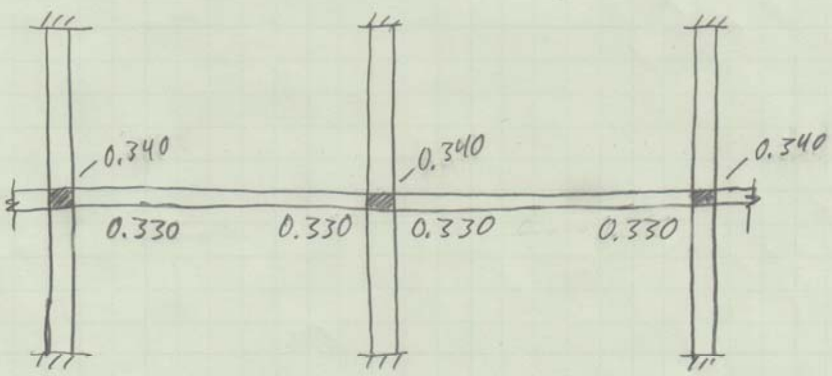
$k = 4.75$

$k_c = \sum \frac{kEI}{\ell_c} = 2\left[\frac{4.75\left(\frac{24 \cdot 24^3}{12}\right)E}{180}\right] = 1459E$

$\frac{1}{k_{ec}} = \frac{1}{k_1} + \frac{1}{k_t} = \frac{1}{1459E} + \frac{1}{859E} \longrightarrow k_{ec} = 541\ E$

$DF_{col} = \frac{541}{541 + 524 \cdot 2} = 0.340$

$DF_{slab} = \frac{524}{541 + 524 \cdot 2} = 0.330$



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FEM:

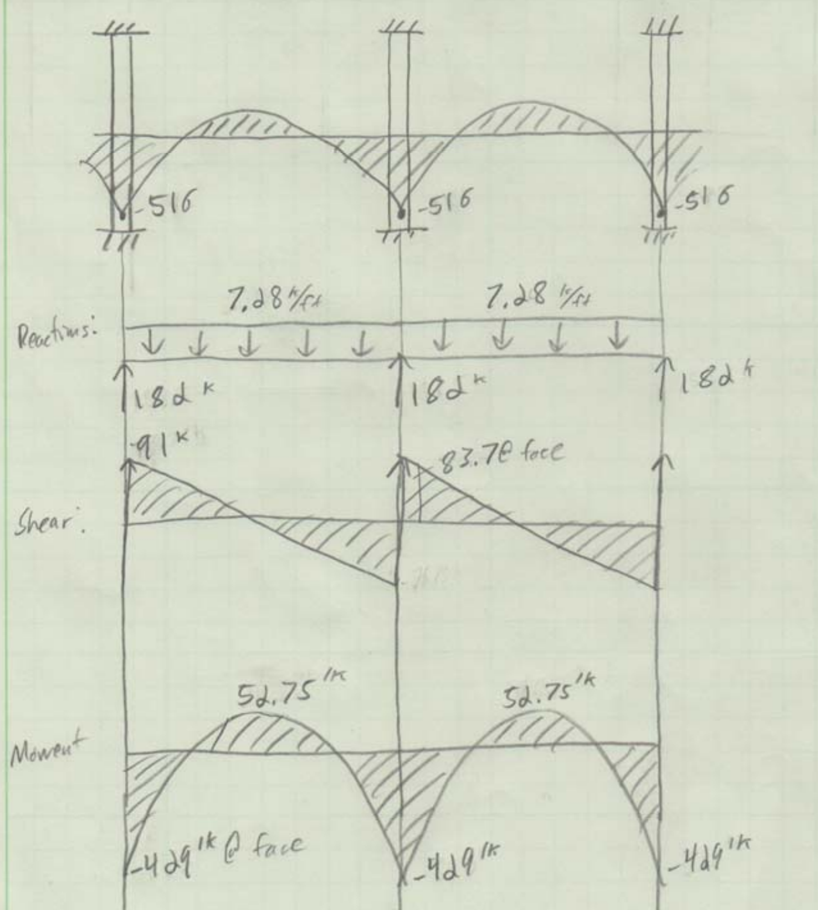
$w = 383$ psf

$M = 0.084\, w\, \ell_2\, \ell_1^2 = 0.084(0.383)(22)(25)^2$

$M = 442$ ft-k

Moment Distribution:

Joint	CO-17		CO-20			CO-23	
DF	0.340	0.330	0.330	0.340	0.330	0.330	0.340
	Col	Slab	Slab	Col	Slab	Slab	Col
COF		0.510			0.510		
FEM		442	-442		442	-442	
BAL	-150	-146				146	150
CO			-74.5		74.5		
BAL	0	0	0	0	0	0	0
Total	-150	296	-516		516	-296	150



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Moment Distribution per DDM:

Exterior Negative Moment:

$\alpha_{f_1} = 0 \therefore$

$M^-_{u_{col}} = 1.00 \cdot 429 = -429$ ft-k

Positive Moment:

$\alpha_{f_1} = 0 \therefore$

$M^+_{u_{col}} = 0.6 \cdot 52.75 = 31.7$ ft-k

$M^+_{u_{mid}} = 0.4 \cdot 52.75 = 21.1$ ft-k

Reinforcement:

Exterior Neg. Moment Column Strip:

$d = 10.94''$

$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{429 \cdot 12000}{(0.9)(60000)(0.95)(10.94)}$

$A_{s_{req}} = 9.17$ in²

$A_{s_{min}} = 0.0018(11 \cdot 12)(12) = 2.85$ in² < 9.17 in² ✓

Use #6 @ 6" $= 0.6 \cdot 11 \cdot 2 = 9.68$ in² > 9.17 in² ✓

Positive Moment Column Strip:

$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{31.7 \cdot 12000}{(0.9)(60000)(0.95)(10.94)} = 1.25$ in² < 2.85 in²

Use #6 @ 18" = 3.23 in² > 2.85 in² ✓

~~Interior Neg. Moment Middle Strip:~~

~~$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{93 \cdot 12000}{(0.9)(60000)(0.95)(10.94)} = 1.98$ in² < 2.85 in²~~

~~Use #6 @ 18" = 3.23 in² > 2.85 in²~~ ✓

Positive Moment Middle Strip:

$A_{s_{req}} = \frac{M_u}{\phi f_y j d} = \frac{21.1 \cdot 12000}{(0.9)(60000)(0.95)(6.813)} < 3.28$ in²

Use #6 @ 18" = 3.23 in² > 2.85 in² ✓

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Punching Shear Check:

$w = 383$ psf

$d = 35''$

$V_u = 0.383\left(19.25 - (35/12)^2\right) = 178.7k$

$\phi V_n = \phi 6\sqrt{f'_c}\, b_o d = 0.75(6)\sqrt{4000}(140)(10.94)$

$\phi V_n = 436\,k > 178.7\,k$

$\phi V_c = \phi 2\lambda\sqrt{f'_c}\, b_o d = 0.75(2)\sqrt{4000}(140)(10.94)$

$\phi V_c = 145.3k$

$$A_v = \frac{(178.7 - 145.3)(3'')}{(0.75)(10.94)(60)}$$

$A_v = 0.20\ in^2$

Use #4 @ 3" to distance:

$V_u \le \phi 2\lambda\sqrt{f'_c}\, b_o d$

$150700 = 0.75(2)\sqrt{4000}\cdot 4(24 + a\sqrt{2})(10.94)$

$a = 25.7'' \longrightarrow 27''$

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Deflection Check:

$w = 383$ psf

Dead Load = 159 psf

Service Load = 229 psf

Const. Load = 200 psf

$M_u = 229/383 = 0.598$

$M_{cr} = \frac{f_r I_g}{y} = \frac{403 \cdot 19008}{6} = 106$ ft·k

Neg. Moment = 0.598 · 429 = 257 ft·k

Pos. Moment = 0.598 · 31.7 = 19 ft·k

$n = 8.1$

Neg. Moment:

$\rho = \frac{9.68}{11 \cdot 12 \cdot 10.94} = 0.007$

$\rho n = 0.0567$

$k = 0.285$

$$\text{Neg } I_{cr} = \frac{bk^3d^3}{3} + nA_s(d-kd)^2$$

$$= \frac{(11 \cdot 12)(0.285 \cdot 10.94)^3}{3} + (8.1)(9.68)(10.94 - 0.285 \cdot 10.94)^2$$

Neg $I_{cr} = 6129$ in^4

$$\text{Neg } I_e = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3 = 6129 + (19008 - 6129)\left(106/257\right)^3$$

Neg $I_e = 7123$ in^4

Pos. Moment:

$M_a = 106/14.7 = 7.2 > 1.0$ ∴ Pos $I_e = 19008$ in^4

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Deflection Check:

$(I_e)_{avg} = 0.7(19008) + 0.15(7123 + 7123)$

$(I_e)_{avg} = 15443 \text{ in}^4$

$w_D = 0.159 \cdot 19 \cdot 0.6 = 1.81 \text{ k/ft}$

$w_\ell = 0.120 \cdot 19 \cdot 0.6 = 1.37 \text{ k/ft}$

$$\Delta_{D_{max}} = 0.0026 \frac{(1.81)(25)^4(1728)}{(3600)(15443)} = 0.057 \text{ in}$$

$$\Delta_{\ell\, max} = 0.0048 \frac{(1.37)(25)^4(1728)}{(3600)(15443)} = 0.080 \text{ in}$$

Long-Term:

$\lambda = 2.0$

$\Delta_{cp+sh} = \lambda (\Delta_i)_{sus} = 2.0(0.5 \cdot 0.080 + 0.057) = 0.194$

$\Delta_{long\text{-}term} = 0.194 + 0.080 = 0.274 \text{ in} < \ell/480 = 0.625 \text{ in}$

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Frame 1

Re-Design w/ 12" thick slab

Slab-Beam stiffness:

$$I_1 = \frac{\ell_2 h^3}{12} = \frac{(18\cdot 12)(12)^3}{12} = 31104 \text{ in}^4$$

$k = 4.183$

$COF = 0.513$

$$K_{sb} = \frac{k E I_1}{\ell_1} = \frac{(4.183)(31104)E}{22\cdot 12} = 493E$$

Column Stiffness:

$C = 9469 \text{ in}^4$

$$k_t = \Sigma \frac{9EC}{\ell_2(1-c_2/\ell_2)^3} = 2\left[\frac{9\cdot 9469E}{18\cdot 12(1-24/216)^3}\right] = 1124E$$

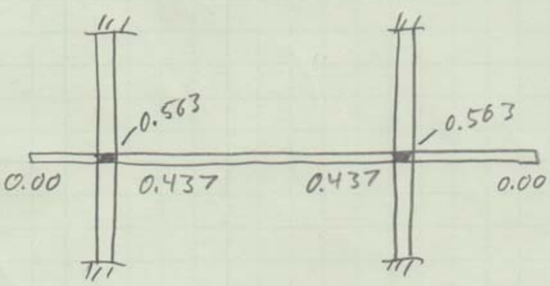
$K = 4.75$

$$k_c = \Sigma \frac{kEI}{\ell_c} = 2\left[\frac{4.75\left(\frac{24\cdot 24^3}{12}\right)E}{180}\right] = 1459E$$

$$\frac{1}{K_{ec}} = \frac{1}{k_c} + \frac{1}{k_t} = \frac{1}{1459E} + \frac{1}{1124E} \rightarrow K_{ec} = 635E$$

$$DF_{col} = \frac{635}{493+635} = 0.563$$

$$DF_{slab} = 1 - 0.563 = 0.437$$



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FEM:

$w = 383$ psf

$M = 0.084 \cdot (0.383)(18)(22)^2$

$M = 280$ ft·k

Left Cantilever FEM:

$M = 1.2(18 \cdot 0.1)(6.5') + 1.2(0.159 \cdot 18 \cdot 6.5)(3.25) + 1.6(0.12 \cdot 18 \cdot 6.5)(3.25)$

$M = 160$ ft·k

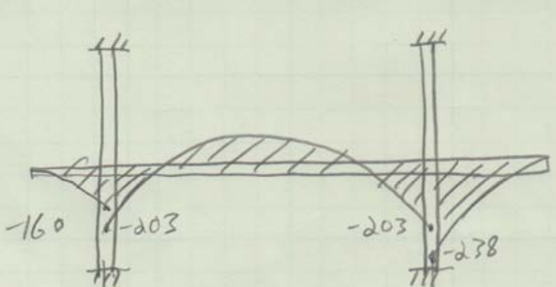
Right Cantilever FEM:

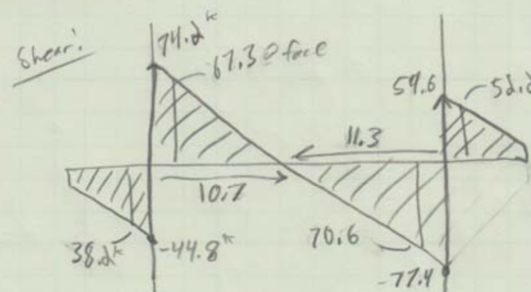
$M = 1.2(18 \cdot 0.1)(8) + 1.2(0.159 \cdot 18 \cdot 8)(4) + 1.6(0.12 \cdot 18 \cdot 8)(4)$

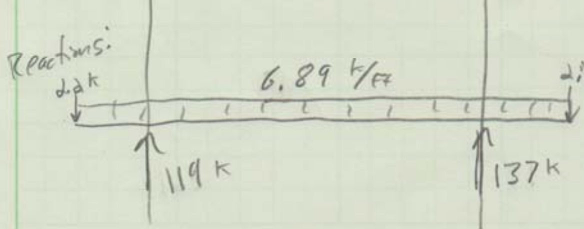
$M = 238$ ft·k

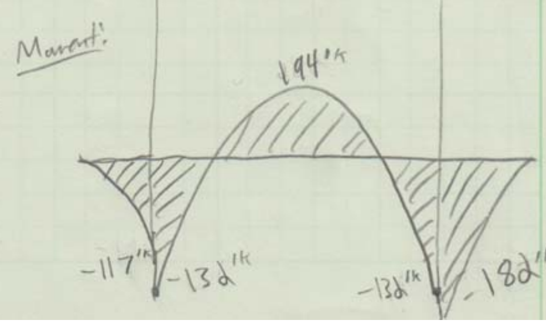
Moment Distribution:

Joint	C0-16			C0-17		
DF	0	0.563	0.437	0.437	0.563	0
	Cant.	Col	Slab	Slab	Col	Cant
COF			0.513			
FEM	-160		280	-280		-238
BAL		-158	-122	122	158	
CO			63	-63		
BAL		-35.5	-27.5	27.5	35.5	
CO			14.1	-14.1		
BAL		-7.9	-6.2	6.2	7.9	
CO			3.2	-3.2		
BAL		-1.8	-1.4	1.4	1.8	
Total	-160	-203	203	-203	203	-238









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Moment Distribution per DDM:

Left Cantilever:

$M_{u\,col}^- = 0.75 \cdot 117 = 88$ ft·k

$M_{u\,mid}^- = 0.25 \cdot 117 = 29$ ft·k

Right Cantilever:

$M_{u\,col}^- = 0.75 \cdot 182 = 137$ ft·k

$M_{u\,mid}^- = 0.25 \cdot 182 = 45$ ft·k

Interior Neg. Moment:

$M_{u\,col}^- = 0.75 \cdot 132 = 99$ ft·k

$M_{u\,mid}^- = 0.25 \cdot 132 = 33$ ft·k

Positive Moment:

$M_{u\,col}^+ = 0.6 \cdot 194 = 116$ ft·k

$M_{u\,mid}^+ = 0.4 \cdot 194 = 78$ ft·k

Reinforcement:

$d = 12'' - 3/4'' - 0.625 - 0.625/2 = 10.31''$

Left Conti. Col. Strip:

$A_{s\,reqd} = \dfrac{88 \cdot 12000}{0.9 \cdot 60000 \cdot 0.95 \cdot 10.31}$

$A_{s\,reqd} = 2.00$ in^2

$A_{s\,min} = 0.0018(9 \cdot 12)(12) = 2.33$ in^2 > 2.00 in^2

Use #6 @ 12" $= 3.96$ in^2 > 2.33 in^2 ✓

Left Conti. Middle Strip:

$A_{s\,reqd} = \dfrac{29 \cdot 12000}{\phi f_y jd} < 2.33$ in^2

Use #6 @ 12"

Right Conti. Col. Strip:

$A_{s\,reqd} = \dfrac{137 \cdot 12000}{0.9(60000)(0.95)(10.31)} = 3.11$ in^2

Use #6 @ 12"

Reinforcement:

Right Canti Middle Strip:

Use #6 @ 12" = $3.96 \text{ in}^2 > 2.33 \text{ in}^2$

Interior Neg. Moment Col. Strip:

$$A_{s_{reqd}} = \frac{99 \cdot 12000}{0.9(60000)(0.95)(10.31)} = 2.24 \text{ in}^2 < 2.33 \text{ in}^2$$

Use #6 @ 12"

Interior Neg. Moment Middle Strip:

Use #6 @ 12"

Positive Moment Column Strip:

$$A_{s_{reqd}} = \frac{116 \cdot 12000}{0.9(60000)(0.95)(10.31)} = 2.63 \text{ in}^2$$

Use #6 @ 12"

Positive Moment Middle Strip:

Use #6 @ 12"

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Punching Shear Check:

$w = 383$ psf

$d = 10.31/2 = 5.2'' \cdot 2 + 24 = 34.4''$

$V_u = 0.383\left(17.5 \cdot 18 - (34.4/12)^2\right)$

$V_u = 117$ k

$\phi V_n = \phi 6 \sqrt{f'_c}\, b_o d = 0.75(6)(\sqrt{4000})(34.4 \cdot 4)(10.31)$

$\phi V_n = 404$ k > 117 k

$\phi V_c = \phi 2 \lambda \sqrt{f'_c}\, b_o d = 0.75(2)\sqrt{4000}(34.4 \cdot 4)(10.31)$

$\phi V_c = 135$ k > 117 k

No shear reinforcement necessary

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Deflection Check:

$w = 383$ psf

Dead $= 159$ psf

Service $= 229$ psf

Const. $= 200$ psf

$M_a = 229/383 = 0.598$

$I_g = 15552$ in^4

$$M_{cr} = \frac{(403)\left(\frac{9 \cdot 12 \cdot 12^3}{12}\right)}{6} = 87 \text{ ft·k}$$

Neg. Moment $= 0.598 \cdot 137 = 82$ ft·k

Pos. Moment $= 0.598 \cdot 45 = 27$ ft·k

$n = 8.1$

$$\rho = \frac{3.96}{9 \cdot 12 \cdot 10.31} = 0.004$$

$\rho n = 0.0324$

$k = 0.224$

M_{cr}/M_a for Pos. & Neg. $\therefore$ $(I_e)_{avg} = 15552$ in^4

$w_D = 0.159 \cdot 18 \cdot 0.6 = 1.72$ k/ft

$w_\ell = 0.120 \cdot 18 \cdot 0.6 = 1.30$ k/ft

$$\Delta_{D\,max} = 0.0026 \frac{(1.72)(22)^4(1728)}{(3600)(15552)} = 0.032 \text{ in}$$

$$\Delta_{\ell\,max} = 0.0048 \frac{(1.3)(22)^4(1728)}{(3600)(15552)} = 0.045 \text{ in}$$

Long-Term:

$\lambda = 2.0$

$\Delta_{cp+sh} = \lambda(\Delta_i)_{sus} = 2.0(0.5 \cdot 0.045 + 0.032) = 0.109$ in

$\Delta_{long\text{-}term} = 0.109 + 0.045 = 0.154$ in $> \ell/480 = 0.55''$ ✓

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Development Length:

-per ACI 318-11, Figure 13.3.8

Column Strip requires continuous bottom reinforcement.:
Make all bottom reinforcement continuous.
Terminate at edge of slab with a 90° hook at least 6" - Section 13.3.3 and 13.3.5

Column Strip top reinforcement development length:
$0.30 \ell_n$ for slabs w/out drop panels.

Frame Line 1: $22 \cdot 12 - 30 = 234'' \cdot 0.3 = 70.2/12 = 5.85' \longrightarrow 6'$

Frame Line 2: $18 \cdot 12 - 30 = 186'' \cdot 0.3 = 55.8/12 = 4.65' \longrightarrow 5'$

Frame Line 3: $25 \cdot 12 - 30 = 270'' \cdot 0.3 = 81/12 = 6.75' \longrightarrow 7'$

Top bar hooks are designed per Section 12.5:

$$\ell_{dh} = \left(\frac{0.2 \psi_e f_y}{\lambda \sqrt{f'_c}}\right) d_b = \left[\frac{(0.02)(1)(60000)}{\sqrt{4000}}\right](0.75'') \longleftarrow \text{diameter of \#6 bar}$$

$\ell_{dh} = 14'' \cdot 0.7 \longleftarrow$ reduction factor per 12.5.3 (a)

$\ell_{dh} = 9.8''$ down as a 90° hook

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Dead Load: 258 psf for green roof
150 psf for 12" slab
408 psf

Live Load: 100 psf

High loads make a two-way flat plate slab impractical!
Try one-way slab w/ beams along interior & exterior column rings.

Slab Design:

Min thickness $= \ell/28 = 22 \cdot 12/28 = 9.4" \rightarrow$ use 12" slab to be consistent with other slabs

$d = 11"$

$w_u = 1.2(408) + 1.6(100) = 650$ psf

$M_u = \frac{w_u \ell_n^2}{16} = \frac{(650)(1)(19.5)^2}{16} = 15.4$ ft-k/ft

$A_s \cong \frac{M_u}{\phi f_y (jd)} = \frac{15.4 \cdot 12}{0.9 \cdot 60 \cdot 0.95 \cdot 11} = 0.327$ in²/ft

$A_{s\,min} = 0.0018\,bh = 0.0018 \cdot 12 \cdot 12 = 0.259 < 0.327$ in²/ft ✓

$\boxed{\text{Use \#6 @ 12"} \rightarrow A_s = 0.44 \text{ in}^2}$

Check Spacing:

$s = 15\left(\frac{40000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40000}{f_s}\right)$

$s = 15\left(\frac{40000}{40000}\right) - 2.5(0.75) \leq 12\left(\frac{40000}{40000}\right) = 12"$ ✓

Check Shear:

$V_u = \frac{1.15 w_u \ell_n}{2} = \frac{1.15(650)(19.5)(1)}{2} = 7288$ lb/ft

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{4000}(12)(11) = 16700$ lb/ft $\cdot 0.75 = 12525 > 7288$ ✓

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Check Deflections:

$w_u = 650$ psf Service Load $= 408 + 100 = 508$ psf

$DL = 408$ psf Constr. Load $= 2(100) = 200$ psf

$M_a = 508/650 = 0.782$

$$M^- = \frac{w_u l_n^2}{16} = \frac{(650)(19.5)^2(1)}{16} = 15.4 \text{ ft-k/ft} \cdot 0.782 = 12.0 \text{ ft-k/ft}$$

$$M^+ = \frac{w_u l_n^2}{16} = \frac{(650)(19.5)^2(1)}{16} = 15.4 \text{ ft-k/ft} \cdot 0.782 = 12.0 \text{ ft-k/ft}$$

$$I_g = \frac{(12)(12)^3}{12} = 1728 \text{ in}^4/\text{ft}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{403 \cdot 1728}{6} / 12000 = 9.67 \text{ ft-k/ft}$$

$n = 8.1$ $\rho = \frac{0.44}{12 \cdot 11} = 0.0033$ $n\rho = 0.027$ $k = \sqrt{2\rho n + (\rho n)^2} - \rho n$

$k = 0.207$

$$I_{cr}^- = \frac{b k^3 d^3}{3} + nA_s(d - kd)^2$$

$$= \frac{(12)(0.207 \cdot 11)^3}{3} + 8.1(0.44)(11 - 0.207 \cdot 11)^2 = 315 \text{ in}^4$$

$$I_e^- = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_a}\right)^3 = 315 + (1728 - 315)\left(\frac{9.67}{12.0}\right)^3 = 1054 \text{ in}^4$$

$I_{cr}^+ = 315 \text{ in}^4$

$$I_e^+ = 315 + (1728 - 315)\left(\frac{9.67}{12}\right)^3 = 1054 \text{ in}^4$$

$$I_{e_{avg}} = 0.7(1054) + 0.15(1054 + 1054) = 1054 \text{ in}^4$$

$w_D = 0.408 \cdot 1 \cdot 0.6 = 0.25$ k/ft

$w_L = 0.100 \cdot 1 \cdot 0.6 = 0.06$ k/ft

$$\Delta_{D_{max}} = 0.0026 \frac{w_D l^4}{EI} = 0.0026 \frac{(0.25)(19.5)^4(1728)}{(3600)(1054)} = 0.04''/\text{ft}$$

$$\Delta_{L_{max}} = 0.0048 \frac{w l^4}{EI} = 0.0048 \frac{(0.06)(19.5)^4(1728)}{(3600)(1054)} = 0.02''/\text{ft}$$

Long-Term:

$\xi = 2.0$ $\Delta_{cp+sh} = 2.0(0.5 \cdot 0.02 + 0.04) = 0.1''$

$\Delta_{long\text{-}term} = 0.1 + 0.02 = 0.12'' < l/480 = 0.49''$ ✓

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Beam Design:

25' long beam $h = \ell/21 = 25 \cdot 12/21 = 14.28''$

Due to large loads try $b = 30''$, $h = 24''$, $d = 21''$

Loads:

$w_u = 1.2(408 \cdot (8+11)) + 1.6(100 \cdot (8+11)) = 12.3$ klf

Positive Midspan Moment:

$M_u = \frac{w_u \ell_n^2}{16} = \frac{(12.3)(22.5)^2}{16} = 389$ ft-k

Negative Moment @ Face:

$M_u = \frac{w_u \ell_n^2}{11} = \frac{(12.3)(22.5)^2}{11} = 566$ ft-k

Midspan Bottom Reinforcement:

$A_s = \frac{M_u}{4d} = \frac{389}{4(21'')} = 4.63$ in²

Try (4) #10 = 5.08 in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5.08(60)}{0.85(4)(30)} = 2.99''$

$c = 2.99/0.85 = 3.52''$

$\varepsilon_s = \frac{0.003}{3.52}(21 - 3.52) = 0.015 \geq 0.002$ ✓

$\phi M_n = 0.9 A_s f_y (d - a/2)$

$\phi M_n = 0.9(5.08)(60)(21 - 2.99/2)$

$\phi M_n = 446$ ft-k > 389 ft-k ✓

Check min & max reinforcement:

$A_{s_{min}} \geq \begin{cases} \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3\sqrt{4000}}{60000}(30)(21) = 1.99 \text{ in}^2 \\ \frac{200bd}{f_y} = \frac{200(30)(21)}{60000} = 2.4 \text{ in}^2 \end{cases} \leq 5.08$ in²

$A_{s_{max}} = 0.018(30)(21) = 11.34$ in² ≥ 5.08 in² ✓ $\boxed{\text{Use (4) \#10's}}$

Shear Reinforcement:

$V_u = \frac{w_u \ell_n}{2} = \frac{(12.3)(22.5)}{2} = 138^k$

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2\sqrt{4000}(30)(21) = 80^k$ → need shear reinforcement

$V_s = V_u/\phi - V_c = 138/0.75 - 80 = 104^k$

$V_{s_{max}} = 8\sqrt{f'_c}\, b_w d = 8\sqrt{4000}(30)(21) = 319^k > 104^k$ ✓

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Max spacing: $V_s \le 4\sqrt{4000}(30)(21) = 159\,k$

$$s_{max} = \min\begin{cases} 21/2 = 10.5'' \\ 24'' \end{cases}$$

$$A_v = \frac{V_s}{f_{yt}\,d/s} = \frac{104}{60 \cdot 21/10.5} = 0.87\ in^2 \rightarrow \text{too large, use (2) \#4}$$

$$s = \frac{A_v f_{yt} d}{V_s} = \frac{0.4(60)(21)}{104} = 4.8'' \rightarrow \boxed{\text{use (2) \#4 @ 4''}}$$

Negative Reinforcement:

$$A_s = \frac{M_u}{4d} = \frac{566}{4(21)} = 6.74\ in^2$$

Try (6) #10's $= 7.62\ in^2$

$$a = \frac{(7.62)(60)}{0.85(4)(30)} = 4.48''$$

$$c = 4.48''/0.85 = 5.27''$$

$$\varepsilon = \frac{0.003}{5.27}(21 - 5.27) = 0.009 > 0.002 \checkmark$$

$$\phi M_n = 0.9(7.62)(60)(21 - 4.48/2)$$

$$\phi M_n = 643\ k\text{-}ft > 566\ k\text{-}ft \checkmark$$

$$\boxed{\text{Use (6) \#10's}}$$

Beam Deflection Check:

$f_r = 474$ psi

$E_c = 3600$ ksi

$n = 8.1$

$I_g = \frac{bh^3}{12} = \frac{(30)(24)^3}{12} = 34560 \text{ in}^4$

$\bar{y} = \frac{bh(h/2) + (n-1)(A_s)(d)}{bh + (n-1)(A_s)} = \frac{30 \cdot 24(12) + (7.1)(5.08)(21)}{(30)(24) + (7.1)(5.08)} = 23.7''$

$I_{cr} = \frac{bh^3}{12} + bh(h/2 - \bar{y})^2 + (n-1)(A_s)(d-\bar{y})^2$

$I_{cr} = \frac{(30)(24)^3}{12} + (30)(24)(12-23.7)^2 + (7.1)(5.08)(21-23.7)^2$

$I_{cr} = 133380 \text{ in}^4$

$M_{cr} = \frac{f_r I_{cr}}{y_{bot}} = \frac{(474)(133380)}{23.7} = 222$ ft-k

$I_e = (M_{cr}/M_a)^3 I_g + [1-(M_{cr}/M_a)^3] I_{cr}$

$I_e = (222/389)^3(34560) + [1-(222/389)^3]133380$

$I_e = 115000 \text{ in}^4$

$\Delta_{Dmax} = 0.0026 \frac{(7.75)(22.5)^4(1728)}{(3600)(115000)} = 0.02''$

$\Delta_{Lmax} = 0.0048 \frac{(1.9)(22.5)^4(1728)}{(3600)(115000)} = 0.01''$

Long-Term:

$\lambda = 2.0$ $\quad \Delta_{cp+sh} = 2.0(0.5 \cdot 0.01 + 0.02) = 0.05''$

$\Delta_{long-term} = 0.05 + 0.01 = 0.06'' < \ell/480 = 0.56''$ ✓

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Design Column -16

Tributary Area = $(11' + 6.5')(18') = 315 \text{ ft}^2$

Roof Load:

DL Green Roof = 258 psf

DL Slab = 150 psf

LL = 30 psf

Roof Snow = 20 psf

$P_u = 1.2(.408_{psf}) + 1.6(.03) + 0.5(.02) = 0.548 \text{ ksf}$

$P_u = 0.548 \cdot 315 = \underline{173 \text{ k}}$

Floor Load:

w = 383 psf

$P_u = 0.383 \cdot 315 = 121 \text{ k}$

CO-16 supports roof and 4 levels

$P_u = 173 + 4(121) = 657 + 60 \text{ k}$ → estimated column self weight.
$.150 \cdot 2' \cdot 2' \cdot 101' = 60 \text{ k}$

$P_u = 717 \text{ k}$

M_u from slab design = 457k

Input P_u and M_u into spColumn:

- Circular Section 42" ⌀
- 41' clear height
- analyzed as a non-sway frame

Analysis found 42" ⌀ column w/ (11) #10 bars to be sufficient for gravity loads.

$A_s = 13.97 \text{ in}^2$ $\rho = 1.008\%$

For columns above use 30" × 30" w/ (8) #10 bars

$A_s = 10.16 \text{ in}^2$ $\rho = 1.129\%$

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Slenderness Effects: ACI 318-11 Section 10.10

For 15' tall sections:

$k = 1$

$\ell_u = 15' \cdot 12 = 180''$

$r = 0.3(30'') = 9$

$\frac{k\ell_u}{r} = \frac{180}{9} = 20 \leq 22$ ∴ slenderness can be neglected

For 41' tall section:

$k = 1$

$\ell_u = 41' \cdot 12 = 492''$

$r = 0.25(42'') = 10.5''$

$\frac{k\ell_u}{r} = \frac{492}{10.5} = 46.9 > 22$ ∴ slenderness must be included.

Check if nonsway:

$Q = \frac{P_u \Delta_o}{V_{us} \ell_c} \leq 0.05 = \frac{(717)(0.3617)}{(198)(492)} = 0.003 \leq 0.05$ ∴ Nonsway

Column has to support P_u and M_c as per Section 10.10.6

$I_g = \frac{0.7 \pi r^4}{4} = \frac{0.7\pi(21)^4}{4} = 106{,}922 \text{ in}^4$

$E_c = 57000\sqrt{4000} = 3605 \text{ ksi}$

$EI = \frac{0.4 E_c I_g}{1 + \beta_{dns}}$ ← assume $\beta_{dns} = 0.6$ → $0.25 E_c I_g$

$EI = 0.25(3605)(106922) = 96{,}363{,}453$

$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2(96363453)}{(492)^2} = 3929 \text{ k}$

$C_m = 0.6 + 0.4\left(\frac{199}{204}\right) = 0.99$

$\delta = \frac{0.99}{1 - \frac{717}{0.75(3929)}} = 1.31$

$M_{2,min} = P_u(0.6 + 0.03h) = 717(0.6 + 0.03(492)) = 11013 \text{ k-in}$

$M_c = \delta M_2 = (1.31)(11013) = 14317 \text{ k-in} / 12 = 1193 \text{ k-ft}$

Check P_u and M_c in spColumn interaction diagram

Floor Slab:

Original Slab Area = $117240\ ft^2 \cdot 8/12 = 78160\ ft^3$

New Slab Area = $106370\ ft^2 \cdot 12/12 = 106370\ ft^3$

$$1 - \frac{78160\ ft^3}{106370\ ft^3} = 0.27 = 27\%$$ increase in slab concrete used

Columns:

36" Ø Cross Sectional Area = $\pi(18)^2 = 1018\ in^2$

42" Ø Cross Sectional Area = $\pi(21)^2 = 1385\ in^2$

$$1 - \frac{1018\ in^2}{1385\ in^2} = 0.26 = 26\%$$ increase in concrete used

$24" \times 24" = 576\ in^2$

$30" \times 30" = 900\ in^2$

$$1 - \frac{576\ in^2}{900\ in^2} = 0.36 = 36\%$$ increase in concrete used

Original: (11)#9 = $11 \cdot 1.0 = 11\ in^2$

New: (11)#10 = $11 \cdot 1.27 = 13.97\ in^2$

$$1 - \frac{11}{13.97} = 0.21 = 21\%$$ increase in steel used

Original: (8)#8 = $8 \cdot 0.79 = 6.32\ in^2$

New: (8)#10 = $8 \cdot 1.27 = 10.16\ in^2$

$$1 - \frac{6.32}{10.16} = 0.38 = 38\%$$ increase in steel used

Appendix B: Wind and Seismic Calculations

Wind Load Calculations

- Based on ASCE 7-10
  - Risk Category III (Table 1.5-1)
  - Basic Wind Speed, $V = 120$ mph (Fig 26.5B)
  - Directionality Factor, $K_d = 0.85$ (Table 26.6-1)
  - Exposure Category: B (Sect. 26.7)
  - Topographic Factor, $K_{zt} = 1.0$ (Sect. 26.8)
  - Gust Effect Factor, $G = 0.85$ (Sect. 26.9)
  - Enclosure Classification: Partially Enclosed (Sect. 26.10)
    → Based on the numerous openings between the exterior of the drum and the interior courtyard
  - Internal Pressure Coefficient; $GC_{pi} = \pm 0.43$ (Sect. 26.11)
    → Reduction Factor:

$$R_i = 0.5\left[1 + \frac{1}{\sqrt{1 + \frac{V_i}{22.8 A_{og}}}}\right] \leq 1.0$$

→ Applicable for partially enclosed bldgs containing a single unpartitioned large volume; in this case the courtyard.

V_i = Volume of space = 326755 ft³

A_{og} = total area of openings = 7462 ft³ (includes roof & wall slots)

$$R_i = 0.5\left[1 + \frac{1}{\sqrt{1 + \frac{326755}{22.8 \cdot 7462}}}\right] = 0.79 \leq 1.0$$

→ $GC_{pi} = \pm 0.55 \cdot 0.79 = \underline{\pm 0.43}$

from Table 26.11-1

- Refer to Excel spreadsheets for
  → Velocity pressure exposure coefficients, K_z
  → Velocity pressure, q_z
  → Wind pressure, p

- Wind pressures were analyzed on 2 faces of the building, due to their unique heights and openings.

See Excel spreadsheets for further calculations.

Wind Design Pressures:

North-South MWFRS							
Level	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)
1	0	0.57	17.86	31.96	19.83	-27.33	0.00
2	14	0.57	17.86	31.96	19.83	-27.33	2160.00
3	27	0.68	21.31	31.96	23.65	-27.33	2160.00
4	41	0.76	23.81	31.96	26.43	-27.33	2320.00
5	56	0.83	26.01	31.96	28.87	-27.33	2400.00
6	72	0.89	27.89	31.96	30.96	-27.33	2400.00
7	88	0.95	29.77	31.96	33.04	-27.33	2400.00
Roof	103	1	31.33	31.96	34.78	-27.33	2000.00
Tower Roof	113	1.02	31.96	31.96	35.48	-27.33	615.00

East-West MWFRS							
Level	z	K_z	q_z	q_h	Windward P (psf)	Leeward P (psf)	Tributary Area (ft^2)
1	0	0.57	17.86	31.33	19.83	-26.79	0.00
2	14	0.57	17.86	31.33	19.83	-26.79	2160.00
3	27	0.68	21.31	31.33	23.65	-26.79	2160.00
4	41	0.76	23.81	31.33	26.43	-26.79	2320.00
5	56	0.83	26.01	31.33	28.87	-26.79	2400.00
6	72	0.89	27.89	31.33	30.96	-26.79	2400.00
7	88	0.95	29.77	31.33	33.04	-26.79	2400.00
Roof	103	1	31.33	31.33	34.78	-26.79	1200.00

ETABS User Defined Input for Wind Cases:

Case 1 Y	Case 3 Y	Case 2 Y	
Story Force (k)	Story Force (k)	Story Force (k)	Torsional Moment (k-ft)
0.00	0	0	0
101.85	76.38	76.38	29250
110.11	82.58	82.58	30589
124.72	93.54	93.54	33901
134.87	101.15	101.15	36016
139.87	104.91	104.91	36828
144.88	108.66	108.66	37639
124.21	93.16	93.16	31929
38.62	28.97	28.97	9888

Case 1 X	Case 3 X	Case 2 X	
Story Force (k)	Story Force (k)	Story Force (k)	Torsional Moment (k-ft)
0.00	0	0	0
100.68	75.51	75.51	28810
108.95	81.71	81.71	30149
123.47	92.60	92.60	33428
133.57	100.18	100.18	35527
138.58	103.94	103.94	36338
143.59	107.69	107.69	37150
73.88	55.41	55.41	18913

Case 4		
Story Force (k)	Torsional Moment (++)	Torsional Moment (+-)
0	0	0
57.34	51793	378
61.99	54473	378
70.22	60602	406
75.93	64586	420
78.75	66211	420
81.57	67835	420
69.93	46195	11811
21.75	8988	8988

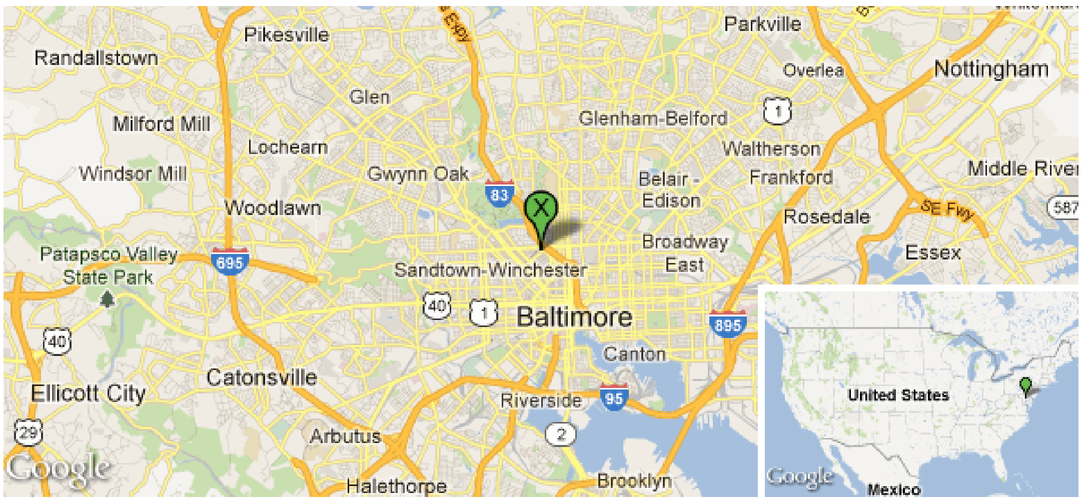
Seismic Design Information from USGS:

USGS Design Maps Summary Report

Print View Detailed Report

User–Specified Input

Building Code Reference Document	2012 International Building Code (which makes use of 2008 USGS hazard data)
Site Coordinates	39.31°N, 76.625°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

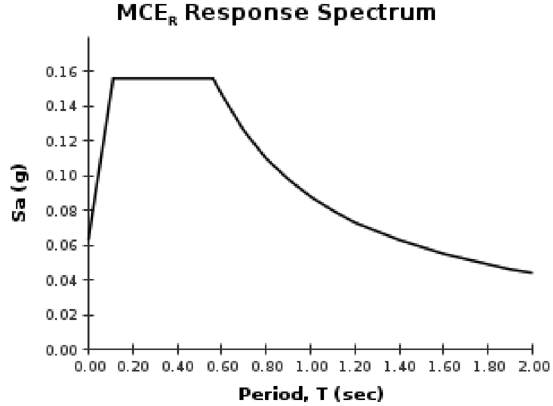


USGS–Provided Output

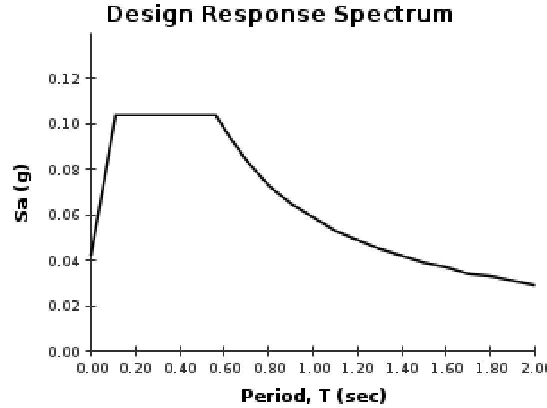
S_S = 0.130 g	S_{MS} = 0.156 g	S_{DS} = 0.104 g
S_1 = 0.052 g	S_{M1} = 0.088 g	S_{D1} = 0.059 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



Seismic Load Calculations:

- Based on ASCE 7-10
- From USGS website: U.S. Seismic Design Maps: $S_s = 0.130g$, $S_1 = 0.052g$, $S_{MS} = 0.15g$, $S_{M1} = 0.088g$, $S_{DS} = 0.104g$, $S_{D1} = 0.059g$
- Based on this data and Tables 11.6-1 and 11.6-2, the building falls into Seismic Design Category A, contrary to the building plans which state Seismic Design Category B. SDC B will be used for the remainder of the calculations.
- Assume building has ordinary reinforced concrete shear walls:
  - Response Modification Coefficient, R=4, per Table 12.2-1
  - Importance Factor = 1.25, per Table 1.5-2
  - No height limitations per Table 12.2-1
- $T_L = 6s$ per Fig. 22-12
- $C_u = 1.7$ per Table 12.8-1
- $C_t = 0.02$ per Table 12.8-2
- $x = 0.75$ per Table 12.8-2
- $h = 113$ ft

$$T_a = C_t h_n^x = 0.02(113)^{0.75} = 0.69s$$

$$T = C_u \cdot T_a = 1.7 \cdot 0.69 = 1.18s$$

$$C_s \le \begin{cases} S_{DS}/R/I = 0.104/(4/1.25) = 0.021 \\ S_{D1}/[T(R/I)] = 0.059/[1.18(4/1.25)] = 0.016 \end{cases}$$

$$k = 1 + (1.18-1)\left(\frac{2-1}{2.5-0.5}\right) = 1.09$$

See following page for building mass estimations.

See Excel spreadsheet for Story Force computations

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Slab Weight:

Level 1: 16750 ft² · 150 pcf · 12/12 = 2513 k
Level 2: 10060 ft² · 150 pcf · 12/12 = 1509 k
Level 3: 15660 ft² · 150 pcf · 12/12 = 2349 k
Level 4: 12600 ft² · 150 pcf · 12/12 = 1890 k
Level 5: 12600 ft² · 150 pcf · 12/12 = 1890 k
Level 6: 12900 ft² · 150 pcf · 12/12 = 1935 k
Level 7: 12900 ft² · 150 pcf · 12/12 = 1935 k
Roof: 12900 ft² · 150 pcf · 12/12 = 1935 k
16296 k

Tower Roof:
1973 · 150 = 296 k

Column Weight:

42" Ø column = 9.62 ft² · 41 ft · 150 pcf = 59 k · 14 = 826 k
30" Ø column = 4.91 ft² · 103 ft · 150 pcf = 76 k · 4 = 304 k
30" x 30" column = 6.25 ft² · 113 ft · 150 pcf = 106 k · 4 = 424 k
30" x 30" column = 6.25 ft² · 62 ft · 150 pcf = 58 k · 24 = 1392 k
24" x 24" column = 4 ft² · 113 ft · 150 pcf = 68 k · 5 = 340 k

$\frac{3290\text{ k}}{7} = 470$ k/floor

Other Dead Loads:

Level 2: 10060 · 10 psf = 100 k
Level 3: Planters: 2967 · 258 = 765 k
Plaza: 4073 · 38 = 155 k
Other: 8620 · 10 = 86 k
Level 4: 12600 · 10 = 126 k
Level 5: 12600 · 10 = 126 k
Level 6: 12900 · 10 = 129 k
Level 7: 12900 · 10 = 129 k
Roof: 12900 · 10 = 129 k
Tower Roof: 1973 · 10 = 20 k

Appendix C: Shear Wall Calculations

N-S Shear Walls Shear and Moments:

Shear Wall 1			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	0	0
Roof	103	114	11742
7	88	106.2	9345.6
6	72	86.3	6213.6
5	56	65.6	3673.6
4	41	49.9	2045.9
3	27	44.2	1193.4
2	14	15	210
		ΣMoment	**34424.1**

Shear Wall 2			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	14.3	1615.9
Roof	103	32.6	3357.8
7	88	30.4	2675.2
6	72	24.7	1778.4
5	56	18.8	1052.8
4	41	14.3	586.3
3	27	12.6	340.2
2	14	4.3	60.2
		ΣMoment	**11466.8**

N-S Shear Wall Shear and Moments:

Shear Wall 3			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	14.3	1615.9
Roof	103	32.6	3357.8
7	88	30.4	2675.2
6	72	24.7	1778.4
5	56	18.8	1052.8
4	41	14.3	586.3
3	27	12.6	340.2
2	14	4.3	60.2
		ΣMoment	**11466.8**

Shear Wall 4			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	12.3	1389.9
Roof	103	28	2884
7	88	26	2288
6	72	21.2	1526.4
5	56	16	896
4	41	12.2	500.2
3	27	10.9	294.3
2	14	3.6	50.4
		ΣMoment	**9829.2**

E-W Shear Wall Shear and Moments:

Shear Wall 5			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	14.7	1661.1
Roof	103	74.6	7683.8
7	88	69.5	6116
6	72	33.9	2440.8
5	56	25.7	1439.2
4	41	19.6	803.6
3	27	17.4	469.8
2	14	5.9	82.6
		ΣMoment	**20696.9**

Shear Wall 6			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	14.7	1661.1
Roof	103	74.6	7683.8
7	88	69.5	6116
6	72	33.9	2440.8
5	56	25.7	1439.2
4	41	19.6	803.6
3	27	17.4	469.8
2	14	5.9	82.6
		ΣMoment	**20696.9**

E-W Shear Wall Shear and Moments:

Shear Wall 7			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	11.5	1299.5
Roof	103	58	5974
7	88	54	4752
6	72	26.3	1893.6
5	56	20.1	1125.6
4	41	15.2	623.2
3	27	13.4	361.8
2	14	4.5	63
		ΣMoment	**16092.7**

Shear Wall 8			
Story	Height (ft)	Story Force (k)	Moment (k-ft)
Tower Roof	113	0	0
Roof	103	0	0
7	88	0	0
6	72	62.8	4521.6
5	56	47.7	2671.2
4	41	36.3	1488.3
3	27	32.1	866.7
2	14	10.9	152.6
		ΣMoment	**9700.4**

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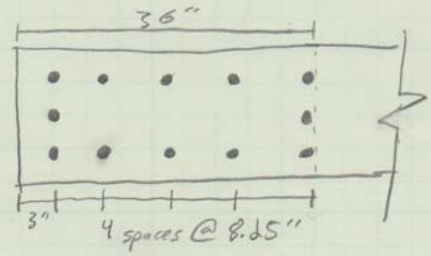
Shear Walls 5, 6, 7 → in X-direction

Min vertical reinforcement: $0.0015 = \rho_\ell$ (ACI 318-11, 14.3.2 (b))

Keep 3' End Zone reinforcement



$\ell_w = 23'$
$h_w = 113'$
$h = 12''$

Check Moment Strength @ Base:

$M_u = 20700$ k-ft

Try (12) #10's

$T = A_s f_y = 12 \cdot 1.27 in^2 \cdot 60 ksi = 914$ k

$N_u = \frac{150 pcf \cdot 1' \cdot 23' \cdot 113'}{1000} = 390$ k

$d = \ell_w - (3'' + 2 \cdot 8.25'') = 276'' - 19.5'' = 256.5''$

$a = \frac{T + N_u}{0.85 f'_c b} = \frac{914 + 390}{0.85(4)(12'')} = 32''$

$c = a/\beta_1 = 32''/0.85 = 37.6''$

$0.375d = 96.2'' > 37.6''$ ∴ Tension Controlled, $\phi = 0.9$

$\phi M_n = \phi\left[T(d - a/2) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$

$\phi M_n = 0.9\left[914(256.5 - 32/2) + 390\left(\frac{276 - 32}{2}\right)\right]$

$\phi M_n = 240700$ k-in$/12 = 20060$ k-ft < 20700 k-ft X No good

Try (12) #11's

$T = A_s f_y = 12 \cdot 1.56 in^2 \cdot 60 = 1123$ k

$N_u = 390$ k

$d = 256.5$

$a = \frac{1123 + 390}{0.85(4)(12)} = 37.1''$

$\phi M_n = 0.9\left[1123(256.5 - 37.1/2) + 390\left(\frac{276 - 37.1}{2}\right)\right]$

$\phi M_n = 282400$ k-in$/12 = 23530$ k-ft > 20700 k-ft ✓

$\rho_\ell = \frac{A_{v,vert}}{h s_1} = \frac{2 \cdot 1.56}{(12)(8.25)} = 0.032 > 0.0015$ ✓

Use (12) #11's in above configuration for end zone.

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Shear 5, 6, 7:

For vertical reinforcement between of end zones:

Try #4's

$\rho_\ell = \frac{A_{v,vert}}{h s_2} = 0.0012 = \frac{2 \cdot 0.2}{(12) s_1} \rightarrow s_1 = 27.8" > 18"$ ← code max (14.3.5)

Use #4 @ 18" in two layers for vertical reinforcement.

Horizontal Shear Strength:

$\rho_t = 0.0020$ (ACI 318-11, 14.3.3 (a))

$V_u = 261\,k$ $d = 0.8\ell_w = 221"$ (11.9.4)

Check V_c w/ ACI 318-11 Equations 11-27 & 11-28

11-27: $V_c = 3.3\lambda\sqrt{f'_c}\,hd + \frac{N_u d}{4\ell_w}$

$V_c = 3.3\sqrt{4000}(12)(221) + \frac{(390)(221)}{4(276)} = 632\,k$

11-28: $V_c = \left[0.6\lambda\sqrt{f'_c} + \frac{\ell_w\left(1.25\lambda\sqrt{f'_c} + 0.2\frac{N_u}{\ell_w h}\right)}{\frac{M_u}{V_u} - \frac{\ell_w}{2}}\right] h \cdot d$

$V_c = \left[0.6\sqrt{4000} + \frac{276\left(1.25\sqrt{4000} + 0.2\frac{(390)}{(276)(12)}\right)}{\frac{20700}{261} - \frac{276}{2}}\right](12)(221) =$

Term is negative ∴ Eq. (11-28) does not apply

$V_c = 632\,k$

$\phi 0.5 V_c = 0.75(0.5)(632) = 237\,k < 261\,k = V_u$ ∴ reinforcement required.

Try #5's $\rightarrow \rho_t = \frac{A_{v,horiz}}{h s_2} = \frac{2 \cdot 0.31}{(12)(s_2)} = 0.0025 \rightarrow s_2 = 21" > 18"$ ← controls

$V_s = \frac{A_v f_y d}{s} = \frac{(2 \cdot 0.31)(60)(221)}{18"} = 460\,k$

$\phi V_n = \phi(V_c + V_s) \geq V_u = 0.75(632 + 460) \geq 261 = 819^k \geq 261^k$ ✓

Use #5 @ 18", two layers

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Shear Walls 2, 3, 4 → in Y-direction:

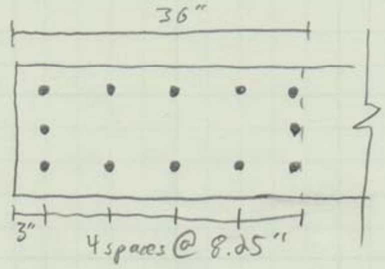
$\rho_\ell = 0.0015$

3' End zone reinforcement

$\ell_w = 10.5'$

$h_w = 113'$

$h = 12''$



Check Moment Strength @ Base:

$M_u = 10400$ k-ft

Try (12) #10's

$T = A_s f_y = (12)(1.27)(60) = 914$ k

$N_u = \dfrac{150 \cdot 1' \cdot 10.5' \cdot 113'}{1000} = 178$ k

$a = \dfrac{T + N_u}{0.85 f'_c b} = \dfrac{914 + 178}{0.85(4)(12)} = 26.8''$

$c = a/\beta_1 = 26.8/0.85 = 31.5''$

$0.375d = 39.9'' > 31.5''$ ∴ Tension controlled, $\phi = 0.9$

$d = 126'' - (3 + 2 \cdot 8.25) = 106.5''$

$\phi M_n = \phi\left[T(d - a/2) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$

$\phi M_n = 0.9\left[914(106.5 - 26.8/2) + 178\left(\frac{126 - 26.8}{2}\right)\right]$

$\phi M_n = 84530$ k-in/12 $= 7040$ k-ft < 10400 k-ft ✗ No good

Try (12) #11's

$T = (12)(1.56)(60) = 1123$

$a = \dfrac{1123 + 178}{0.85(4)(12)} = 31.9''$

$c = 31.9/0.85 = 37.5'' < 39.9''$
∴ $\phi = 0.9$

$\phi M_n = 0.9\left[1123(106.5 - 31.9/2) + 178\left(\frac{126 - 31.9}{2}\right)\right]$

$\phi M_n = 99060$ k-in/12 $= 8250$ k-ft < 10400 k-ft ✗ No good

Increase Shear Wall's 2, 3, 4 4 feet in length to increase N_u

Change ETABS model and re-analyze.

New Moments recorded in Excel.

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Shear Walls 2,3,4:

3' End zone reinforcement

$\ell_w = 14.5'$

$h_w = 113'$

$h = 12''$

Check Moment Strength @ Base:

$M_u = 11500$ k-ft

Try (12) #11's

$T = 12 \cdot 1.56 \cdot 60 = 1123$ k

$N_u = \dfrac{150 \cdot 1' \cdot 14.5' \cdot 113'}{1000} = 246$ k

$a = \dfrac{T + N_u}{0.85 f'_c b} = \dfrac{1123 + 246}{0.85(4)(12)} = 33.6''$

$c = 33.6/0.85 = 39.5''$

$0.375d = 57.9'' > 39.5''$

$\therefore \phi = 0.9$

$d = 174 - 19.5 = 154.5$

$\phi M_n = \phi\left[T\left(d - a/2\right) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$

$\phi M_n = 0.9\left[1123\left(154.5 - 33.6/2\right) + 246\left(\frac{174 - 33.6}{2}\right)\right]$

$\phi M_n = 154716$ k-in$/12 = 12900$ k-ft ≥ 11500 k-ft ✓

$\rho_\ell = \dfrac{2 \cdot 1.56}{(12)(8.25)} = 0.032 > 0.0015$ ✓

For vert. reinforcement between end zones:

Try #4's

$\rho_\ell = \dfrac{A_{vert}}{h s_2} = 0.0012 = \dfrac{2 \cdot 0.2}{(12) s_2} = 27.8'' > 18''$ ← controls

Use #4@18" in 2 layers

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Shear 2, 3, 4:

Horizontal Shear Strength:

$\rho_t = 0.002$

$V_u = 152\ k$

$d = 0.8\ell_w = 0.8(174) = 139''$

Eq (11-27): $V_c = 3.3\lambda\sqrt{f'_c}\,hd + \frac{N_u d}{\ell_w}$

$V_c = 3.3\sqrt{4000}(12)(139) + \frac{246(139)}{4(174)} = 397\ k$

Eq (11-28): $\frac{M_u}{V_u} - \frac{\ell_w}{2} = \frac{11500}{152} - \frac{174}{2} = -11.3 \therefore$ Eq (11-28) does not apply

$V_c = 397\ k$

$\phi 0.5 V_c = (0.75)(0.5)(397) = 149^k < 152^k = V_u \therefore$ reinforcement required

Try #5's $\rightarrow \rho_t = \frac{A_{v\,horiz}}{h s_2} = \frac{2 \cdot 0.31}{(12)s_2} = 0.0025 \rightarrow s_2 = 21'' > 18'' \leftarrow$ controls

$V_s = \frac{A_v f_y d}{s} = \frac{(2 \cdot 0.31)(60)(139)}{18} = 287^k$

$\phi V_n = \phi(V_c + V_s) \geq V_u = 0.75(397 + 287) \geq 152^k = 513^k \geq 152^k$ ✓

use #5 @ 18", two layers

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Shear Wall 1:

Built w/ C0-25 and C0-29 as end zones

$\ell_w = 30'$

$h_w = 113'$

$h = 12''$

Check Moment Strength @ Base:

$M_u = 34400$ k·ft

Try (11) #10's

$T = A_s f_y = 11 \cdot 1.27 in^2 \cdot 60 = 840$ k

$$N_u = \frac{150 \cdot 1' \cdot 23' \cdot 103'}{1000} + \frac{150 \cdot \pi (1.75')^2 \cdot 103'}{1000} = 504 \text{ k}$$

$d = \ell_w - 1/2$ col. diameter $= 360'' - 21'' = 339''$

$$a = \frac{T + N_u}{0.85 f'_c b} = \frac{840 + 504}{0.85(4)(12)} = 32.9''$$

$c = a/\beta_1 = 32.9/0.85 = 38.8''$

$0.375d = 135'' > 38.8''$ ∴

$\phi = 0.9$

$$\phi M_n = \phi \left[T\left(d - a/2\right) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$$

$$\phi M_n = 0.9\left[840\left(339 - 32.9/2\right) + 504\left(\frac{360 - 32.9}{2}\right)\right]$$

$\phi M_n = 320735$ k-in $/12 = 26730$ k-ft < 34400 k-ft ✗ No good

Try (15) #11's

$T = 15 \cdot 1.56 \cdot 60 = 1404$ k

$N_u = 504$ k

$d = 339''$

$$a = \frac{1404 + 504}{0.85(4)(12)} = 46.7''$$

$$\phi M_n = 0.9\left[1404\left(339 - 46.7/2\right) + 504\left(\frac{360 - 46.7}{2}\right)\right]$$

$\phi M_n = 469910$ k-in $/12 = 39159$ k-ft > 34400 k-ft ✓

Use (15) #11's in Col 25 and Col 29

Use #4 @ 18" in 2 layers for other vertical reinforcement

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Horizontal Shear Strength:

$V_u = 481\ k$

$d = 0.8(360) = 288''$

Eq. (11-27): $V_c = 3.3\lambda\sqrt{f'_c}\,hd + \dfrac{N_u d}{4 l_w}$

$$V_c = 3.3\cdot\sqrt{4000}(12)(288) + \frac{(504)(288)}{4(360)} = 822\ k$$

Eq. (11-28): $\dfrac{M_u}{V_u} - \dfrac{l_w}{2} > 0 = \dfrac{34400}{481} - \dfrac{360}{2} = -108 < 0$ ✗ does not apply

$V_c = 822\ k$

$\phi 0.5 V_c = 0.75(0.5)(822) = 312\ k < 481\ k = V_u$ ∴ reinforcement required

Try #5's $\rightarrow \rho_t = \dfrac{A_{v\,horiz}}{h s_2} = \dfrac{2\cdot 0.31}{(12) s_2} = 0.0025 \longrightarrow s_2 = 21'' > 18'' \leftarrow$ controls

$$V_s = \frac{A_v f_y d}{s} = \frac{(2\cdot 0.31)(60)(288)}{18} = 595\ k$$

$\phi V_n = \phi(V_c + V_s) = 0.75(832 + 595) = 1070\ k > 481\ k$ ✓

Use #5 @ 18'', two layers

Appendix D: Foundation Calculations

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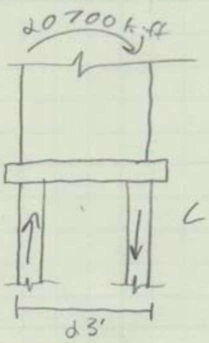
Caisson supporting Shear Wall 5:

Overturning Moment = 20700 k-ft

Axial Load of Shear Wall = 390 k



$C = \frac{20700}{23} + \frac{390}{2} = 1095\ k$

$f'_c = 3000$ psi:

Based on CRSI Handbook Ch. 13:

Shaft diameter w/ 1095 k @ $0.3f'_c$ = 3'-6"

Bell diameter w/ 1095 k and 10 ksf bearing load = 12'-0"

Minimum Shaft size recommended w/ 12' bell is 4'-0"

Bell height = 7.93'

Recommended reinforcement for 4'-0" shaft:

(7) #10 verticals

#4 @ 18" ties

Caisson is 50' deep