WEST SHORE PINNACLEHEALTH HOSPITAL
FINAL REPORT: CONCRETE REDESIGN

1992 Technology Parkway, Mechanicsburg, PA, 17050

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Option: **Structural**
Advisor: **Dr. Aly M. Said**

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PINNACLEHEALTH West Shore Hospital

Project Information

Building Location: 1995 Technology Parkway, Mechanicsburg, PA

Occupancy Type: Hospital

Floor Area: 188,000 sq. ft.

Construction Cost: $120,000,000 | $638.30 per sq. ft.

Building Team

Owner: PinnacleHealth

Structural Engineer: O'Donnell & Naccarato

Architect: Stantec

General Contractor: Quandel Construction Group

Image Credit: Google Earth Pro 2015

Structure

- Composite steel frame design
- Lateral Force Resisting System: "Steel systems not specifically detailed for seismic resistance"
- Spray fireproofing and typical spread footing foundation

Construction Management

- Design-Build project with very successful implementation of BIM
- Prefabricated panels able to be made ahead of time, steel erected in 2 months, 208 precast panels hung in 33 days.
- Project constructed ahead of schedule and under price guarantee

Mechanical

- VAV ductwork distribution
- Control system designed to save energy with variable speed pumps and motors
- Starting system up slowly decreases peak demand on system

Lighting/Electrical

- 480Y277 V electrical line fed from 2 different substations through a transfer switch
- Emergency generators for power loss, and operating rooms have battery backups for continuous power through delay from primary power to generator backup
- Energy Efficient LED lighting with parking lot lighting on photo sensors with timer override

Aaron King | Structural Option   Advisor: Dr. Aly Said

EXECUTIVE SUMMARY

This report covers the design and included methods and results of the redesign from a steel superstructure to concrete. The gravity and lateral systems were both redesigned and two breadths, construction management and the building enclosure, are investigated.

Pinnacle Health West Shore Hospital is a 188,000 sq. ft. hospital located in Mechanicsburg, PA. Construction was finished on the hospital in May 2014. The building is 5 stories with surgery suites on the first two floors, along with short visit rooms and areas for the hospital employees. The top three floors comprise a bed tower with 180 beds to service the people of the surrounding area. It is located within a health service campus also owned by Pinnacle Health.

A two way slab with drop panels and concrete columns are the gravity elements that were designed with the design goals in mind of having a thinner floor slab for more mechanical space and resulting a less expensive construction, as concrete buildings are more prevalent in the area. Shear walls comprise the redesigned lateral system, with a model created in RAM and verified for the design forces, which were wind controlled even though the buildings weight was increased significantly by the use of a concrete superstructure.

The construction management breadth looks at the cost and construction time difference between the two superstructure alternatives, and verifying if the redesigned alternative is feasible for actual construction. The enclosures breadth looks at the building envelope for efficiency and proposed changes where possible or necessary through a critical analysis.

In order to fulfill the requirements of the integrated BAE/MAE degree, work utilizing the information from MAE classes was used to analyze the connections, the enclosure and aid in creation of a 3D model. This includes classes such as AE 530 Computer Modeling, AE534 Steel Connections and AE542 Building Enclosures Science and Design.

The redesign poses evidence that the concrete alternative is a feasible method of construction, with a cost and timeline similar to the steel structure achievable, and the material being a popular choice for buildings and contractors in the area. The thinner floor plate of the two way slab increases the space for equipment placement and duct runs, an essential opportunity for a hospital that may be upgraded or expanded in the future.

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INTRODUCTION

West Shore PinnacleHealth Hospital, owned by PinnacleHealth, is located off of Wertzville Rd on Technology Parkway in Mechanicsburg, PA. It is contained within a PinnacleHealth campus containing other health centers, such as the Ortenzio Cancer Center and the Fredricksen Outpatient Center. This was the first new hospital in the central PA region in quite a long time. The first two floors of the building contain waiting rooms, visitor areas and staff areas, along with beds, examination rooms and surgery suites.

BUILDING LAYOUT AND DESIGN

Figure 1 gives an architectural floor plan of the lower floor of the hospital. Any heavy medical equipment is placed on a slab on grade on the ground floor, preventing any difficulties in designing elevated slabs for these large loads.

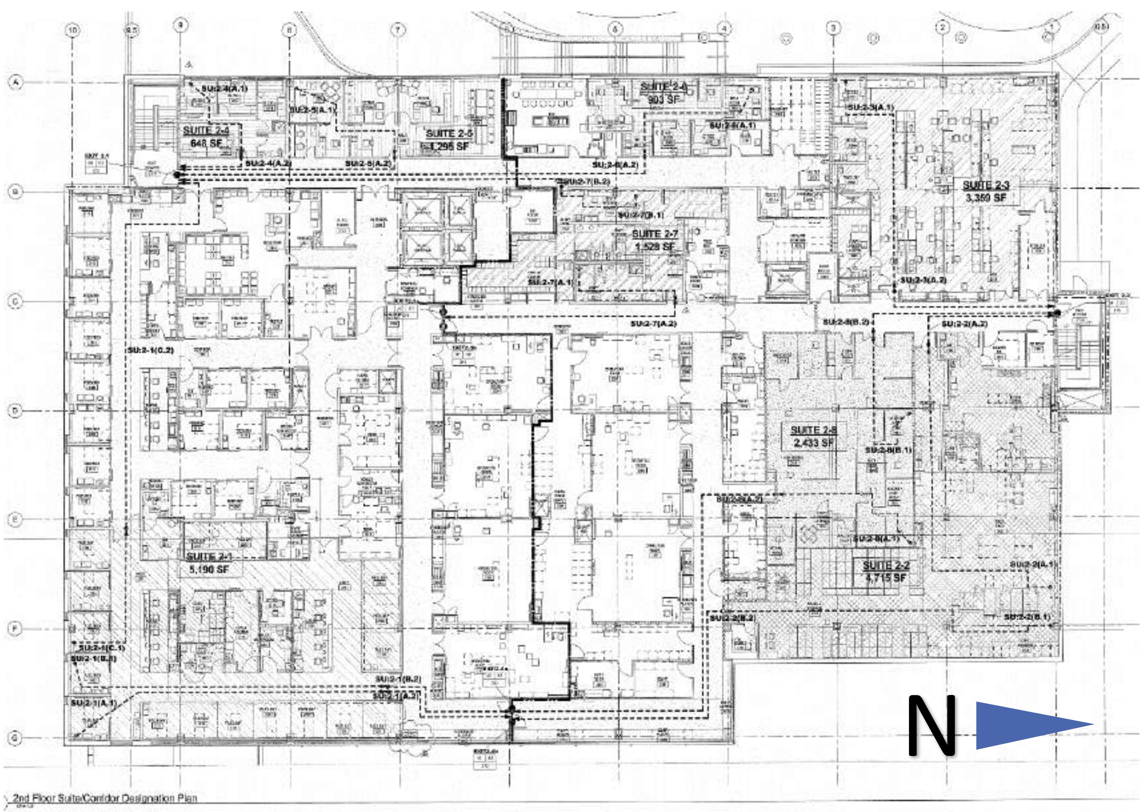

Figure 1: 2nd Floor Plan (Similar to 1st)

The third floor and above have a smaller footprint and are vertically located on the west face of the building. These floors contain typical suites for long term hospital visits. Figure 2 gives an architectural floor plan of the upper floors.

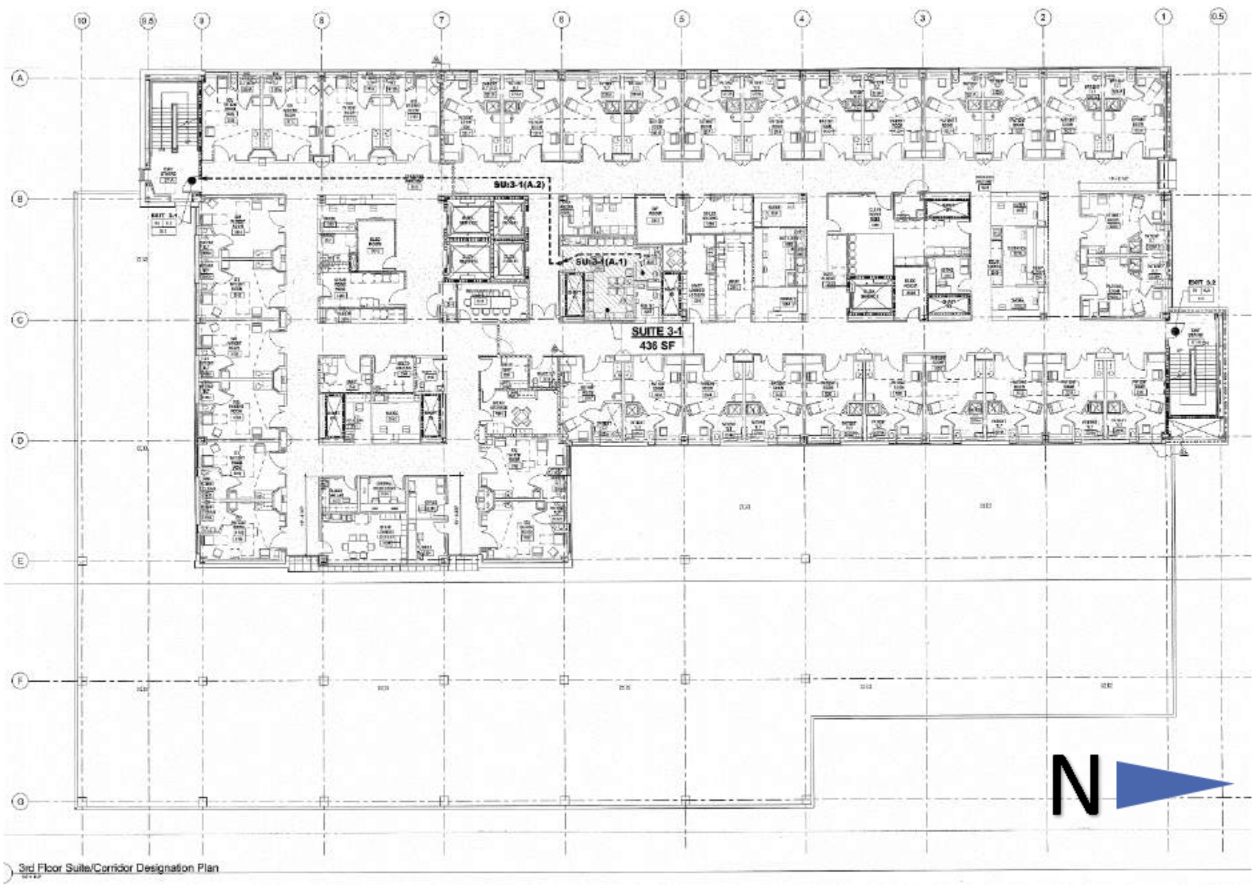

Figure 2: 3rd Floor Plan (Similar to 4th and 5th)

Parking is not included in the building, but is available on almost all sides on the campus. Because of the open campus, there were few horizontal limitations, and it also allowed the designers to think in terms of horizontal and vertical expansion, which was incorporated in all of the floor designs. Figure 3 contains the site plan of the hospital in its healthcare campus.

Figure 3: Aerial View of the PinnacleHealth campus centered on West Shore Hospital

EXISTING BUILDING

The existing building was designed by O'Donnell & Naccarato as a steel wide flange structure. W12 columns are typical along with W16, W21 or W24 beams and girders. The floors were determined to be partially composite, limited by the number of shear studs present on the beams. The gravity and lateral system are the same frames, which is an interesting design decision. The whole structure rests on spread footings on a soil with a bearing capacity of 6,000 psf. The enclosure is composed of two architectural features, one being a precast panel and the other being architectural aluminum panels.

GRAVITY SYSTEM

LOADS

A typical floor was designed with a 100 psf reducible live load, and 65 psf dead load. Floor areas used for medical imaging have an increased 100 psf dead load. Floor slabs are 3-1/4" LW concrete partially composite slabs on steel beams. Typical floor bays are virtually identical to the roof bays. They are modular 30' by 30' square bays with 2 infill beams spaced at 10' o.c. differing only where floor penetrations may be required for mechanical equipment. Rooftop mechanical equipment were approximated as a distributed 100 psf additional dead load. Because the building could be vertically expanded in the future, the roof is not of a lighter construction, but is designed using the same members and partially composite slabs as the floors.

DESIGN LOADS

1. GRAVITY

a) 3 ¼" LW CONC ON 3' METAL DECK	49 PSF
b) STEEL	8 PSF
c) CEILING	2 PSF
d) COLLATERAL	4 PSF
e) MEP	2 PSF
f) ROOF & INSULATION	10 PSF

2. LIVE

a) TYPICAL FLOOR	100 PSF
b) ROOF	50 PSF

TYPICAL FLOOR TOTAL
65 PSF DEAD LOAD
100 PSF REDUCIBLE LIVE LOAD

TYPICAL ROOF TOTAL
75 PSF DEAD LOAD
50 PSF LIVE LOAD

Flat roof snow load was determined from Notebook Submission A to be 21 psf flat roof snow load with a maximum drift on the lower roof of 60 psf diminishing over approximately 11 ft in the windward direction, and 90 psf in the leeward direction diminishing over a distance of approximately 15 ft.

The building enclosure has two designs, one being a precast wall panel, and the other an aluminum architectural wall panel. Calculations from Submission A resulted in the precast wall panel weighing 85 psf, and the aluminum panel weighing 30 psf. Floor heights are 15' on the lower levels, and 14' at the upper level bed tower.

DESIGN

West Shore Hospital's steel structure design is fairly simple, with a typical bay sized at 30' by 30' with two infill beams spaced at 10' o.c. Infill beams are typically W21x44's between columns and W16x26's between girders. The larger steel size between beams is a result of the whole building flexible moment connections. Other than framing around openings or irregular portions of architectural features, this bay size is used repeatedly throughout the entire design. Columns reaching up to the fifth floor are mainly W12x50's with a few W12x58's in certain areas. All main columns from rooftop to basement in the building are W12's. The first and second floor both have a 15’ floor to floor height, and the third floor and above is 14’ high. Figure 4 is an image of the typical upper floor framing plan.

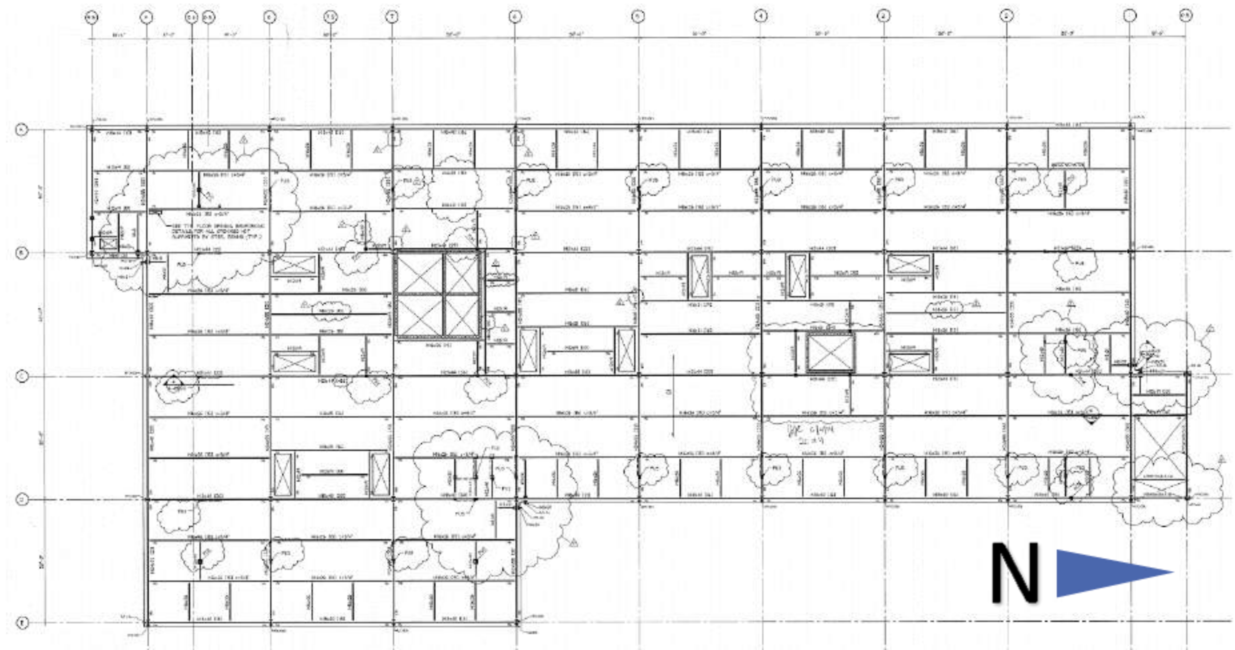

Figure 4: Upper Floors Framing Plan

Serviceability checks were done based on IBC 2009 deflection limits. The limit for live load deflection is L/360 which results in a 1” maximum allowable deflection in a 30’ bay, and with the IBC stating, “For steel structures, dead load shall be taken as 0”, the live load deflection controls. Beams were designed with a ¾” camber and the deflection was calculated as 0.903”, within the 1” limit.

The load from the structure is transferred to the foundation at the column bases. West Shore Hospital has a split level basement because it is built on a slight incline. There are two foundation plans, one for the south entrance area and one for the northern portion of the building. This foundation system consists of 66 typical spread footings under all areas of the building. Soil load capacity noted in the structural documents is 6,000 psf soil bearing pressure. Typical sizes used in the foundation system range from 3’ by 3’ by 1’2” thick footings with #4 reinforcing bars to 11’ by 11’ by 3’ thick with #8 reinforcing bars.

LATERAL FORCE RESISTING SYSTEM

LOADS

Wind hand calculations produced a 223k base shear in the N/S direction, and a 404k base shear in the E/W direction. When compared to the 3D RAM model's outputs on load combinations for the lateral system, this very closely matches the wind values divided by 1.6 within the combinations. This verified both the values within RAM and manual methods.

Lateral Loads – Wind

a) Basic Wind Speed (3 Second Gust) – 90 mph
b) Wind Importance Factor – 1.15
c) MWFRS Exposure Category – C
d) Internal Pressure Coefficient - +- 0.18

Seismic shear values were determined using 0.01*Fx, in accordance with ASCE 7-05 Section 11.7.2. Calculated values of the buildings mass resulted in a 130k base shear. This also matches the RAM output report, again verifying the methods and model. See Appendix pg. 18 (Document pg. 48) for the RAM Report on seismic building shears.

Lateral Loads – Seismic

a) Seismic Importance Factor – 1.5
b) Occupancy Category – IV
c) Seismic Design Category – A
d) Seismic Force Resisting System – Steel System not Seismically Detailed
e) Response Modification Factor – 3
f) Analysis Procedure – Equivalent Lateral Force Procedure

Upon examination of the structural typical connections details, it shows flexible moment connections used at all columns unless noted otherwise. This led to the conclusion that the building frame may work with the two elevator cores as a composite primary lateral force resisting system in the building. Figure 5 shows the core layout on floors 3 and above.

Of the two elevator shafts present in the building, the larger one houses four elevators, two oversized for moving beds and equipment and two smaller elevators for normal transport. The single shaft is a typical elevator for patients, visitors or employees. The current layout and flexible moment connection design may not have been sufficient if two single elevator shafts were present in the building. When analyzed in the buildings lateral assignment, the interaction of the larger elevator shafts was later added to determine if the steel structure itself could resist the loads and to compare those values. The difference of stiffness of the smaller elevator walls was significant, so it was not utilized in the lateral system. It was determined that the deflection of the steel structure itself was within the code limit, but was not within the H/400 design deflection, which was 2.16" for this building. After analyzing the building with the large elevator shaft included, the deflection was within the H/400 limit, at 1.93" of deflection at roof level.

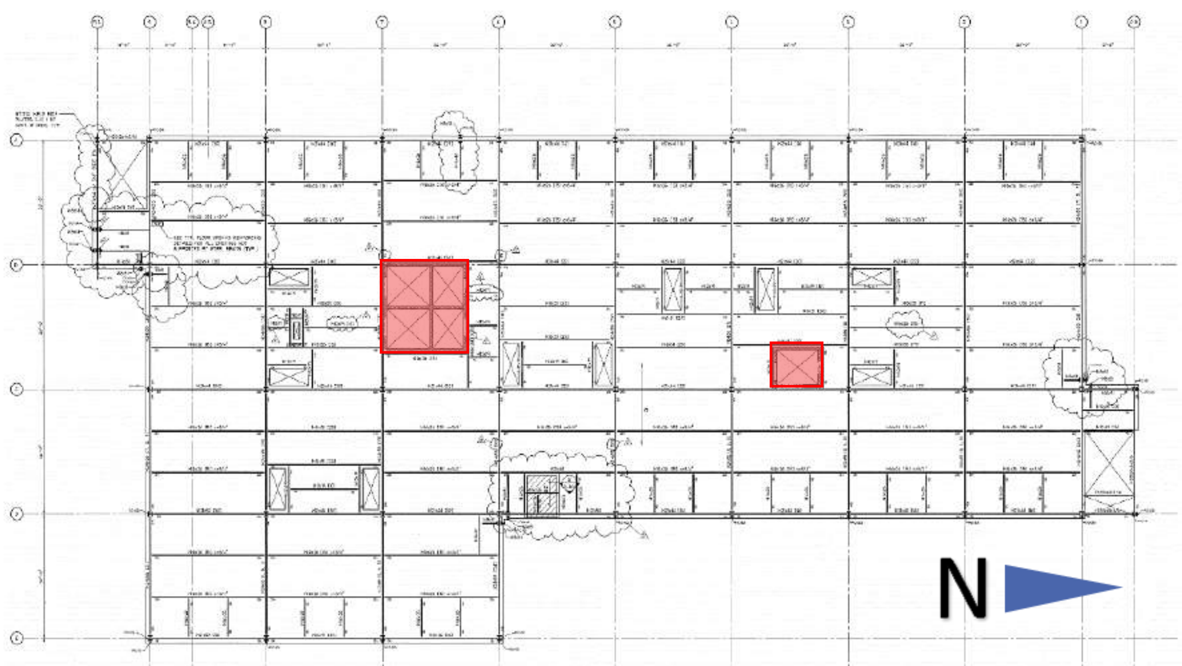

Figure 5: Elevator Cores on Upper Floors

The typical column to beam or girder connection used throughout most of the building is a flexible moment connection, as shown in Figure 6. Even though this is not a full moment connection designed to take all of the developed beam or column moment, these connections still participate in the primary lateral force resisting system. The connection was analyzed for how much moment it could take compared with the capacity of the beam, and this percentage of fixity was used when modeling the structure in RAM. See Notebook Submission C for hand calculations on the flexible moment connection.

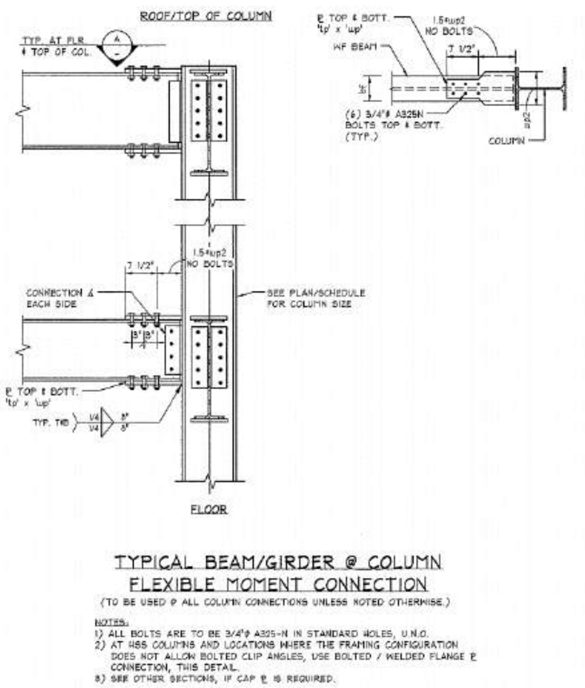

Figure 6: Typical Flexible Moment Connection

STRUCTURAL DEPTH

Redesigning the hospital brought up several goals that should be met with the new structure. Expansion and rearrangement of the hospital is possible, and floors should not be deeper than the existing steel. Thinner floors would provide a good point for making the system feasible. A system that is familiar to the area would also provide a good possibility, as the construction experience is readily available and can result in a comparable price point. In order to meet these goals, a two way slab with drop panels was chosen in order to create thinner floors, facilitate a type of construction that is common in the area and be competitive with the steel design, and as a learning goal of a direct comparison between the two systems.

GRAVITY SYSTEM

LOADS

Loads for the redesigned concrete system are similar to the loads used for the existing system. The main difference is in the dead load increase to 135 psf from 65 psf for the floor and roof structural system. This is a direct result of changing the floor to a 10" thick slab. Columns are also larger, which increases their self-weight and the force on the foundations below them. All live loads, enclosure loads and snow loads remain the same.

DESIGN LOADS

1. GRAVITY

a)	10" CONCRETE SLAB	125 PSF
b)	CEILING	2 PSF
c)	MEP	2 PSF
d)	FLOOR	2 PSF
e)	COLLATERAL	4 PSF

2. LIVE

a)	ANY LEVEL	100 PSF

TYPICAL FLOOR & ROOF TOTAL

135 PSF DEAD LOAD

100 PSF REDUCIBLE LIVE LOAD

Floor Slab Design

With the loads determined, the CRSI manual was used to come up with a preliminary design to validate and refine in RAM concept using a model of the floor. Using an f'c of 4000 psi, and 30' spans with the loads defined, a 10" flat slab was chosen with 19" thick drop panels, 10'x10' around columns. This slab thickness is above the minimum required by ACI 318-11 Tbl. 9.5(c) for a further analysis of the deflection of the system.

The slab design was modeled in 3D using RAM Concept. Initially, a two way slab with beams was used, but hand calculations showed that thickness of up to 10" could not hold the design moments. The slabs with drop panels was recommended by Dr. Said as a way to mitigate this failure, but keep the floor plate from being excessively large. The design was determined to be acceptable after modeling, and verifying punching failure with hand checks.

Column and middle strips consist of 15' wide strips, thanks to the square 30' bays. This is true for both the N/S and E/W directions. The area of steel per strip output by RAM Concept was confirmed using moments to calculate required steel areas without going below minimum requirements. Middle strips interior positive moment reinforcement have #5's @ 16" O.C. (minimum area) and column strips have #5's @ 12" O.C. Figures 7 and 8 below show middle and column strip end and interior span final reinforcement designs.

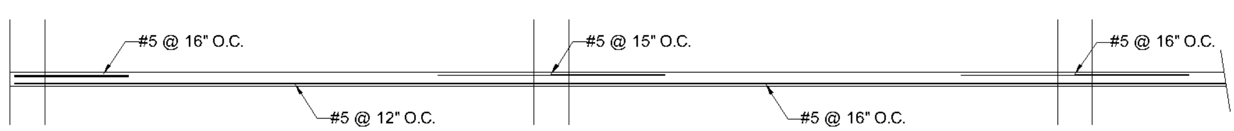

Figure 7: Middle Strip End and Interior Span Reinforcing (Typical)

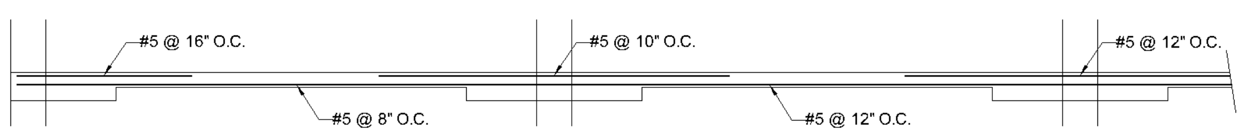

Figure 8: Column Strip End and Interior Span Reinforcing (Typical)

Appendix pgs. 1-5 (Document pgs. 31-35) provides the comparison calculations and verifications of the steel areas in the strips. While decreasing bar size and using a closer spacing provides better performance, it also makes construction much easier by only using 1 size bar throughout the floor slab. This will lower the amount of potential mistakes on the project.

Openings in the slab system are controlled by ACI 318-11 Section 13.4, specifically, sections 13.4.2.1 through 13.4.2.4. This requirements state that openings in middle strips are allowed as long as the steel that has been removed due to the opening is placed around the opening to maintain the same amount in the slab. In the column strip, openings no more than 1/8th the width of the strip can be allowed as long as the removed reinforcing is added on the sides of the opening. In the area between column and middle strips, no more than ¼ of the reinforcement in either strip can be interrupted. Again, the reinforcement removed can be added to the sides of the opening to maintain the same amount of steel as removed.

When viewing the goals of this part of the structure, to be at most the same depth as the steel design, it is easy to see that goal has been surpassed. With W21 and W26 sizes being used from column to column, and the portion of the slab between the drop panels being 10" deep, 1 ft of floor depth has been saved, with an increase in area to run equipment of at least 20 sq. ft. between each set of columns. This design successfully met the floor thickness goal.

Column Design

It was expected that the gravity columns would increase in size, because of the heavier structure and weaker strength than steel. The columns were sized in RAM and iterated until a passing solution was found. The interaction diagram was then plotted to double check the value. With the increased load from the concrete structure on the columns, to keep them within a reasonable size (not above 24"x24") a concrete with f'c of 6,500 psi was used. The table below shows the forces and strength of the 24"x24" columns. Figure 9 shows the interaction diagram for the column.

24"x24" column forces and capacity				
Pu	1,600 k	**ϕPn**	1,733 k	✓
Mu	45 k·ft	**ϕMn**	230 k·ft	✓

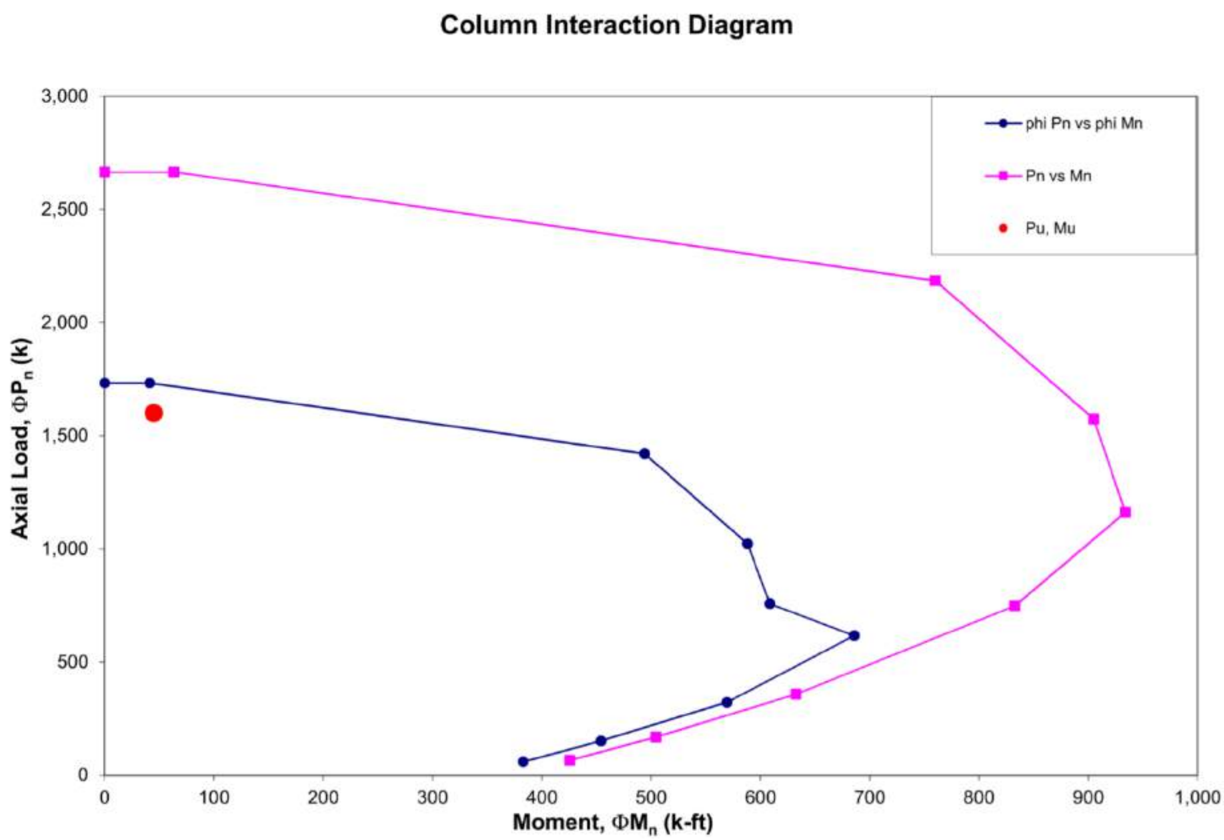

Figure 9: Concrete Column Interaction Diagram

These columns are used throughout the structure and not sized down where they are under capacity to allow for vertical expansion of the lower floors to reach the 5 story height of the bed tower. The square column shape also works well for horizontal expansion, because it is unknown which direction the building could be expanded, and rectangular columns may end up having high gravity moments in their weak direction. Figure 10 shows the final reinforcement design for the columns. See Appendix pgs. 9-13 (Document pgs. 39-43) for column design information and calculations.

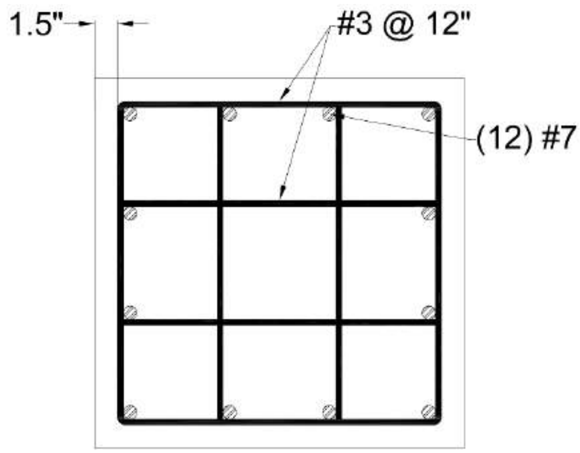

Figure 10: Typical Column Reinforcement Section

Foundation Design

The foundations for this system can remain spread footings, as there is plenty of available room and soil capacity is good. Although, with the increased load of the concrete, they were required to be increased. Utilizing the CRSI manual and backwards calculations to find the increased area needed, the 11'x11'x3' footings with (11) #8 bars EW were upsized to 15'x15'x3'9' footings with (12) #10 bars EW. These have a max allowed factored column capacity of 1,738 k. This is a 182% increase in capacity over the previous footings, which had a max allowed factored column capacity of 955 k. See Appendix pgs. 10 & 14 (Document pgs. 40 & 44) for the footing calculations and the CRSI pages referenced.

LATERAL FORCE RESISTING SYSTEM

LOADS

In order to be informed with the use of the current codes, the use of ASCE 7-10 for the redesigned lateral system was required. This results in different forces for wind, which still control the design, as in the existing structure. Seismic forces increase because of the increased weight of the structure, which was obviously expected when redesigning to a concrete system, but the Equivalent Lateral Force Procedure did not change, therefore does not result in different seismic forces as a result of that specific method. The lateral design parameters from ASCE 7-10 are summarized below.

Lateral Loads – Wind

a) Risk Category – IV
b) Basic Wind Speed (3 Second Gust) – 120 mph
c) Wind Importance Factor – 1.0
d) MWFRS Exposure Category – C
e) Internal Pressure Coefficient - +- 0.18

Lateral Loads – Seismic

a) Seismic Importance Factor – 1.5
b) Occupancy Category – IV
c) Seismic Design Category – A
d) Seismic Force Resisting System – Ordinary Concrete Shear Walls
e) Response Modification Factor – 4
f) Analysis Procedure – Equivalent Lateral Force Procedure

Seismic floor forces were scaled up by approximately 200%. Seismic base shear was calculated as 299k, matching values in RAM's outputs from the computer model. Wind base shear also matched RAM output reports.

Since the structure redesign already uses concrete, cast in place shear walls were chosen as the redesigned lateral system. The architectural plans were utilized to place shear walls where they wouldn't interfere with changes from floor to floor. The elevator cores could be utilized and additional shear walls were placed wherever possible to decrease eccentricities. Figure 11 shows the initial shear wall layout, aligned and placed to minimize eccentricities. After running the analysis and several verification checks, all walls were under-utilized and only required minimum reinforcement. Deflections were much too small, with values around 1/3" when 2.16" was the allowable h/400 design limit. 12" walls were possibly oversized for the building size and forces therein. Figure 12 shows the final shear wall layout, with roof level deflections increased to approximately 1.3". This design, as expected with shear walls, still had less deflections because of the inherent stiffness of the concrete, and the wall thicknesses were decreased to 8", which requires much less concrete and only a single curtain of reinforcement instead of two.

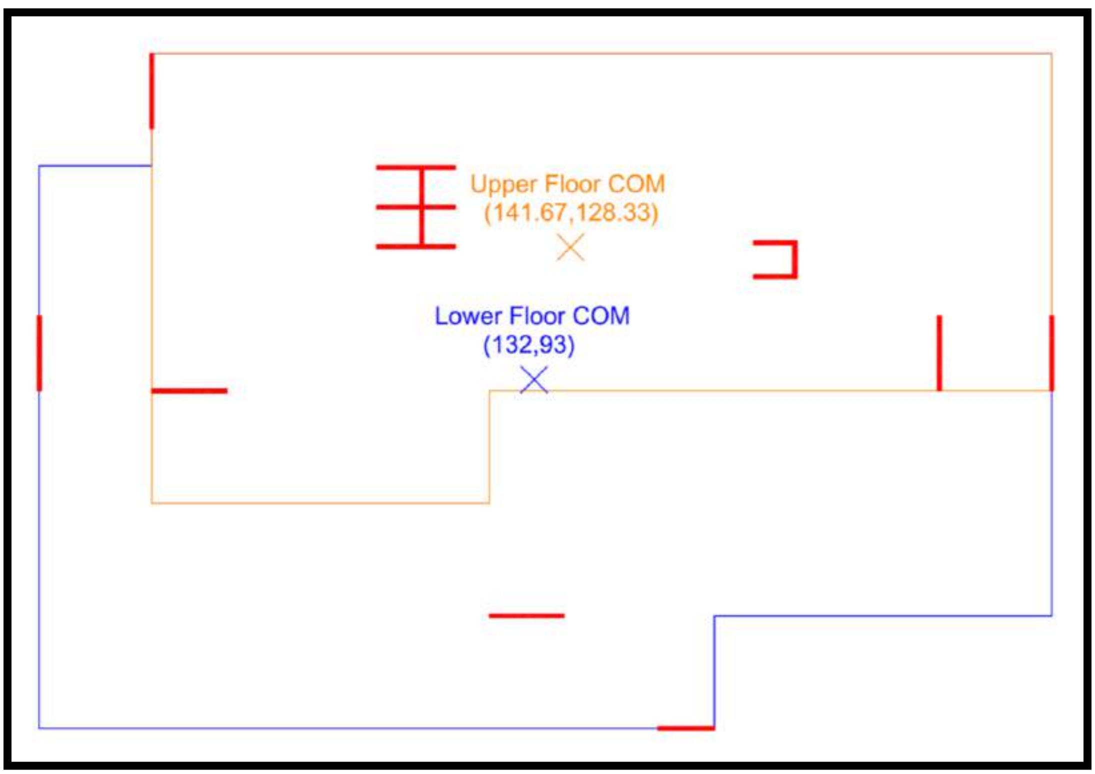

Figure 11: Initial Shear Wall Layout

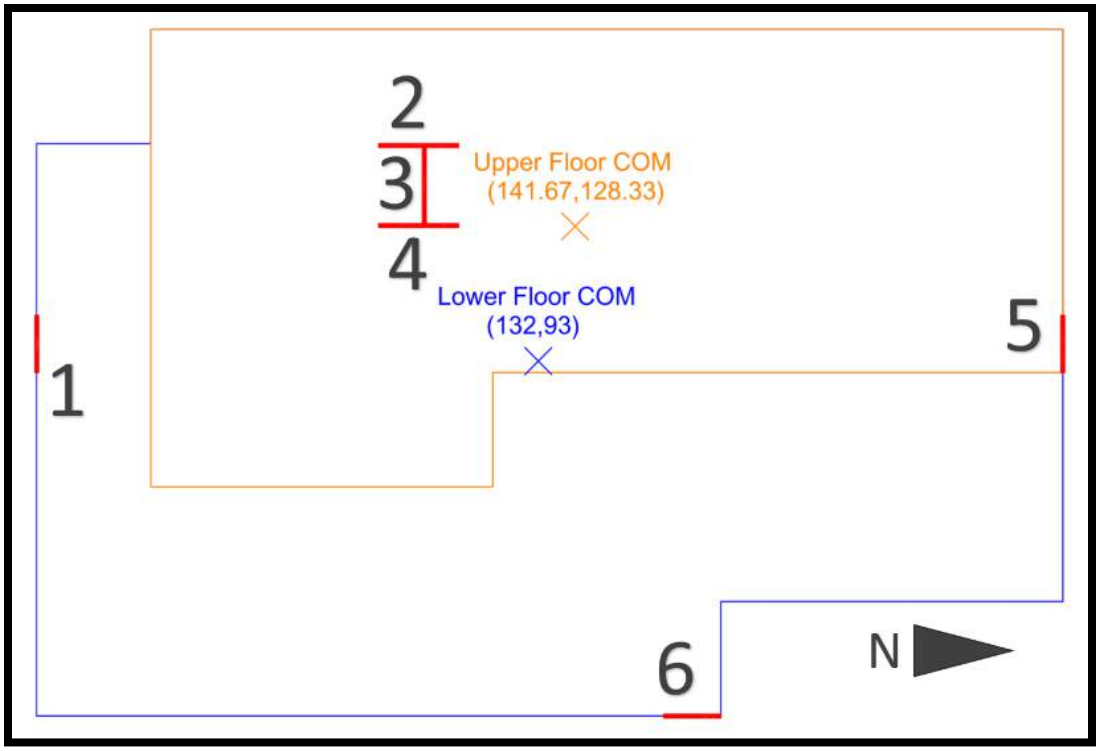

Figure 12: Final Shear Wall Layout

The final design of the shear walls was determined to be an 8" thick wall using concrete with an f'c of 4000 psi and a single curtain of reinforcement. Figures 13 and 14 show final reinforcement designs for walls 3 and 5, the controlling walls from load combination 0.9D+1.0W. All walls are 8" thick and use concrete with f'c of 4000 psi and have a single curtain of reinforcement. Appendix pgs. 27-36 (Document pgs. 57-66) gives the calculations for the shear wall capacity and comparisons from RAM to hand calculations.

Wall 3 is 72' tall and 20' deep. See Figure 13 and the following table for the reinforcement section cut and capacity and forces. When comparing with the values from RAM, it output a reinforcement schedule for vertical bars of #9's at 16" O.C. This was excessive and resulted in a ϕMn value of 8,934 k·ft. Excel calculations allowed a solution to be found through goal seek, resulting in the more efficient design shown below.

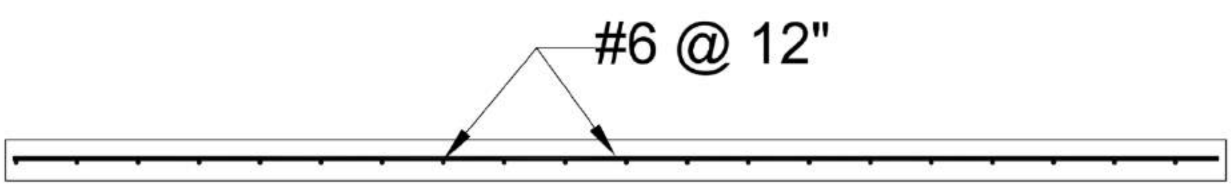

Figure 13: Wall 3 Reinforcement Section

Wall 3 Forces and Capacities Summary		
ϕPn (k)	ϕMn (k·ft)	ϕVn (k)
OK by observation	5905	432.5
Pu (k)	Mu (k·ft)	Vu (k)
260	5859	400

Wall 5 is 72' tall and 15' deep. See Figure 14 and the following table for the reinforcement section cut and capacity and forces. When comparing with the values from RAM, it output a reinforcement schedule for horizontal bars of #6's at 8" O.C. This was excessive and resulted in a ϕVn value of 467 k. Excel calculations allowed a solution to be found through goal seek, resulting in the more efficient design shown below. The vertical reinforcement matched excel calculations.

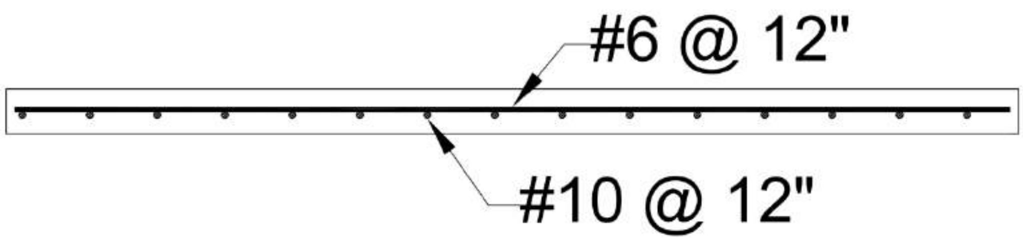

Figure 14: Wall 5 Reinforcement Section

Wall 5 Forces and Capacities Summary		
ϕPn (k)	ϕMn (k·ft)	ϕVn (k)
OK by observation	7170	226.4
Pu (k)	Mu (k·ft)	Vu (k)
376.7	7164	196

With the shear walls slightly altered from RAM, but towards more efficiency, the designs from RAM for the remaining shear walls are shown to be acceptable. See Appendix pgs. 37-38 (Document pgs. 67-68) for the reinforcement design of all shear walls.

BREADTH STUDIES

The two breadth studies covered in this thesis project are the construction management aspects of building the redesigned structure, and the study of the precast and aluminum panel enclosure. The cost and timeline of the concrete system are analyzed and compared to the steel structure in the construction management breadth. The enclosures breadth will view the exterior enclosures, analyze them for their properties and make any adjustments if necessary.

CONSTRUCTION MANAGEMENT

Concrete structures can be competitive to a steel structure in many ways. Steel can be put up very quickly, but also requires a lot of lead time in order to ensure the correct pieces are fabricated and delivered on time. Concrete does not need as much lead time, but care must be taken when delivering, as the construction site needs to be ready to place the concrete as soon as the trucks arrive. With the right crew, equipment and phasing, a concrete structure can go up just as quickly as steel.

To price out the two systems, RS Means 2015 Construction Cost Data was used, after pulling takeoffs from the 3D models. The steel building used a tonnage estimate, along with floor placing and finishing and foundations. The total cost for the steel superstructure was estimated to be approximately $4,950,000. This is a cost per square foot of $30.40. When compared against the total building cost, the steel structure accounts for 4.8% of the building cost. Concrete estimates used the columns, floor slabs, foundations and shear walls in terms of placing, forming and finishing if necessary. The total cost for the concrete superstructure was estimated to be approximately $4,650,000, which is a square foot cost of $28.55. This equates to 4.5% of the building cost. Both of these prices are very similar, and show that these two systems are comparable in terms of cost. Appendix page 47 (Document pg. 77) includes the RS Means and cost information.

The original steel structure was erected in two months. This was a very fast construction schedule that was aided by the utilization of building information modeling and having the entire design team involved at the beginning of the project. This two month schedule was also the timeframe for the concrete system. If it could be completed within the same time, and as shown above, at a similar cost, then this is a feasible system.

With information from RS Means daily output and other sources, the construction schedule was determined to be just under two months. Using a fictional start date of May 1, 2016, it was determined to take 79 days to construct the superstructure, with a completion date of June 18, 2016. See figure 15 for the construction dates and task durations. Footing and column completion schedules were determined using RS Means unit data with a typical crew as chosen. The floor schedule was determined using industry information that a typical crew can do approximately 10,000 sq. ft. per day, on a two day schedule. See Appendix pgs. 45-54 (Document pgs. 75-84) for construction timeline and schedule information and calculations.

Task Name	Start	End	Duration (days)
Footings	5/1/2016	5/10/2016	9
First Floor Columns	5/7/2016	5/18/2016	11
Second Floor	5/16/2016	5/21/2016	5
Second Floor Columns	5/20/2016	5/31/2016	11
Third Floor	5/28/2016	6/2/2016	5
Third Floor Columns	5/30/2016	6/4/2016	5
Fourth Floor	6/3/2016	6/6/2016	3
Fourth Floor Columns	6/5/2016	6/10/2016	5
Fifth Floor	6/9/2016	6/12/2016	3
Fifth Floor Columns	6/11/2016	6/16/2016	5
Roof	6/15/2016	6/18/2016	3
			0
			0
			0

Figure 15: Task Durations

BUILDING ENCLOSURE

The enclosure of the hospital is precast concrete panels with punched window openings and aluminum architectural panels. Behind the precast and aluminum panels are a steel stud wall with 5/8" gypsum board. Figure 16 shows the precast assembly and figure 18 shows the aluminum assembly. These two wall assemblies enclose the whole hospital. The design question that is posed is do these two walls have the same temperature resistance, and can any improvements be made to the enclosure, resulting in better performance? Figures 17 and 19 below show the temperature profiles calculated for the walls. An interior temperature of 68˚ was used with a 58˚ drop in temperature to the outside. Appendix pgs. 55-58 (Document pgs. 85-88) contains full calculations on the enclosure and its properties.

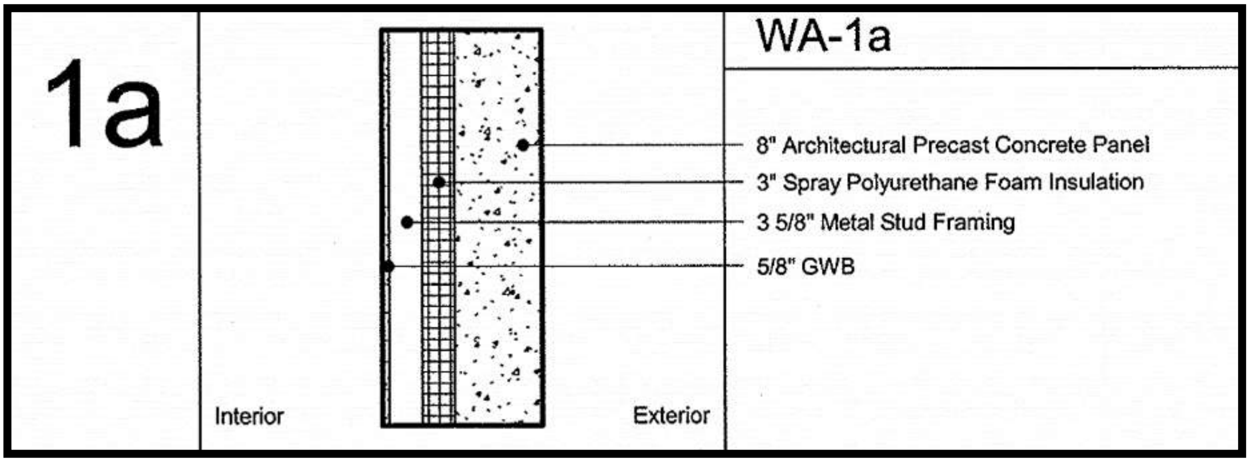

Figure 16: Precast Wall Assembly

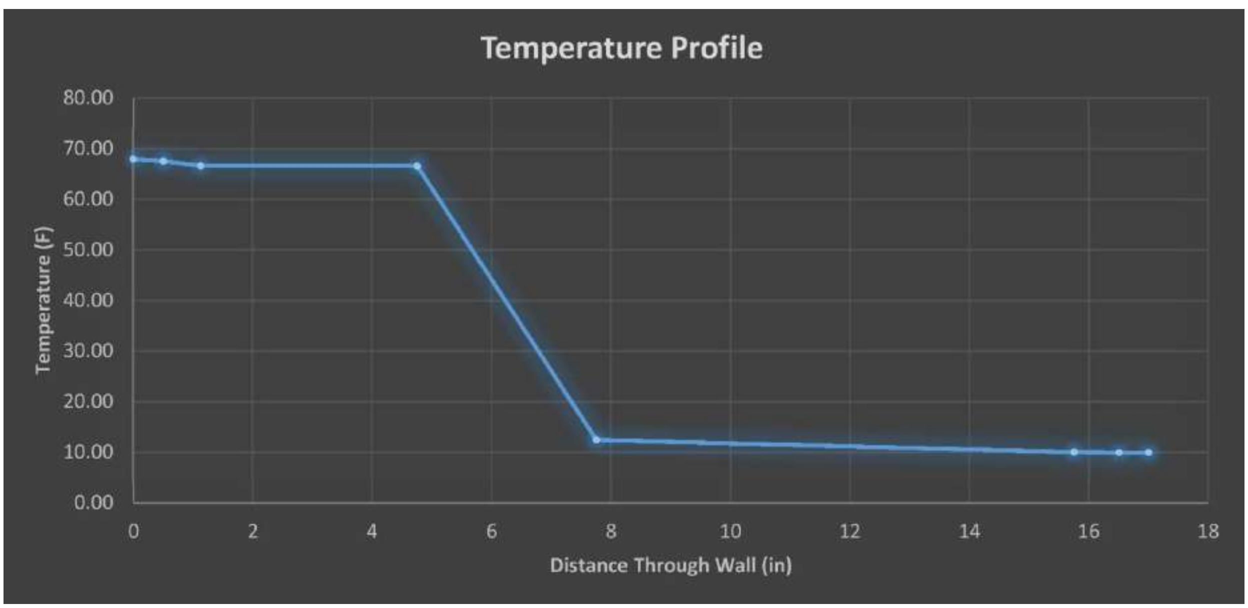

Figure 3: Precast Concrete Wall Section Temperature Profile

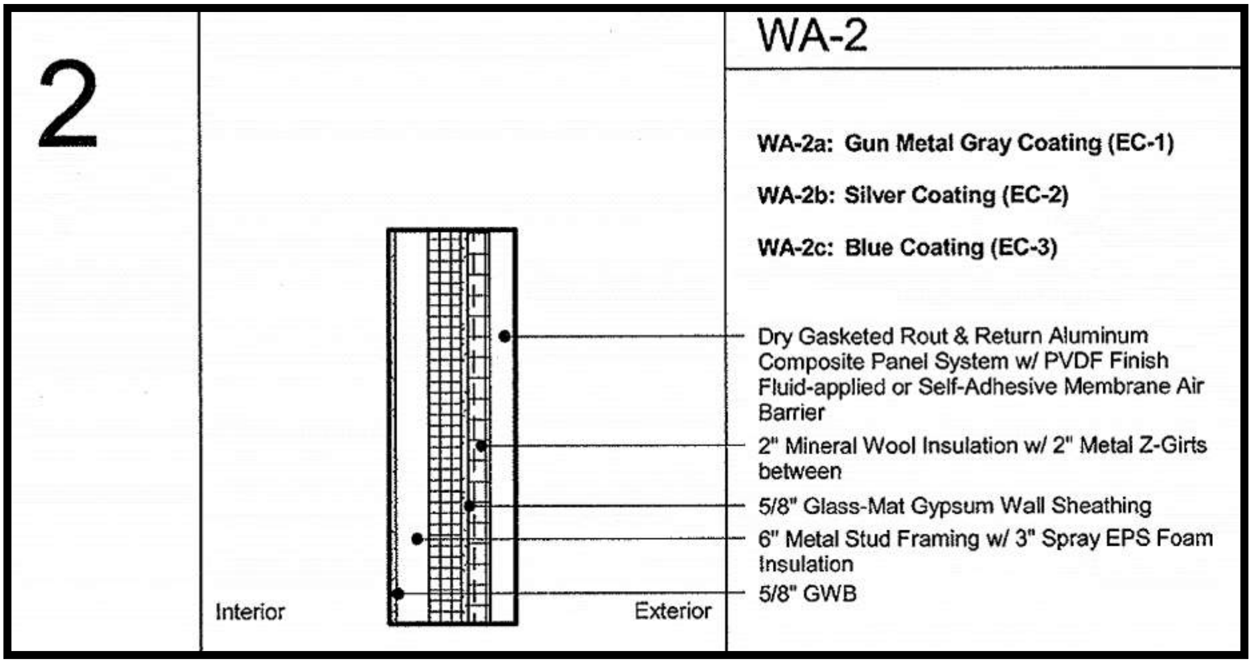

Figure 18: Aluminum Wall Assembly

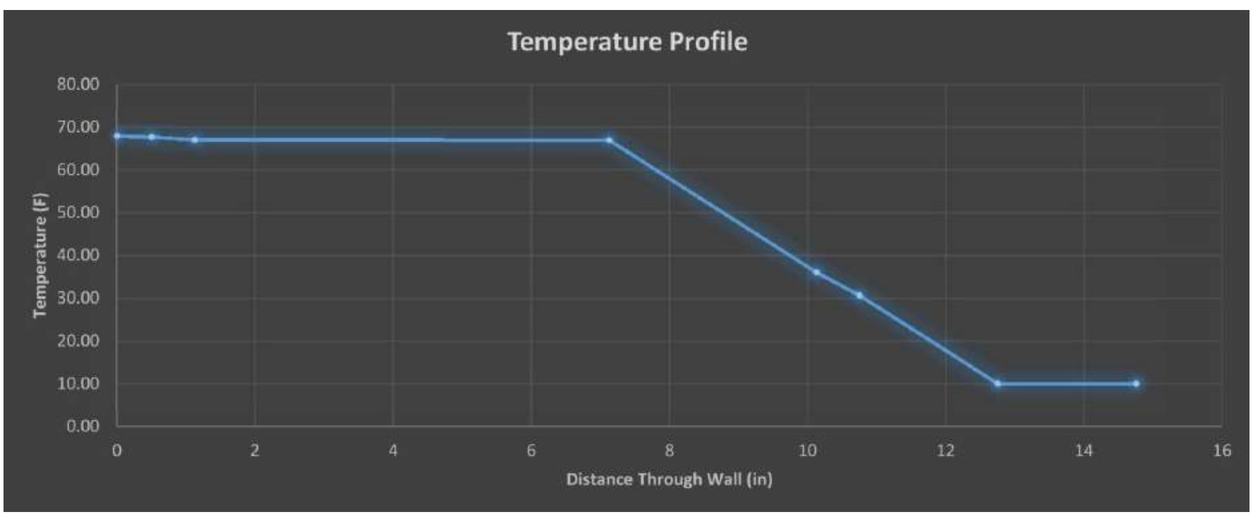

Figure 4: Aluminum Wall Section Temperature Profile

A surprising conclusion was drawn from these profiles and calculations, and that is the aluminum panel having very similar thermal properties as the precast panel. There is no doubt the double insulation in the aluminum panel aided this, and as such, the initial thought that it would be recommended to use precast panels to enclose the entire structure is not true. Special attention needs to be paid to the vertical construction of these panels, because of the difference in thickness.

MAE COURSEWORK REQUIREMENT

As discussed in the executive summary, this thesis included information to fulfill the MAE integrated degree requirements. Fulfillment of this requirement will include utilizing a 3D model in RAM to further analyze and refine the buildings systems using information from AE 530 – Computer Modeling. After finding initial sizes by hand and manual verification, modeling will allow a quick way to iterate through load cases and expedite the refinement process. Also, in order to be able to analyze the existing lateral system, information from AE 534 – Steel Connections was used to analyze the flexible moment connections. The building enclosure analysis utilized information from AE 542 – Building Enclosures Science and Design.

CONCLUSION

West Shore Hospital is currently designed as a composite structural steel gravity system and flexible moment frame lateral system. The work from this thesis has shown that a concrete system consisting of a two way flat slab with drop panels, concrete gravity columns and reinforced concrete shear walls would be feasible as an alternative structure for this hospital.

The redesigned structure is heavier than the existing steel, but the foundations are able to be increased in size without any negative impacts because of the distance between the columns. The heavier structure also has increased seismic forces, with a base shear increased from 140k to 300k, essentially doubling. This still did not control over wind, so the increased weight was not detrimental to the structure.

Floor depth was an important aspect of this design and was significantly improved on in the redesigned concrete structure. The steel design uses large members in between all columns as part of the lateral system, which increases the size of the beams, since they are not gravity only. W21 and W24 beams are used, which have depths of 21 and 24 inches, respectively. All mechanical equipment must be run underneath these, giving the floor system a total thickness of almost 3 feet or more in some places. The advantage of the concrete system is the 10” thick slab that extends through the entire floor, except where the drop panels are present around columns. Mechanical equipment can be routed underneath the 10” slab portion only, therefore decreasing the depth of the floor by at least 1 foot. This gives an extra 20 sq. ft. to run equipment compared to the steel system.

Cost is always a big factor in designing and constructing a building. The construction management analysis done showed that there is not a significant cost difference between the two systems, and they can even be constructed within similar timeframes. All of these statements point toward a successful redesign, with feasibility for construction in the real world.

REFERENCE MATERIAL

The following list contains the documents used and referenced for this thesis project.

- AE Curriculum Course Notes and Materials (various sources)

Louis F. Geschwindner (1991) "A Simplified Look at Partially Restrained Beams." Engineering Journal, American Institute of Steel Construction.

Louis F. Geschwindner, Robert O. Disque (2005) "Flexible Moment Connections for Unbraced Frames Subject to Lateral Forces – A Return to Simplicity." Engineering Journal, American Institute of Steel Construction.

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MacGregor, James G. *Reinforced Concrete: Mechanics and Design*. Englewood Cliffs, NJ: Prentice Hall, 1988. Print.

Geschwindner, Louis F. *Unified Design of Steel Structures 2nd Edition*: John Wiley, 2012. Print.

Steel Construction Manual. Milwaukee, WI: AISC, 2011. Print.

Williams, Alan. *Civil & Structural Engineering: Design of Reinforced Concrete Structures Review for the PE Exam*. Chicago: Kaplan, AEC Education, 2007. Print.

McCormac, Jack C., and Russell H. Brown. *Design of Reinforced Concrete*. 9th ed. N.p.: John Wiley, 2014. Print.

Appendix

Pg. 29

Aaron King – Final Thesis Report Supplement

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Since 2 way slab w/ beam was insufficient, utilize CRSI manual to begin design of two way flat slab system with drop panels.

span = 30 ft   superimposed factored load: $1.2(15) + 1.6(100) = 178$ psf
use 200 psf in table

$f'_c = 4000$ psi $f_y = 60$ ksi

From CRSI chapter 10 Tables

Interior Panel: 20" square columns ✓ acceptable
10" slab thickness ✓
8.25" drop panel depth ✓ may be increased to meet typical sizes
10' drop panel width ✓

Reinforcing

Each Way
- Column Strip < Top: (16) #6's
  Bot: (14) #6's
- Middle Strip < Top: (15) #5's
  Bot: (13) #5's

3.24 psf steel
0.91 cu.ft/ft² concrete

Edge Panel: Use the same concrete sizes

Reinforcing

Each Way
- Column Strip < Top Ext. +: (14) #5, 3
  Bottom: (16) #7
  Top Int: (18) #6
- Middle Strip < Top Int.: (12) #6
  Bottom: (14) #6

3.78 psf steel

Moments ft-kip

Edge (-)	Bot (+)	Int (-)
279.2	558.5	751.8

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER
COMET

RAM Concept V8i Design

Restricted Bar Sizes to #5 & #6

Typical Interior Panel:

Each Way

Column Strip
- Top: (8) #6's
- Bot.: (10) #6's

Middle Strip
- Top: (12) #5's
- Bot: (12) #5's

16-25 #6's on Top of Drop Slabs, each way. Depends on location.

Loads Used

- 150 pcf conc. self-weight
- 15 psf SDL
- 100 psf LL
- 1.19 k/ft wall line load
- 50 psf RLL
- 26 psf snow load

– Typical Bay Statical Moment, M_o

$q_u = 1.2(135)+1.6(100) = 0.322\,ksf$ w/o LL reduction

$L_o = 100$ $L = 50psf$ $q_u = 0.242\,ksf$ w/ LL reduction

$\ell_n = 28ft$ $\ell_2 = 30ft$

$M_o = \frac{0.322 \cdot 30 \cdot 28^2}{8} = 947\,k \cdot ft$ w/o reduction

$M_o = 712\,k \cdot ft$ w/ reduction

Column Strip = Middle Strip = 15' wide

– Excel Spreadsheet created to expedite process –

Typical Bay

RAM

Column Strip:

(10) #6 , $A_s = 4.4 in^2$

#6 @ 18" O.C.

Middle Strip:

(12) #5 , $A_s = 3.72 in^2$

#5 @ 15" O.C.

* smaller bars @ closer spacing can be used.

Excel/Hand Calcs

Column Strip:

+Mom: $A_{s\,req} = 4.14 in^3$ (RAM OK)

Instead, use (15) #5's $A_s = 4.65 in^2$

∴ #5 @ 12" O.C.

Middle Strip

+Mom: $A_{s\,req} = 2.79\ in^2$ (RAM OK, conservative)

Instead, use (9) #5's $A_s = 2.79\ in^2$

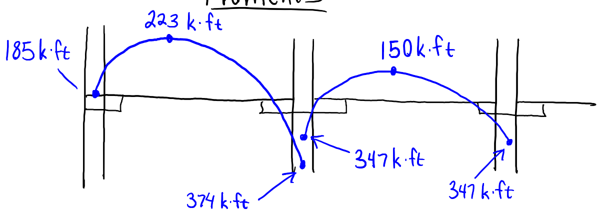
∴ #5 @ 20" O.C.

$As_{min} = 0.0018 \cdot 10 \cdot 12 = 0.22 in^2/ft$ ACI 318-11 7.12.2.1

#5 @ 16 $A_s = 0.23 in^2/ft$, do not go above 16" spacing

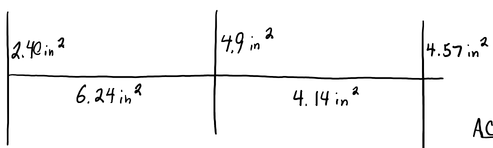
Column Strip 15' wide

Moments



10" slab, 19" thick drop panels @ columns

Req'd Steel Area



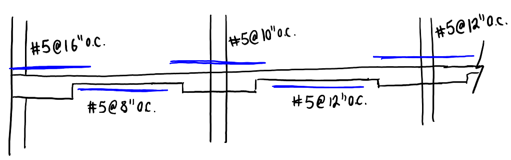
8 bars	21 bars	16 bars	15 bars	15 bars
#5@22.5" N.G. use min spacing #5@16"	#5@8" $A_s = 6.98 in^2$	#5@10" ~18 bars $A_s = 5.58 in^2$	#5@12" $A_s = 4.65 in^2$	#5@12" $A_s = 4.65 in^2$

using #5's ($0.31 in^2$)

ACI 13.3.2

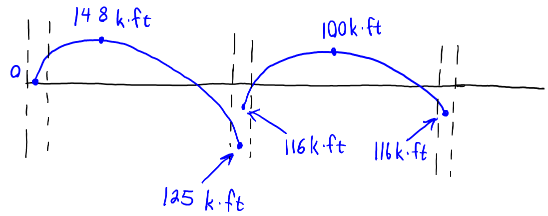
slabs, $S_{max} = 2h = 20''$

does not control over min. reinf. requirements



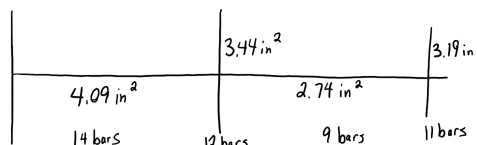
Middle Strip 15' wide

Moments

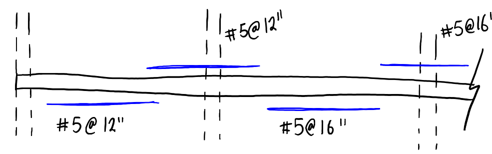


10" thickness entire strip

Req'd Steel Area



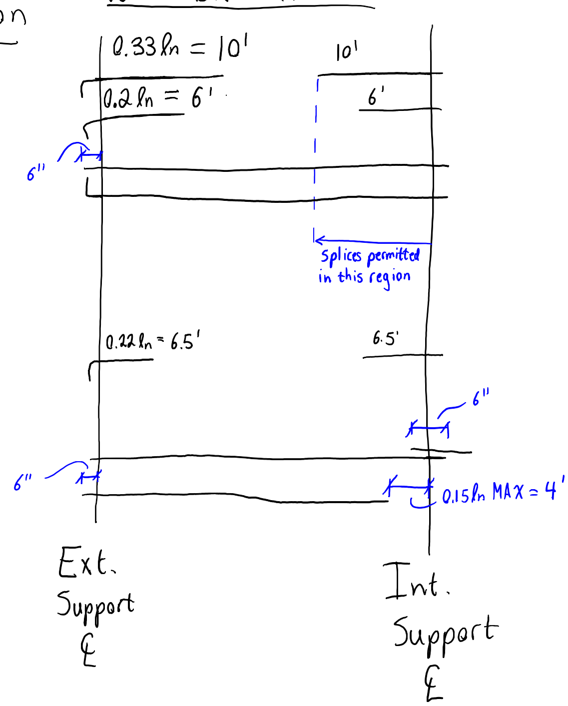
14 bars	12 bars	9 bars	11 bars
use #5@12" 15 bars $A_s = 4.65 in^2$	use #5@14" $A_s = 3.72 in^2$	#5@20" N.G. use min #5@16"	#5@16" 12 bars $A_s = 3.72 in^2$



ACI 318-11 Fig. 13.3.8 – Reinforcement Extensions

WITH DROP PANELS

		Min A_s @ Section
COL. STRIP	TOP	50% remainder
	BOTTOM	100%
MID. STRIP	TOP	100%
	BOTTOM	50% remainder



Allowable Slab Openings according to ACI 318-11 Section 13.4

- Middle strips
  - any size allowed, provided total panel reinforcing is required
- Column strips
  - no more than 1/8 strip width allowed → 1.875 ft or 1'-10.5" max
- Area common to both strips
  - no more than 1/4 reinforcement interrupted → 3'9" max

q_u =	0.242	ksf
l_n =	28.00	ft
l_2 =	30.00	ft
M_o =	711.48	k·ft

Interior Span

M - =	462.46	k·ft
M + =	249.02	k·ft

End Span to Use in Calcs

Int. -	498.04	k·ft
+	369.97	k·ft
Ext. -	184.98	k·ft

End Span Reference (Choose Appropriate Situation)

	1	2	3	4	5
	Ext. Edge Unrestrained	Slab w Beams Between all supports	Slab w/o beams between interior supports		Ext. Edge fully restrained
			w/o Edge beam	w/ Edge beam	
Int. - Factored Moment	533.61	498.04	498.04	498.04	462.46
"+ Factored Moment"	448.23	405.54	369.97	355.74	249.02
Ext. - Factored Moment	0	113.84	184.98	213.44	462.46

Column Strip

Interior Span

Interior Negative Factored Moment (k·ft)

l_2/l_1	0.5	1	2
$\alpha_{f1}l_2/l_1 = 0$	346.85	346.85	346.85
$\alpha_{f1}l_2/l_1 \geq 1$	416.22	346.85	208.11

Positive Factored Moment (k·ft)

l_2/l_1	0.5	1	2
$\alpha_{f1}l_2/l_1 = 0$	149.41	149.41	149.41
$\alpha_{f1}l_2/l_1 \geq 1$	224.12	186.76	112.06

End Span

Interior Negative Factored Moment (k·ft)

l_2/l_1	0.5	1	2
$\alpha_{f1}l_2/l_1 = 0$	373.53	373.53	373.53
$\alpha_{f1}l_2/l_1 \geq 1$	448.24	373.53	224.12

Positive Factored Moment (k·ft)

l_2/l_1	0.5	1	2
$\alpha_{f1}l_2/l_1 = 0$	221.98	221.98	221.98
$\alpha_{f1}l_2/l_1 \geq 1$	332.97	277.48	166.49

Exterior Negative Factored Moment (k·ft)

l_2/l_1		0.5	1	2
$\alpha_{f1}l_2/l_1 = 0$	$\beta_t = 0$	184.98	184.98	184.98
	$\beta_t \geq 2.5$	138.735	138.735	138.735
$\alpha_{f1}l_2/l_1 \geq 1$	$\beta_t = 0$	184.98	184.98	184.98
	$\beta_t \geq 2.5$	166.482	138.735	83.241

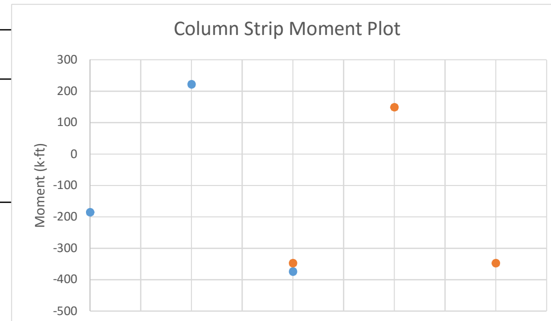
Column Strip Final Values

Interior Span

Neg. Mom. =	346.85	k·ft
Pos. Mom. =	149.41	k·ft

End Span

Int. Neg. =	373.53	k·ft
Pos. =	221.98	k·ft
Ext. Neg. =	184.98	k·ft



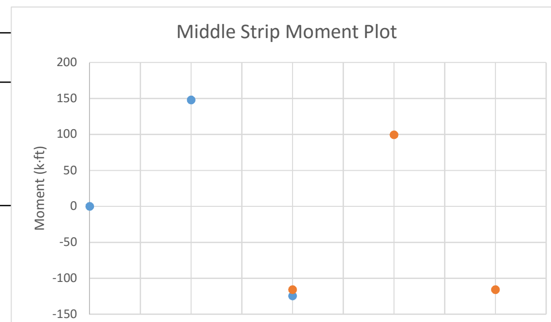
Middle Strip Final Values

Interior Span

Neg. Mom. =	115.61	k·ft
Pos. Mom. =	99.61	k·ft

End Span

Int. Neg. =	124.51	k·ft
Pos. =	147.99	k·ft
Ext. Neg. =	0	k·ft



Design Status: Reinforcement Plan

Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Top Bars; Span Design Bottom Bars; Span Design Shear Bars; Span Design Bar Descriptions; Latitude DS Designs; Longitude DS Designs; DS Design Top Bars; DS Design Bottom Bars; DS Design Shear Bars;
Drawing Import: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Reinforcement: Top Face Concentrated Reinf.; Bottom Face Concentrated Reinf.; Both Faces Concentrated Reinf.; Auto Face Concentrated Reinf.; Concentrated Reinf. Descriptions; Top Face Distributed Reinf.; Bottom Face Distributed Reinf.; Both Faces Distributed Reinf.; Auto Face Distributed Reinf.; Distributed Reinf. Descriptions; Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.;
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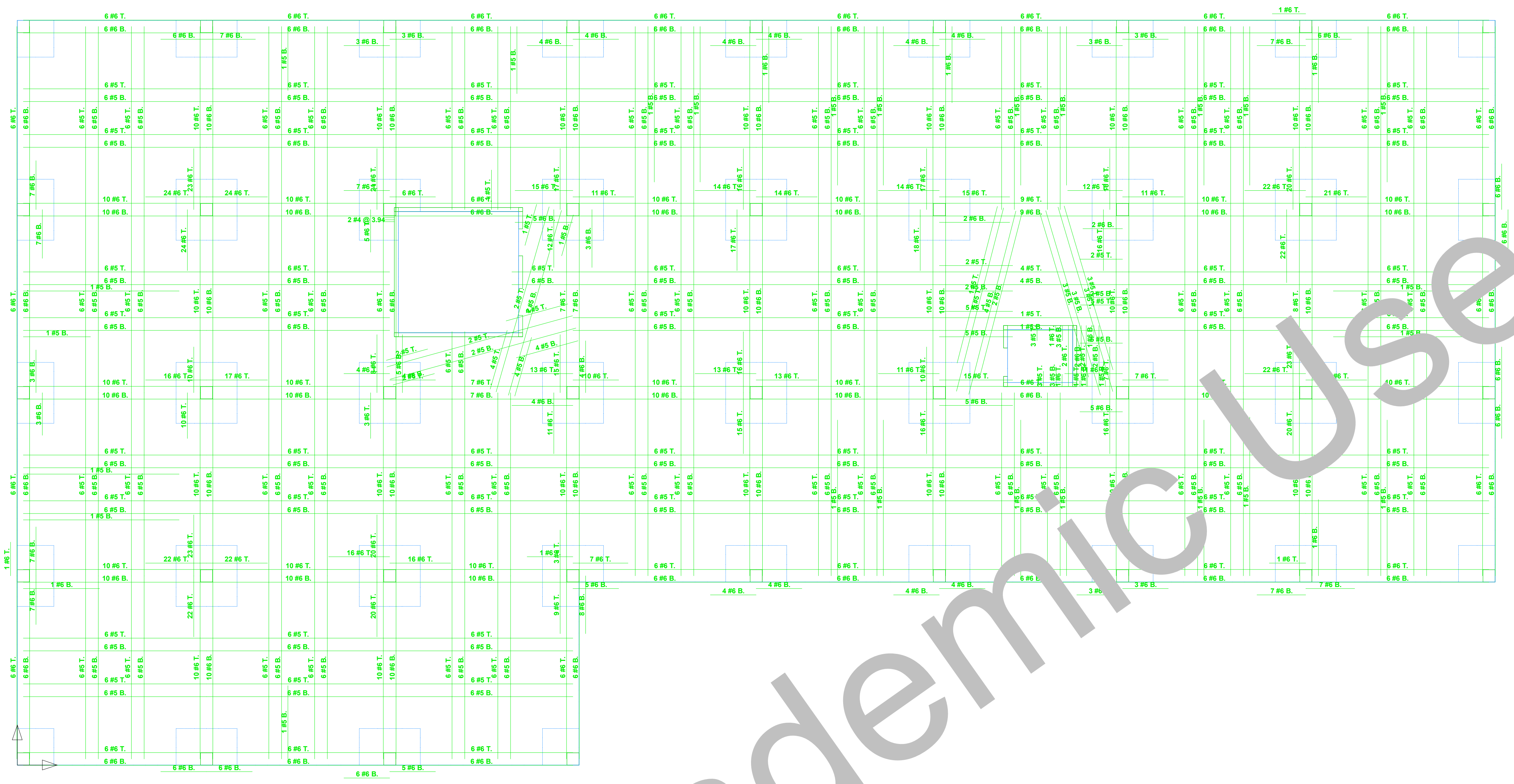

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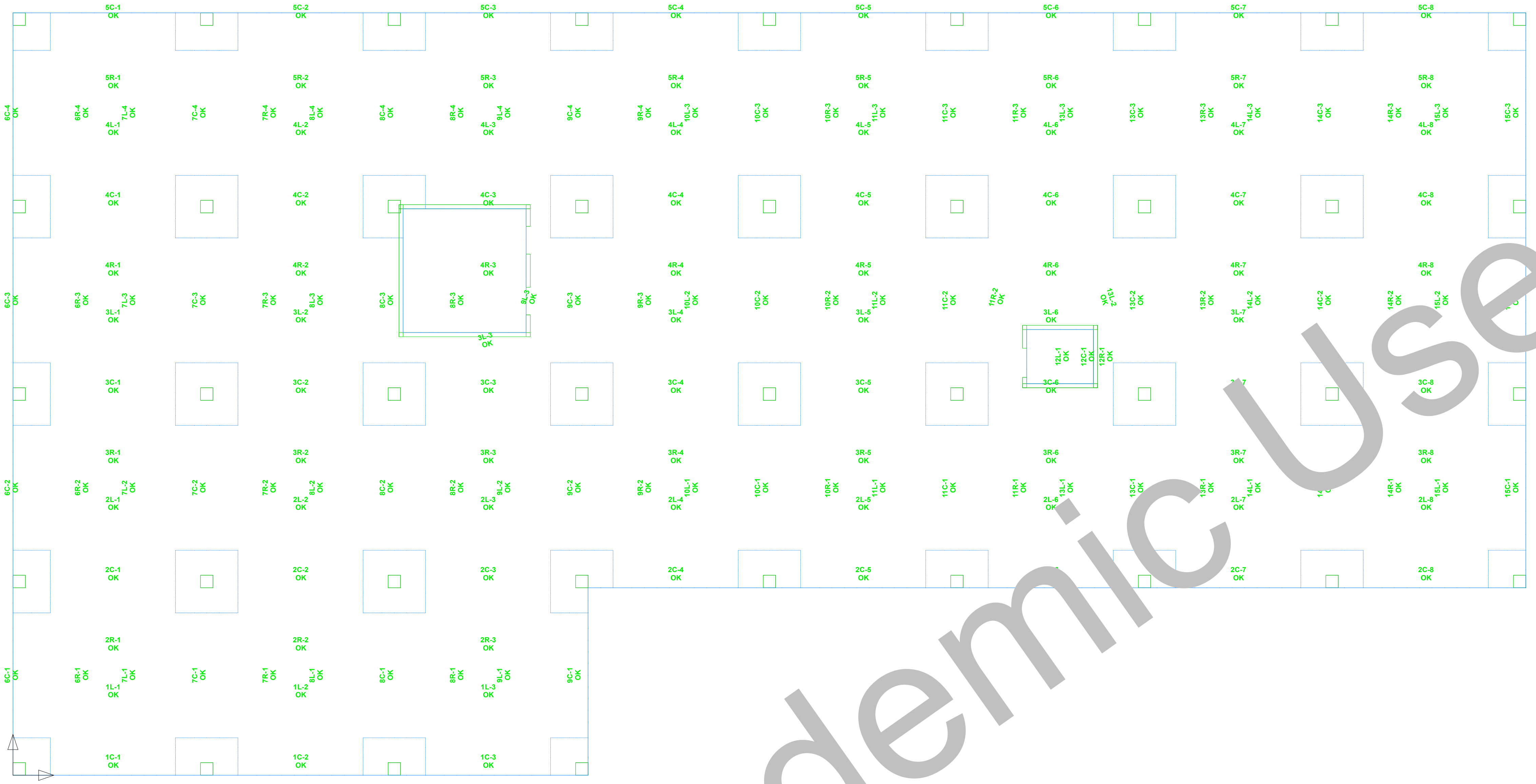
Design Status: Status Plan

Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status;
Drawing Import: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250



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3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Two Way Slab w/ drop panels check

Initial values from CRSI, confirmed in RAM Concept

10" flat slab, $f'_c = 4000\,psi$ 19" thick drop panels, 10' square $f'_c = 4000\,psi$

24" square columns, $f'_c = 6{,}000\,psi$

ACI 13.2.5 - Drop panel must extend below slab by 1/4 slab thickness. ✓ OK

Drop panel must extend at least 1/6 span in a direction from center of support

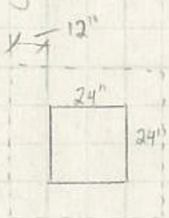
$30/6 = 5' \Rightarrow 10' \times 10'$ panel ✓ OK

ACI Tbl. 9.5(c) - Minimum slab thickness, with drop panels

interior: $\ell_n = 28'$ $\frac{28 \cdot 12}{36} = 9.33'' < 10''$ ✓ OK

exterior: same as above ✓ OK

- Punching shear (drop panels)



$t = 19''$

$b_o = (24 \cdot 2) \cdot 4 = 192''$

$A = t \cdot b_o = 3{,}648\,in^2$

ACI Eqn 11-33

$V_c = 4\sqrt{f'_c} \cdot A = 4\sqrt{4000} \cdot 3{,}648 = 922{,}879\,lb$
$= 922.8k$

$\phi V_c = 0.75 \cdot V_c = 692\,k$

Dead Load: $0.150(30 \cdot 30 \cdot (10/12) + 10 \cdot 10 \cdot (9/12)) = 123.75k$

Live load: $0.075(30 \cdot 30) = 67.5k$

$1.2D + 1.6L = 256.5k \ll \phi V_c = 692\,k$

$V_u > 0.5\phi V_c$ not req'd - ACI 11.4.6.1
· solid slab

• flexural steel present may act as shear reinforcement

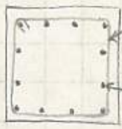
$V_u \ll \phi V_c \therefore$ slab ok for punching ✓

Gravity Columns

RAM showing column 8B as highest capacity with Load/Cap = 0.91

- For future expansion purposes, keep all columns 24" x 24"

Utilized bar pattern of (12) #5-#8 bars w/ #3 ties for RAM design purposes. Since columns are not lateral, square configuration is the most efficient.



- #5-#8 chosen for ease of constructability

— Column 8B RAM information —

Controlling Combination: 1.2D+1.6L+0.5S

Axial: 1684.4k
Top moment: 46.09 k·ft
Bottom moment: -11.85 k·ft

Direct concrete axial capacity: (24"x24" col w/ (12) #7 long. reinf.)

$A_c = (24\cdot24) - 7.2 = 568.8\ in^2$
$A_s = 7.2\ in^2$

$P_n = 0.85\cdot6000\cdot568.8 + 0.9\cdot60{,}000\cdot7.2 = 3{,}290\,k$
$\phi P_n = 0.65\cdot3290 = 2{,}138\,k$

Axial Load Check

$A_T = 900ft^2$ LL = 100psf Reduced LL = 75psf

Dead: column self-weight: 2·2·75·150 = 45k
drop panel s-w: 10·10·9/12·150 = 11.25k/panel (5 panels) = 56.25k
slab self weight: 30·30·10/12·150 = 112.5k/floor (5 floors) = 562.5k
finishes/MEP: 30·30·15 = 13.5k/floor (5 floors) = 67.5k
Air Handling Unit: 13·53·100 = 68.9k
$\Sigma = 800.2\,k$

Live: Typical Floor: 30·30·75 = 67.5k (4 floors) = 270k
Roof Live: 30·30·50 = 45k (1 floor) = 45k
$\Sigma = 315k$

Snow: Roof Snow: 30·30·26 = 23.4k (1 floor) = 23.4k
$\Sigma = 23.4k$

$1.2D + 1.6L + 0.5S = 1.2(800.2) + 1.6(315) + 0.5(23.4) = 1{,}460k$

- Comparing to RAM (see Column Design Forces printout) RAM does not reduce the floor LL, which accounts for the load difference

  - non reduced - $1.2(800.2) + 1.6(410) + 0.5(23.4) = 1628k$ vs. $1684.4k$ ✓

	Hand values	RAM values
Dead:	800.2k	841.12k
Live:	410 k	413.4k
Snow:	23.4 k	27k

Discrepancy in column loading will not change size of columns ✓

- Impact of increased weight due to material change on foundations -

Using Column 8B as example column...

From drawings - 6,000 psf soil bearing pressure

Existing Footing size: 11' x 11' x 3' w/ (11) #8 EW

Using CRSI 10^{th} Ed (ACI 318-08) - Factored Column Capacity = 955k ✗ N.G.

Footing will need to be redesigned to accomodate increased structure weight.

Plenty of room is available for increased spread footing sizes. No overlapping will occur.

- Square footing
- $P_u = 1685k$
- $q_a = 6000 psi$
- $f'_c = 3{,}000 psi$
- 24"x24" pier

Assuming live load ≈ ½ dead load...

$1.2(0.5D) + 1.6(D) = 1685$

$D \approx 840k \therefore L \approx 420k$ ✓

$\therefore P = 840 + 420 = 1260k$

$6ksf = \frac{1260}{B^2}$ $B \approx 14.5$ ft, use <u>15ft</u>

- Comparing to CRSI, 15' sq. footing, 45" thick, 20" minimum column size

(12) #10 EW Max Allowed Factored Column Capacity - 1,738k

1738k < 1650k ✓ acceptable design for max footing size

Comparison:	Existing	Re-designed	
Capacity:	955k	1738k	182% ↑
Steel:	561 lb	1,497 lb	267% ↑
Conc:	12.7 CY	31.3 CY	247% ↑

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER
COMET

Column Design Forces

RAM Concrete Analysis v14.07.01.01
Database: WSH Concrete V3 Two Way With Drop Panels 03/16/16 14:31:15
Building Code: IBC Concrete Code: ACI 318-11
Academic License. Not For Commercial Use.

Story:2nd Floor

Column No	End	Case	Condition	Axial (kip)	Mmajor (kip-ft)	Mminor (kip-ft)
77	Top	Dead Load		841.12	0.43	20.33
		Live Load	Max Mmajor+	413.40	0.71	13.59
			Max Mmajor-	0.00	0.00	0.00
			Max Mminor+	413.40	0.71	13.59
			Max Mminor-	0.00	0.00	0.00
			Sum All Axial	413.40		
		Roof Load		27.23	-0.01	-0.10
	Bot	Dead Load		841.12	-0.21	-9.90
		Live Load	Max Mmajor+	413.40	0.00	0.00
			Max Mmajor-	0.00	-0.35	-6.61
			Max Mminor+	413.40	0.00	0.00
			Max Mminor-	0.00	-0.35	-6.61
		Roof Load		27.23	0.01	0.05

Live Load Reduction Factors: Reducible LL = 0.00% Storage LL = 0.00%
Live Load Reduction Factor: Roof LL = 0.00%

Concrete Column Design

RAM Concrete Column v14.07.01.01
Database: WSH Concrete V3 Two Way With Drop Panels — 02/16/16 19:30:32
Building Code: IBC — Concrete Code: ACI 318-11

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COLUMN INFORMATION:

Level	2nd Floor		
Column Number:	77	Grid Location:	(8-B)
Size:	**24"x24"**	Depth x Width (in)	24.00x24.00
Reinforcement			
Longitudinal:	**12-#7 (4 x 2)**	As (in^2)	7.20 (1.25%)
Transverse:	**#3@ 12.0" 0'-0"-15'-0"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.00

MATERIAL PROPERTIES:

f'c (ksi):	6.50	fy Long (ksi):	60.00
fct (ksi):	0.00	fy Shear (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	4645.39	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	15.00	15.00
K	1.91	1.91
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp + 0.500 Sp

Axial		Load (kip)	1684.40
Moment	Top	Major(kip-ft)	1.65
		Minor(kip-ft)	46.09
Moment	Bottom	Major(kip-ft)	-0.25
		Minor(kip-ft)	-11.85

Calculated Parameters (Angle = 87.94 degrees): Ld/Cap = 0.91

0.65 Pn(kip): 1684.40

0.65 Mn Major(kip-ft): 16.76 — 0.65 Mn Minor(kip-ft): 467.05

	Major	Minor
Kl/r	49.63	49.63
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (1) 1.400 D

	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	0.16	157.46	24.75	0.75	136.66	0.00
1 Minor:	4.57	157.46	24.75	0.75	136.66	0.03

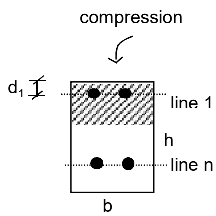
TORSION CAPACITY:

0.75 Tn (kip-ft): 0.00 — Tu (kip-ft): 0.00

Column Interaction Diagrams

for rectangular, tied columns with symmetric reinforcement

Material Properties		
f'_c	6,000	psi
f_y	60,000	psi



Design Loads		
	Pu, k	Mu, k-ft
LC I	1600	45
LC II		

Column Dimensions		
b	24	in
h	24	in

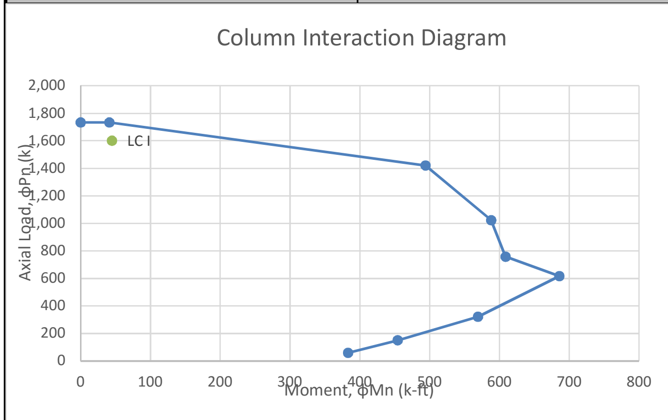
ε_s = strain in steel layer furthest from compression side
+'ve = tension

Reinforcement Requirements			
$A_{s,min}$	5.76	in^2	OK
$A_{s,max}$	46.08	in^2	OK

Calculate ----------->

Type of Shear Reinforcement ties

Interaction Diagram Data



☑Show Mini-Interaction Diagram

Reinforcement Detail				
Line	# of Bars	Bar Size	Line Depth (in)	A_s (in^2)
1	4	7	2.5	2.40
2	2	7	8.83	1.20
3	2	7	15.16	1.20
4	4	7	21.5	2.40
5				
6				
7				
8				
9				
10				
			Total A_s (in^2)	7.2

line 1 is the line of reinforcement closest to the compression side
line n is the line of reinforcement furthest from the compression side

Pn_max	-2,666	k
ϕ Pn_max	-1,733	k

Pn	Mn	ΦP_n	ΦM_n
k	k-ft	k	k-ft
2,666	0	1,733	0
2,666	63	1,733	41
2,186	760	1,421	494
1,573	905	1,023	588
1,162	934	757	609
749	833	617	686
358	633	322	569
168	505	151	454
65	426	59	383
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A
#N/A	#N/A	#N/A	#N/A

CONCRETE REINFORCING STEEL INSTITUTE

CONCENTRICALLY LOADED SQUARE SPREAD FOOTINGS

$f_c' = 3{,}000$ psi $f_y = 60{,}000$ psi

FACTORED SOIL BEARING CAPACITY 8,400 psf (SAFE BEARING CAPACITY 6,000 psf)

Size B	Thick-ness (in.)	Min. Col. Size (in.)	Bars Each Way (No.-Size)	Required Steel Area (sq.-in.)	Weight of Bars (pounds)	Volume of Concrete (cu. yd.)	Factored Column Capacity (kips)
4'-0"	14	10	5-#4 *	0.91	23	0.7	131
4'-6"	15	10	4-#5 *	1.24	33	0.9	166
5'-0"	17	10	5-#5	1.50	47	1.3	204
5'-6"	19	10	6-#5	1.83	63	1.8	245
6'-0"	20	10	8-#5	2.25	92	2.2	292
6'-6"	22	10	6-#6	2.62	108	2.9	341
7'-0"	23	10	8-#6	3.17	156	3.5	395
7'-6"	25	10	6-#7	3.60	172	4.3	451
8'-0"	26	11	7-#7	4.13	215	5.1	513
8'-6"	27	12	8-#7	4.70	262	6.0	578
9'-0"	29	12	9-#7	5.19	313	7.3	645
9'-6"	30	13	10-#7	5.82	368	8.4	717
10'-0"	31	14	11-#7	6.49	427	9.6	794
10'-6"	33	14	9-#8	7.09	481	11.2	872
11'-0"	34	15	10-#8	7.82	561	12.7	955
11'-6"	35	16	11-#8	8.59	646	14.3	1041
12'-0"	37	16	12-#8	9.23	737	16.4	1130
12'-6"	38	17	13-#8	10.06	833	18.3	1223
13'-0"	39	18	11-#9	10.96	935	20.3	1321
13'-6"	41	18	12-#9	11.67	1061	23.1	1419
14'-0"	42	19	10-#10	12.64	1162	25.4	1523
14'-6"	43	19	11-#10	13.79	1325	27.9	1630
15'-0"	45	20	12-#10	14.38	1497	31.3	1738
15'-6"	46	21	13-#10	15.40	1678	34.1	1852
16'-0"	47	21	14-#10	16.65	1868	37.1	1970
16'-6"	48	22	14-#10	17.74	1928	40.3	2091
17'-0"	49	23	15-#10	18.87	2130	43.7	2215
17'-6"	51	23	13-#11	19.82	2348	48.2	2338
18'-0"	52	24	14-#11	21.00	2603	52.0	2469
18'-6"	54	24	15-#11	21.93	2869	57.0	2598
19'-0"	55	25	15-#11	23.16	2949	61.3	2735
19'-6"	56	26	16-#11	24.43	3230	65.7	2875
20'-0"	57	26	17-#11	25.98	3523	70.4	3018

Notes: 1. Epoxy-coated bars of the same size and number may be used except where an "*" is printed. In which case, use: smaller bars (with increased number of bars); hooked or headed bars; increased f_c', or larger footing.
2. Reinforcing steel quantities do not include footing dowels.

CONCENTRICALLY LOADED SQUARE SPREAD FOOTINGS

$f_c' = 3{,}000$ psi $f_y = 60{,}000$ psi

FACTORED SOIL BEARING CAPACITY 9,100 psf (SAFE BEARING CAPACITY 6,500 psf)

Size B	Thick-ness (in.)	Min. Col. Size (in.)	Bars Each Way (No.-Size)	Required Steel Area (sq.-in.)	Weight of Bars (pounds)	Volume of Concrete (cu. yd.)	Factored Column Capacity (kips)
4'-0"	14	10	5-#4 *	0.97	23	0.7	142
4'-6"	16	10	4-#5 *	1.23	33	1.0	179
5'-0"	18	10	4-#6 *	1.54	54	1.4	221
5'-6"	19	10	5-#6 *	1.97	75	1.8	267
6'-0"	21	10	6-#6	2.32	99	2.3	316
6'-6"	23	10	7-#6	2.70	126	3.0	370
7'-0"	24	10	8-#6	3.27	156	3.6	428
7'-6"	26	11	9-#6	3.61	189	4.5	490
8'-0"	27	12	7-#7	4.19	215	5.3	556
8'-6"	28	12	9-#7	4.89	294	6.2	627
9'-0"	30	13	9-#7	5.31	313	7.5	701
9'-6"	31	14	10-#7	5.97	368	8.6	779
10'-0"	33	14	11-#7	6.55	427	10.2	861
10'-6"	34	15	13-#7	7.28	531	11.6	947
11'-0"	35	16	14-#7	8.05	601	13.1	1038
11'-6"	37	16	9-#9	8.78	673	15.1	1130
12'-0"	38	17	10-#9	9.63	782	16.9	1228
12'-6"	39	18	11-#9	10.51	898	18.8	1330
13'-0"	41	18	12-#9	11.25	1020	21.4	1434
13'-6"	42	19	10-#10	12.24	1119	23.6	1544
14'-0"	43	20	11-#10	13.23	1278	26.0	1657
14'-6"	45	20	12-#10	14.05	1446	29.2	1771
15'-0"	46	21	12-#10	15.10	1497	31.9	1892
15'-6"	47	22	13-#10	16.19	1678	34.9	2017
16'-0"	49	22	14-#10	17.08	1868	38.7	2141
16'-6"	50	23	12-#11	18.29	2040	42.0	2273
17'-0"	51	24	13-#11	19.48	2279	45.5	2409
17'-6"	53	24	14-#11	20.44	2529	50.1	2543
18'-0"	54	25	14-#11	21.69	2603	54.0	2686
18'-6"	55	26	15-#11	22.98	2869	58.1	2832
19'-0"	57	26	16-#11	24.01	3145	63.5	2976
19'-6"	58	27	17-#11	25.36	3432	68.1	3129
20'-0"	59	27	18-#11	26.99	3730	72.8	3286

Notes: 1. Epoxy-coated bars of the same size and number may be used except where an "*" is printed. In which case, use: smaller bars (with increased number of bars); hooked or headed bars; increased f_c', or larger footing.
2. Reinforcing steel quantities do not include footing dowels.

– ASCE 7-10 Equivalent Lateral Force Procedure –

Values Used in RAM

Seismic Design Category A

Importance Factor – 1.5 (Risk Cat. IV)

$C_t = 0.02$, $T = T_a$

$S_s = 0.133$ $S_1 = 0.053$, $T_L = 6$

$R = 4$, Ordinary concrete shear walls

Seismic Design Category A

ASCE 7-10 § 11.7: SDC A, comply only with req'ments of § 1.4

ASCE 7-10 § 1.4.3 Eqn. 1.4-1 → $F_x = 0.01 \cdot W_x$

Floor mass determination

• Lower floor

$A = 45{,}000 \text{ ft}^2$

10" slab – 125 psf   Misc DL – 15 psf

(66) 24" sq. columns , $h = 15'$

(66) drop panels, adding 9" of thickness. size = 10'x10'

$W_x = 45000(125) + 66 \cdot 2^2 \cdot 15 \cdot 150 + 66 \cdot 10 \cdot 10 \cdot 9/12 \cdot 150 + 45000(15) = 7{,}636.5\text{k}$

$F_x = 0.01 \cdot W_x = \underline{76.4 \text{ k}}$

• Upper Floor

$A = 24{,}300 \text{ ft}^2$

same slab, (40) columns , $h = 14'$ (40) drop panels

$W_x = 24{,}300(125) + 40 \cdot 2^2 \cdot 14 \cdot 150 + 40 \cdot 10 \cdot 10 \cdot 9/12 \cdot 150 + 24{,}300(15) + 640 \cdot 14 \cdot 85 = 4{,}950\text{k}$

$F_x = \underline{49.5 \text{ k}}$

• Roof

same as Upper Floor, except 7' columns & AHU loads $W_x = 4{,}780 \text{ k}$ $F_x = \underline{47.8 \text{ k}}$

Seismic Base Shear = 299.6k compare to RAM's 298k ✓ confirmed

- Apply F_x's @ C.O.M.

Mass Comparison:

	Manual Value	RAM Value
Roof	4,780k	4,500 k
5th Floor	4,950k	4,940.6k
4th Floor	"	"
3rd Floor	7,636.5k	7,555.6k
2nd Floor	"	"

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5th Floor	1	56.44	-122.01
4th Floor	1	92.15	-199.59
3rd Floor	1	137.10	-279.69
2nd Floor	1	188.20	-360.14

Summary - Total Story Shears

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
Roof	19.34	19.34	-41.49	-41.49
5th Floor	56.44	37.09	-122.01	-80.52
4th Floor	92.15	35.71	-199.59	-77.58
3rd Floor	137.10	44.95	-279.69	-80.10
2nd Floor	188.20	51.10	-360.14	-80.45

Load Case: E1 ASCE 7-10 Seis EQ_ASCE710_X_+E_F

Level	**Diaph. #**	**Shear-X kips**	**Shear-Y kips**
Roof	1	46.48	-0.03
5th Floor	1	97.45	-0.05
4th Floor	1	148.03	-0.07
3rd Floor	1	224.13	-0.08
2nd Floor	1	298.02	-0.04

Summary - Total Story Shears

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
Roof	46.48	46.48	-0.03	-0.03
5th Floor	97.45	50.97	-0.05	-0.03
4th Floor	148.03	50.58	-0.07	-0.02
3rd Floor	224.13	76.10	-0.08	-0.00
2nd Floor	298.02	73.89	-0.04	0.04

Load Case: E2 ASCE 7-10 Seis EQ_ASCE710_X_-E_F

Level	**Diaph. #**	**Shear-X kips**	**Shear-Y kips**
Roof	1	46.47	-0.00
5th Floor	1	97.42	-0.01
4th Floor	1	147.97	-0.02
3rd Floor	1	224.08	0.02
2nd Floor	1	298.01	0.01

Summary - Total Story Shears

Level	**Shear-X kips**	**Change-X kips**	**Shear-Y kips**	**Change-Y kips**
Roof	46.47	46.47	-0.00	-0.00

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5th Floor	1	0.02	-45.57
4th Floor	1	0.15	-72.43
3rd Floor	1	0.76	-100.93
2nd Floor	1	0.37	-125.77

Summary - Total Story Shears

Level	**Shear-X** kips	**Change-X** kips	**Shear-Y** kips	**Change-Y** kips
Roof	0.07	0.07	-19.31	-19.31
5th Floor	0.02	-0.05	-45.57	-26.26
4th Floor	0.15	0.12	-72.43	-26.86
3rd Floor	0.76	0.61	-100.93	-28.50
2nd Floor	0.37	-0.39	-125.77	-24.84

Load Combination: 1.280 D - 1.000 E4

Level	**Diaph. #**	**Shear-X** kips	**Shear-Y** kips
Roof	1	0.07	-19.29
5th Floor	1	-0.09	-45.52
4th Floor	1	-0.38	-72.31
3rd Floor	1	-0.30	-100.88
2nd Floor	1	-0.30	-125.74

Summary - Total Story Shears

Level	**Shear-X** kips	**Change-X** kips	**Shear-Y** kips	**Change-Y** kips
Roof	0.07	0.07	-19.29	-19.29
5th Floor	-0.09	-0.15	-45.52	-26.22
4th Floor	-0.38	-0.29	-72.31	-26.79
3rd Floor	-0.30	0.08	-100.88	-28.58
2nd Floor	-0.30	-0.00	-125.74	-24.85

Load Combination: 0.820 D + 1.000 E1

Level	**Diaph. #**	**Shear-X** kips	**Shear-Y** kips
Roof	1	20.34	0.04
5th Floor	1	48.83	0.05
4th Floor	1	80.95	-0.27
3rd Floor	1	114.11	0.13
2nd Floor	1	136.33	0.05

Summary - Total Story Shears

Level	**Shear-X** kips	**Change-X** kips	**Shear-Y** kips	**Change-Y** kips
Roof	20.34	20.34	0.04	0.04

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

– ASCE 7-10 Wind Loads –

Directional Procedure, Ch. 27

- Risk Category IV → Fig. 26.5-1B → $V = 120$ mph $I = 1.0$

  Exposure Category C

- Wind Directionality Factor, K_d, Tbl. 26.6-1

  $K_d = 0.85$

- Topographic Factor, K_{zt}, § 26.8.2

  $K_{zt} = 1.0$

- Gust-effect Factor, G, § 26.9.1

  Use $G = 0.85$

- Enclosure Classification, § 26.10

  Enclosed

- Internal Pressure Coefficient, § 26.11

  Tbl. 26.11-1, $GC_{pi} = \pm 0.18$

Because of large surface area difference, separate calculations will be used for large lower floor and smaller upper floors in E-W direction.

Floor Wind Forces

see excel printouts for wind psf values

Lower Floors E/W Direction

Chapter 26 Parameters			ASCE 7-10 Section
Basic Wind Speed, V =	120	mph	*Section 26.5*
Wind Directionality Factor, K_d =	0.85		*Section 26.6*
Topographic Factor, K_{zt} =	1		*Section 26.8*
Gust-effect factor, G =	0.85		*Section 26.9*
Internal Pressure Coefficient, GCpi =	0.18	(+ / -)	*Tbl. 26.11-1*
Exposure Category:	C		

Section 27.4 Parameters

Automatic Cp Calculation			*Calculated Cp values*		
Dimension normal to wind, B:	270	ft.	Windward Wall:	0.8	
Dim. parallel to wind, L:	180	ft.	Leeward Wall:	-0.5	
Mean roof height, h:	75	ft.	Side Wall:	-0.7	
Roof angle from horizontal, θ:	0	degrees	Windward Roof:	-0.90	*0-h/2 only*
			Leeward Roof:	-0.48	

Floor	z (ft)	Kz	qz (psf)
2	15	0.9	28.20
3	30	1.04	32.59
4	45	1.13	35.41
5	60	1.17	36.66
Roof	75	1.24	38.85
		-	-
		-	-
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		-	-

Calculated final pressures, p, for building face listed (psf)				
Windward Wall	Leeward Wall	Side Wall	Windward Roof	Leeward Roof
26.17	-23.51	-30.11	-36.72	-22.96
29.15	-23.51	-30.11	-36.72	-22.96
31.07	-23.51	-30.11	-36.72	-22.96
31.92	-23.51	-30.11	-36.72	-22.96
33.42	-23.51	-30.11	-36.72	-22.96
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
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**Roof Values valid for 0-h/2 or conservative for all roof*

Lower Floors N/S Direction

Chapter 26 Parameters			ASCE 7-10 Section
Basic Wind Speed, V =	120	mph	*Section 26.5*
Wind Directionality Factor, K_d =	0.85		*Section 26.6*
Topographic Factor, K_{zt} =	1		*Section 26.8*
Gust-effect factor, G =	0.85		*Section 26.9*
Internal Pressure Coefficient, GCpi =	0.18	(+ / -)	*Tbl. 26.11-1*
Exposure Category:	C		

Section 27.4 Parameters

Automatic Cp Calculation			*Calculated Cp values*		
Dimension normal to wind, B:	180	ft.	Windward Wall:	0.8	
Dim. parallel to wind, L:	270	ft.	Leeward Wall:	-0.4	
Mean roof height, h:	75	ft.	Side Wall:	-0.7	
Roof angle from horizontal, θ:	0	degrees	Windward Roof:	-0.90	*0-h/2 only*
			Leeward Roof:	-0.46	

Floor	z (ft)	Kz	qz (psf)
2	15	0.9	28.20
3	30	1.04	32.59
4	45	1.13	35.41
5	60	1.17	36.66
Roof	75	1.24	38.85
		-	-
		-	-
		-	-
		-	-
		-	-
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Calculated final pressures, p, for building face listed (psf)				
Windward Wall	Leeward Wall	Side Wall	Windward Roof	Leeward Roof
26.17	-20.20	-30.11	-36.72	-22.04
29.15	-20.20	-30.11	-36.72	-22.04
31.07	-20.20	-30.11	-36.72	-22.04
31.92	-20.20	-30.11	-36.72	-22.04
33.42	-20.20	-30.11	-36.72	-22.04
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
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-	-	-	-	-
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-	-	-	-	-

**Roof Values valid for 0-h/2 or conservative for all roof*

Upper Floors E/W Direction

Chapter 26 Parameters			ASCE 7-10 Section
Basic Wind Speed, V =	120	mph	*Section 26.5*
Wind Directionality Factor, K_d =	0.85		*Section 26.6*
Topographic Factor, K_{zt} =	1		*Section 26.8*
Gust-effect factor, G =	0.85		*Section 26.9*
Internal Pressure Coefficient, GCpi =	0.18	(+ / -)	*Tbl. 26.11-1*
Exposure Category:	C		

Section 27.4 Parameters

Automatic Cp Calculation			*Calculated Cp values*		
Dimension normal to wind, B:	240	ft.	Windward Wall:	0.8	
Dim. parallel to wind, L:	120	ft.	Leeward Wall:	-0.5	
Mean roof height, h:	75	ft.	Side Wall:	-0.7	
Roof angle from horizontal, θ:	0	degrees	Windward Roof:	-1.00	*0-h/2 only*
			Leeward Roof:	-0.55	

Floor	z (ft)	Kz	qz (psf)
2	15	0.9	28.20
3	30	1.04	32.59
4	45	1.13	35.41
5	60	1.17	36.66
Roof	75	1.24	38.85
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
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		-	-
		-	-
		-	-

Calculated final pressures, p, for building face listed (psf)				
Windward Wall	Leeward Wall	Side Wall	Windward Roof	Leeward Roof
26.17	-23.51	-30.11	-40.02	-25.16
29.15	-23.51	-30.11	-40.02	-25.16
31.07	-23.51	-30.11	-40.02	-25.16
31.92	-23.51	-30.11	-40.02	-25.16
33.42	-23.51	-30.11	-40.02	-25.16
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
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-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-

**Roof Values valid for 0-h/2 or conservative for all roof*

Upper Floors N/S Direction

Chapter 26 Parameters			ASCE 7-10 Section
Basic Wind Speed, V =	120	mph	*Section 26.5*
Wind Directionality Factor, K_d =	0.85		*Section 26.6*
Topographic Factor, K_{zt} =	1		*Section 26.8*
Gust-effect factor, G =	0.85		*Section 26.9*
Internal Pressure Coefficient, GCpi =	0.18	(+ / -)	*Tbl. 26.11-1*
Exposure Category:	C		

Section 27.4 Parameters

Automatic Cp Calculation			*Calculated Cp values*		
Dimension normal to wind, B:	120	ft.	Windward Wall:	0.8	
Dim. parallel to wind, L:	240	ft.	Leeward Wall:	-0.3	
Mean roof height, h:	75	ft.	Side Wall:	-0.7	
Roof angle from horizontal, θ:	0	degrees	Windward Roof:	-0.90	*0-h/2 only*
			Leeward Roof:	-0.46	

Floor	z (ft)	Kz	qz (psf)
2	15	0.9	28.20
3	30	1.04	32.59
4	45	1.13	35.41
5	60	1.17	36.66
Roof	75	1.24	38.85
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
		-	-
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		-	-
		-	-
		-	-

Calculated final pressures, p, for building face listed (psf)				
Windward Wall	Leeward Wall	Side Wall	Windward Roof	Leeward Roof
26.17	-16.90	-30.11	-36.72	-22.27
29.15	-16.90	-30.11	-36.72	-22.27
31.07	-16.90	-30.11	-36.72	-22.27
31.92	-16.90	-30.11	-36.72	-22.27
33.42	-16.90	-30.11	-36.72	-22.27
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
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-	-	-	-	-
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-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-

**Roof Values valid for 0-h/2 or conservative for all roof*

Floor Level Wind Forces

- Lower Floors (2-3rd)

2nd Floor

- East/West: $h = 15'$ $w = 270'$ $p = 26.17 + 23.51 = 50 psf$

  $F = 15 \cdot 270 \cdot 50 = \underline{202.5k}$

- North/South: $h = 15'$ $w = 180'$ $p = 26.17 + 20.2 = 46.5 psf$

  $F = 15 \cdot 180 \cdot 46.5 = \underline{125.6k}$

3rd Floor

- East/West: $h = 15'$ $w = 270'$ $p = 29.15 + 23.51 = 52.75 psf$

  $F = 15 \cdot 270 \cdot 52.75 = \underline{213.7k}$

- North/South: $h = 15'$ $w = 180'$ $p = 29.15 + 20.2 = 49.5 psf$

  $F = 15 \cdot 180 \cdot 49.5 = \underline{133.7k}$

- Upper Floors (4-Roof)

4th Floor

- E/W: $h = 15'$ $w = 240'$ $p = 31.07 + 23.51 = 54.6 psf$

  $F = 15 \cdot 240 \cdot 54.6 = \underline{196.7k}$

- N/S: $h = 15'$ $w = 120'$ $p = 31.07 + 16.9 = 48 psf$

  $F = 15 \cdot 120 \cdot 48 = \underline{86.4k}$

5th Floor

- E/W: $h = 15'$ $w = 240'$ $p = 31.92 + 23.51 = 55.5 psf$

  $F = 15 \cdot 240 \cdot 55.5 = \underline{199.8k}$ (say $\underline{\underline{200k}}$)

- N/S: $h = 15'$ $w = 120'$ $p = 31.92 + 16.9 = 49 psf$

  $F = 15 \cdot 120 \cdot 49 = \underline{88.2k}$

Roof $h \approx 10'$

- E/W $h = 10'$ $w = 240'$ $p = 33.42 + 23.51 = 57$

  $F = 10 \cdot 240 \cdot 57 = \underline{136.8k}$

- N/S $h = 10'$ $w = 120'$ $p = 33.42 + 16.9 = 50.5$

  $F = 10 \cdot 120 \cdot 50.5 = \underline{60.6k}$

- Summary of Forces

E/W		N/S	
2 -	202.5k	2 -	125.6k
3 -	213.7k	3 -	133.7k
4 -	196.7k	4 -	86.4k
5 -	200k	5 -	88.2k
Roof -	136.8k	Roof -	60.6k

Base Shear: 949.7k   494.5k

Wind controls over seismic, as expected.

Controlling wall will be E/W direction. It has highest forces and least walls.

Possibility: Wall 5 or 16

- RAM X direction → Building N/S
- " Y " → " E/W

Load Combinations - IBC 2012/ASCE 7-10, use reduced snow in combination with seismic.

– RAM Shear Wall Reinforcing Design Bar Patterns –

- # of curtains - 1 - ACI 318-11 §14.3.4 (< 10" thick)
- Min. bar size - #6
- Max. bar size - #10
- Max. Spacing - §14.3.5

  $S_{max} = 3t$ or 18" - use 18"

- Min. Spacing - 4"
- Spacing increment - 4" (same as min spacing)

– RAM Shear Wall Section Cuts –

- Max spacing of cut, vertical and horizontal - 60 in
- Offset from edge of wall - 6 in

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5th Floor	1	-61.54	135.77
4th Floor	1	-106.04	224.28
3rd Floor	1	-153.07	306.62
2nd Floor	1	-198.65	379.99

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-20.00	-20.00	44.84	44.84
5th Floor	-61.54	-41.54	135.77	90.93
4th Floor	-106.04	-44.49	224.28	88.51
3rd Floor	-153.07	-47.04	306.62	82.34
2nd Floor	-198.65	-45.57	379.99	73.37

Load Combination: 0.900 D + 1.600 W1

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	35.76	0.04
5th Floor	1	110.16	0.06
4th Floor	1	192.30	-0.32
3rd Floor	1	280.93	0.12
2nd Floor	1	358.56	0.04

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	35.76	35.76	0.04	0.04
5th Floor	110.16	74.40	0.06	0.02
4th Floor	192.30	82.14	-0.32	-0.38
3rd Floor	280.93	88.62	0.12	0.44
2nd Floor	358.56	77.63	0.04	-0.08

Load Combination: 0.900 D + 1.600 W2

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.04	79.77
5th Floor	1	-0.26	241.71
4th Floor	1	-1.26	400.04
3rd Floor	1	-2.21	545.24
2nd Floor	1	-1.43	675.73

Summary - Total Story Shears

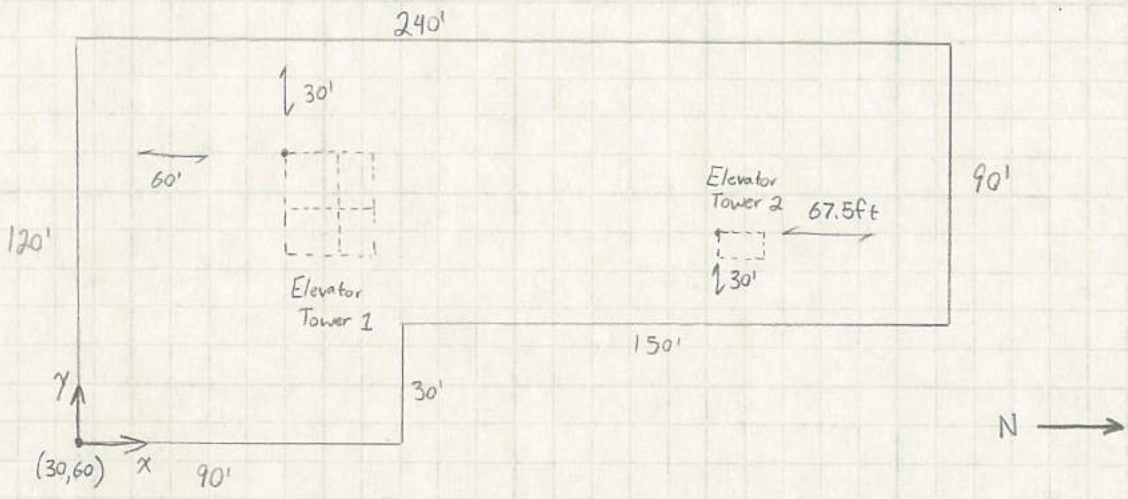
Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.04	0.04	79.77	79.77

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER

COMET

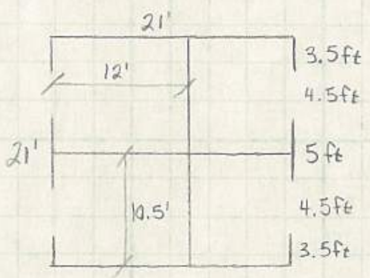
Lateral System Investigation

- Upper floor layout



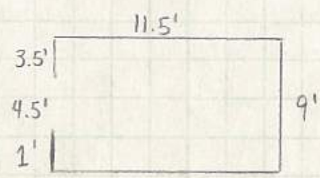
Tower 1

- ~~12" thick walls~~ 8" thick



Tower 2

~~12" thick walls~~ 8" thick



COM of floor

x direction: $\dfrac{150 \cdot 21600 + 75 \cdot 2700}{24300} = 141.67'$

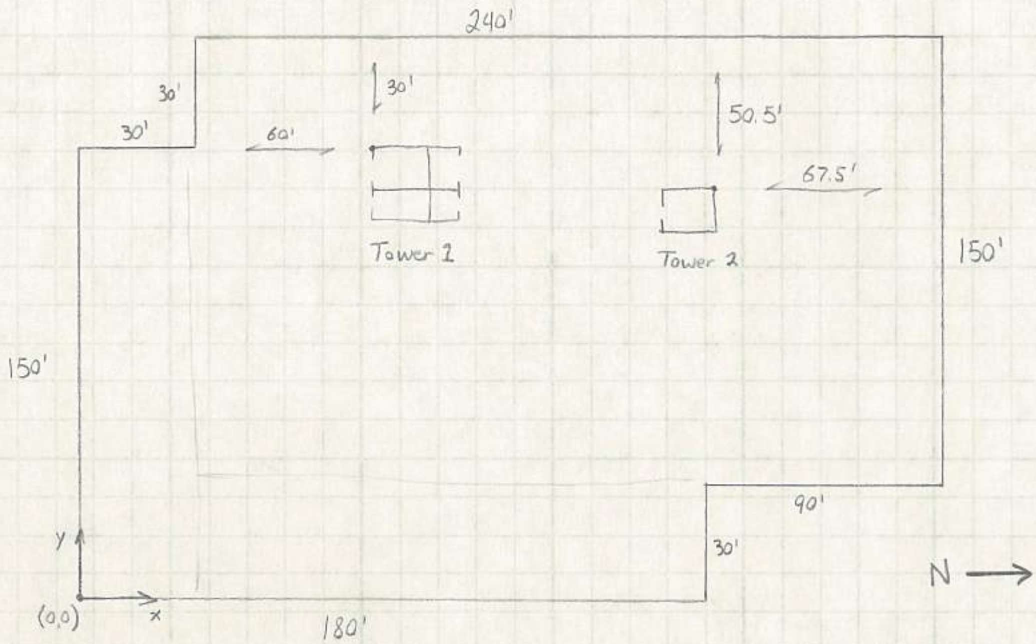
y direction: $\dfrac{135 \cdot 21600 + 75 \cdot 2700}{24300} = 128.33'$

(141.67, 128.33)

verified with RAM ✓

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

• Lower Floor Layout



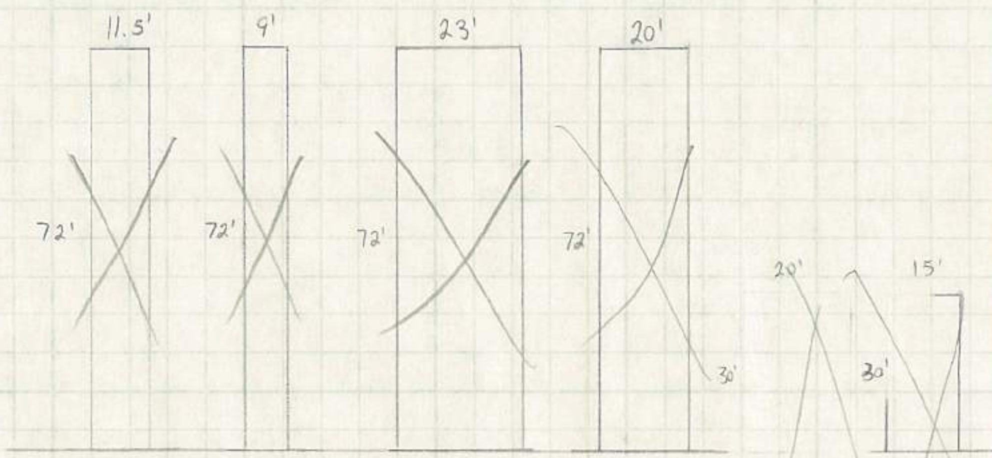
COM of floor

x-direction: $\frac{36{,}000 \cdot 150 + 4500 \cdot 15 + 4500 \cdot 105}{45000} = 132 \text{ ft}$ (132, 93)

y-direction: $\frac{36{,}000 \cdot 105 + 4500 \cdot 75 + 4500 \cdot 15}{45{,}000} = 93 \text{ ft}$ Verified with RAM ✓

Rigidity of elements 1' ~~thick~~ walls

Roof Level:



126.74 ft^4 60.75 ft^4 1013.92 ft^4 666.67 ft^4 666.67 ft^4 281.25 ft^4

$f'c = 6{,}000$ psi $E \approx 57000\sqrt{6000} = 4.415 \text{E}6$ psi $\Delta_{max} = \frac{w\ell^4}{8EI}$ $k = \frac{1}{\Delta} = \frac{8EI}{w\ell^4}$ $w = 1$ klf

0.1666 0.0798 1.3326 0.8762 29.070 12.264

After decreasing size and number of shear walls...

- Existing elements -

$f'c = 4{,}000$ psi $E \approx 57000\sqrt{4000} = 3.605E6$ psi

$3{,}605$ ksi $\times 144\,in^2/ft^2$

$= 519{,}120$ ksf

Wall 5:

$t = 8''$ $l = 20'$ $h = 72'$

$I = \frac{8/12 \cdot 20^3}{12} = 444.44 ft^4$ $(9.21591E6\ in^4)$

Wall 16:

$t = 8''$ $l = 15'$ $h = 72'$

$I = \frac{8/12 \cdot 15^3}{12} = 187.5 ft^4$ $(3.888E6\ in^4)$

Wall 15, 14:

$t = 8''$ $l = 15'$ $h = 30'$

$I = 187.5 ft^4$ $(3.888E6\ in^4)$

N/S → Walls 2, 4, 14

E/W → Walls 5, 15, 16

Walls 2, 4:

$t = 8''$ $l = 23'$ $h = 72'$

$I = \frac{8/12 \cdot 23^3}{12} = 675.94 ft^4$ $(1.40164E7\ in^4)$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARE
3-0137 — 200 SHEETS — FILLER
COMET

Wall 5: (Wall 3 in Report)

$P_u = 260k$ $V_u = 400k$ $M_u = 5859\ k\cdot ft$

$t = 8''$ $\ell = 240''$ $h_w = 864$ in

utilizing spreadsheet –

$f_y = 60$ ksi

SEE EXCEL PRINTOUT

RAM	Excel Hand
Vert	$A_v = 0.44\ in^2$
#9 @ 16"	$\rho_t = 0.00417$
$0.75\ in^2/ft$	$\rho_\ell = 0.00411$
$\rho_\ell = 0.008$	$0.4\ in^2/ft$ req
Horiz	#6 @ 12" horiz $A_v = 0.44\ in^2$
#6 @ 12"	
$A_v = 0.44\ in^2$	#6 @ 12" vert $A_s = 0.44\ in^2$

* ϕM_n higher than necessary

Wall 16: (Wall 5 in Report)

$P_u = 376.7k$ $V_u = 195.62k$ $M_u = 7,163.6k$

$t = 8''$ $\ell = 180''$ $h_w = 864''$

utilizing spreadsheet

SEE EXCEL PRINTOUT

RAM	Excel Hand
Vert:	$A_v = 0.44\ in^2$
#10 @ 12	$\rho_t = 0.00458$
$A_s = 1.27\ in^2/ft$	$\rho_\ell = 0.0115$
$\rho_\ell = 0.013$	
	$1.11\ in^2/ft$ req'd
Horiz:	
	#6 @ 12" horiz $A_v = 0.44\ in^2$
#6 @ 8"	
$A_v = 0.66\ in^2$	#10 @ 12" vert $A_v = 1.27\ in^2/ft$

* ϕV_n higher than necessary

Horizontal Section Cut Forces

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Wall 5 (3 in Report) Forces

WDG	Story	Section	LC ID	P	Vmaj	Vmin	Mmaj	Mmin	T
	2nd Floor	SC5H:25	1	342.18	1.32	0.00	66.34	0.00	-0.00
			2	393.30	-0.41	-0.00	62.52	0.00	0.00
			3	390.58	-0.37	-0.00	60.63	0.00	0.00
			4	332.41	0.52	0.00	64.09	0.00	-0.00
			5	302.01	0.98	0.00	62.91	0.00	-0.00
			6	374.21	4.53	0.00	92.48	-0.01	-0.00
			7	268.80	200.41	0.00	2962.38	0.00	-0.00
			8	268.42	155.27	0.00	2227.35	0.00	0.00
			9	315.81	118.99	0.00	1695.98	-0.00	0.00
			10	229.81	-2.56	-0.00	33.34	0.01	0.00
			11	335.23	-198.44	-0.00	-2836.56	0.00	-0.00
			12	335.61	-153.30	-0.00	-2101.53	-0.00	-0.00
			13	288.22	-117.03	-0.00	-1570.16	0.01	-0.00
			14	470.82	7.71	0.00	119.08	-0.02	-0.00
			15	259.99	399.47	0.00	5858.87	0.00	-0.00
			16	259.23	309.18	0.00	4388.81	0.01	0.00
			17	354.01	236.64	0.00	3326.07	-0.01	0.00
			18	182.02	-6.47	-0.00	0.79	0.02	0.00
			19	392.85	-398.23	-0.00	-5739.00	-0.00	-0.00
			20	393.61	-307.95	-0.00	-4268.95	-0.00	-0.00
			21	298.83	-235.40	-0.00	-3206.21	0.01	-0.00
			22	468.10	7.75	0.00	117.19	-0.02	-0.00
			23	257.27	399.51	0.00	5856.98	0.00	-0.00
			24	256.51	309.23	0.00	4386.92	0.01	0.00
			25	351.29	236.68	0.00	3324.19	-0.01	0.00
			26	179.30	-6.43	-0.00	-1.10	0.02	0.00
			27	390.13	-398.19	-0.00	-5740.89	-0.00	-0.00
			28	390.89	-307.90	-0.00	-4270.84	-0.00	-0.00
			29	296.11	-235.36	-0.00	-3208.10	0.01	-0.00
			30	440.42	8.17	0.00	117.90	-0.02	-0.00
			31	229.59	399.93	0.00	5857.69	0.00	-0.00
			32	228.83	309.65	0.00	4387.63	0.01	0.00
			33	323.61	237.10	0.00	3324.90	-0.01	0.00
			34	151.62	-6.01	-0.00	-0.39	0.02	0.00
			35	362.45	-397.77	-0.00	-5740.18	-0.00	-0.00
			36	363.21	-307.48	-0.00	-4270.12	-0.00	-0.00
			37	268.43	-234.94	-0.00	-3207.39	0.01	-0.00
			38	437.70	8.22	0.00	116.01	-0.02	-0.00
			39	226.87	399.98	0.00	5855.80	0.00	-0.00
			40	226.11	309.69	0.00	4385.75	0.01	0.00
			41	320.89	237.15	0.00	3323.01	-0.01	0.00
			42	148.90	-5.96	-0.00	-2.28	0.02	0.00
			43	359.73	-397.72	-0.00	-5742.07	-0.00	-0.00
			44	360.49	-307.44	-0.00	-4272.01	-0.00	-0.00
			45	265.71	-234.89	-0.00	-3209.27	0.01	-0.00

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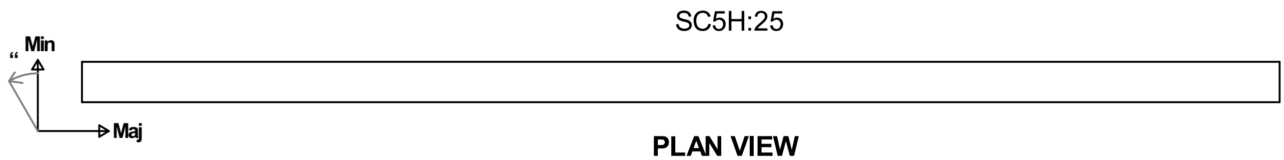
Section Cut ID: SC5H:25 (Horizontal)
Story: 2nd Floor
Ag = 1920 in2 Imaj = 9215999 in4 Imin = 10240 in4
Major Axis Orientation: 90.00 degrees (CCW from global X-axis)
Wall Design Group: 5
Design Status: PASS



Axial/Flexural Results:
Interaction: 0.618 OK
Pu = 153.54 kips phiPn = 248.46 kips
Mu = 5841.6 kip-ft at Beta = 0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D + 1.000 W2 (LC 47)
Code Ref: 10.3.7

Shear Results:
Segment SC5H:25:
Length = 20.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #9@16" oc Horiz Bar Pat: #6@12" oc
Vu = 400.0 kip phiVn = 463.8 kip OK
Controlling Load Combo: 1.200 D + 1.000 W2 (LC 39)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.833% (11.9.9.4) OK
Segment SC5H:25:
Max Vert Bar Spacing Limit: 18.00 in Actual: 16.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.13 in Actual: 14.87 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Wall 5 (3 in Report) Concept Design

Wall 5 (3 in Report) Excel/Hand Calcs

Shear

User Inputs

f'c =	4000	psi
h =	8	in.
lw =	240	in.
d =	192	in.
Vu =	400	k
Nu =	260	k
Mu =	5859	k·ft
h_w =	864	in

Calculations

	320.63	k
Vc = MIN of:	581.03	k
Vc =	320.63	k
φVc =	240.47	k
0.5·φVc =	120.24	k
Vu	>	0.5·φ·Vc

Use ACI 318-11 Sec. 11.9.9

Vs,req =	212.70	k
fy =	60	ksi
Av =	0.4	in^2
s =	21.66	in.
s_{max} =	18.00	in.
Use s =	18.00	in.

Using s from previous column...

Vs =	256.00	k
Vs + Vc =	576.63	k
φVn =	432.47	k
φVn	>	Vu
ρ_t =	0.00417	

Design OK!

Flexure

User Inputs

f'c =	4000	psi
β1 =	0.85	
fy =	60000	psi
h =	8	in.
lw =	240	in.
d =	192	in.
Nu =	260	k
Mu =	5859	k·ft
ρ_l =	0.00411	
$\rho_{l,min}$ =	0.00250	

Calculations

ω =	0.061707	
α =	0.034	
c =	27.11	in
Ast =	7.898465	in^2
T =	420.37	k
φMn =	5859.00	k·ft

Checks

c/d =	0.14121
φ =	0.9

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WDG	Story	Section	LC ID	P	Vmaj	Vmin	Mmaj	Mmin	T
			44	226.57	-35.52	-0.01	-3903.22	0.05	-0.03
			45	226.57	-14.84	-0.01	-3488.85	0.05	-0.03
			46	169.93	-17.94	-0.01	565.72	0.03	-0.02
			47	169.93	58.37	-0.01	3076.83	0.02	-0.02
			48	169.93	45.90	-0.01	3971.11	0.02	-0.01
			49	169.93	25.22	-0.01	3556.73	0.02	-0.01
			50	169.93	26.84	-0.01	-507.53	0.04	-0.02
			51	169.93	-49.47	-0.01	-3018.64	0.04	-0.02
			52	169.93	-37.00	-0.01	-3912.92	0.04	-0.02
			53	169.93	-16.32	-0.01	-3498.55	0.04	-0.02
	2nd Floor	SC16H:21	1	330.85	3.05	-0.01	78.83	-0.03	-0.01
			2	376.70	7.33	-0.01	102.53	-0.06	-0.02
			3	374.11	7.56	-0.01	102.96	-0.06	-0.02
			4	320.15	3.43	-0.01	77.26	-0.04	-0.01
			5	291.86	1.88	-0.01	66.21	-0.03	-0.01
			6	291.86	3.64	-0.01	242.51	-0.04	-0.01
			7	291.86	97.61	-0.01	3261.23	-0.03	-0.01
			8	291.86	95.65	-0.01	3614.45	-0.03	-0.01
			9	291.86	78.87	-0.01	3055.49	-0.04	-0.01
			10	291.86	0.12	-0.01	-110.09	-0.02	-0.01
			11	291.86	-93.85	-0.01	-3128.82	-0.03	-0.01
Wall 16 (5 in Report) Forces			12	291.86	-91.89	-0.01	-3482.04	-0.02	-0.01
			13	291.86	-75.11	-0.01	-2923.07	-0.02	-0.01
			14	314.46	7.46	-0.01	430.80	-0.06	-0.01
			15	314.47	195.39	-0.01	6468.25	-0.05	-0.01
			16	314.47	191.47	-0.01	7174.70	-0.05	-0.01
			17	314.47	157.92	-0.01	6056.76	-0.06	-0.01
			18	314.46	0.41	-0.01	-274.40	-0.02	-0.01
			19	314.46	-187.52	-0.01	-6311.85	-0.03	-0.02
			20	314.46	-183.60	-0.01	-7018.29	-0.03	-0.02
			21	314.46	-150.05	-0.01	-5900.36	-0.02	-0.02
			22	311.88	7.69	-0.01	431.23	-0.06	-0.01
			23	311.88	195.62	-0.01	6468.68	-0.05	-0.01
			24	311.88	191.70	-0.01	7175.12	-0.05	-0.01
			25	311.88	158.15	-0.01	6057.19	-0.06	-0.01
			26	311.88	0.64	-0.01	-273.97	-0.02	-0.01
			27	311.87	-187.29	-0.01	-6311.42	-0.03	-0.02
			28	311.87	-183.37	-0.01	-7017.87	-0.03	-0.02
			29	311.87	-149.82	-0.01	-5899.93	-0.02	-0.02
			30	286.18	5.91	-0.01	419.74	-0.05	-0.01
			31	286.18	193.84	-0.01	6457.19	-0.04	-0.01
			32	286.18	189.92	-0.01	7163.64	-0.04	-0.01
			33	286.18	156.37	-0.01	6045.70	-0.05	-0.01
			34	286.18	-1.14	-0.01	-285.45	-0.01	-0.01
			35	286.17	-189.07	-0.01	-6322.91	-0.02	-0.01

Section Cut Design Summary

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3
Design Code: ACI 318-11
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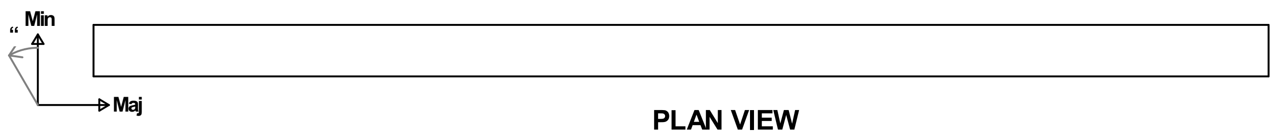
03/31/16 14:33:54

Section Cut ID: SC16H:21 (Horizontal)
Story: 2nd Floor
Ag = 1440 in2 Imaj = 3887999 in4 Imin = 7680 in4
Major Axis Orientation: 90.00 degrees (CCW from global X-axis)
Wall Design Group: 16
Design Status: PASS

SC16H:21



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.907 OK
Pu = 212.70 kips phiPn = 234.40 kips
Mu = 7147.2 kip-ft at Beta = -0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D + 1.000 W5 (LC 48)
Code Ref: 10.3.7

Shear Results:
Segment SC16H:21:
Length = 15.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #10@12" oc Horiz Bar Pat: #6@8" oc
Vu = 195.6 kip phiVn = 467.1 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.000 W2 (LC 23)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 1.408% (11.9.9.4) OK
Segment SC16H:21:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.27 in Actual: 10.73 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Wall 16 (5 in Report) RAM Design

Wall 16 (5 in Report) Excel/Hand Calcs

Shear

User Inputs

f'c =	4000	psi
h =	8	in.
lw =	180	in.
d =	144	in.
Vu =	195.62	k
Nu =	370	k
Mu =	7164	k·ft
h_w =	864	in

Calculations

	240.51	k
Vc = MIN of:	90.66	k
Vc =	90.66	k
ϕVc =	67.99	k
0.5·ϕVc =	34.00	k
Vu	>	**0.5·ϕ·Vc**

Use ACI 318-11 Sec. 11.9.9

Vs,req =	170.17	k
fy =	60	ksi
Av =	0.4	in^2
s =	20.31	in.
s_{max} =	18.00	in.
Use s =	18.00	in.

Using s from previous column...

Vs =	192.00	k
Vs + Vc =	282.66	k
ϕVn =	211.99	k
ϕVn	>	Vu
ρ_t =	0.00417	

Design OK!

Flexure

User Inputs

f'c =	4000	psi
β1 =	0.85	
fy =	60000	psi
h =	8	in.
lw =	180	in.
d =	144	in.
Nu =	370	k
Mu =	7164	k·ft
ρ_l =	0.01150	
$\rho_{l,min}$ =	0.00250	

Calculations

ω =	0.172536	
α =	0.064	
c =	39.92	in
Ast =	16.56343	in^2
T =	773.39	k
ϕMn =	7164.00	k·ft

Checks

c/d =	0.277232
ϕ =	0.9

Wall Panel Reinforcing

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3 03/31/16 14:33:54
Design Code: ACI 318-11
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Wall Design Group: **15**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
3rd Floor	62	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
2nd Floor	63	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%

Wall Design Group: **2**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
Roof	6	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
5th Floor	7	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
4th Floor	8	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
3rd Floor	9	(1) #6@12" oc	0.46%	(1) #5@16" oc	0.24%
2nd Floor	10	(1) #7@16" oc	0.47%	(1) #5@16" oc	0.24%

Wall Design Group: **16**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
Roof	64	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
5th Floor	65	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
4th Floor	66	(1) #8@16" oc	0.61%	(1) #5@16" oc	0.24%
3rd Floor	67	(1) #8@16" oc	0.61%	(1) #5@16" oc	0.24%
2nd Floor	68	(1) #10@12" oc	1.32%	(1) #6@8" oc	0.69%

Wall Design Group: **4**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
Roof	16	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
5th Floor	17	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
4th Floor	18	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
3rd Floor	19	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
2nd Floor	20	(1) #6@12" oc	0.46%	(1) #5@16" oc	0.24%

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Wall Design Group: **5**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
Roof	21	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
5th Floor	22	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
4th Floor	23	(1) #6@16" oc	0.35%	(1) #5@16" oc	0.24%
3rd Floor	24	(1) #8@16" oc	0.61%	(1) #4@4" oc	0.61%
2nd Floor	25	(1) #9@16" oc	0.78%	(1) #6@12" oc	0.46%

Wall Design Group: **14**
Bar Pattern Template: ACI 318-11 Reinforcing

Bar Patterns

Story	Wall Panel	Vertical	rho	Horizontal	rho
3rd Floor	60	(1) #7@16" oc	0.47%	(1) #5@16" oc	0.24%
2nd Floor	61	(1) #6@8" oc	0.69%	(1) #5@16" oc	0.24%

Section Cut Design Summary

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3 03/31/16 14:33:54
Design Code: ACI 318-11
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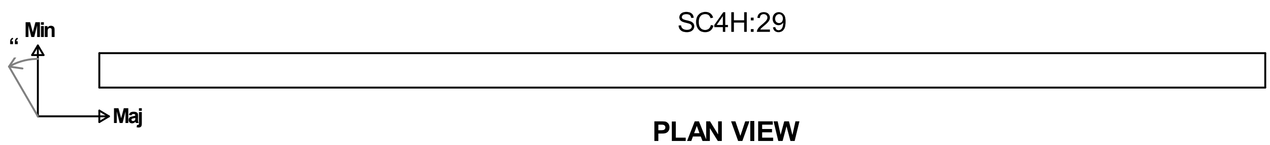
Section Cut ID: SC4H:29 (Horizontal)
Story: 2nd Floor
Ag = 2208 in2 Imaj = 14016383 in4 Imin = 11776 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 4
Design Status: PASS



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.854 OK
Pu = -426.88 kips phiPn = -500.13 kips
Mu = 700.5 kip-ft at Beta = -0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D + 1.000 W2 (LC 47)
Code Ref: 10.3.7

Shear Results:
Segment SC4H:29:
Length = 23.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #6@12" oc Horiz Bar Pat: #5@16" oc
Vu = 159.3 kip phiVn = 358.1 kip OK
Controlling Load Combo: 1.200 D + 0.500 Sp - 1.000 W1 (LC 34)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.480% (11.9.9.4) OK
Segment SC4H:29:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.25 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Section Cut Design Summary

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3
Design Code: ACI 318-11

03/31/16 14:33:54

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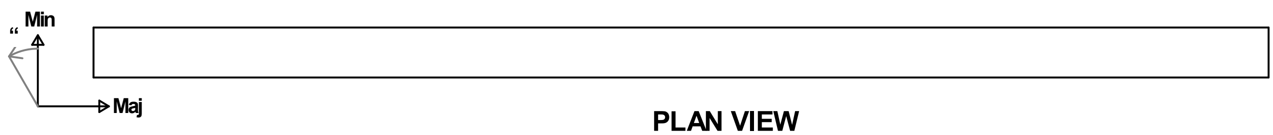
Section Cut ID: SC14H:6 (Horizontal)
Story: 2nd Floor
Ag = 1440 in2 Imaj = 3887999 in4 Imin = 7680 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 14
Design Status: PASS

SC14H:6



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.993 OK
Pu = 72.41 kips phiPn = 72.94 kips
Mu = 4237.4 kip-ft at Beta = 0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D - 1.000 W10 (LC 53)
Code Ref: 10.3.7

Shear Results:
Segment SC14H:6:
Length = 15.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #6@8" oc Horiz Bar Pat: #5@16" oc
Vu = 119.1 kip phiVn = 233.5 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.000 W10 (LC 25)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.736% (11.9.9.4) OK
Segment SC14H:6:
Max Vert Bar Spacing Limit: 18.00 in Actual: 8.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 7.25 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Section Cut Design Summary

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3
Design Code: ACI 318-11
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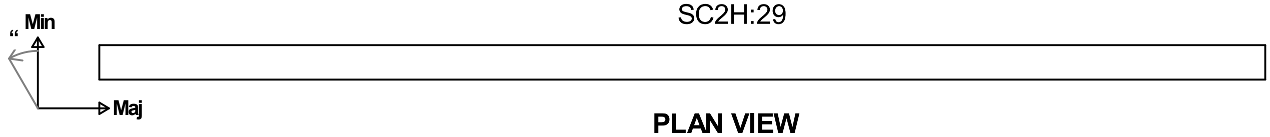
03/31/16 14:33:54

Section Cut ID: SC2H:29 (Horizontal)
Story: 2nd Floor
Ag = 2208 in2 Imaj = 14016383 in4 Imin = 11776 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 2
Design Status: PASS

SC2H:29



PLAN VIEW

Axial/Flexural Results:

Interaction:	0.962	OK	
Pu =	-476.82 kips	phiPn =	-495.52 kips
Mu =	1324.1 kip-ft	at Beta =	-0.0 deg CCW from Major axis
Controlling Load Combo:	0.900 D - 1.000 W2 (LC 51)		
Code Ref:	10.3.7		

Shear Results:

Segment SC2H:29:
Length = 23.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #7@16" oc Horiz Bar Pat: #5@16" oc
Vu = 159.7 kip phiVn = 358.1 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.000 W1 (LC 22)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:

Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.517% (11.9.9.4) OK
Segment SC2H:29:
Max Vert Bar Spacing Limit: 18.00 in Actual: 16.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 15.13 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Section Cut Design Summary

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3
Design Code: ACI 318-11

03/31/16 14:33:54

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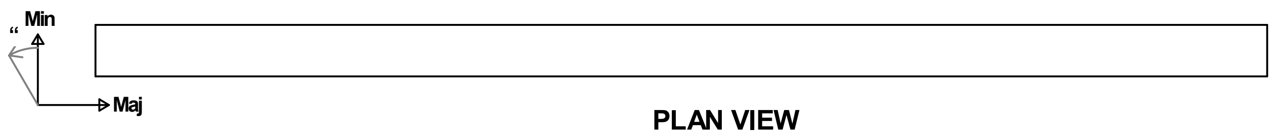
Section Cut ID: SC15H:6 (Horizontal)
Story: 2nd Floor
Ag = 1440 in2 Imaj = 3887999 in4 Imin = 7680 in4
Major Axis Orientation: 90.00 degrees (CCW from global X-axis)
Wall Design Group: 15
Design Status: PASS

SC15H:6



PLAN VIEW

Axial/Flexural Results:

Interaction:	0.362	OK	
Pu =	89.73 kips	phiPn =	247.64 kips
Mu =	1348.6 kip-ft	at Beta =	0.0 deg CCW from Major axis
Controlling Load Combo:	0.900 D + 1.000 W2 (LC 47)		
Code Ref:	10.3.7		

Shear Results:

Segment SC15H:6:
Length = 15.00 ft Thick = 8.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #6@16" oc Horiz Bar Pat: #5@16" oc

Vu =	100.5 kip	phiVn =	233.5 kip	OK
Controlling Load Combo:	1.200 D + 0.500 Lp + 1.000 W2 (LC 23)			
Code Ref:	14.2.3 & 11.9.5			

Reinforcement Checks:

Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.399% (11.9.9.4) OK
Segment SC15H:6:
Max Vert Bar Spacing Limit: 18.00 in Actual: 16.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 15.25 in (7.6.1) OK
Min Number of Reinf Curtains: 1 Actual: 1 (14.3.4) OK

Center of Rigidity

RAM Frame 14.07.01.01
DataBase: Less Walls Lateral V3 03/31/16 13:07:01
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CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors

			Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Type**	**Xr**	**Yr**	**Xm**	**Ym**
			ft	**ft**	**ft**	**ft**
Roof	1	Rigid	113.72	132.87	142.07	127.88
5th Floor	1	Rigid	113.51	130.65	142.32	127.80
4th Floor	1	Rigid	114.08	127.00	142.32	127.80
3rd Floor	1	Rigid	113.01	120.38	132.35	93.78
2nd Floor	1	Rigid	117.81	115.23	131.94	92.87

			Story Lateral Stiffness	
Level	**Diaph. #**	**Type**	**KX**	**KY**
			kips/ ft	**kips/ ft**
Roof	1	Rigid	7411.80	10779.58
5th Floor	1	Rigid	11328.72	15722.62
4th Floor	1	Rigid	18741.64	22284.59
3rd Floor	1	Rigid	34801.40	36868.31
2nd Floor	1	Rigid	105796.71	87643.56

NOTES:
Center of rigidity (CR) values given above are only used for load cases that require explicit calculation of CRs for use in calculation of load eccentricities (for example, ASCE 7-05 Wind Load Case).

Note that this information is never used for analysis. On the other hand, it should be noted that analysis results always include any torsional effects due to having center of rigidity and mass center at different locations. In other words, the analysis always accounts for locations and stiffnesses of frame members and diaphragms. Hence, any torsional effects of the masses being offset from the stiffnesses (i.e., CR) are implicitly and correctly accounted in the analysis.

The reported story stiffness is the inverse of the interstory drift that is calculated according to a unit load applied at the story.

Story Displacements

RAM Frame 14.07.01.01
DataBase: Less Walls Lateral V3 03/31/16 13:07:01
Building Code: IBC
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CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	ASCE 7-10 Wind	Wind_ASCE710_1_X
W2	ASCE 7-10 Wind	Wind_ASCE710_1_Y
W3	ASCE 7-10 Wind	Wind_ASCE710_2_X+E
W4	ASCE 7-10 Wind	Wind_ASCE710_2_X-E
W5	ASCE 7-10 Wind	Wind_ASCE710_2_Y+E
W6	ASCE 7-10 Wind	Wind_ASCE710_2_Y-E
W7	ASCE 7-10 Wind	Wind_ASCE710_3_X+Y
W8	ASCE 7-10 Wind	Wind_ASCE710_3_X-Y
W9	ASCE 7-10 Wind	Wind_ASCE710_4_X+Y_CW
W10	ASCE 7-10 Wind	Wind_ASCE710_4_X+Y_CCW
W11	ASCE 7-10 Wind	Wind_ASCE710_4_X-Y_CW
W12	ASCE 7-10 Wind	Wind_ASCE710_4_X-Y_CCW
E1	ASCE 7-10 Seis	EQ_ASCE710_X_+E_F
E2	ASCE 7-10 Seis	EQ_ASCE710_X_-E_F
E3	ASCE 7-10 Seis	EQ_ASCE710_Y_+E_F
E4	ASCE 7-10 Seis	EQ_ASCE710_Y_-E_F

Level: Roof, Diaph: 1

Center of Mass (ft): (142.07, 127.88)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.01882	0.01171	0.00002
Lp	-0.01034	0.00351	0.00001
Sp	-0.00123	0.00011	0.00000
W1	0.88990	0.05979	0.00017
W2	0.03484	1.30629	0.00106
W3	0.66569	-0.01504	-0.00004
W4	0.66916	0.10473	0.00029

CONSTRUCTION MANAGEMENT BREADTH

- Construction Timeline

Assume May 1, 2016 as start date

– Footings

Total C.Y. = 1684 CY

w/ (1) C-14C crew, @ 75 C.Y. / day

23 days

Use (2) C-14C crews to decrease time to 12 days

	with weekends		w/o weekends	
start:	5/1/16	+12 days	5/2/16	+12 weekdays
end:	5/13/16		5/17/16	

– Floor 1 columns

Assuming columns can begin 1 week after footings started

Total C.Y. = 146 CY

w (1) C-14A crew @ 23 CY/day

7 days (increase as necessary to give adequate curing time of footing)

	with weekends		w/o weekends	
start:	5/7/16	+7 +4(curing)	5/9/16	+7 +3 (to finish week)
end:	5/18/16		5/20/16	

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

- Floor 2 elevated slab

Total S.F. = 45,000 ft^2 Total CY = 1,556 CY

Pour in 4 sections, 11,250 ft^2 / pour

w (1) C-14B crew @ 51 CY / day

8 days / pour (seems high)

From Buildings.com article - "on 2 day cycle, can pour up to 20,000 ft^2 every 2 days"

Use 5 days / floor for schedule (lower floors)

* Floor 2 columns & Floor 3 elevated slab will require the same time period as Floor 1 columns & Floor 2 elevated slab.

- Upper Floor Columns

40 columns ∴ $\frac{66}{40}$ (7 days) = 4.25 ⇒ 5 days

- Upper Floor Elevated Slabs

24,300 ft^2 w 20,000/2day use 3 days

RS Means #	Description	Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equip.	Total	Total + O&P
03 30 53.40 - Cast-In-Place Concrete (includes forms, rebar, concrete, placing and finishing)										
900	24”x24” columns	C14A	23.66	8.453	C.Y.	241	400	32	673	915
1950	Elevated Flat Slab w/Drops 30’ span	C-14B	50.99	4.079	C.Y.	276	192	14.75	482.75	615
05 12 23.77 - Structural Steel Project (3-6 story hospital, bolted cnxns)										
800	Hospital 3-6 stories	E-6	14.4	8.889	Ton	2700	465	128	3,293	3,900
5700	3” deep 22ga floor decking	E-4	3200	0.01	S.F.	2.1	0.53	0.05	2.68	3.28
03 31 13.70 - Placing Concrete										
1400	Elevated slab less than 6” thick, pumped	C-20	140	0.457	C.Y.		18.45	5.6	24.05	34
350	Power screed, bull float, machine float and trowel, ride on	C-10E	4000	0.006	S.F.		0.26	0.06	0.32	0.45
2400	Footings under 1 C.Y. Direct Chute	C-6	55	0.873	C.Y.		34	1.13	35.13	53.5
2600	Footings over 5 C.Y. Direct Chute	C-6	120	0.4	C.Y.		15.65	0.52	16.17	24.5
03 30 53.40 Misc. Cast-In-Place Concrete										
3800	Footing < 1 CY	C-14C	28	4	C.Y.	166	180	1.12	347.12	405
3825	Footing 1-5 CY	C-14C	43	2.605	C.Y.	201	117	0.73	318.73	310
3850	Footing > 5 CY	C-14C	75	1.493	C.Y.	185	67.5	0.42	252.92	300
03 31 13.70 - Placing Concrete										
4950	8" thick pumped walls	C-20	100	0.64	C.Y.	107	26	7.85	140.85	166
03 21 11.60 - Reinforcing in Place										
700	Walls, #3-#7	4 Rodman	2.1	15.238	Ton	970	800		1770	2325
03 11 13.85 - Forms In Place Walls										
9460	Steel Framed Plywood	C-2	400	0.12	SFCA	0.74	5.5		6.24	9.25

Concrete Column Takeoff

RAM Concrete Column v14.07.01.01
Database: WSH Concrete V3 Two Way With Drop Panels — 02/16/16 19:30:32
Building Code: IBC — Concrete Code: ACI 318-11

Academic License. Not For Commercial Use.

Longitudinal Reinforcement

Grade fy (ksi): 60.00

Size	Quantity	Length (ft)	Weight (lb)
#7	3024	47721.88	97582
	3024	47721.88	97582

Transverse Reinforcement

Grade fy (ksi): 60.00

Size	Shape	Quantity	Length (ft)	Weight (lb)
#3	20.63x20.63	3786	26028.75	9758
		3786	26028.75	9758

Concrete

Strength f'c (ksi): 6.50 Unit Weight (pcf): 145.00

Size	Quantity	Length (ft)	Volume (yds³)	Weight (lb)
24"x24"	252	3660.00	542.22	2122800
	252	3660.00	542.22	2122800

Building Statistics - All Stories

Total Reinforcement Weight (lb): 107339
Total Column Lengths (ft): 3660 Total Floor Surface Area (ft²): 162627
Total Reinforcement Weight In Columns / Total Column Lengths (lb/ft): 29.33
Total Reinforcement Weight In Columns / Total Floor Area (lb/ft²): 0.66

Concrete Shearwall Takeoff

RAM Concrete Shearwall 14.07.01.01
Database: Less Walls Lateral V3 03/31/16 14:33:54
Design Code: ACI 318-11
Academic License. Not For Commercial Use.

Concrete

Conc Strength (psi)	Unit Weight (pcf)	Volume (yds3)	Weight (lbs)
4000	150	166.22	673200
		166.22	673200

Vertical Reinforcing

Size	Quantity	Length (ft)	Weight (lbs)
#6	305	4348.9	6538
#7	32	476.0	974
#8	42	611.8	1635
#9	16	238.0	809
#10	16	238.0	1026
	411	5912.6	10982

Horizontal Reinforcing

Size	Quantity	Length (ft)	Weight (lbs)
#4	46	914.2	611
#5	261	5054.4	5277
#6	40	675.0	1015
	347	6643.6	6902

Confined Reinforcing

Size	Quantity	Length (ft)	Weight (lbs)
	0	0.0	0

Estimate

Concrete Costs

Materials:	100 per yd³	x	838.8 yd³	=	83880
Labor:	50 per yd³	x	838.8 yd³	=	41940
Total:	150 per yd³	x	838.8 yd³	=	125800

Post-Tensioning Costs

Materials:	1 per pounds	x	0 pounds	=	0
Labor:	0.5 per pounds	x	0 pounds	=	0
Total:	1.5 per pounds	x	0 pounds	=	0

Formwork Costs

Materials:	1 per ft²	x	24540 ft²	=	24540
Labor:	1 per ft²	x	24540 ft²	=	24540
Total:	2 per ft²	x	24540 ft²	=	49080

Mild Steel Reinforcing Costs

Materials:	1000 per tons	x	56.53 tons	=	56530
Labor:	500 per tons	x	56.53 tons	=	28270
Total:	1500 per tons	x	56.53 tons	=	84800

SSR Costs

Materials:	2 per stud	x	0 studs	=	0
Labor:	1 per stud	x	0 studs	=	0
Total:	3 per stud	x	0 studs	=	0

Total Costs

Materials:	6.721 per ft²	x	24540 ft²	=	164900
Labor:	3.861 per ft²	x	24540 ft²	=	94750
Total:	10.58 per ft²	x	24540 ft²	=	259700

For Academic Use only

Concrete Cost Estimate

Floor	24"x24" Columns	Column Volume (CY)	Floor Surface Area (sf)	Floor Concrete Volume (CY)
2	66	145.2	45000	1556
3	66	145.2	45000	1556
4	40	88	24300	840
5	40	88	24300	840
Roof	40	88	24300	840
Total:	252		162900	

2.2 CY/column

Floor	Days to Build Columns (1 crew)	Columns Cost	Days to Build Floor (1 crew)	Floor Cost
2	6.31	$ 132,858.00	30.50980392	$ 956,940.00
3	6.31	$ 132,858.00	30.50980392	$ 956,940.00
4	3.83	$ 80,520.00	16.47058824	$ 516,600.00
5	3.83	$ 80,520.00	16.47058824	$ 516,600.00
Roof	3.83	$ 80,520.00	16.47058824	$ 516,600.00

Foundations	Volume (CY)	Quantity	Cost/Footing	Total Cost
F30	-	0	-	-
F60	-	0	-	-
F66	-	0	-	-
F76	10.4	12	$ 3,120.00	$ 37,440.00
F96	20.52	12	$ 6,156.00	$ 73,872.00
F110	31.25	42	$ 9,375.00	$ 393,750.00
			Total:	$ 505,062.00

	Volume (CY)	Contact Area (SFCA)	Rebar (Ton)	Placing (per C.Y.)	Forming (per SFCA)	Rebar Placement(Ton)
Shear Walls	166.2	13500	9	$ 166.00	$ 9.25	$ 2,325.00
			Subtotal:	$ 27,589.20	$ 124,875.00	$ 20,925.00
				Total:	$ 173,389.20	

Total Cost:	$	4,649,407.20
Cost/SF:	$	28.54

Compare to $19.20/S.F. from RS Means S.F. Estimate

Steel Cost Estimate

Total Steel Weight (lbs)	Tons	Cost/Ton	Total
989896	495	3,900	$ 1,930,500.00

Floor	Floor Surface Area (sf)	Decking	Finishing	Concrete Volume	Concrete Placing
2	45000	$147,600.00	$ 20,250.00	17820.00	$ 605,880.00
3	45000	$147,600.00	$ 20,250.00	17820.00	$ 605,880.00
4	24300	$ 79,704.00	$ 10,935.00	9622.80	$ 327,175.20
5	24300	$ 79,704.00	$ 10,935.00	9622.80	$ 327,175.20
Roof	24300	$ 79,704.00	$ 10,935.00	9622.80	$ 327,175.20
	Totals:	$534,312.00	$ 73,305.00		$ 2,193,285.60

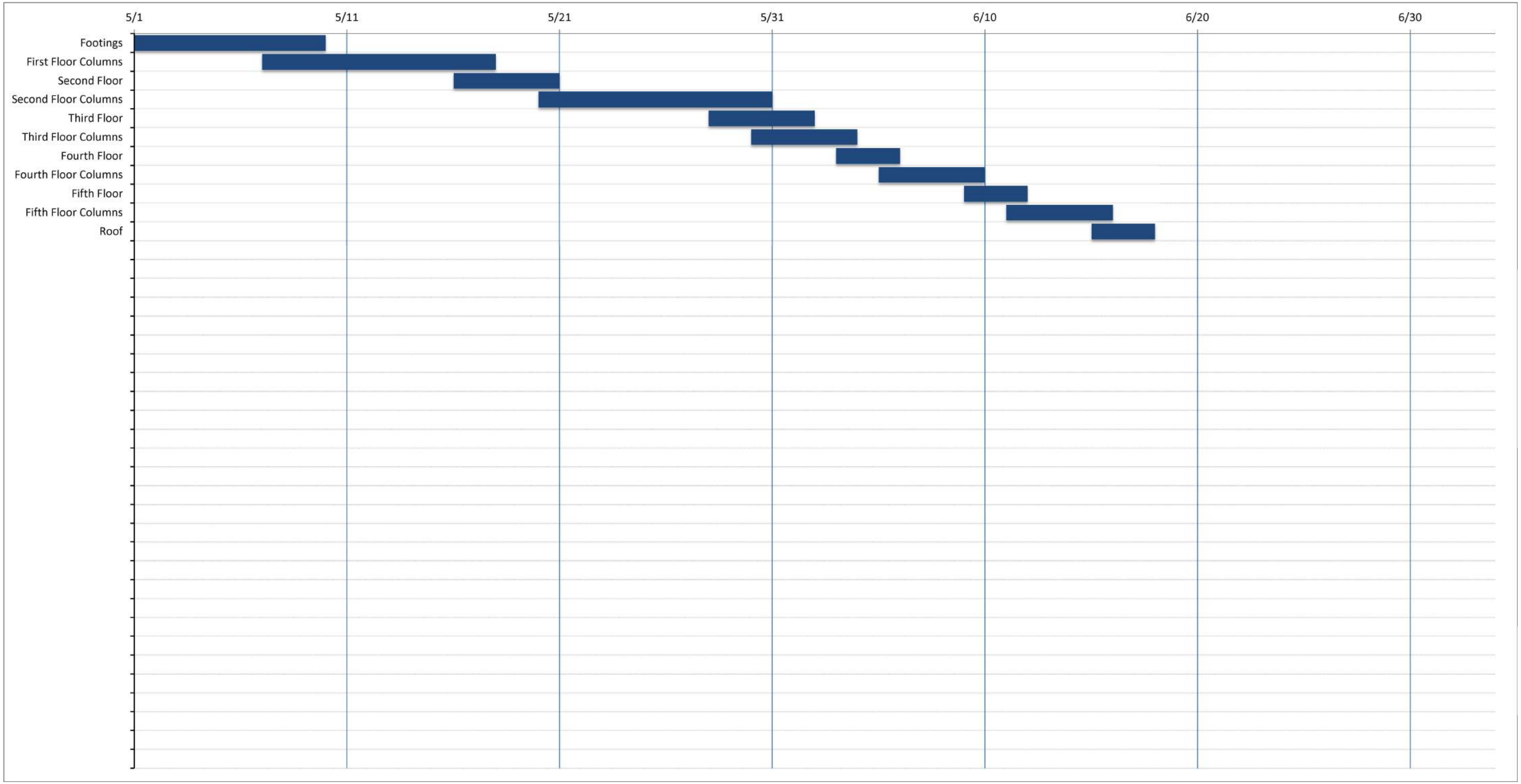
Foundations	Volume (CY)	Quantity	Cost/Footing	Total Cost
F30	0.39	0	$ 157.95	$ -
F60	2.22	0	$ 688.20	$ -
F66	2.67	0	$ 827.70	$ -
F76	4.514	12	$ 1,399.34	$ 16,792.08
F96	8.92	12	$ 2,676.00	$ 32,112.00
F110	13.44	42	$ 4,032.00	$ 169,344.00
			Total:	$ 218,248.08

Structure Total: $ 4,949,650.68
Cost/SF: $ 30.38

Compare to $33.20/S.F. from RS Means S.F. Estimate

Gantt Chart

Task Name	Start	End	Duration (days)
Footings	5/1/2016	5/10/2016	9
First Floor Columns	5/7/2016	5/18/2016	11
Second Floor	5/16/2016	5/21/2016	5
Second Floor Columns	5/20/2016	5/31/2016	11
Third Floor	5/28/2016	6/2/2016	5
Third Floor Columns	5/30/2016	6/4/2016	5
Fourth Floor	6/3/2016	6/6/2016	3
Fourth Floor Columns	6/5/2016	6/10/2016	5
Fifth Floor	6/9/2016	6/12/2016	3
Fifth Floor Columns	6/11/2016	6/16/2016	5
Roof	6/15/2016	6/18/2016	3
			0
			0
			0


5/1
5/11
5/21
5/31
6/10
6/20
6/30
Footings
First Floor Columns
Second Floor
Second Floor Columns
Third Floor
Third Floor Columns
Fourth Floor
Fourth Floor Columns
Fifth Floor
Fifth Floor Columns
Roof

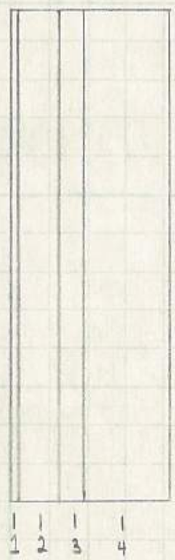
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

ENCLOSURES BREADTH

• 2 types of panels: 1) concrete precast 2) aluminum pre-built

1- Precast Concrete

Interior



1: 5/8" GWB

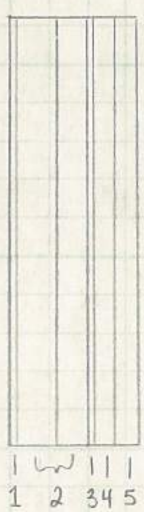
2: 3 5/8" metal stud framing

3: 3" spray polyurethane foam

4: 8" architectural precast concrete

2- Aluminum

Interior



1: 5/8" GWB

2: 6" metal stud w/ 3" spray EPS foam

3: 5/8" glass-mat gypsum wall sheathing

4: 2" mineral wool insulation w/ 2" metal Z-girts between

5: Aluminum panel w/ fluid applied or self-adhesive air barrier

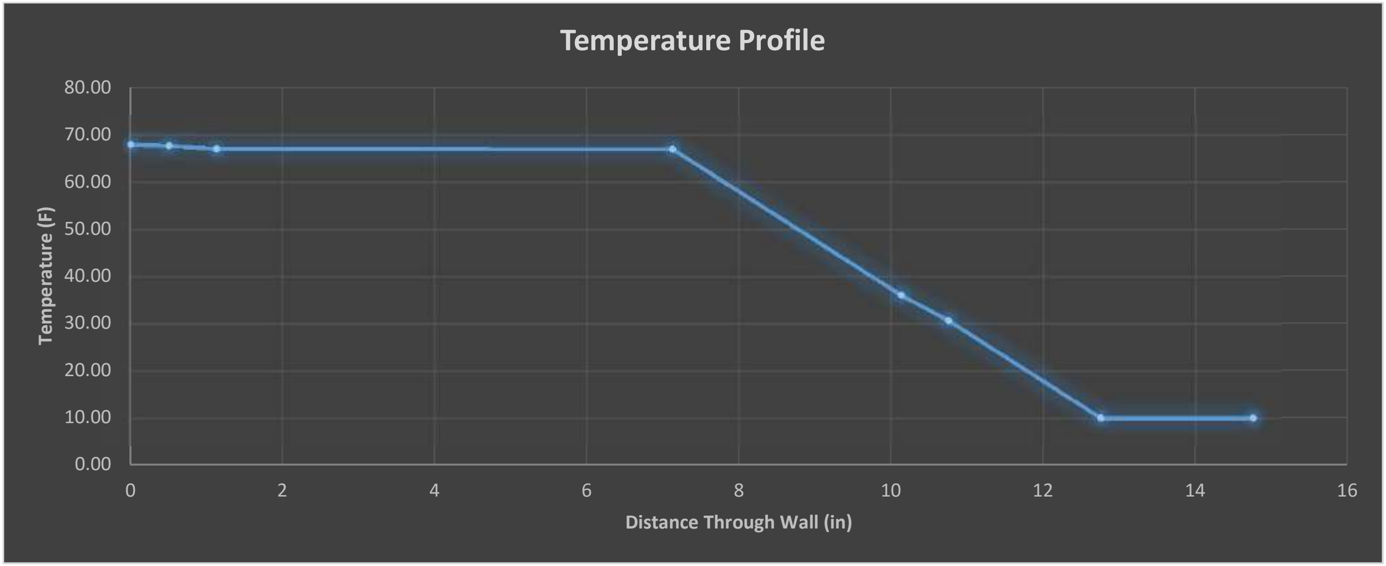
Virtualmaths
www.virtualmaths.org

TYPICAL THERMAL CONDUCTIVITY OF BUILDING MATERIALS: STRUCTURAL AND FINISHING MATERIALS (Always check manufacturer's details – variation will occur depending on product and nature of materials).	THERMAL CONDUCTIVITY (W/mK)
Acoustic plasterboard	0.25
Aerated concrete slab (500kg/m3)	0.16
Aluminium	237
Asphalt (1700kg/m3)	0.50
Bitumen-impregnated fibreboard	0.05
Brickwork (outer leaf 1700kg/m3)	0.84
Brickwork (inner leaf 1700kg/m3)	0.62
Dense aggregate concrete block 1800 kg/m3 (exposed)	1.21
Dense aggregate concrete block 1800 kg/m3 (protected)	1.13
Calcium silicate board (600 kg/m3)	0.17
Concrete general	1.28
Cast concrete (heavyweight 2300 kg/m3)	1.63
Cast concrete (dense 2100 kg/m3 typical floor)	1.40
Cast concrete (dense 2000 kg/m3 typical floor)	1.13
Cast concrete (medium 1400 kg/m3)	0.51
Cast concrete (lightweight 1200 kg/m3)	0.38
Cast concrete (lightweight 600 kg/m3)	0.19
Concrete slab (aerated 500kg/m3)	0.16
Copper	390
External render sand/cement finish	1.00
External render (1300 kg/m3)	0.50
Felt - Bitumen layers (1700kg/m3)	0.50
Fibreboard (300 kg/m3)	0.06
Glass	0.93
Marble	3
Metal tray used in wriggly tin concrete floors (7800 kg/m3)	50.00
Mortar (1750 kg/m3)	0.80
Oriented strand board	0.13
Outer leaf brick	0.77
Plasterboard	0.21
Plaster dense (1300 kg/m3)	0.50
Plaster lightweight (600 kg/m3)	0.16
Plywood (950 kg/m3)	0.16
Prefabricated timber wall panels (check manufacturer)	0.12
Screed (1200kg/m3)	0.41
Stone chippings (1800 kg/m3)	0.96
Tile hanging (1900 kg/m3)	0.84
Timber (650 kg/m3)	0.14
Timber flooring (650 kg/m3)	0.14
Timber rafters	0.13
Timber roof or floor joists	0.13
Roof tile (1900kg/m3)	0.84
Timber blocks (650 kg/m3)	0.14
Web of I stud timber	0.15
Wood wool slab (500kg/m3)	0.10
Cellular glass	0.038-0.050
Expanded polystyrene	0.030-0.038
Expanded polystyrene slab (25 kg/m3)	0.035
Extruded polystyrene	0.029-0.039
Glass mineral wool	0.031-0.044
Mineral quilt (12 kg/m3)	0.040
Mineral wool slab (25 kg/m3)	0.035
Phenolic foam	0.021-0.024
Polyisocyanurate	0.022-0.028
Polyurethane	0.022 –0.028
Rigid polyurethane	0.022-0.028
Rock mineral wool	0.034 –0.042

Virtualmaths

Layer Material	Conductivity (k)	Thickness (t) (in)	*C=k/t* Conductance (C)	*R=1/C* Resistance (R)	ΔT (˚F)	T (˚F)
Interior Temperature			-	-		68.00
Interior Film		0.5	8	0.125	0.27	67.73
5/8" GWB	2.08	0.625	3.328	0.300480769	0.65	67.09
6" metal stud	346.67	6	57.77833333	0.017307526	0.04	67.05
3" EPS foam insulation	0.208	3	0.069333333	14.42307692	30.97	36.08
5/8" glass mat GWS	0.2496	0.625	0.39936	2.50400641	5.38	30.71
2" mineral wool insulation w/ metal girts	0.208	2	0.104	9.615384615	20.64	10.06
Aluminum panel w/ air barrier	1643	2	-	0	0.00	10.06
Exterior Film		0.75	34	0.029411765	0.06	10.00
Exterior Temperature	-	0.5	-	-		10.00
			∑R (RSI):	27.01		
	Total ΔT: 58		Coeff. Of Heat, U:	0.037016927	also heat flow rate	

∑Thickness (in)
0
0.5
1.125
7.125
10.125
10.75
12.75
14.75
15.5
16



Layer Material	Conductivity (k)	Thickness (t) (in)	*C=k/t* Conductance (C)	*R=1/C* Resistance (R)	ΔT (˚F)	T (˚F)
Interior Temperature			-	-		68.00
Interior Film		0.5	8	0.125	0.39	67.61
5/8" GWB	2.08	0.625	3.328	0.300480769	0.94	66.67
3 5/8" metal stud	346.67	3.625	95.63310345	0.01045663	0.03	66.64
3" spray polyurethane	0.173	3	0.057666667	17.34104046	54.15	12.49
8" precast concrete	10.4	8	1.3	0.769230769	2.40	10.09
Exterior Film		0.75	34	0.029411765	0.09	10.00
Exterior Temperature	-	0.5	-	-		10.00

∑Thickness (in)
0
0.5
1.125
4.75
7.75
15.75
16.5
17

∑R (RSI): 18.58

Total ΔT: 58

Coeff. Of Heat, U: 0.053834003 also heat flow rate