
AEI Team Number:
05-2014
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Structural Report
2014 Charles Pankow Foundation
Annual AE Student Competition

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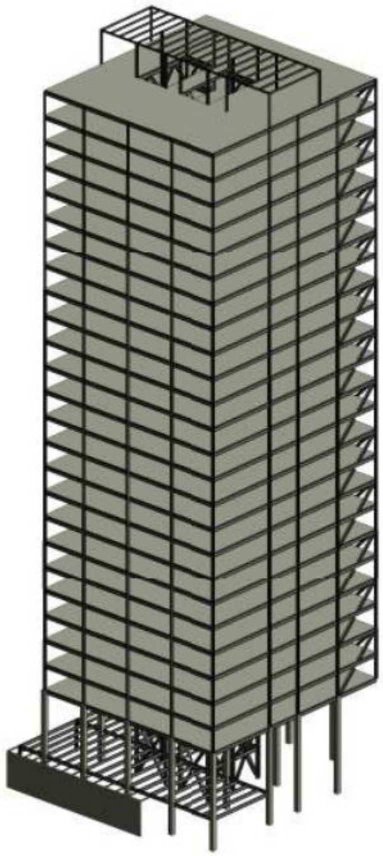
Executive Summary:

The main goal of the Charles Pankow Foundation Design Competition is to design a building that improves upon the quality, efficiency, and value of large buildings. These ideas are to be developed through new and innovative design ideas via construction, building systems, and structural components.



The following report summarizes the strategies, rationale, and steps the structural design team took when designing the structural systems for San Francisco's 350 Mission Street Project. The report also contains an appendix and supporting drawings that also help to summarize the results of the team.

The subsequent paragraphs summarize the main design concepts that the structural team implemented in conjunction with all other disciplines to create an efficient and high quality building for San Francisco's business district.

The main goals of the structural team for this project was to create a lateral force resisting system capable of complying to one half the code allowed drift during a design level earthquake and to have the ability to be occupied immediately after that same earthquake. Because of the latter goal of immediate occupancy the design team first decided to use Occupancy Category III to determine the seismic loading on the structure. They felt that in order to truly set a goal of immediate occupancy then the structure should be designed the same way that hospitals, fire stations, and police stations are designed. When determining the seismic loads on the structure using Occupancy Category III the loads are higher than if we were to use the traditional Occupancy Category II for a building of this type. The goal of immediate occupancy can be realistically achieved by designing a structure to withstand these higher loads. Table 1-1 Appendix A.

The structural design team decided to use a reinforced concrete sub-structure and a steel super-structure. The main focus of the team was on the super-structure. The two main goals of the project really pushed the design team more and more towards an all steel system because they did not feel that a concrete system would behave better damage wise than a steel system. By this we mean that concrete, being such a stiff material, does not allow for any type of flexibility when it comes to movement. This stiffness would cause the concrete to be more easily damaged than a ductile steel system. The loss of stiffness would be made up for in the steel system and the design team felt that this was the best option.

The design team did a lot of researching and a core consisting of Special Braced Frames were chosen to act as the main lateral force resisting. The design team did not arrive at this conclusion right away though. They went through a few different ideas which will be explained in detail in the following report. Due to code requirements the structure cannot only consist of just the core as the only system. The code required that the structure have moment frames around the exterior bays of the structure.

The design team used a structural analysis program called Etabs to perform the drift analysis of the structure. When the analysis was complete the design of the structure proved to be satisfactory. The building complies with one half the code allowed drift limit and is designed so that it can be immediately occupied after an earthquake.

Introduction

Project Requirements

This project creates a structural challenge from several different perspectives as it encompasses the design of a 30 story high-rise building in an urban area of California. The design project is centered around 350 mission street (see Figure A), located in the city of San Francisco, CA. The city is a sprawling urban center, and, in 2011, was the sixth most-visited tourist site in the United States. This is especially important to the structural design team as they consider the safety of the building occupants and patrons to be of utmost importance. The team must provide an office building that not only addresses several crucial design parameters set by the competition, but will also be a cultural facelift for the city. This submittal responds to the following guidelines set forth by the AEI competition for the structural design of the building:

1. The building is able to be immediately occupied after a design level earthquake.
    a. For the purposes of this competition the structural design team has assumed this to mean that the building may undergo repairs during regular business hours while being occupied by the regular tenants.
2. The building is to be designed to have an increased life span and enhanced performance even after the onset of a major design level earthquake.
3. The drift of the building under seismic and wind conditions is limited to one-half of the code specified drift limit.
4. The integration of all systems, and as such the structural team must constantly consult with the other disciplines in order to maintain overall efficiency throughout the design process.

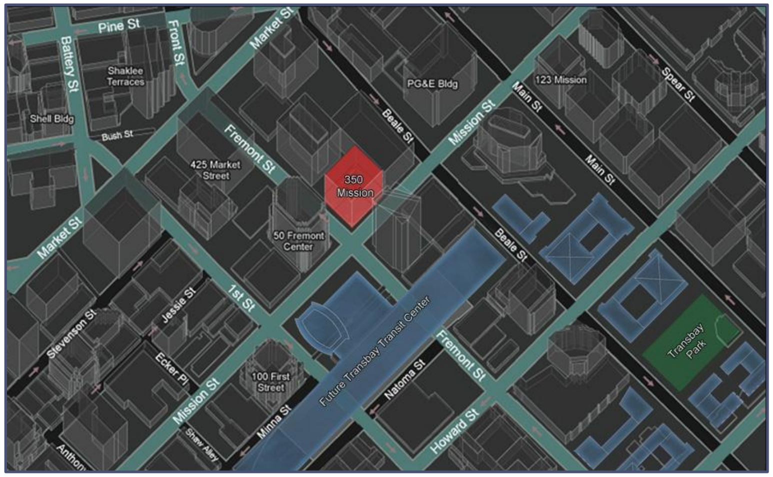


Figure A: 350 Mission Street site located in urban San Francisco. (From www.350mission.com)

Structural Goals

Before beginning the design of 350 mission the structural team developed several critical goals that they wished to achieve by the end of the competition. They are as follows:

1. Design a structural system conforming to the strict limits set by the competition without sacrificing efficiency or impeding the other disciplines.
2. Complete a design that focuses on the safety of the occupants in the event of a major design level earthquake, while ensuring the integrity of the structure.
3. Complete all of these goals with economy and the environment in mind.

The results of these goals and early programming/ planning are presented in the subsequent sections of the introduction.

Design Strategy

These goals would provide quite a challenge for any building design and even more so for one to be constructed in an urban and seismically controlled region. In order to attain these long term goals the building was divided into gravity and seismic considerations.

Gravity System

The gravity system was the first tackled by the design team before moving on to the seismic system. As seen in **TABLE 1.1** the gravity loading put on to the system was very straight forward.

Table 1.1	
Description	**Load (psf)**
Live Load	100
Raised floor	10
Mech Allowance	10

The design team had to consider the significant dead loads imposed by things such as the under floor mechanical systems as well as the modular kitchen units used in each office floor, as well as the weight of the 30 stories themselves. After completing the design of a preliminary gravity system with these dead loads and live loads from governing codes the team recognized that the girder depths leading from the core of the building to the west side of the building (see figure B) were excessive. In order to combat this issue, additional columns were placed along these girder lines in order to lower girder depths.

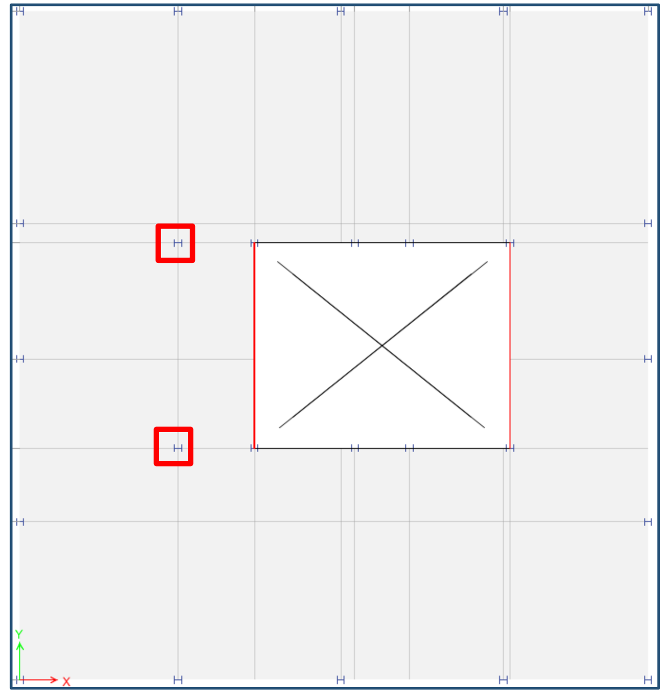


Figure B: Seen above is a plan of a typical floor section of the 350 Mission Street building with added columns highlighted.

The seismic design for this project proved to be the most challenging for the design team and required the most due to its core functions to lateral resistance. Much of the research time spent on the structural portion of this project was put into considerations for the best seismic system to use. The team considered many options including various types of damping systems, moment and braced frames, thick concrete shear walls, steel plate shear walls, base isolation, and outrigger systems. In the end the team decided that a combination of special concentric braced frames and moment frames would best meet the requirements set forth by the competition. The design team had originally planned to use steel plate shear walls as well; until they were removed for reasons discussed later in this section during the latter portion of design.

Preliminary Lateral System Investigation

During the design of the lateral system the structural team investigated several configurations and options. These systems included the use of braced frames and steel plate shear walls. The structural team's justifications for exploring these options are contained below. The lateral system would change several times before a final configuration (which can be found in the dedicated lateral system section) was defined.

Braced and Moment Frames

The design team decided upon the use of special concentrically braced frames for the core and moment frames on the outer perimeter of the building in order to satisfy the drift requirements. The choice to use both these and steel plate shear walls came from Table 12.2-1 in ASCE7-05(**Appendix A Page S-3**) governing the use of dual systems based on building height. The design team decided upon an "x" configuration for the braced frames (see Figure 4) due to the fact that a chevron configuration in either direction would cause excessive supporting beam depths during analysis.

Steel Plate Shear Walls

Once again, the design team at one point had decided to use steel plate shear walls in the core for the entire height of the building (see Figure E). The steel plate shear walls are used in order to provide stiffness along its length. Steel shear walls were decided upon over concrete shear walls in order to comply with the near immediate occupancy provision of the competition guidelines. It is the opinion of the design team that the steel plate shear walls would have held well during a design level earthquake where a concrete shear wall would crack and require extensive repairs.

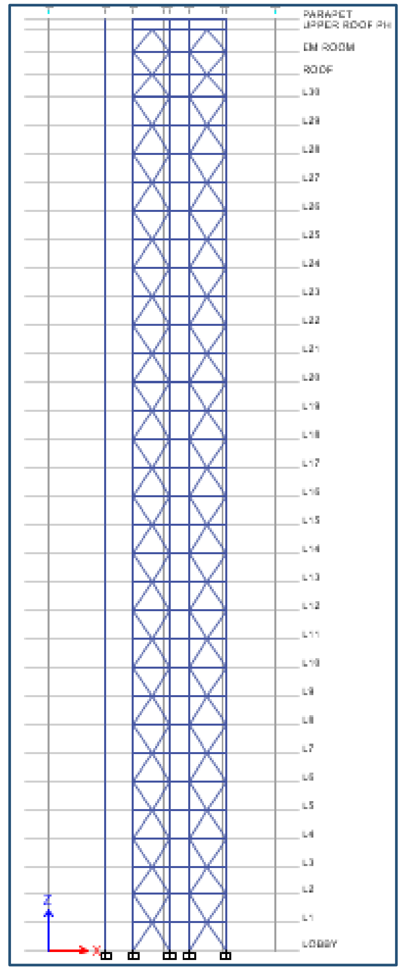
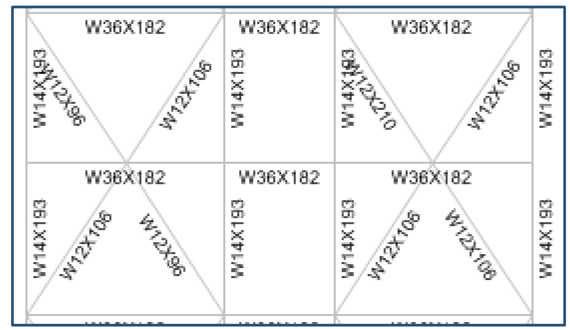


Figure C:

Shown left is an elevation of the braced frames contained within the core of the 350 Mission Street building. Note the "x" formation of the braces used to reduce supporting beam depth.

Figure C.1:

Below is a blown up detail.



SITE

The site of the project is located at 350 Mission Street in San Francisco, California which is the intersection of Mission Street and Fremont Street. This intersection finds itself in a densely populated urban area of the city. The building is located very near to the site of the new Transbay Transit Center (shown in figure D). As such the building will be in close proximity to many of the professionals commuting to, and working in, urban San Francisco. As the building is located in California it is at a high risk for seismic activity. This marks 350 Mission Street as an intersection between dangerous natural conditions and a high population density. This makes the competition provision for seismic control and immediate occupancy after a design-level earthquake extremely important as it will allow the business(es) taking residence in the building to continue working almost uninterrupted after such an event.

The climate in the San Francisco area is very temperate. The weather typically ranges anywhere from 50^{0}F-70^{0}F throughout the year. This is an ideal climate for the structural team as it means that the structural steel used throughout the building will not often experience the expansion and contraction brought upon by extreme changes in temperature.

Figure D: The urban Transbay Transit Center (shown mid-page to the left) located in San Francisco, California. As shown in the photo above, 350 Mission Street is located in a populated city center. As such it is imperative that the systems are design with the citizens' safety in mind. (Taken from www.transbaycenter.org)

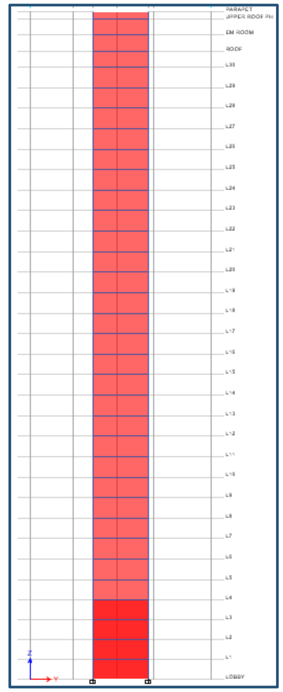


Figure E:

Shown right is an elevation of the 350 Mission Street high rise with the steel plate shear walls shown in red. The shear walls the entire height of the building. They are braced at every floor by two columns (one on each side) and a beam running along the level of the floor. This design was revised in the final phase to only include braced and moment frames in the core.

FOUNDATION

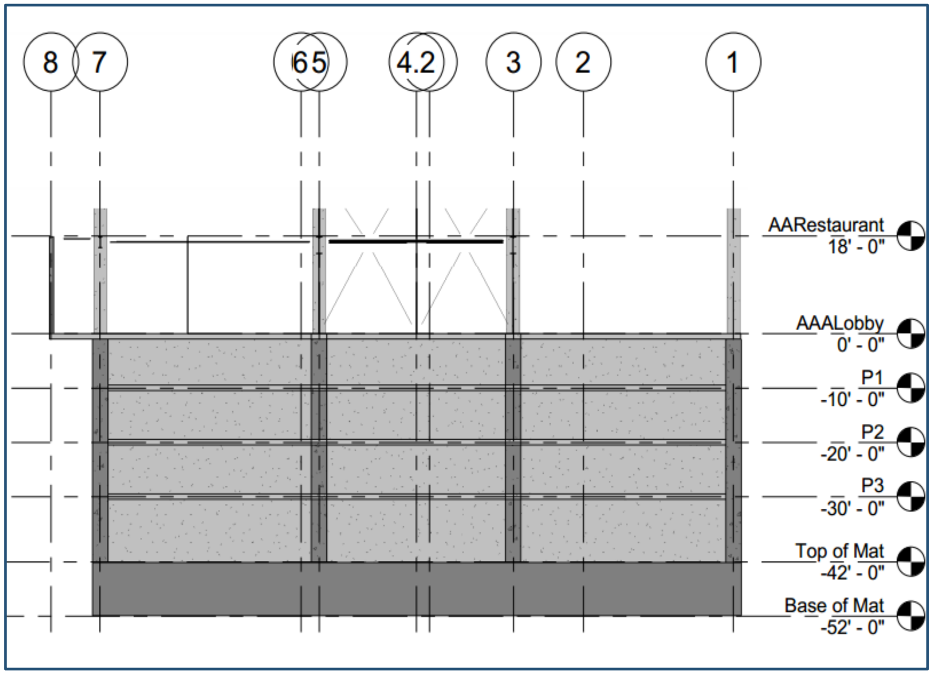


Figure F:

Section of Proposed foundation system

As can be seen on **S-13 in the Appendix** the equivalent lateral force (ELF) procedure was used in order to determine an approximate overturning moment (OTM) for the building. This preliminary overturning moment was found to be approximately 703,000 kip*ft. To accommodate this extremely large moment the design team has decided that the original foundation proposed for the site to be the best solution. This foundation consists of a 10 foot deep mat foundation encompassing the footprint of the building. According to the geotechnical report provided to the design team for the 350 Mission site, the excavation elevation is below that of the existing water table. As saturated soil is undesirable for the construction of the mat foundation, a smaller working foundation is to be constructed on which the mat foundation can then be poured. A section elevation can be seen in **FIGURE F.** **FIGURE G** shows the proposed configuration of the excavation, working foundation, and mat foundation.

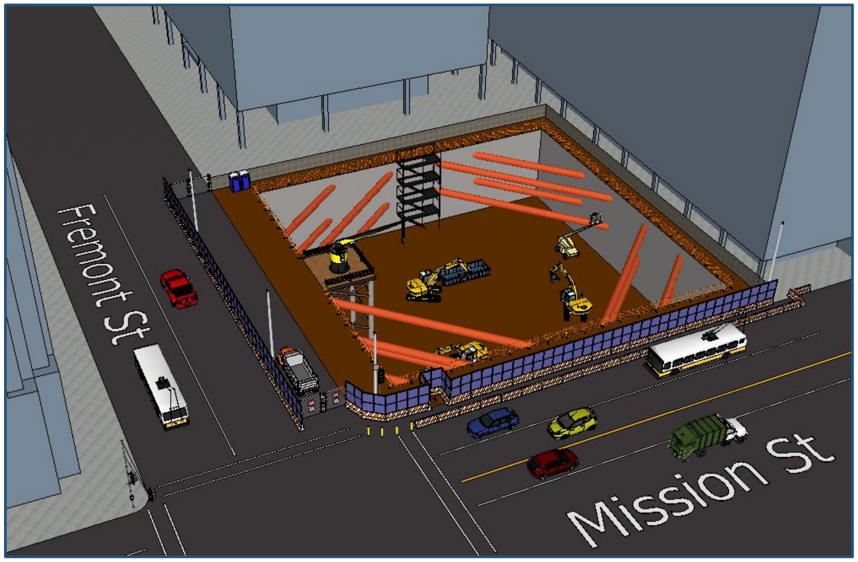


Figure G:

Seen above is the proposed construction of the mat foundation for the 350 Mission site, to be constructed upon a working foundation due to water table complications.

Gravity System

The building is composed of two different types of gravity systems in the form of the sub-structure and the super-structure. The sub-structure is composed of reinforced concrete to meet parking load demands and environmental conditions. The super-structure is composed of composite steel beams and girder system as well as steel columns.

The reinforced concrete system was chosen for the sub-structure because it was the ideal solution to work well with the foundation system and for construction of a parking structure. The design team as a whole decided that the design by SOM is efficient for the purpose of time and other limitations that drive more detail into other areas of the structural system. The steel system chosen for the super-structure was not based on the gravity requirements but for lateral concerns regarding the need for immediate occupancy after a design level seismic event. The design team decided that the ductile steel would make for less immediate damage than concrete. The steel system also reduced the weight of each floor significantly, on average 44%. This reduced weight makes for less seismic loading on the lateral system, therefore making the drift limit more attainable. Also, if repairs were ever needed steel makes for ease of replacement, schedule time, and accessibility for occupants during repair. The gravity loads were calculated using ASCE7-05 and are shown in Table 4.1.

Table 4.1	
Description	**Load (psf)**
Live Load	100
Raised floor	10
Mech Allowance	10

The layout for the super-structure steel system was based on the architectural drawings provided, code requirements, material limitations and constructability parameters. Challenges addressed for this system during the design were excessive beam depths due to long spans, high floor to floor height on levels above the lobby level, and the cantilevered corner of the building.

The first design addressed for the super-structure was the slab on metal deck. The Vulcraft Manufacturer deck catalog was used to choose a deck and topping thickness that met a two hour fire rating, an appropriate span condition, and deflection criteria. The deck chosen was 2VLI20 set at a 3 span condition with a topping thickness of 4 ½ inches of normal weight concrete. This limited our beam spacing to no more than nine feet from center line to centerline of beam, this also helped to limit our beam depths by keeping this spacing under ten feet. Another reason for choosing to keep this spacing less than ten feet is to have no shoring needed during construction.

After the deck design and load cases were determined the design team chose to use Bentley RAM Structural System to lay out the beam design and begin to get preliminary sizes. The typical office floor layout was the starting point and it was determined that there needed to be columns added on the floor plan to reduce the beam depths in some areas. These additional columns are shown in **FIGURE H**.

The addition of these columns on the typical office level gave the design team the ability to limit beam depths to no more than 24 inches when they are spanning over occupied areas. Limiting the beam depths to this height allowed the team to set the floor to floor height of the building at the office levels to 14-feet. The beams are set to be composite beams with no camber. RAM was used to finalize the size of the beams and set the number of studs per beam. The beams range in size from W12's to W36's with an average composite action of 25 percent. **FIGURE I** shows a typical section of our composite beam section.

After the typical office level was modeled and designed in RAM the model was then expanded to complete the super-structure. This completed super-structure model consisted of the restaurant level, level 5 (level above the lobby), typical office level, penthouse, and roof all modeled as different floor types. A picture of the completed RAM model is shown in **FIGURE 13.** The final beam designs and layouts of all levels can be found in the **DRAWINGS.** The final composite beams were then spot checked by hand for capacity and deflection limit state against ASCE 7-05.

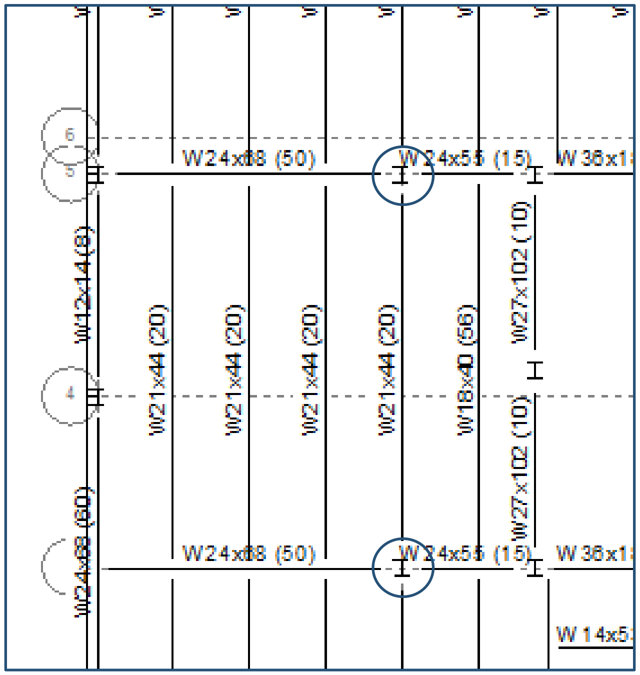


Figure H:
Shows the additional columns circled in red

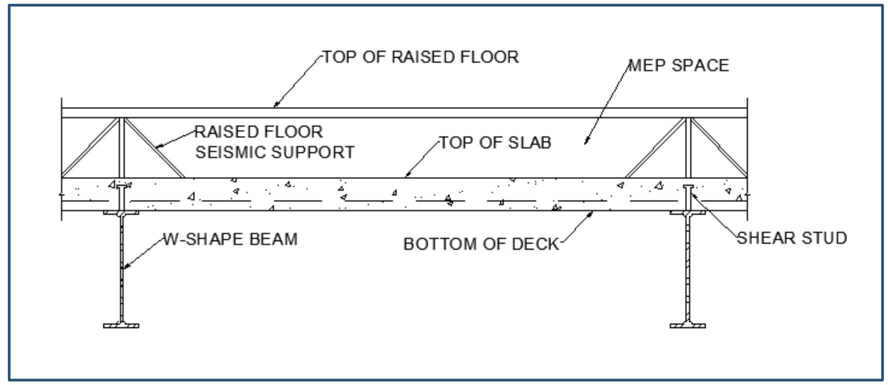


Figure I:
Typical Floor Section

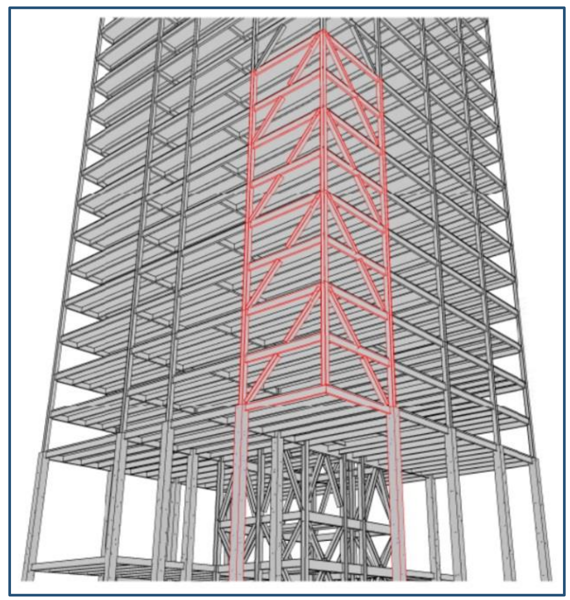


Figure J:
Corner Perspective

The column design of the building was also verified using RAM and hand calculations. W14 sections were chosen to be the default size for all columns. These were chosen because the design wanted to use a consistent and efficient column size for the entire building. A W14 shape has very compatible section properties for our building's size and function to satisfy these two goals. After running the RAM model it was realized by the design team that W14's alone do not have the capacity to carry the load at the lower levels of the building. The main problem that needed to be addressed was the large un-braced length of the columns at the lobby level. The large un-braced length of 54 feet in some areas causes the interaction equation to fail during analysis. The design team decided that built up columns would be used for these levels. The final design chosen for these columns was a studded W14x398 encased in a 36"x36" concrete column. The concrete column will be made of 8000 psi concrete and have (12) #18 bars. The detail for these columns can be found in **DRAWING S-2**. The calculations for these columns can be found in **Appendix C page S-12.** Columns will be spliced every 2 levels, which comes to 28 feet and is an efficient length for erection and transportation to the site.

As the design team was finalizing the column designs a new problem arose. The addition of the column in the south west (plan direction) corner of the building created an aesthetic problem at the lobby level. The team wanted the iconic corner of the building to remain as open as possible (shown in **FIGURE J**). This was addressed by eliminating the column at the lobby level. The design team decided that they would use a series of transfer braces and girders to transfer the load away from that corner of the building thus creating the possibility of eliminating that corner column at the lobby level. The braces had to have the capacity to transfer two stories of load away from the cantilevered corner and into other exterior columns. These braces were decided to be W14x257's on every level above the lobby level. The design check of these transfer braces can be found in **Appendix C Page S-12. FIGURE J** shows these braces highlighted in red. **FIGURE K** shows these braces on the building in the RAM model.

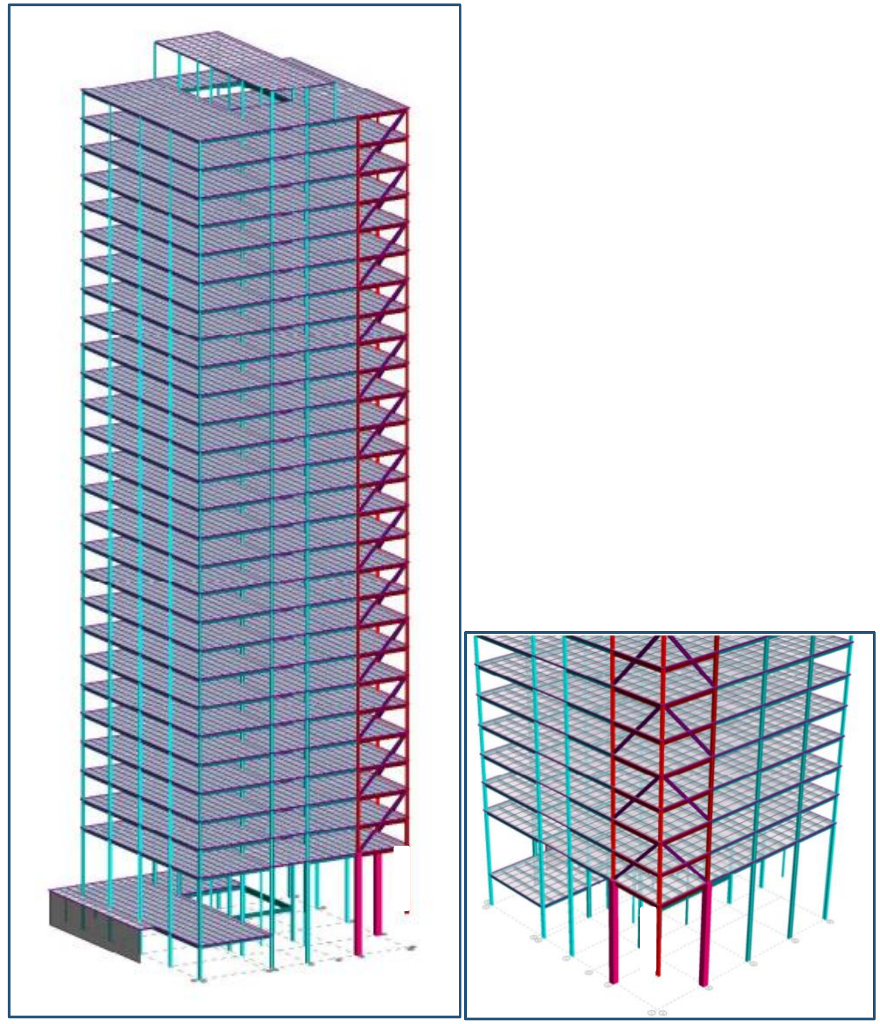


Figure K:

3D Perspective of RAM Model

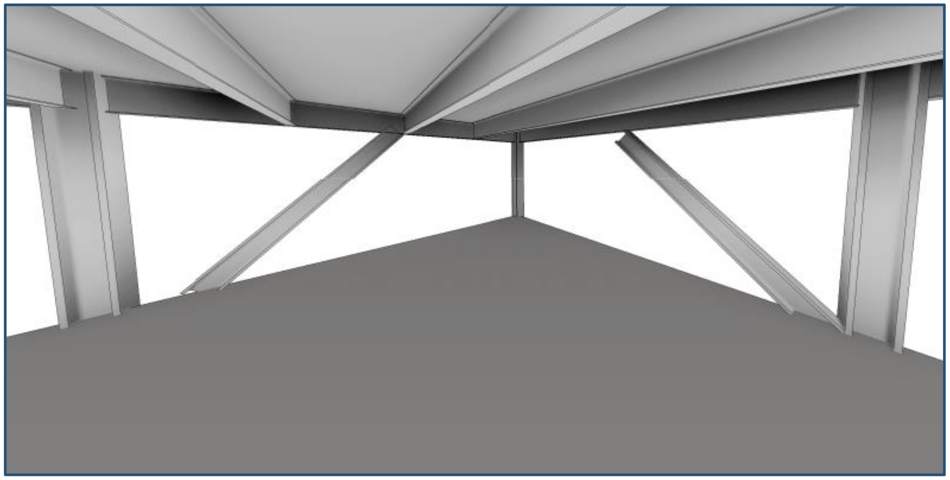


Figure L:

Interior 3-D View of Corner Bracing without finishes

Figure M:

Interior 3-D View of Corner Bracing with finishes

Lateral System

The structural team determined the lateral system to be up the utmost importance due to the restrictions placed by the competition on drift of the building. The design of the lateral system proved particularly challenging for the structural design team. As mentioned earlier in the introduction of the report the lateral design hinged upon several things:

1 **Conforming to one-half the code allowed drift limit**

2 **Attaining near immediate occupancy after a design-level seismic event.**

2 **Designing a lateral syst m that is both durable and easily repairable.**

The team began this process by m ting with all other disciplines to discuss possible complications between the lateral system and things such as the mechanical and electrical equipment. Further information on the collaboration between the various teams can be found in the integration report. After determining the possible complications from a lateral system in a building this size the structural design team decided to attempt to limit the lateral system of the building primarily to the core and perimeter of the building.

After coming to this conclusion the team began to consider different options for the material of the building. While the long spans and drift limit could certainly have warranted a concrete design the team ultimately decided to go with a primarily steel structure. This came about after discussing the merits of repairing a steel system as opposed to the extensive effort that it would take to repair any failed portion of a concrete structure. (It was also noted that the construction of a steel system would potentially be more costly. While efforts were made to keep the cost of the structure reasonable, the primary concerns were those imposed by the competition.) This may have made the near immediate occupancy portion of the competition very difficult to attain. On top of these considerations the team did research on both damping and shear wall systems that led them to believe that it would indeed be possible to attain an appropriate drift limit in steel design.

As can be seen below the team then performed a new equivalent lateral force procedure with the gravity system and the estimated weight of a steel core system. These calculations provided the forces and moments required to perform a lateral analysis of the system being designed in ETABS.

10	1748	129	4480.44	7831801	0.010824	26.59	3429.76	
9	1748	114	3617.81	6323927	0.00874	21.47	2447.39	
8	1748	99	2834.32	4954391	0.006847	16.82	1665.09	
7	1748	84	2133.06	3728590	0.005153	12.66	1063.25	
6	1834	69	1517.78	2783609	0.003847	9.45	652.03	
5	1834	54	993.21	1821548	0.002518	6.18	333.92	
rest	1160	18	148.46	172218.3	0.000238	0.58	10.52	
Lobby	3190	0	0.00	0	0	0.00	0	
	50342			7.24E+08	1	2456.3	**812433.396**	**OTM**

TABLE 5.1
Revised ELF procedure for lateral loads (First 10 floors, the entire excel sheet can be found in **Appendix C page S-13**)

After performing this set of calculations the only other metric needed to begin quantitative design of the lateral system would be the allowed drift for the competition. Upon investigation of preliminary sizing of the gravity and lateral systems the design team determined the overall height of the building would not exceed 450 feet. This number was used in accordance with ASCE 7-05 Table12.12-1**, which can be found in Appendix A page S-4**, to determine the new allowable drift limit to be no more than 41.5 inches at the full height of the building.

Knowing this metric, a preliminary model was constructed in ETABS containing a core made up of braced frames in the north/south directions and steel plate shear walls in the east/west directions The design also contained moment frames on the perimeter of the building as per **ASCE 7-05 table 12.2-1, which can also be located in Appendix A page S-3.** The shear walls were chosen for their ability to provide excess stiffness, and the braced frames were chosen in order to meet the requirement of being replaced with minimal downtime.

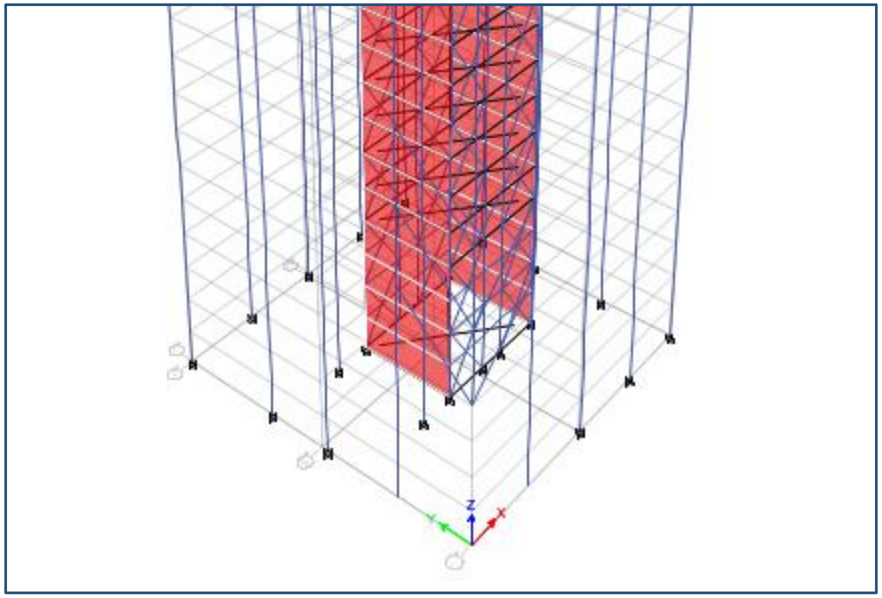


Figure N:
Preliminary ETABS model showing the locations of the braced frames and shear walls in the core. (Full building height in **Appendix B page S-8**)

During the period that this model was being constructed the design team also researched ways to further limit drift in the event that the building could not conform to the drift limits set upon by the competition. This research came primarily in the form of viscous fluid dampers, which could replace some of the cross bracing in the braced frames in order to dissipate energy and thus lower building drift. It was decided after all of the research was gathered that the team would only attempt this in the event that the drift was substantially higher than anticipated. This was due to the fact that the research showed that these damping elements could be very expensive and in fact lowered the strength of the lateral system by replacing a bracing element, but providing no lateral stiffness in return.

Upon completion of the first iteration of the ETABS model, it was run with the loads provided from the updated ELF procedure to obtain the drift and period with the steel plate shear wall and braced frame configuration. The results indicated that the steel plate shear walls were actually providing far too much stiffness and giving a result of roughly one-half to three-quarters the drift allowed by the competition based on the thickness of the shear walls. This was a significant problem as the cost of the shear walls was justifiable as a solution only if the drift was close to that of the competition guidelines. As it stood with these results the walls provided an excess limit on the drift and excess in cost for the system.

This realization eventually led to the final design of the lateral system. The design team discovered that by removing the shear walls on the east and west sides of the building and replacing them with two braced frames and adjusting the size of the members accordingly that they were able to achieve a drift of 30.9 inches at the full height of the building. This number matches the competition drift limit very well (in fact 2 inches lower than the requirement), and provides an excellent design change as the braced frames are both economical and easily replaceable.

Upon placing these criteria and variables in ETABS and accounting for various lateral forces (primarily seismic in nature) a final lateral model containing the core and perimeter moment frames was produced. This final model composed of two braced frames on the north and south faces, as well as the two braced frames on the east and west faces as mentioned above. All of the braced frames are concentrically braced and alternate in direction, creating an “X” pattern along the faces of the core, every two stories. This pattern was chosen because a typical chevron formation created an excessive force, and as such, an excessive beam depth in the supporting horizontal members.

In the north and south directions the core spans roughly 47 feet. This span is broken down into three sections, two braced frames spanning 18.5 feet on either end connected by one beam spanning 10 feet between them. The team discovered, while in talks with a professional engineer who has over 40 years of tall building experience, that making this center beam sufficiently large would allow these braced frames to work in conjunction with one another to resist lateral forces, while allowing for space beneath for an entrance to the elevators.

In the east and west directions the core spans ~ 38 feet. As the east and west faces of the core were not being used as main entrances to the elevators it was feasible to make this span into two braced frames spanning roughly 19 feet each and sharing a center column.

As per ASCE 7-05 a dual system such as this requires moment frames on the perimeter of the building. These were included as two-bay moment frames in the center two bays on each face of the building. Although they are required it was found that these moment frames provide negligible lateral reinforcement and that the allowable drift was taken care of simply with the reinforcement in the core.

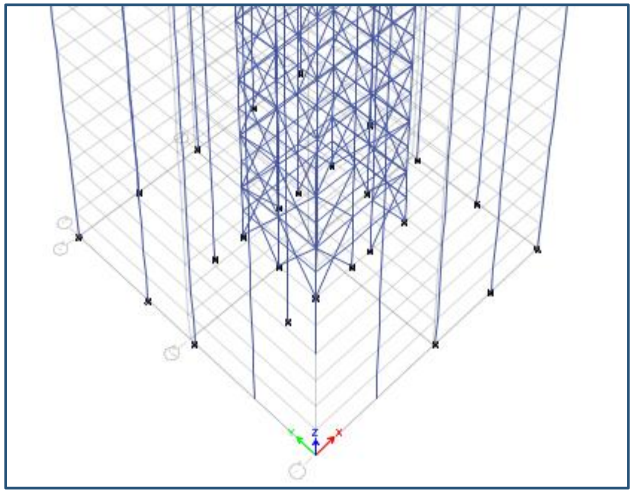


Figure O:
Final lateral model in etabs. (Full building height can be found in **Appendix B page S-9)**

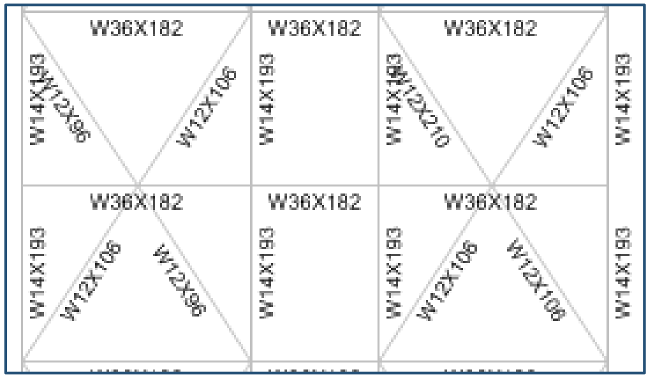


Figure P:
Callout of the braced frames showing typical beam sizes. (Full building height for frames in both directions can be found in **Appendix B page S-10**)

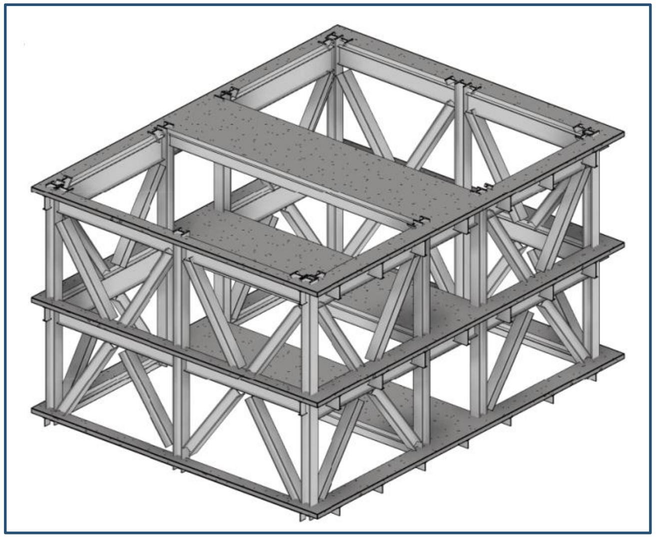


Figure Q:
Two Floor 3-D Perspective of Core
(Full building height can be found in **Appendix B page S-9)**

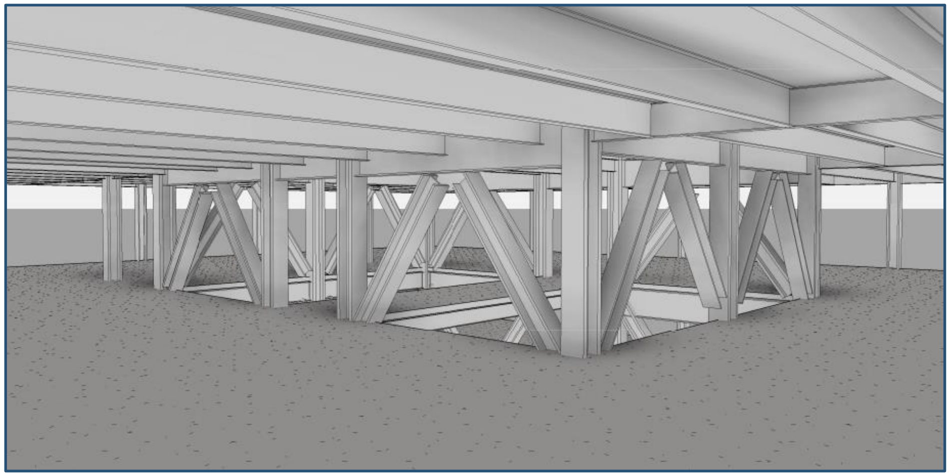


Figure R:
Interior perspective of Core Framing with No Finishes

ENCLOSURE

The enclosure system chosen for the project was a double layer façade. This means that there are two layers of façade instead of the traditional one. The façade will consist of individual panels. The panels will be 5 feet wide by 14 feet tall. The enclosure has been schematically designed and a detail of it can be found in **FIGURE S.** For visual purposes the figures of people modeled are 6'-3" in height.

After the Façade requirements were designed by the Mechanical team, they were handed off to the Construction and Structural teams to work together to a final design that would satisfy constructability and structural requirements. The façade panels will be assembled as a whole off site and brought to site as they are needed in the schedule. We, as a design team, took advantage of the offsite fabrication and structurally designed these panels with this in mind.

Each panel is the full double façade. This means each panel will have two panes of glass and two sets of louvers held together by mullions. Each layer of the façade consists of one pane of glass and one set of louvers. The louvers act as our ventilation for the double façade as well as the covering needed for the plenum space and structural framing.

The panels will be hung from the slab by steel angles. They will have kickbacks to maintain stability. These will also be steel angles. The beams that have these kickbacks framing into them will need torsional support. This support will come from the gravity members where they frame in perpendicular to the exterior beam. Where the gravity beams are framing parallel there will be torsional support framing added spanning between these two beams acting as bridging.

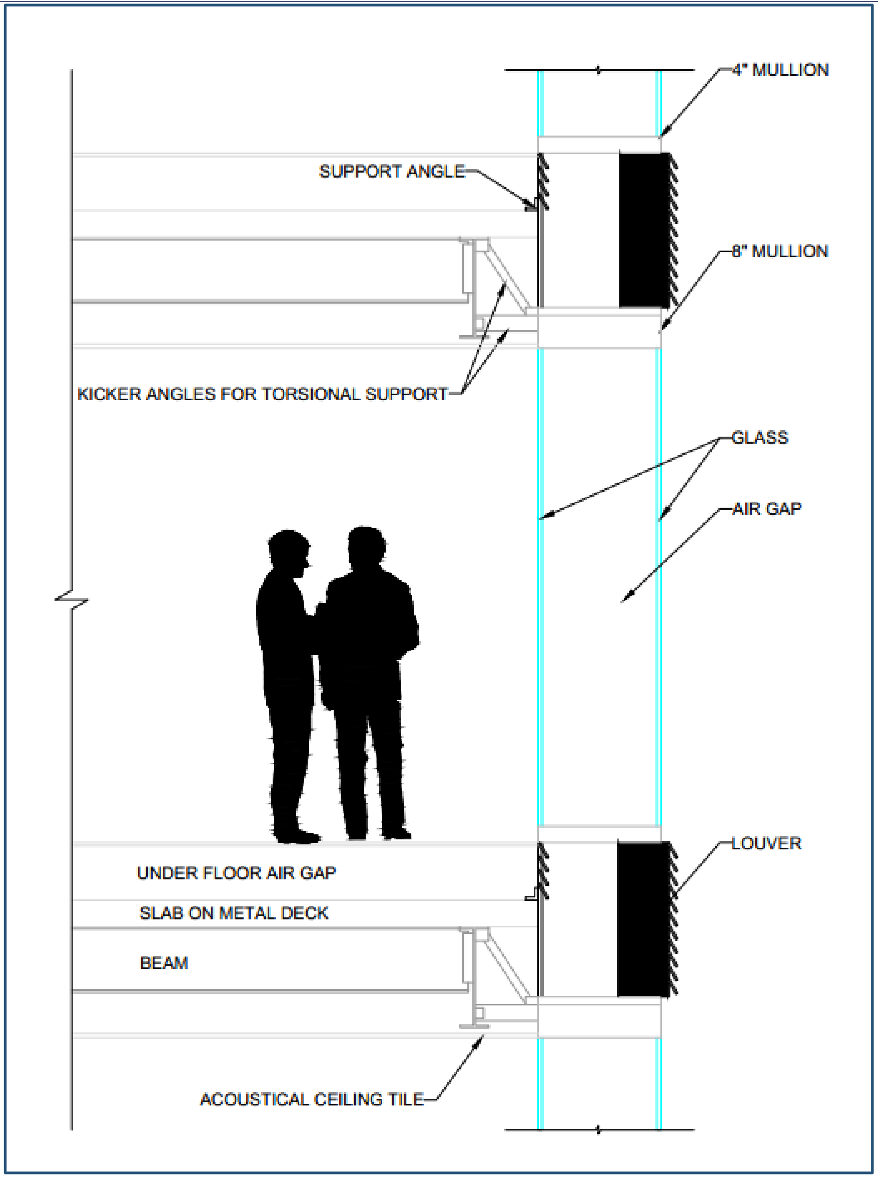


Figure S:

Floor to floor section of the façade and its structural components

Conclusion

Overall the goals set forth by the competition and those self-imposed by the structural design team were accomplished across the board. The foundation and gravity system were designed with the mixed use of the building as well as the desire to have to office spaces be modular, in mind. The design of the system in steel allows for the building to be easily repaired while maintaining strength and being flexible in the modular design of the building. The design of the lateral system will perform extremely well even under extreme seismic conditions. Under the conditions imposed by the ASCE 7-05 code the building will displaced a maximum of 39.0 inches at the full height of the building, well under the 41.5 inches imposed by the AEI competition. This system has also been designed in an all-steel fashion. This again lends to the strength and ease of repair of the building. The enclosure of the building is a double glass façade supported primarily by steel angles. This will allow for the façade to be braced every floor and will also reinforce the lateral system slightly.

The structural design not only satisfied the goals of the competition and structural design team but also the goals of the Project team. The steady collaboration between the structural design team and the other design teams resulted in the structural system achieving countless other common goals of every building project such as; elimination of system clashes, constructability goals, and architectural aesthetics. This collaboration can also be found detailed in the integration report from our project team.