Structural Systems

Submitted: Integration Structural Mechanical Electrical Construction

350 Mission St.

ASCE Charles Pankow Foundation Architectural Engineering Student Competition

AEI Team 2-2014

Contents

Narrative

Structural System Highlights

50% code required drift allows for **near-immediate occupancy** post seismic event

Multi-story diagonal bracing relieves lateral demands on core resulting in **9100 ft²** usable space increase

Accentuated architecture via structural coordination

Efficient façade design and multi-disciplinary integration

Appendices

Drawings

Executive Summary. The structural members of AEI Team 2 have addressed the many challenges facing the design of 350 Mission Street. This submittal includes a project overview, project performance goals, description of design process, rationale for our chosen design, analysis and design modeling summaries, and all associated analyses and justifications. Additionally, the submittal includes appendices and drawings containing supporting documentation of detailed calculations, floor plans, sections, elevations, modeling data and references.

Project Goals, Requirements and Introduction. In accordance with the project guidelines set forth by the competition, the structural discipline was responsible for:

- A design that limits structural damage from earthquake events
- A structure that limits building drift to ½ of the code allowed value
- Solutions that increase building life cycle cost and efficiency
- Consideration of architectural features and the structure's impact on them
- The design and detail of gravity and lateral systems
- A thoroughly detailed design of one typical floor
- Representative drawings and model documentation
- A design of the building enclosure with details to achieve a high-performance standard
- A foundation design

Personal goals were developed around these requirements. These goals are outlined in Section 2.0

Integration. Throughout the course of the project, the structural team used BIM technology, workflow and communication strategies to create an efficient structural solution for 350 Mission, and seamlessly integrate with the other disciplines. Structural design solutions were conceptualized by the structural team, discussed and analyzed by the entire project team, and carried to fruition by the structural designers. The structural discipline was additionally called upon to support the other disciplines in maximizing the potential for the whole building design.

Lateral System. The lateral force resisting system utilizes a concentrically braced frame core, coupled with diagonal exterior bracing. These were designed to reduce building drift and increase seismic efficiency to meet the project requirements. The system meets the ½ Code Allowable drift limit with a total drift of 26 inches at the top of the building, with all floors meeting the ½ Code allowable inter-story drift limit. This design effectively handles the seismic loads and allows the tenant to have immediate occupancy post Maximum Considered Earthquake (MCE). Additionally, the exterior mega bracing mobilizes the perimeter of the building in lateral resistance which relieves lateral loads on the core. This allows the concentrically braced frame core to be thinner and lighter, creating a 9100 sf increase of rentable space in the building over the initial 33-inch concrete shear wall design.

Gravity System. The gravity system was a multidisciplinary effort to design an efficient structure while ensuring the other systems present would not be limited. An efficient gravity system was formulated that limited beam and girder depths to a maximum of 24 inches. This allowed for a large floor-to-ceiling height of 9 feet 10 inches, and a 32 ½ inch plenum space for MEP coordination. Optimized member sizes and placement create open views which enhance working conditions for the tenant and cement the building as part of the urban fabric.

Lobby Area. A major structural consideration in the lobby was the potential for soft story behaviors created by the 5-story high space. Built-up sections were designed in order to handle the 54 foot unbraced column length. The design required custom sections consisting of a W14x730 with 1-inch steel plates welded between the flanges. The structural team also investigated methods of designing the 29 foot South West cantilever in order to preserve the inviting nature of the lobby space.

Façade System. The structural team coordinated closely with the other disciplines to design an attractive and efficient façade system to account for the seismic behavior. This process resulted in an extensive movements and tolerance report as well as in depth glass fracture and fallout drift studies. Ultimately a unitized system consisting of 1 ½ inch double pane glass supported by 2 ½" inch deep mullions spaced at 57 ½ inch intervals was identified and its structural anchorage detailed.

Sustainability. Optimizing the structural member sizes limited fabrication and shipping waste over the project timeframe. The structural team worked closely with the construction team to achieve the least amount of steel weight for the building. Using factors such as steel weight and schedule duration, a carbon efficiency study of the structural design was conducted using SOM's Environmental Analysis Tool.

1.0 Project Goals

The structural team outlined specific project goals at the beginning of the project in order to guide their decisions as they moved through the design process.

The over-arching ideals of the ASCE Charles Pankow Foundation Student Competition included improving quality, efficiency, value, and performance of buildings by advancing integration, collaboration, communication and work-flow efficiency. The design team was to accomplish these by using innovative new tools and technologies, and by advanced means and methods. Competition goals, required of each team, spawned out of these ideals and centered on increasing expected life-cycle efficiency of the building, complimenting architectural features with engineered systems, and executing a high-performance, Near Net-Zero solution.

The competition goals led the design team to identify early on what a "high-performance" building means in conjunction with their goals. The competition description defines "high-performance" buildings in relation to the Energy and Independence Security Act of 2007 stating: "a building that integrates and optimizes on a life cycle basis all major high performance attributes, including energy conservation, environment, safety, security, durability, accessibility, cost-benefit, productivity, sustainability, functionality, and operational considerations." AEI Team 2 agreed to adhere closely to this definition, as the Integration Report describes in further detail. Figure 1 below, highlights in red the ways the Structural team helped impact the "High Performance" design.

While the above goals and challenges applied to the team as a whole; the structural discipline had some specific requirements and performance goals the competition asked them to meet. Enhancing the building performance during and after an earthquake was the main focus. To this end, the building drift was required to be limited to 50% of the code allowable drift.

Figure 1: Structural Impact on High Performance Design

The owner also preferred that the design would limit the structural and non-structural damage and repair required from a design level earthquake in order to allow expedited post-event occupation. Looking at all of these competition goals allowed the structural team to then align these goals with their own objectives for the project and to the specific responsibilities the design must incorporate.

The competition's emphasis on a high-performing, durable, sustainable, and life-cycle focused design drove the structural team to develop the following goal hierarchy for the project:

1. Near Immediate occupancy after major earthquake
2. Innovative and efficient lateral system
3. Optimized lightweight gravity system
4. Increased lifecycle efficiency
5. Reduced mat slab foundation size for ease of construction and MEP coordination
6. Structural design that enhances the building architecture.
7. Implementation and full leverage of BIM technology and methods to increase collaboration, integration, and innovation.

Allowing these objectives to guide and aid in their decision making, the structural team has managed to develop the structural systems and solutions, shown in Figure 2 (on page 2), that not only satisfy owner goals, competition aims, and team objectives but also improve the quality, efficiency, and overall value of the new 350 Mission Street.

2.0 Integration

In order to achieve the optimum solution for the whole building design of 350 Mission, the entire project team adopted a work culture of collaboration, integration and communication. The design team set out three main ideals of Performance, Endurance, and Connectivity. All design decisions were made based on providing the best case solution to achieve these three team objectives. Please see the Integration Report for description of the metrics used to measure the team's success. Figure 3, shows how the responsibilities of the Structural Team related to the three team ideals.

Discipline Goals	Solutions
Net Zero Energy	***Locally Produced Steel***
100% Elastic Design	***Reliable Level of Redundancy***
Net Zero Waste	***Optimized Member Design***
Earthquake Resilience	***High Performance Lateral System***
Landmark Architecture	***Exterior Structural Bracing for Architectural Enhancement***
Life Cycle Performance	***No Structural Damage after Major Design Level EQ***
Community	***Engaging, Interactive Design***
Sustainability	***Reduced EQ Maintenance***

Figure 2: Discipline Goals and Related Solutions

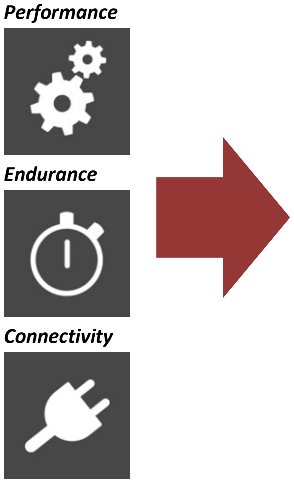
When faced with a design decision, the structural team came up with possible solutions that fit their specific needs and pitched them to the rest of the design team. Constructive criticism was received and the entire team analyzed whether one of the proposed solutions could be utilized. If no solution appeared workable, the structural team re-addressed the initial problem and came up with alternative solutions to discuss with the entire team.

Integrated Project Ideals

Performance

Endurance

Connectivity



Responsibilities

50% Code Required Drift Limit

Immediate Occupancy Post Design Level EQ

Eliminate Structural Damage due to MCE

Coordinate with MEP Special Systems

Mat Slab Reduction

Building Façade Attachment

Figure 3: Integrated Project Ideals and Structural Responsibilities

Once a solution was decided on, the structural team carried out modeling, calculation and other design tasks, while keeping the entire team up to date with progress and any changes. The structural team was also involved in discussing the design decisions of the other disciplines. Open, involved communication practices made this type of workflow possible.

To increase efficiency and productivity of the integrated design process, the entire design team used BIM based software which could communicate with specialized analysis and design software of all disciplines involved. Revit was used as a central modeling tool, with ETABS and Ram Structural System providing the primary analysis and design functions for the Structural Team. These tools added value to the project by allowing all disciplines to see how the building systems worked together in real-time throughout the design phase. Please see the Integration Report for a detailed view of how all disciplines worked collaboratively to achieve the desired design.

3.0 Building and Site

One of the first activities that all team members engaged in was an in-depth site analysis. Please see the Integration Report for a more detailed explanation of the interdisciplinary results and impacts of this site analysis, this report will only focus on the results that specifically affect the structural content presented herein.

The 350 Mission site is located south of Market Street at the corner of Mission and Fremont streets. The site footprint is approximately 19,000 sf and is heavily constrained by public infrastructure on all sides. Some context to the situation of the site within the Mission District of San Francisco can be seen in Figure 4 on the next page.

Site analysis, depicted in Figure 4 on the next page, reveals the 1,100 foot Transbay Tower and 650 foot Millennium Tower directly southwest of the building along with the Transbay Transit Center. 350 Mission houses a single client, SalesForce.com, and will provide the public with restaurant and leisure services on the ground floor lobby. The building's proximity to the Transbay Terminal will further increase the pedestrian traffic and interest, in and around the building site along with Mission St and Market St. connecting the building to other important parts of the city.

Integrating the structure within the architecture in an unobtrusive manner was a key goal of the structural team; this coupled with site analysis gave rise to the following considerations:

- Preserve southwest cantilever to maintain architectural connection to the new urban center being constructed
- Maintain an open and tall lobby space to directly engage the public.
- Assist in providing opportunity for architectural enhancement and a unique identity to the new 350 Mission.

Furthermore, an analysis of the geotechnical report prepared by Treadwell & Rollo revealed the presence of the Colma sand layer at a variable distance of 40-95ft below the ground surface, relating to approximate elevations of -37 to -92 feet. The Colma sand layer is suitable to support the expected loads of the existing design via a 10ft thick mat slab. The structural team took note of this for the foundation design phase discussed in Section 4.4.

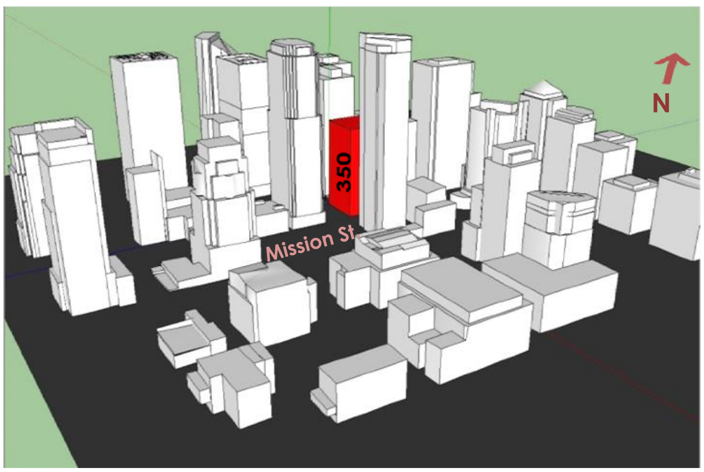


Figure 4: Site Context in Downtown San Francisco

4.0 Structural Systems and Solutions

4.1 Code Analysis

In implementing design processes for 350 Mission, the structural team considered all applicable codes and standards as required by the San Francisco Building Industry Commission (BIC). A full description of all relevant codes, standards and amendments can be found in Appendix B.

4.2 Gravity System Design

4.2.1 Specific Gravity System Goals: The first system design the team completed was the analysis and design of the building gravity system. It was decided through discussion with the design team, that a lightweight efficient gravity system would be most beneficial. Quickly establishing a gravity system allowed the structural team to focus on the high performing lateral system.

The Structural Team began by laying out gravity system specific goals and considerations by reviewing their list of project goals/responsibilities. These gravity system specific goals included:

1. Structural:
   a. Reduce seismic weight
   b. Post-earthquake safety
2. Architectural:
   a. Maintain high floor to ceiling height for occupant comfort
   b. Avoid column penetrations in the occupied space surrounding the core.
   c. Maintain Southwest cantilever for connection to the important urban fabric of the Transbay Terminal area.
   d. Provide stunning views to outdoors.
   e. Open lobby to engage community.
3. MEP:
   a. Reduce in-fill member sizes throughout the floor plan as smaller "tap-off" service ducts may frequently cross their span.
   b. Maintain high floor to ceiling height.
   c. Maintain ample plenum space
   d. See Mechanical and Electrical Reports for detailed plans and designs
4. Construction:
   a. A lightweight, durable system
   b. Organized layout for simple erection
   c. Optimized members for reduced waste

4.2.2 Gravity Loads: The gravity system was designed with the required loading conditions per ASCE7-10. The un-factored gravity loads for a typical floor design were: Dead Load = **58psf** and Live Load = **100psf**. A detailed description of all loading can be found in Appendix C.

4.2.3 Comparison of Alternative Systems: The structural designers first analyzed the current gravity system (flat plate post tensioned concrete slab) design to achieve baseline performance values which could be used later for comparison purposes. Alternative systems such as concrete beam-slab and flat plate slab without post-tensioning were considered but quickly dismissed based on their depth, weight, and need of additional interior columns.

In evaluation of the current post tensioned slab system, the team acknowledged the added value of the extreme floor to ceiling heights to enhance user comfort. However, the structural team wanted a system that reduced the seismic weight of the building, so they quickly decided on steel as the material of choice. Another positive of a steel system is the reduction of carbon pollution during manufacture, compared to a similar concrete design. It became clear that in order to meet their high-performance requirements, including a Net-Zero design, the structural team would choose a composite slab-on-metal deck/composite steel beam solution for the gravity structure.

Changing the design from a PT concrete slab to steel led to careful coordination with the MEP designers, since steel beams would greatly reduce available plenum space, as seen in Figure 5. Discussions were held to weigh the pros and cons; and it was decided that the systems would be possible to integrate, and a steel design would provide the best solution moving forward. The team considered the possibility of both acoustic tile drop ceilings and exposed ceilings being implemented over the building's lifetime.

In order to achieve the required 2-hour fire rating between floors, it was determined that a 3 ¼" lightweight concrete topping on a 1 ½" metal deck (4.75" thickness total) would suffice for the typical floor. The specific type chosen was a Vulcraft 1.5VL19 (or allowed equivalent).

The structural team decided to enforce a minimum floor beam size of W10X12 in order to reduce vibration and connection issues. To optimize the beam and member sizes, the team chose only sections from a list of economical, common W-shapes provided by AISC, reducing waste and increasing efficiency.

When laying out the composite steel framing for the gravity system the structural team looked at a two possible configurations and analyzed their positive and negative features. The team used RAM Structural System (RAM) to optimize the layout and beam sizes, and performed manual calculations to check the design. (see Appendix E for detailed explanation)

Angular Layout: First, the team considered what they called an "angular layout" due to the angular nature in which the girders, running to the columns supporting the perimeter cantilever beams, framed into the core, as shown in Figure 6 on Page 5. The perimeter column locations were not changed from the design provided by SOM. Ultimately, this layout was proven unwieldy based on constructability issues and large beam depths. The full description and a detailed plan view of this layout can be found in Appendix E and on Drawing S3.

Revised Column Layout: The next layout investigated was what became known as the "revised column" layout, in which perimeter columns were moved in-line with the core for a more regular bay spacing, and ease of purely orthogonal construction. The major pro of this system was the heightened constructability from the lack of angular connections to columns. However, because it increased the cantilever span to almost 42ft. the structural team decided it would not be an efficient design. See Appendix E and Drawing S3 for a detailed plan view of this layout.

Steel Joist System: At the indication of the Mechanical team, both potential gravity layouts were analyzed to check the possibility of using a long span steel joist system. Joists were considered because of the possibility of running MEP systems through the web openings. Quick calculations showed the joists would be quite large, approaching 40" deep in some locations. These sizes proved too large for the team to consider joists as a viable option. Please see Appendix E for verification of this analysis.

4.2.4 Framing Design and Optimization:

To optimize the gravity layout, the structural team decided to meld the positives of both considered beam layouts to create the most effective structural design, as well as seamless interaction with the other

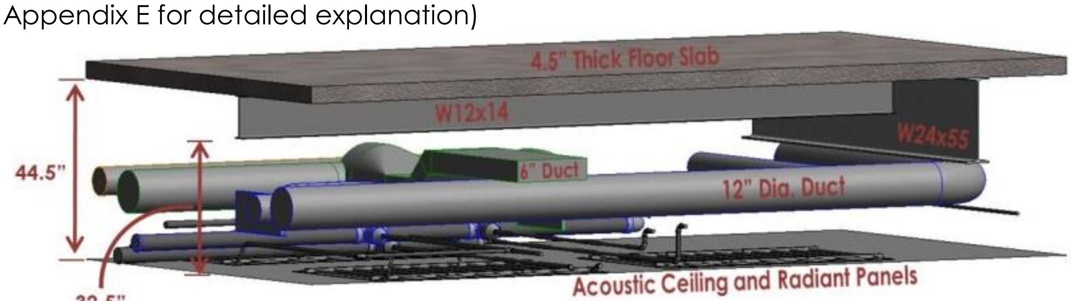


Figure 5: Plenum Spacing and MEP Coordination

Figure 6: Two Potential Floor Beam Layouts and Finalized Layout (l-r: Angular, Revised Column, Final)

building systems. This final layout combined the efficient cantilever framing of the "angular layout" as well as the other perimeter column adjustments for regular orthogonal framing of the "revised layout".

Through the use of Ram Structural System, design shapes, stud quantities, and camber values were again obtained for our final gravity layout. Please see Drawing S4 for a full representation of the final layout features. The new steel design greatly reduced the seismic and gravity loads compared to a concrete structure. Figure 10, on page 7, shows a table outlining the force and weight reductions of the structural team's gravity design versus a similar concrete structure.

Additionally, designing the spandrel beams to not function as girders was important to the project team as it allows for the members to be as small in depth as possible, creating maximum outward views.

During this stage, the structural team coordinated with the mechanical team to delineate the location and path of potential duct runs. Please see Appendix E for some of the calculation checks. Part of the optimization process included incorporating good design practices to aid in the constructability of the framing. This was partly accomplished by having no instances where deep beams frame into shallow girders; which decreases labor costs and built in stress risks associated with fit up issues during construction. Constructability was also taken into account by grouping beams and girders into only a few groups of same sizes to speed up fabrication. Furthermore, framing was optimized by following recommendations from the industry according to Structure Magazine's April 2009 issue:

1. No camber was specified less than ¾" and always in ¼" increments.
2. The camber of beams less than 24' in span, or with webs less than ¼" thick was avoided as they tend to incur damage from local stresses during the camber process.
3. No camber was specified for spandrel members to avoid complications in the connection of the cladding system

Please see Drawing S4 for the final optimized results of our gravity framing layout.

Southwest Corner Considerations:

To accommodate the architectural desire of inspiring views from the Southwest corner of the building, the structural team strove to minimize the sight obstruction of the cantilevered spandrel beams. Two studies were completed: the first analyzing the effects of adding moment connections in the back-span bays behind the cantilever; the second eliminating cantilever action with a tension hanger.

Cantilever Study 1: Connections

Simple 2D models of the cantilever and the potential anchor spans were created in RISA using the section properties optimized by RAM Structural System and checked by hand. These were loaded and analyzed 4 times, adding an additional moment connection in the anchor span column line each time. The team found that making the beam continuous across Column C1 creates the optimal moment reduction on the supporting cantilevered column. Figure 7, on

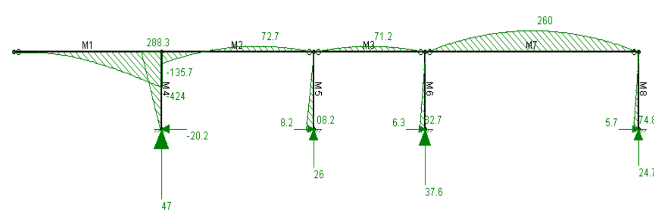


Figure 7: Moment Connection Study – Moment Diagram

the next page, depicts the resulting moment diagram. Please see Appendix E for the details of these analysis results for each case.

Cantilever Study 2: Tension Hangers

Again simple 2D models of the cantilever spans and their anchor spans were created in RISA using the section properties optimized by RAM Structural System. Reaction forces were used to preliminarily size an unobtrusive tension rod to eliminate cantilever action. This was accomplished by tying the tension rod back to Column lines 1 and A, at the above story level. Figure 8 depicts the cantilever spandrels investigated and the geometry of the potential tension hangers. Please see Appendix E for details of this analysis.

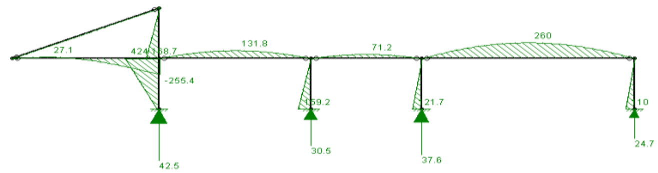


Figure 8: Tension Hanger Study – Moment Diagram

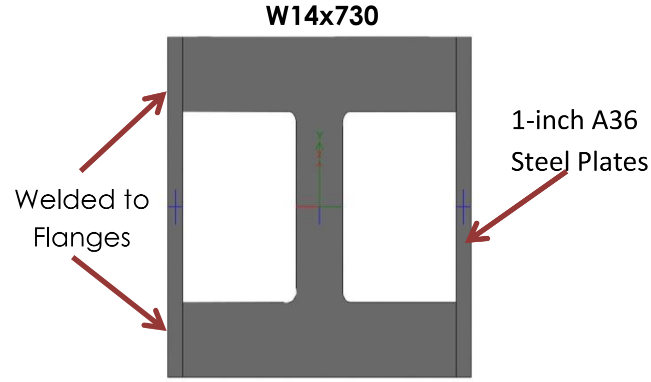


Figure 9: Custom Built-Up Column Section

4.2.5 Column Design:

Perimeter gravity column design was facilitated by both Ram Structural System and ETABS. All columns were optimized to achieve an efficient 75-90% Demand/Capacity ratio, when applicable. The columns were designed to be spliced every two stories (every 26'-4"). This is common practice in high-rise construction to coordinate with the construction lift sequence and schedule duration of a typical floor. They were also checked to make sure columns of this length could reasonably fit on a standard truck bed bringing materials to the site.
Typical column sizes ranged from W14x43 at the roof level to W14x730 at the lobby level. Smaller section depths were possible at the top two levels; however with the switch to a smaller cross section comes a more expensive splice connection. The structural team was advised by the construction team that the costs of those connections generally outweigh the savings achieved by using a smaller section.

A W14x730 column size was not sufficient to support the large bending and axial force interaction induced by the Southwest cantilever, so the structural team designed a custom built up section to handle the loads. The custom section consisted of a W14x730 with two 1 inch steel plates welded to the exterior as depicted in Figure 9. This design was verified with a 2D ETABS model. Please see Appendix E for detailed calculations.

4.2.6 Connection Design:

Connections were designed according to AISC recommendations, including seismic provisions. Two columns, with multiple beams or girders framing in at the same level, were chosen as critical connection locations. Please see Appendix E for specific locations of the designed connections. The design strategies were heavily influenced by cost, constructability, and seismic behavior. The resultant shear double angle designs allow for significant shop fabrication, decreasing construction time and cost, and increasing safety benefits. Tekla Structures was used to assist in the detailing and design of the connections to a fabrication level of detail, providing descriptive 3D connection details (Figure 18 on Page 13 shows Lateral example). Expanded details, calculation summaries, and visual representations of the gravity connections can be found in Appendix E and in the drawings.

4.2.7 Gravity System Summary:

By closely following the goals set at the beginning of the project, the structural team was able to efficiently design a gravity structural system that met all structural requirements and allowed other building systems to reach their full potential. The steel framing system achieved a much lower weight than a comparable concrete system, greatly reducing the seismic forces acting on the lateral system. This reduction data is displayed in Figure 10, on the next page. The optimized floor beam system allows the MEP designers to run duct work and utilities without experiencing conflicts over plenum space. This coordination is achieved despite the long beam spans desired for creating an open and architecturally pleasing space.

4.2.8 Software Utilization: To aid in the team's project goals of collaboration and integration, the structural team worked to keep the gravity design model current in Revit 2014. This practice allowed the team to assign parameters to all members such as: framing tags, sizes, stud quantities, connection types, and camber values. The designers were then able to easily develop plans, sections, schedules and details for the structural design.

Each discipline was able to coordinate the evolution of their systems with the gravity framing. Good communication was important, as all team members needed to be aware of system updates in the model. Descriptive and systematic file naming was also important in keeping the entire team working on the most recent and correct models. Similar processes were completed for the lateral and foundation designs, as discussed in later sections. For examples of these documents, please see Drawing S2, S6, S7.

4.3 Lateral System Design

4.3.1 Lateral System Goals: With the gravity system designed, the structural team then focused their attention on the high performance lateral force resisting system. The process continued again by reviewing our project goals/responsibilities to determine our specific aims for the lateral system. These included:

1. Structural
   a. No structural damage and immediate occupancy post major design level earthquake
   b. 50% code allowable Drift.
   c. Architectural enhancement
2. Architectural
   a. Accommodate all existing core openings
   b. Do not impede interior layout
   c. Reduce thickness of core thereby adding rentable space
   d. Visible structural components should be architecturally appealing
3. Mechanical and Lighting/Electrical
   a. Design to allow for ease of access and coordination to core as compared to existing solid shear wall.
4. Construction Management
   a. Coordinate safe construction of core ahead of gravity system

4.3.2 Comparison of Alternative Solutions: The ultimate arrival and decision of the lateral system was the product of a 2 stage process:

1. ***Empirical Evaluation*** of systems to meet our project goals based off research→choose system(s)
2. ***Reexamine Goals & Additional Preliminary Analysis*** upon the chosen top systems and specific building geometry→Modify and adjust to meet goals, continue with more in-depth verification analysis of best system for all.

Empirical Evaluation:
Immediately after forming the project goals, the structural team began researching potential high performing lateral systems. The team wanted to analyze what systems would meet these goals and work cohesively with the other discipline's designs. For feasibility purposes, the structural team set out to empirically evaluate the suitability of the existing system and various others to meet the team's goals.

The team began by evaluating the geometry of the building and the existing lateral system. The considered centering the core in the footprint of the building in order to decrease torsional irregularities and increase lateral efficiency. Ultimately, the decision was made not to move it as it would cause a loss of 250 sf of floor space directly adjacent to the appealing southwest corner.

Furthermore, the structural team built a preliminary model of the existing structure's lateral system. This was done to gain an understanding of the rough order of magnitude of drifts and forces generated by the existing structure and to serve as a baseline for comparisons of future models and systems analysis. This rough model was verified with an Equivalent Lateral Force (ELF) Procedure. Please see Appendix D for samples of this preliminary analysis.

As information was gathered on numerous systems, the team began formulating a comparison matrix for the pros and cons of various systems. Potential pairings of systems were also brainstormed and empirically analyzed. A detailed overview of this research can be seen in Appendices D & F; a sample of which is shown below in Table 1.

Following this research, the structural team began to look at pairs of systems that would work well together. The team thought about each pair from a viewpoint

Steel Gravity System Comparison		
Category	**Notes:**	**Outcome:**
% Reduction in Seismic Weight	Approximate ELF Procedure	68
% Reduction in Base Shear	Approximate ELF Procedure	68
% Reduction in OTM	Approximate ELF Procedure	68
% Reduction in Gravity Loading to Foundation	Approximate from Revit Model	47
Smallest Beam	Bay Adjacent to Floor Cut-Out Space	W8x10
Typical Beam Size	In Bays	W12x14
Largest Beam	At Cantilever	W24x76
Potential Floor to Dropped Ceiling Height (22"MEP allowance)	At Girder and in Bays	9' 1" at girder location 10' 1" in bays
% Reduction in Carbon Dioxide Emmissions	Using SOM Environemental Analysis Tool	?

Figure 10: Reduction in Weight and Forces for Steel Design vs. Original Concrete Design

of how they could complement each other performance-wise, their impacts for other engineering disciplines, and whether or not research on these systems showed them commonly used together in industry. The team came up with three potential paired systems for the building as seen in Table 1 below.

In examination of these three potential pairings the structural team reexamined their specific goals for the lateral system. Conversations with the MEP members helped lead to the choice of braced frames as the optimal system for the core. Table 2 on the next page lists the main factors for each discipline in settling on a Braced Frame core design.

Reexamine Goals and Preliminary Analysis:
The structural team wanted to start with strength based-design to note size, efficiency at controlling drift, and stress distribution; and then use this knowledge to evaluate their system choices. The structural team continued to analyze the core model with the other two lateral element schemes. The perimeter moment frame scheme and perimeter bracing scheme can be seen in Appendix F. They then compared the analysis results. Figure 12 shows the effectiveness of adding these systems to the core at controlling drift. As expected the exterior diagonal bracing was most efficient.

While the perimeter moment frame and core model was effective, the resulting member sizes (spandrel beams) were 36" deep in multiple locations. This was a direct reflection of our long spans between perimeter columns which was important to the team for architectural and daylighting reasons. For details of this analysis please see Appendix F.

In recognition of the large member sizes necessary to make a perimeter moment frame have a significant effect on the size of the core the structural team moved away from this solution. In regards to the perimeter diagonal bracing, the structural team noted the challenges with developing a working load path for this system (analysis up until now had been all rigid diaphragm analysis). They also noted its extreme effectiveness at controlling drift and alleviating demands on the core as well as the unique opportunity for architectural enhancement. Figure 14 shows the bracing scheme analyzed.

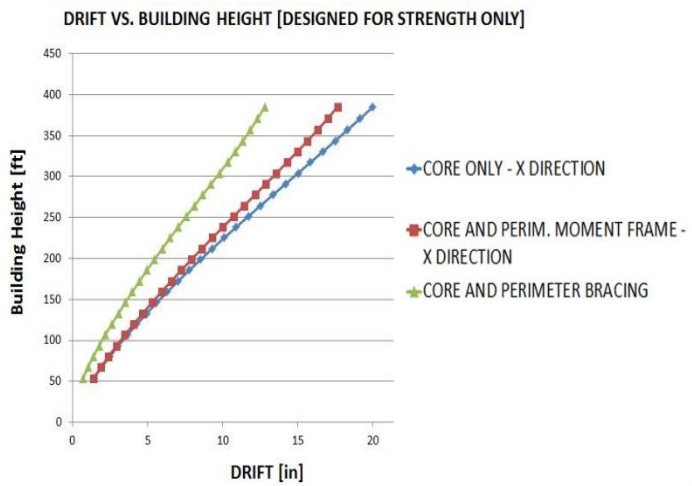


Figure 12: Drift Control for Potential Lateral System Options

Therefore they proceeded by modeling the core, accommodating all necessary penetrations, and running design iterations and Modal Response Procedure (MRP) analysis in ETABS. The results confirmed that a concentrically braced frame core could indeed meet the desired high performance drift requirement. However it would result in large uneconomical members, and a structural system that offers little reduction in core size and architectural enhancement. At that point the structural team felt that while they were meeting the high performance drift requirement, proposed solution had little potential to add much value to the project.

	Concrete Shear Wall with Post-Tension Slab	Base Isolation and Steel Plate Shear Walls	Braced Frame Core with Viscous Dampers
Pros	- Contractor Familiarity - Ease of Construction - Cost Effective - Slab Thickness	- Durable and Reliable - Reduced Overturning Moment - Decreased Drift - Speedy Construction - Decreased Building Weight, - Increased Square Footage	- Reduced Seismic Weight - Steel Floor – Tenant Benefits - Coordination with MEP - Overturning Moment - Passive System Possibilities - Decreased Repair
Cons	- Slab Weight and Drift - Overturning Moment - Tenant Limitations - Slow Construction - Post-Event Occupancy	- Buffer Zone Required - Flexible Utility Entries - Unjustifiable Over-Design - MEP Coordination - Fabricator Issues	- Increased Initial Cost - Special Connections - Architectural Clashes

Table 1: Potential Lateral System Pairings and Pros/Cons of Each

Therefore in order to realize the project goals associated with this system the structural team began looking at methods to mobilize the perimeter and full depth of the building to alleviate the core. Originally three methods were considered to alleviate the core: outrigger at the top of the building, perimeter diagonal bracing, and perimeter moment frames.

The structural team quickly realized that given the geometry of the gravity system and cantilevered south west corner that an outrigger on any level would have significant spatial impacts to the floor plan and would require an overhaul on the perimeter gravity column placement as would a belt truss. Thus, this option was ruled out. See Appendix F for an image displaying this concept.

Structural	Symmetric layout of core conducive to braced frames ; shown in Figure 12
Mechanical	Allows for simple duct system coordination
Electrical	Allows for simple utilities coordination
Construction	Simple construction and modularization ability
Architecture	Avoids clashes with openings into the core

Table 2: Disciplinary Pros of Braced Frame Core Design

The layout of the braces was intended to be as elegant as possible, as seen in Figure 13 on the next page, allowing for an articulation of the buildings high seismic performance and unique identity with respect to surrounding buildings. It should be noted that the braces were configured such that they avoid impeding the southwest entrance. Their multiple floor spans also have less of an impact on exterior views to the occupant then the deep spandrel beams of the perimeter moment frame system.
To arrive at a decision the structural team conversed regularly with the other engineering disciplines about the effects of various systems on design coordination. Furthermore the structural team was not happy with a perimeter moment frame solution as the spandrel members (beams of the perimeter moment frames) were up to 36" in depth. This was not in line with the structural team's goals of preserving large views to the exterior. The exterior diagonal system was also determined to allow the maximum amount of daylight to assist the lighting electrical designers in their interior space design. The system also was purposed to be easily integrated with the façade design and performance requirements of the mechanical engineers. Through dialogue with the construction team it was also determined that the exterior system would pose significant challenges during the erection of the super-structure due to load distribution issues. Please see the Integration Report and Appendix F for more on this matter.

Decision
Therefore, it was decided that the most conducive decision to the project goals was to proceed with a braced frame core and exterior bracing scheme as shown above. This system would be designed and seismically detailed for full strength and to meet the 50% drift requirement; providing an effective structure for handling seismic loads and allowing for Immediate Occupancy post MCE.

4.3.3 Braced Frame Core and Exterior Bracing:

Load Path Determination

Utilizing a mega bracing system about the exterior of the building presented certain challenges in confidently identifying a total structure load path. Figure 14, on the next page, shows the breakdown of the team's schematic understanding of the system.

First lateral loads are excited in the floor diaphragms. They are then transferred to the core ("secondary system"). This secondary system spans the full height of the building but is further restrained at key nodal levels 10, 20, and 30 by the exterior bracing "mega system". Conceptually the core can thus be thought of as a continuous beam simply supported by the mega exterior braces at key nodal levels. These "core modules" act between the key nodal levels. Once lateral load has been "kicked" out at key nodal levels into the exterior braces it follows the braces to the foundation. This method was recommended by an internationally known consultant with 40 years of high rise experience who we periodically conversed with.

The transfer of lateral forces (only at key nodal levels) from the core to the exterior braces is accomplished by a thickened floor diaphragm with two-way reinforcing to serve as a true rigid diaphragm. With the load path now identified, the structural team could begin to break it into its components for preliminary sizing.

Core Design Modules

Preliminary design sizes were obtained by modeling each of the core modules that span between rigid diaphragms connecting them to the exterior bracing at major nodal levels. These modules were built for each 10-story section, and in each primary direction. Loads from a previous Modal Response Spectrum Analysis (MRSA), accounting for accidental torsion, were manually applied to the story diaphragms.

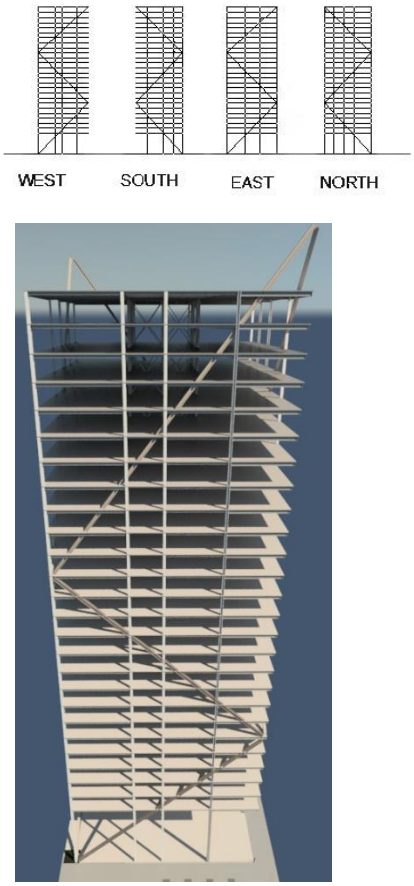


Figure 13: Exterior Bracing Elevations and

The top and bottom story diaphragms were restricted in their movement both in and out of plane. This restraining reaction represents the mega systems restraining effects on the secondary core system. Design iterations were run in ETABS and preliminary sizes of the core braces obtained. Figure 16 depicts these design modules, including the forces transferred to the exterior braces at each nodal level.

Note that only seismically compact and approved ductility sections according to the AISC Seismic Design Manual were used. Also all analysis and design iterations were done from a performance based design philosophy using a response modification factor, R=1. This is because the structural team wished to ensure that the building is not incurring inelastic deformations over its lifetime. A more detailed annotated depiction of these core design modules and assumptions/justification for modeling process can be found in Appendix F.

Preliminary Exterior "Mega" Brace Design

With preliminary sizing of the core complete, next the structural team preliminarily sized the exterior mega bracing. This was completed by taking the restraining reactions of the rigid diaphragms from the ETABS Core Design Modules and resolving those reactions into the brace forces. The braces were sized for both tension yielding and rupture of the net section as well as for compression. It was assumed that the floor diaphragms and spandrel beams that the exterior braces cross at each story will effectively brace it for compression. Figure 15, on the next page, also shows this use of restraining reaction of the floor diaphragm to size the exterior brace. Detailed sample of this analysis along with annotated depiction can be found in Appendix F.

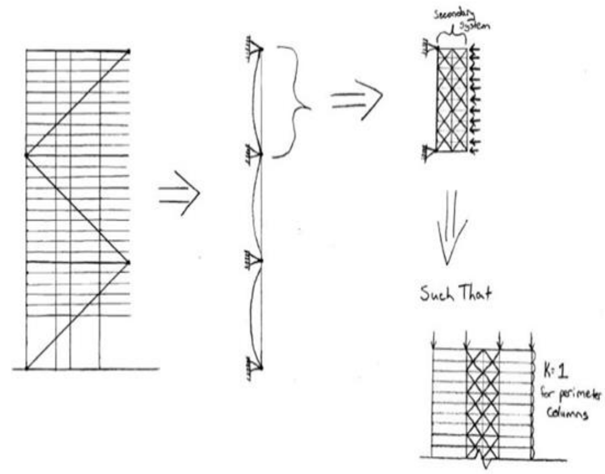


Figure 14: Schematic of Seismic Load Path

Re-Analysis of Total System, Final Sizing, & Hand Checks

Now with preliminary sizing completed of both the "secondary" core and the "mega" exterior braces, final sizing could be made. The structural team could then fully compare the system with the mega exterior braces and without them (where the design began) allowing for verification of reduction of core size, added drift control, and the potential for architectural enhancement.

All of the preliminarily sized core modules were assembled back together into a whole building model in ETABS. The preliminarily sized brace members were put in along with the designed gravity columns from RAM Structural System. Spandrel members were included in the model as well in order to account for their stiffening effects as the exterior braces cross them. Model response spectrum analysis was performed on these models using the Maximum Considered Earthquake (MCE) spectra supplied by the geotechnical report.

Roughly 50% of the preliminarily sized members, mostly ones concentrated in the first 10 stories, were found to be overstressed. These members were adjusted accordingly to obtain final sizes. Figure 16, shows the full reassembled Lateral system model.

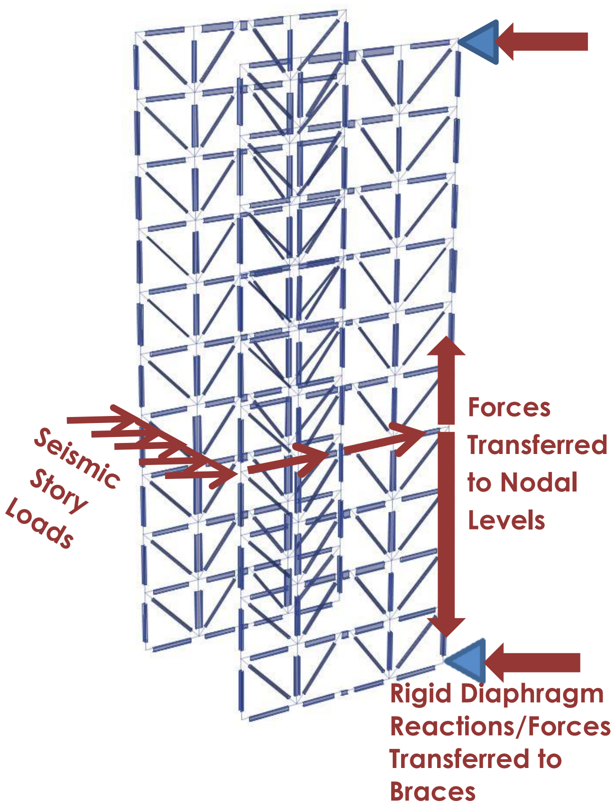


Figure 15: Schematic Load Path of Core Modules

Exterior mega braces ranged from the same built up section used for the columns to W14X120. Most notably the core was relieved enough to allow all brace members to be HSS shapes instead of the thicker W shapes needed for the previous dedicated core lateral system.

With the complete system modeled, analyzed, and design sections adjusted, the structural team fine-tuned some of the perimeter columns and spandrel beams to account for their interface with the exterior mega braces that cross them. Notably, the three perimeter corner columns shown in Figure F.9 in Appendix F were required to engage in significant tension not originally accounted for by the column design completed in RAM Structural System. These columns needed to be increased in section size in order to handle this increased tension. Please see Appendix F for an in depth explanation into the structural teams investigation into their complete lateral system model.

Now the structural team had a complete designed lateral system model for a concentrically braced frame core both with and without exterior mega braces. The structural team now compared the two designs to truly verify that the exterior mega braces were adding value to the system and not just additional strength.

With the addition of the exterior mega braces to the core, the lateral force resisting system as a whole saw a **48%** reduction in steel weight. This in turn represents a $**8.6 million** savings over the core alone system. Furthermore the mega braces relieved lateral demands enough on the core allowing its thickness to be reduced on average by about **24"**. This equates to an increase in rentable square footage of about **9100 ft²**.

While these great numbers do not include the increase in material required to create the rigid diaphragms or the potentially complex connections required by the mega brace system, they are still revealing of the efficiency and applicability not only to this specific building but also to the high performance goals of this project. Achieving these results served as an excellent verification of system choice and concept.

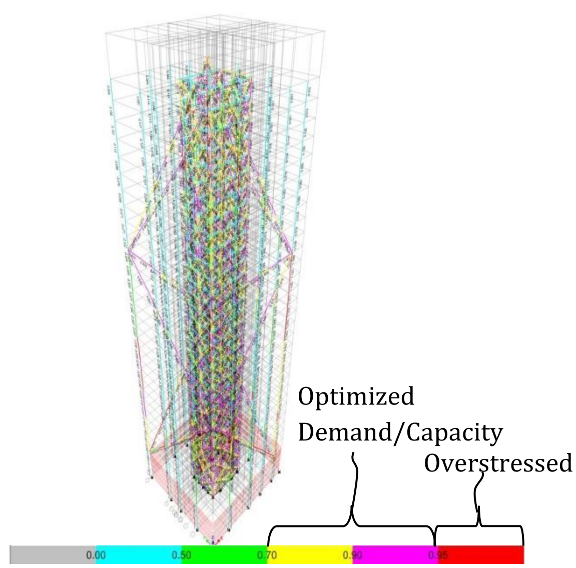


Figure 16: Fully Assembled Lateral

Iterative P-Delta Analysis

A major assumption the structural team made in the gravity design phase was that the perimeter gravity columns are braced at every level. This allowed the team to use an Effective Length Factor of K=1 when considering second order effects on the columns. In order to prove this assumption, the team needed to verify that the core is stiff enough to arrest the translational movement of the diaphragms, so they can provide effective bracing. An Iterative P-Delta Analysis was run in ETABS with gravity loads applied to all columns and a preset convergence tolerance to make sure the perimeter columns were adequately braced. A full description of this process can be found in Appendix F.

Lateral Connection Design
The structural designers developed the scope of the lateral system to include a basic connection design for the concentric braces in the core. Seismic provisions were considered and the full detail of this connection can be found in Appendix F. Tekla Structures was used to model the connections to a fabrication level detail as shown in Figure 17. The accurate detailing and modeling of these connections was not only important from a seismic point of view but also from a coordination point of view because the MEP vertical distribution and associated branch offs occur around these connections in the core.

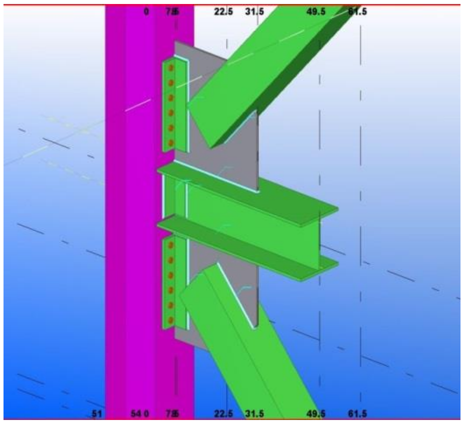


Figure 17: Tekla Model of Connection in Core Braced Frame

First Floor Diaphragm Design
The lobby floor design also presented some unique considerations. The floor slab needed to be sized and reinforced to act as a transfer slab in carrying lateral loads to the foundation walls; as well as house a radiant slab heating system. The structural team worked with the mechanical team to see that both disciplines desires were fully met.

4.4 Foundation System Design

With the superstructure design completed, the structural team moved their attention to the foundation system. Up until now the substructure had remained the same as the existing 350 Mission St: a 1ft 10 in thick perimeter concrete wall resting on a 10ft thick mat slab. The structural team began by examining specific foundation system goals per each discipline:

- Construction: design system with minimum schedule and site impact.
- MEP: design system that allows for an additional 3-10 ft of depth on bottom floor to help house the biomethane digester system without impacting the architectural layout
- Structural: Design system that provides adequate and safe supporting conditions for the building, its systems, and occupants.

As stated in the Building and Site Analysis section, one of the first activities the structural team took part in at the start of this project was a detailed examination of the geotechnical report provided from Treadwell & Rollo. The structural team noted their recommendation that the building be founded on a mat foundation bearing on the dense to very dense Colma Sand Layer.

The structural team proceeded to investigate the legitimacy of resting their proposed system on a mat slab. Switching to a steel superstructure resulted in a 48% reduction of gravity loads (see Figure 10 or Appendix E). However, the team also noted that a reduced mat slab thickness resulting from their gravity loads may present problems in handling overturning moment of the building. The structural team began by using the results from their MRSA model and RAM SS gravity analysis to design a mat slab with the necessary reduced thickness for the housing of the biomethane digester system. The team successfully designed a 6ft mat slab (4ft reduction from original system) in order to provide a larger floor to ceiling height in the basement for the biomethane digester as shown in Figure 18. Please see Appendix G for calculations and validation on this design.

Next the structural team moved to the design of the perimeter subgrade foundation walls. They first schematically laid out how forces were to be transferred from the superstructure to the foundation. While the core was analyzed and intended to continue down to the mat slab the transfer of the large diagonal tensile and compressive forces in the perimeter mega braces into the foundation walls had to be resolved. Schematics of this can be seen in Appendix G.

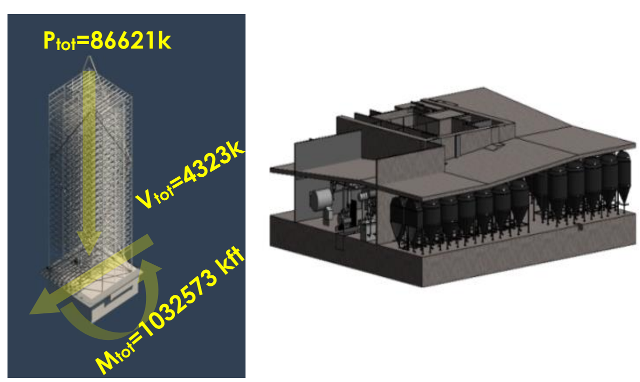


Figure 18: Visualization of Reduced Mat Slab for MEP

A two fold solution was developed. In order to transfer the horizontal component of the large mega braces along the top of the basement walls, the first floor level (Lobby Level) would need to serve as a rigid diaphragm. That or the perimeter areas must at least be rigid enough to ensure this transfer.

This Lobby Level diaphragm however also intended to act as a transfer slab for all lateral loads developed in it to the outer walls as an "indirect outrigger" helping to engage the perimeter subgrade walls in lateral resistance as is commonly done. Furthermore the presence of an in floor radiant heating and cooling system necessitated an increase in thickness to ensure proper housing and protection of this system in the event of an earthquake. This lobby level diaphragm was always intended to be designed as a rigid diaphragm for these reasons so the solution of transferring the large diagonal mega brace forces into it fit well with the other design decisions being made.

Second, the large tensile/compressive vertical component of the mega braces needed to be transferred down through the subgrade walls to the mat slab. In recognition of the fact that such a large concentrated force acting on the end of a wall would surely require an immensely thick wall; the structural team decided upon the elegant solution of simply encasing a steel column in the wall at the four corner locations where braces connect to the top of the wall. This column effectively transfers the vertical component of the brace to the mat slab while the rigid diaphragm effectively transfers the horizontal component along the top of the perimeter walls intended to be mobilized for lateral resistance anyway. Either an encased section at these corners or a heavy amount of reinforcing would have been required any way as boundary elements for the effective shear transfer from the horizontal force component of the mega braces as shown in Appendix G.

Additionally the perimeter foundation walls needed to act as retaining structures. The structural team therefore had to design the foundation walls for the combined effects of the shear along their length and the out of plane soil loads as schematically depicted in Figure 19. The structural team notes that the perimeter foundation walls should be designed as a tank according to ACI 350 due to the fact that the water table is above almost their entire depth. A necessary amount of reinforcing was found in two curtains both for shear wall and retaining wall behavior for a final detailed design (Appendix G).

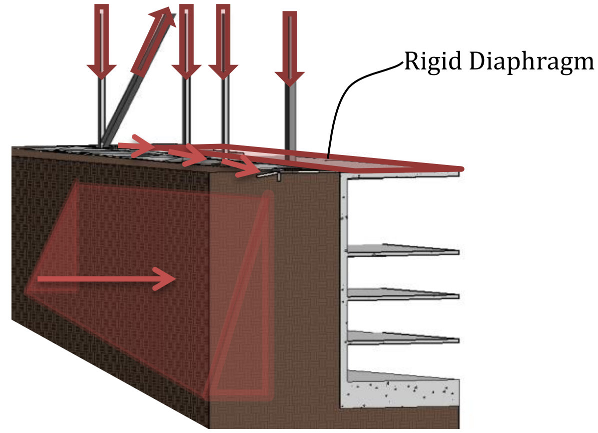


Figure 19: Schematic Loads on Basement Shear Walls

4.5 Building Enclosure Design

In addition to the main building substructure and superstructure the structural team also collaborated with all other project team members to develop a high performing façade system. In regards to the façade design the structural team began by defining their high performance requirements. The structural team identified the following desired characteristics:

- Architectural: The façade should actively engage the public by expressing the slenderness and unique identity provided by the mega braces.
- MEP: The façade should accommodate the necessary glazing thickness and coatings for high performance lighting, electrical, and thermal qualities.
- Construction: The façade should lend itself to quality control, labor sensitivity, and speed of construction.
- Structural: The façade should accommodate all movements necessary to ensure safety and function during and after a major earthquake.

First the structural team set out by garnering an understanding of typical geometries and sizes of high rise curtain wall elements. Through early collaboration with the Electrical team's shading layout a preliminary mullion spacing of 5ft was decided upon. A MWFRS study then allowed for the structural team to calculate an expected mullion depth of 7-8 inches which was further supported by potential product research. These calculations can be found in Appendix H.

The structural team proceeded to investigate, calculate, and document all expected potential movements of the façade. A "Movements and Tolerances Report" was written and presented to the other disciplines. This report calculates expected vertical and horizontal movement from deflections/drift, thermal expansion of framing elements, and construction tolerances and can be viewed in Appendix H.

With the expected movements/tolerances of the façade fully investigate as well as preliminary sizing completed the structural team conducted detailed product specification research. Various testing results conducted by Memari et. al. led the structural team in the direction of selecting a 4 Sided Structural Sealant Glazed (4SSG) system with Rounded Corner Glass (RCG). Specifying rounded corner glass has been shown through American Architectural Manufacturers Association (AAMA) 501.6 testing to increase a curtain wall system's glass fallout drift capacity as much as 50-90%. The functional difference between RCG and the conventional square corner glass can be seen in Figure 20, and is explained in detail in Appendix H. Furthermore using 4SSG systems over common dry glazed or gasketed systems has been shown through similar testing to increase glass fallout drift capacity up to 146% (Memari et. al.).

Figure 20: Square Corner vs. Rounded Corner Glass

In conjunction with the other disciplines criteria the structural team then proceeded to select a curtain wall system: the Kawneer 2500 PG Unitwall. In the absence of AAMA 501.6 racking test data for the chosen product the structural team verified its cracking and fallout drift capacity against the expected movements and tolerances report in accordance with ASCE7-10 section 13.5.9. These calculations and verification can be found in Appendix H.

With a product selected the structural team proceeded to detail its anchorage to the structure. The product cut sheet provided simple representative details of single angle connection from the floor slab to the vertical mullions. A representative view of the connection scheme is shown in Figure 21. The structural team checked not only the anchoring bolts lateral capacity to handle the induced inertial earthquake loads from the façade but also the slab on metal deck overhang's capacity to support the façade without inducing torsion into spandrel members. Finally, the structural team fully detailed the façade's anchorage to the structure as can be seen on Drawing S8. All of these calculations and supporting images can be found in Appendix H.

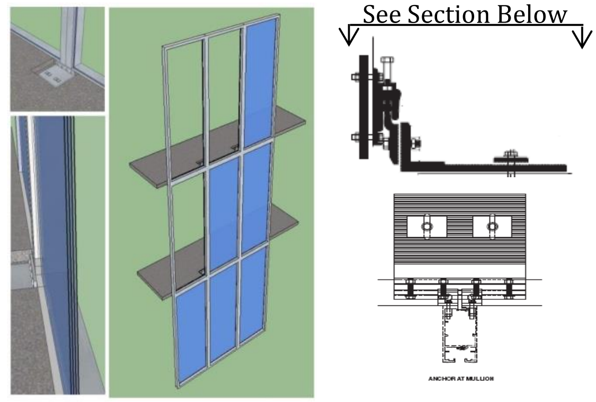


Figure 21: Façade Connection Details (Details from Kawneer)

5.0 Sustainability and Environmental Analysis

With the Near Net-Zero design goal in mind, the structural team considered sustainability issues with all of their design decisions. The team looked at this issue as not only a way to improve building efficiency and lifecycle cost for the owner; but also as an opportunity to engage and educate the public.

Through collaboration with the construction team the structural team was able to reduce the environmental impact of their designed systems by ensuring that all required materials/services could be obtained locally. Figure 22 depicts the locations of various potential concrete sub-contractors, masonry subcontractors, steel erectors, and steel fabricators all within 500 miles of the building site per the

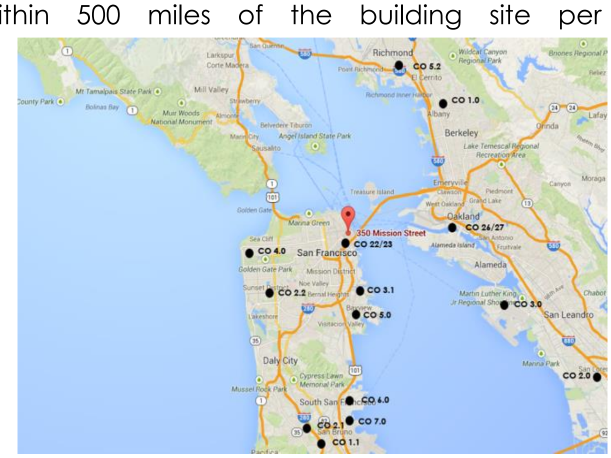


Figure 22: Sub-Contractors and Fabricators Near Building Site

requirements of the Regional Materials LEED credit. The structural design's contribution to the building's LEED rating is outlined in Appendix I.

Additionally, the structural team's conscious design decisions led to a highly constructible system through methods such as prefabricated core segments, prefabricated rebar cages, and reduction of material usage in the foundation (see Construction Report for full explanation of constructability). These design decisions cut down on the energy required to construct this building leading to a reduced environmental impact.

7.0 Conclusion

The tasks set before the structural design team for 350 Mission were diverse, complex and crucial to achieving a high-performance design. The designers analyzed the project guidelines and requirements, and developed goals that ultimately drove their design to success. Not only was the structural system lightweight, efficient and innovative; but it operated seamlessly with the rest of the building's engineered systems and architecture. Table 3 below shows how the structural teams design decisions met their original project goals outlined in **Section 1.0**; and related to the team ideals of Performance, Endurance and Connectivity outlined in **Section 2.0**.

In response to the project requirement of Near Immediate Occupancy after a major earthquake, the structural team designed a system that efficiently handles seismic loads while reducing the building drift to **33-inches** at the top of the building; meeting the **½ of the code allowable limit requirement**. Creating a system that performs in this way helps meet the project goal of improving the life-cycle efficiency of the building. The structural team's elegant exterior mega brace system not only creates a unique identity for 350 Mission but also provides an accentuated connection to the urban fabric of the Transbay Terminal Area. Furthermore, the mega brace system relieves lateral demands on the core allowing for a **9100 square foot** increase in rentable space for the building.

The structural team's efficient gravity system design allowed for an estimated **48% reduction** in the weight of the structure. While this not only helps to alleviate seismic demands and increase the buildings performance from an earthquake standpoint, it allowed for a **4-foot reduction** in thickness of the foundation mat slab. This reduction in thickness not only reduced excavation needs but also increased the usable volume of the subgrade levels in order to house a high performing mechanical system. This integration further improved building life-cycle efficiency and added value to the project.

In addition reducing the weight of the structure, the structural team's typical floor layout considered architectural features, allowing for the preservation of the **29-foot southwest corner cantilever**. This cantilever opens 350 Mission in the southwest direction, further highlighting its connection to its urban environment. It helps enhance the architecture of the building by allowing for an extremely open lobby at the corner of Mission St and Fremont St. This dramatic cantilever effectively engages the streetscape and public in the key direction of the future Transbay Terminal.

The façade performs effectively for both the seismic and thermal situations found in the San Francisco area. Its ability to provide open views with minimal framing, while accommodating all necessary movement, thermal, and glazing characteristics make it truly high performing. Additionally, its semi transparency highlights the **architectural enhancement** of the diagonal mega brace system, creating public interest in the building and connecting 350 Mission to the surrounding urban environment.

The structural team was able to accomplish this signature design while keeping a holistic approach first and foremost in their minds. With all disciplines working collaboratively and capitalizing on the full potential of BIM software and workflow methods; the team produced an exceptional design solution, endorsing quality, efficiency, safety and functionality.

Integrated Project Ideals	Responsibilities/Goals	Design Result
Performance	50% Code Required Drift Limit	33-inch Total Building Drift
	Mat Slab Reduction	4-foot Reduction of Slab Thickness
Endurance	Eliminate Structural Damage	Elastic Design of Structural System
	Immediate Occupancy Post EQ	Limited Drift Minimizes Helps Minimize Damage
Connectivity	Coordinate with MEP Special Systems	Biomethane Plant in Added Basement Space from Mat Slab Reduction
	Building Façade Design	Façade Designed for Seismic Loading
	Exterior Mega Braces	Architectural Identity and Seismic Demand Reduction on Core→9100sf increase

Table 3: Ideals, Goals and Conclusion Summary

Appendix A: Lessons Learned

Throughout the course of designing 350 Mission, the structural team learned many things. These lessons learned have provided vital help throughout the course of the design process. The structural team is confident that these lessons learned will continue to aid them in their current and future professional success. Outlined below are some of these key lessons:

1. File Structure/Organization is extremely important:
   a. Due to the iterative nature of design, being able to quickly access previous information is crucial for an efficient and effective process. Identifying and adhering to a logical organizing strategy makes for much more efficient workflows. Furthermore the interdisciplinary nature of such work really demands a user-friendly organizational system. Models, spreadsheets, images, and other information that is not stored properly with back-up materials can easily be lost or corrupted without a proper file structure.

2. Industry professionals are a wealth of available knowledge:
   a. Throughout the design process for 350 Mission, the structural team realized that industry professionals have an exorbitant amount of information and knowledge and are often very willing to share their experiences. Often times in design it is necessary to make an assumption about an unknown. However the structural team has learned the value of actively seeking these unknowns through others past experiences as opposed to an inefficient and time wasting "guesswork" motivated design process.

3. Analysis and design software is enormously powerful but MUST be used with caution and understanding:
   a. Throughout the design process the structural team employed various structural analysis and design programs to aid in the design of 350 mission. Though these software platforms and their associated BIM workflows greatly aided the design process, it is of the utmost importance to use them with discretion. It is very important to verify all computer output with some form of manual check and investigate all potential areas of error. An incorrectly modeled structure can still produce analysis results without any warnings.
   b. All team members modelling must be knowledgeable and agreeable upon he level of detail to be modeled, type of model, type of analysis, and type of design to be performed. One of the most common errors in computer modeling of structures is miscommunication among multiple modelers (Solnosky, et. al).

4. Grid layout and geometric organization early on is crucial:
   a. Early on the structural team recognized the fact that they would constantly be moving between different analyses, design, and drafting software. Usually these different platforms have different naming conventions/organizational patterns for the structures developed in them. Developing a neat and effective grid system and sticking to a convention proved vital in keeping information organized and being able to discuss project details from remote locations

5. Effective communication is paramount:
   a. The importance of clear, effective communication cannot be understated. When communicating different ideas across an interdisciplinary team you can never be too black and white. Be as plain as possible and always consider the audiences background knowledge and concerns. Huge amounts of time can be wasted making sure all project team members are up to date and knowledgeable about the project's current state. Setting up regular meeting times, as well standardizing information recording can really help keep all team members up to date.

6. BIM tools and technology are essential for graphic representation and information exchanges:
   a. When engaging in interdisciplinary work, the value of an effective info graphic can go a long way when trying to express an idea or concept. Furthermore many forms of BIM technology have helpful tools in place to aid in the effective exchange of information such as Revit, Tekla, Navisworks, and BIM connectivity capabilities in SAP200, ETABS, and RAM Structural System

Appendix B: Applicable Codes, Standards & Software

Codes and Standards:

- International Code Council (ICC). *International Building Code*. International Code Council, Falls Chruch, Va. (2009).
- California Building Standards Commision (CBSC). *2010 California Building Code*. (2010).
- San Francisco Building Industry Commision (BIC). *San Francisco Building Code*. 2010 Edition (2010).
- American Concrete Institute (ACI). "Building Code Requirements for Structural concrete and Commentary." *ACI Standard 318-11*. (2011).
- American Institute of Steel Construction (AISC). *Seismic Design Manual. 2nd Edition*. (2012).
- American Institute of Steel Construction (AISC). *Steel Construction Manual. 14th Edition* (2011).
- American Society of Civil Engineers (ASCE). "Minimum Design Loads for Buildings and Other Structures." *ASCE/SEI Standard 7-05*. (2010).
- National Earthquake Hazards Reduction Program (NEHRP). "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures." *FEMA P-750* (2009).

BIM and Structural Analysis/Design Software:

- "Autodesk Revit 2014." Autodesk. (2014).
- "ETABS 2013 Ultimate." Computers and Structures, Inc. (2013).
- "SAP 2000 Version 15." Computers and Structures, Inc. (2011).
- "RAM Structural System." Bentley Engineering. (2012).
- "RISA-2D Educational." RISA Technologies. (2002).
- "Tekla Structures Educational" Tekla, a Trimble Company. (2012).

IBC 2009

Title 24: 2010 California Building Code

San Francisco Amended Sections:

1603A.1.5●1603A.1.7●1603A.1.10●1604A.3
●1604A.3.7●1604A.10●1607A.1●1607A.11.5
●1609A.1.3●1609A●1613A.2●1613A.5.6
●1613A.5.6.1●1613A.5.6.2●1613A.6.2●1613A.6.3
●1613A.6.4●1614A.2●1615A.1.1●1615A.1.2A
●1615A.1.3●1615A.1.4●1615A.1.5●1615A.1.6
●1615A.1.8●1615A.1.10●1615A.1.16●1615A.1.21
●1615A.1.26●1615A.1.27●1615A.1.31●1615A.1.32
●1615A.1.34●1615A.1.35●1615A.1.36●1615A.1.37

Appendix C: Building Design Loads, Parameters & Preliminary Analysis

Dead Loads:

Typical floor

Type:	Notes:	Value:
Decking	Lightweight Concrete Slab on Composite Metal Deck; Vulcraft 1.5VLI or Approved Equivalent	37 lb/ft^2
Miscellaneous Concrete Overpour	Account for accidental overpour	1 lb/ft^2
Flooring Finish	Superimposed	3 lb/ft^2
Ceiling		2 lb/ft^2
Lighting		5 lb/ft^2
MEP		10 lb/ft^2
Total		58 lb/ft^2

Table C.1: Typical Floor Dead Load Summary

Facade

Type:	Notes:	Value:
Façade	Line load based on 30 lb/ft^2 façade weight	400 lb/ft

Table C.3: Façade Dead Load Summary

Live Loads:

Typical Floor

Type:	Notes:	Value:
Office+Partitions	Open floor plan→design for variability of corridor placement: 80 lb/ft^2 Minimum partition loading is 15 lb/ft^2 ; due to unknown nature of office will use 20 lb/ft^2	100 lb/ft^2
Assembly	Open unlabeled miscellaneous space to the east of the core will be taken as "Other Assembly Areas"	100 lb/ft^2
Stairways	Located in the core	100 lb/ft^2

Table C.2: Typical Floor Live Load Summary

Preliminary Wind Loading Analysis:

One of the first activities the structural team conducted was a lateral loading analysis. They began by examining the magnitude of wind vs. seismic loads to determine the controlling lateral load case. A Main Wind Force Resisting System (MWFRS) study was completed according to ASCE7-10 Ch. 26 to establish the effects of wind pressures on the structure. An example of the main variables for the structure and roof level is presented below along with the worst case pressure distribution.

Structure and Story Variable	Value	
Risk Category	III	
Importance Factor	1.0	
Basic Wind Speed, V	115mph	
Wind Directionality Factor, K_d	.85	
Exposure Category	C	
Velocity Pressure Exposure Coefficient, K_z (max value at roof level shown)	1.712	
Topographic Factor, K_{zt}	1	
Gust Effect Factor	.72	
Enclosure Classification	Enclosed	
Windward/Leeward -Controlliing Internal Pressure Coefficient, GCpi	.18	.18
Velocity Pressure, q_z	49.268 lb/ft^2	
Windward/Leeward -Wall Pressure Coefficients, C_p	.8	-.5
Windward*/Leeward Design Wind Pressure, p	24.6 lb/ft^2	11.6 lb/ft^2

Table C.4: MWFRS Study Variables Summary

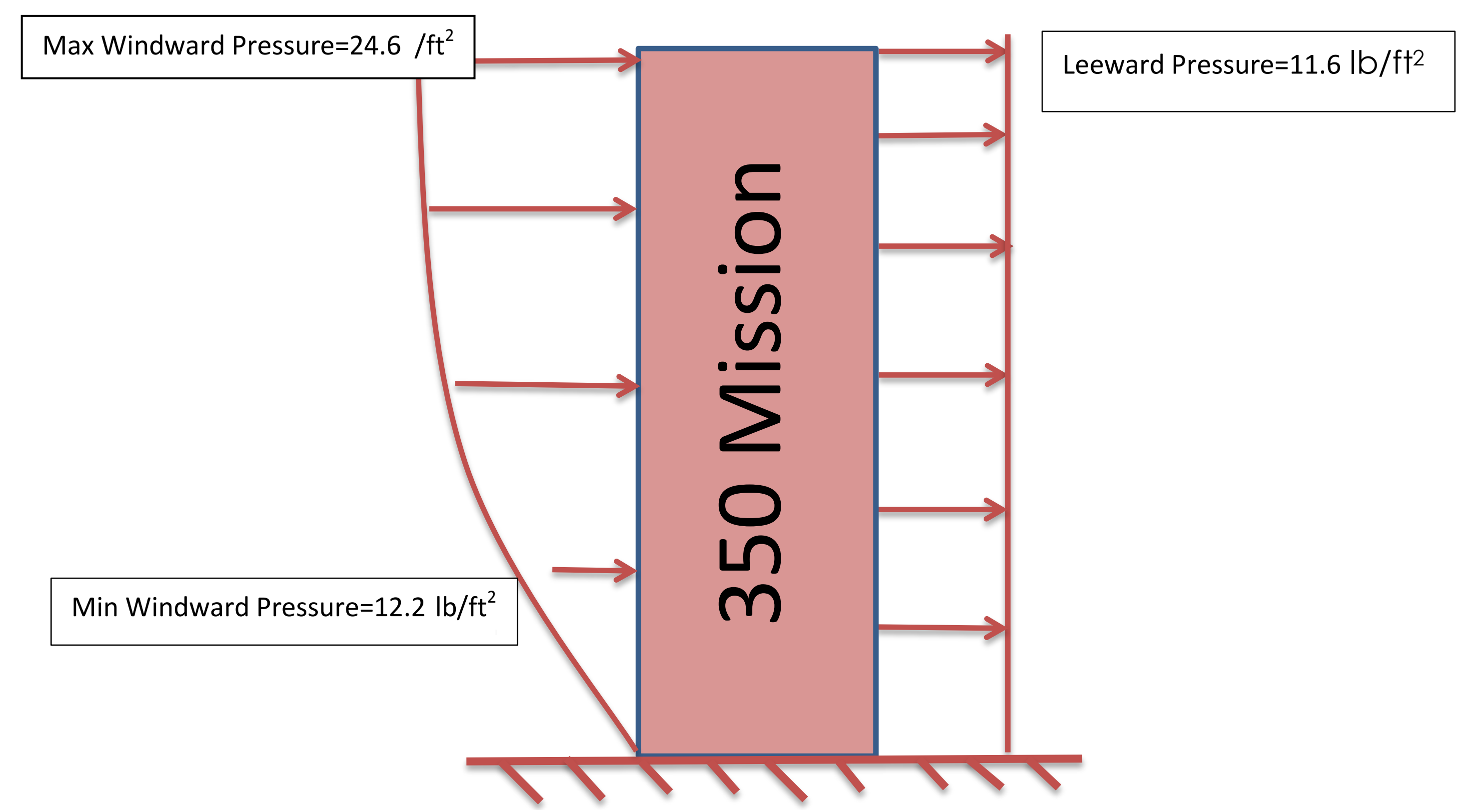


Figure C.1: Schematic Worst Case Scenario Wind Loading

Appendix D: Empirical Evaluation and Precedent Research

Preliminary Seismic Analysis:

Next preliminary seismic analysis was performed on the on the existing structure. This was done to serve as a baseline for comparison of the structural team's future models and systems analysis. A model was made in ETABS and Modal Response Spectrum Analysis (MRSA) was run. Then the structural team verified this rough model with an Equivalent Lateral Force (ELF) Procedure, of which a sample of the results is presented below. The structural team made use of the United States Geological Survey (USGS), Earthquake Hazards Programs Earthquake Ground Motion Parameter to identify the appropriate ground motion parameters of the building site in accordance with ASCE7-10. It was noted that seismic forces were, as expected, the controlling lateral load type.

NOTE: Due to the fact that this building is a core only building with no dual system the structural team recognized the need for a Performance Based Design approach when evaluating it as a system. Therefore all seismic analysis was conducted with an R=1 to examine the worst case demands on the building not knowing its ductility and to account for over-strength. Furthermore, all of the structural team's own design and analysis was completed on a Performance Based Design approach with an **R=1**. This was done to have an accurate comparison to the existing baseline model and because the structural team's solutions (see Appendix F) were ones that required this approach with no established R factor.

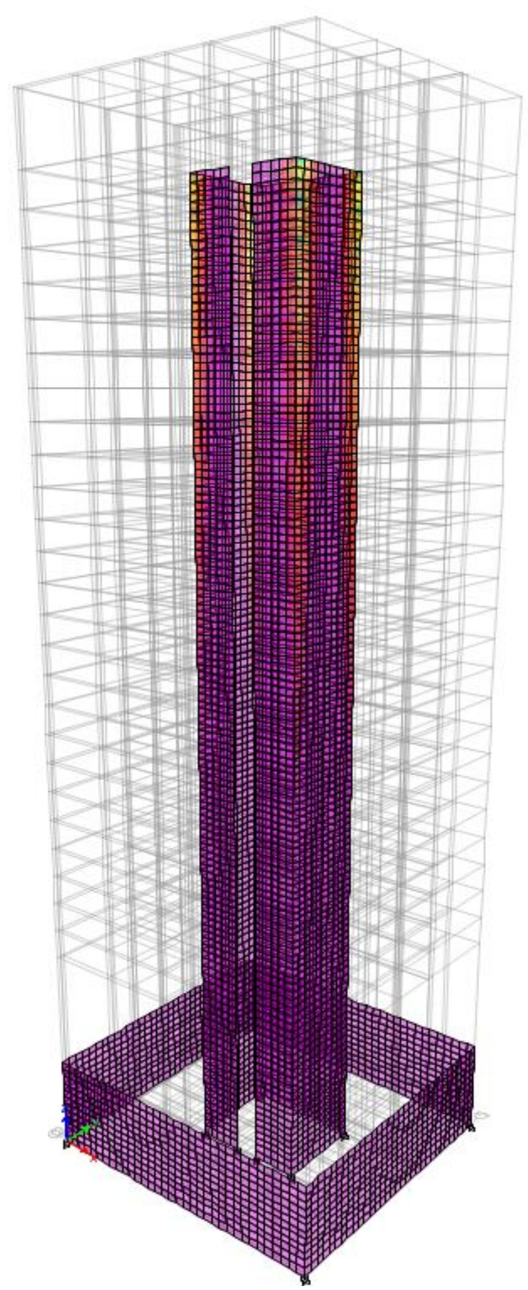


Figure D.1: ETABS Model of Provided Concrete Design

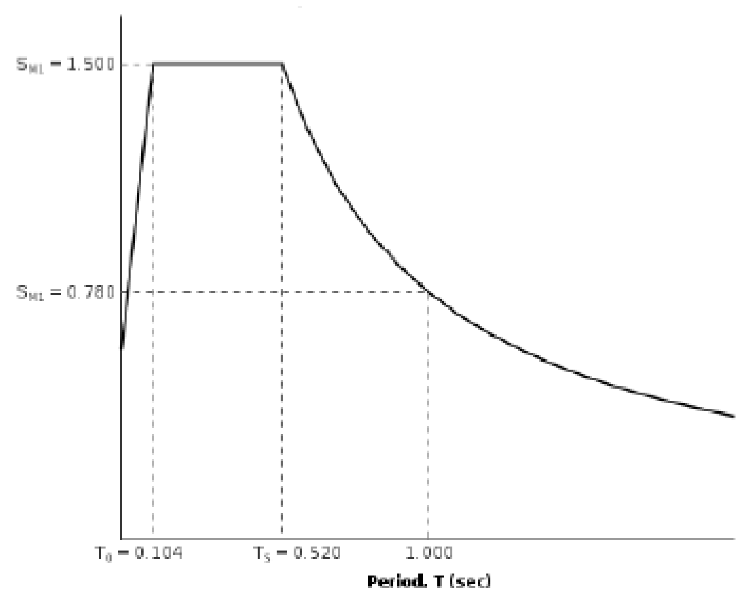


Figure D.2: Response Spectrum Provided by USGS.

Seismic Parameter	Value
R	1
Ss	1.5g
S1	.6g
Sms	1.5g
Sm1	.78g
Sds	1g
Sd1	.52g
TL	12s
Ts	.52s
Ie	1.25
SDC	D

Table D.1: Seismic Analysis Parameters

Level	Story Force [k]	Story Shear [k]	Story Moment [ftk]
30	3605.806	3605.806	1514438.710
29	3369.426	6975.232	1367986.877
28	3141.058	10116.290	1231294.761
27	2920.703	13036.994	1104025.819
26	2708.361	15745.355	985843.510
25	2504.032	18249.387	876411.290
24	2307.716	20557.103	775392.619
23	2119.413	22676.516	682450.955
22	1939.123	24615.639	597249.755
21	1766.845	26382.484	519452.477
20	1602.581	27985.065	448722.581
19	1446.329	29431.394	384723.523
18	1298.090	30729.484	327118.761
17	1157.865	31887.348	275571.755
16	1025.652	32913.000	229745.961
15	901.452	33814.452	189304.839
14	785.265	34599.716	153911.845
13	677.090	35276.806	123230.439
12	576.929	35853.735	96924.077
11	484.781	36338.516	74656.219
10	400.645	36739.161	56090.323
9	324.523	37063.684	40889.845
8	256.413	37320.097	28718.245
7	196.316	37516.413	19238.981
6	144.232	37660.645	12115.510
5	100.161	37760.806	7011.290
4	64.103	37824.910	3589.781
3	36.058	37860.968	1514.439
2	16.026	37876.994	448.723
1	4.006	37881.000	56.090
Base Shear= 37881 k		Overturning Moment= 12128130 ftk	

Table D.2: Seismic Building Forces Summary

Empirical Evaluation and Precedent Research:

Once the structural team had garnered an understanding of the lateral demands on the building they empirically evaluated various high performing lateral systems that could potentially be appropriate for the project teams goals. Extensive research was conducted on precedent buildings using these various lateral systems. Based off this research the structural team assigned each system a series of pros and cons as they related to the specific geometry and goals of 350 Mission. This research and empirical evaluation as it related to the structural team's goals is shown in tabular format below. Please note: All references for this research have been provided in Appendix J.

Goals: / System: (Source)	Reduce Size and Weight of Core		50% Code Mandated Drift		No Post-Seismic Structural Damage	
	Pros	**Cons**	**Pros**	**Cons**	**Pros**	**Cons**
Steel Plate Shear Walls (Seilie and Hopper)(Astaneh)	Reduces Weight of Core (approx. 20% less than concrete shear wall core)	N/A	Effective Drift Control, Very Stiff System, Stiffness Provided by Oversized Perimeter Members	N/A	N/A	Designed to Buckle and Develop Hinges, Amplified Acceleration Due to Stiffness
Base Isolation (Wang)	Reduces/Distributes Lateral Forces Over Floors - Vibrates Like a Rigid Body	Not conducive to our geometry.	Limits Max Seismic Force to Superstructure-Mitigates Overturning Moment	N/A	Endures Deformations & Displacements/No Repair	N/A
Concentrically Braced Frames (Sabelli, et al.)	Reduced Size of Horizontal Steel Members and Columns	N/A	High Elastic Stiffness Due to Braces	N/A	N/A	Ductility is Developed Through Inelastic Action In Braces
Eccentrically Braced Frames (Popov and Engelhardt)	Reduced Size of Horizontal Steel Members and Columns	N/A	Provides Stiffness of CBF's with Ductility of Special Steel Moment Frames	N/A	N/A	Shear Links are Designed to Yield Under Seismic Loading
Buckling Restrained Braced Frames (Sabelli and Lopez)	N/A	Additional Steel Casing Required for Buckling Restraint	Inelastic Demands Distributed to Several Stories	N/A	N/A	Braces Yield During Compression
Special Truss Moment Frames (Chao and Goel)	N/A	Heavy Spanning Members Especially for Wide Bays, Increased Steel Usage	N/A	Story Drifts Not Always Uniformly Distributed	N/A	Special Replaceable Sections Yield Under Seismic Loading
Special Steel Moment Frames (Hamburger, et al.)	N/A	Heavy Spanning Members Especially for Wide Bays, Increased Steel Usage	Design Controlled by Drift, No Code Restrictions for Tall Buildings	N/A	N/A	Connections will Yield, Costly to Repair
Reinforced Concrete Shear Walls (Lombard, et al.)	N/A	Heavy Core Walls and Coupling Beams, Heavy Steel Reinforcement	N/A	Requires Extreme Reinforcement to Achieve	N/A	Seismic Cracking and Loss of Capacity, Extensive Repair Required
Tuned Liquid Damping (Robinson, et al)	N/A	Adds Weight to System Especially Near Top, Increased Steel Usage	N/A	Not Always Effective for Seismic	No Damage, Simple Design and Function	N/A
Viscous Damping (Taylor)	Can Reduce Member Size and Weight When Used with Steel Frame	N/A	Can Drastically Reduce Drift by Reducing Seismic Loads	N/A	Long Lasting, Can Eliminate Yielding In Structure, Low Maintenance	N/A
Friction Damping (Fu and Cherry)	Does Not Contribute Much Weight, Integral with the Connections	N/A	N/A	Effectiveness Can Vary Over Time, Constant Friction Force is Not Always Possible	N/A	Maintenance Needed After Major Event
Tuned Mass Damping (Purdue)	N/A	Adds Weight to System, Large Mass Requires Beefed Up Structure	N/A	Not Always Effective for Seismic, More Often Used for Wind	No Damage, Simple Design and Function	N/A
Viscoelastic Damping (Yokota, et al.)	Does Not Contribute Much Weight, Integral with Structural Members	N/A	N/A	More for Vibration Control Than Drift	Can Sustain Large Shear Deformation, Very Stable and Ages Well	N/A
Outrigger/ Belt Truss (Nair)	Does not Contribute Much	Difficult and Sensitive Connections to Core	Can Significantly Reduce Drift and OTM	Large Mega Columns, Difficult to Construct	Less Drift Less Potential for Non Structural Components Being Damaged	N/A

Table D.3a: Pros/Cons of Potential Lateral Systems Related to Structural Goals

Goals: / System: (Source)	100% Passive Energy Dissipation		Architectural Enhancement		Efficient MEP Coordination		Ease of Construction	
	Pros	**Cons**	**Pros**	**Cons**	**Pros**	**Cons**	**Pros**	**Cons**
Steel Plate Shear Walls (Seilie and Hopper)(Astaneh)	N/A	Large Columns at Core Corners to Provide Flexural Stiffness	Thin Core Wall = Increased Usable Space (Estimates approx. 2% increase)	N/A	N/A	Difficult to Penetrate with MEP, Very Few and Small Areas Where Possible	Decreased Cost and Schedule (Estimated 1-month Savings)	N/A
Base Isolation (Wang)	Dissipates Energy Through Sliding Friction of Components	N/A	N/A	Requires a Buffer Zone Between the Structure and Surrounding (Sidwalk, Plaza, etc.)	N/A	Utilities Entering Building Require Flexibility to Endure Structure Movement	N/A	Construction Uncertainty, Grouting at Beam-Column Joints, Complex Reinforcement Required for Ductility
Concentrically Braced Frames (Sabelli, et al.)	Energy Dissipated Through Inelastic Brace Action	N/A	N/A	Difficult Opening Coordination	Ease of MEP Penetrations to Core	N/A	Able to Pre-Fabricate Modules	N/A
Eccentrically Braced Frames (Popov and Engelhardt)	Shear Links Yield, Potential for Damping	N/A	Eccentricity Allows for Opening Coordination, Increased Usable Floor Area	N/A	Ease of MEP Penetrations to Core	N/A	Able to Pre-Fabricate Modules	N/A
Buckling Restrained Braced Frames (Sabelli and Lopez)	Dissipates Energy in Compression and Tension Since Restrained From Buckling	N/A	N/A	Intrudes on Floor Space	Ease of MEP Penetrations to Core	N/A	Able to Pre-Fabricate Modules	N/A
Special Truss Moment Frames (Chao and Goel)	Yielding Segmant Dissipates Energy	N/A	N/A	Decreased Floor-to-Ceiling Height, Difficulty of Opening Coordination Because of Braces	Ease of MEP Penetrations to Core	N/A	N/A	Costly to Construct Especially Due to Special Connections
Special Steel Moment Frames (Hamburger, et al.)	Yielding in Connections	N/A	Opening Coordination, Increased Usable Floor Area, Large Sightlines Possible	N/A	Ease of MEP Penetrations to Core	N/A	N/A	Costly to Construct Especially Due to Special Connections
Reinforced Concrete Shear Walls (Lombard, et al.)	Dissipates Energy by Cracking	N/A	N/A	Decreased Opening Potential and Floor Area	N/A	Difficult to Penetrate with MEP	N/A	Increased Schedule, Must Wait For Curing
Tuned Liquid Damping (Robinson, et al)	Dissipates Energy by Liquid Movement	N/A	N/A	Decreased Floor Area	N/A	Additional MEP Needed for System	N/A	Extra Structure Required, Additional Components, Sequencing Issues
Viscous Damping (Taylor)	Dissipates Energy Like a Shock Absorber	N/A	N/A	Can Take Up Increased Wall Space	Does Not Interfere With MEP	N/A	Easy to Install	N/A
Friction Damping (Fu and Cherry)	N/A	Constant Dissipation Values Cannot Be Assumed, Based on Slip Force	Do Not Interfere with Architecture	N/A	Does Not Interfere With MEP	N/A	N/A	Unfamiliarity with Construction of System, Expensive Material
Tuned Mass Damping (Purdue)	Dissipates Energy by Mass Movement	N/A	N/A	Takes Up Large Area, Extra Structure Required	N/A	Housing Area Could Interfere with MEP	N/A	Sequence Issues, Difficulty of Placement
Viscoelastic Damping (Yokota, et al.)	Reduces Response Acceleration, Estimates of About 30% Reduction	N/A	Do Not Interfere with Architecture, Integral with Members	N/A	Does Not Interfere With MEP	N/A	N/A	Expensive, Difficult to Find Material

Table D.3b: Pros/Cons of Potential Lateral Systems Related to Structural Goals

Appendix E: Gravity System Methodology, Calculations, and Description:

Once all loading calculations and precedent research was complete, the structural team moved into the Gravity System Design phase. Initially, the structural team decided to design in steel in order to reduce seismic weight, enhance the usable building space and increase life-cycle efficiency. After deciding on a deck type and size, two floor beam layouts were created and the team quickly identified the positives of both layouts and created the final design by melding the two. The team explored the possible use of a steel joist system and carried out numerous studies relating to the design of the Southwest cantilever area. The gravity columns of both the core and perimeter were designed and a built-up section study was completed for the cantilever supporting columns. Basic connection design was implemented for gravity beams and girders in order to complete the scope of the Gravity System Design.

Angular Layout

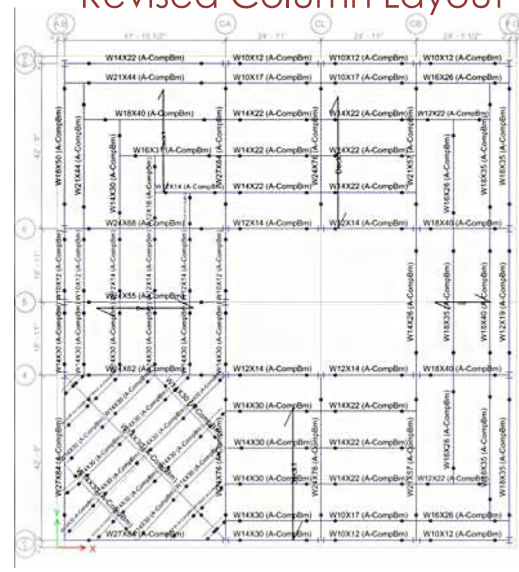
Pros:

- Shorter Cantilever Span
- Decking Direction is Same for Majority of Floor
- Column Locations are Unchanged from Original Design

Cons:

- 5 ft. Span to First Infill Beam
- 'Basket Weave' Framing in Corners
- Typical Beam Sizes in Bays are W16's

Revised Column Layout



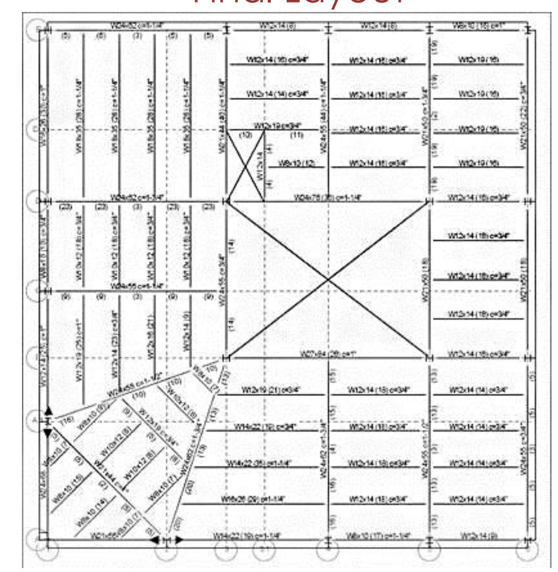
Pros:

- Smaller Infill Beam Sizes in Bays
- Larger Un-obstructed views in Southwest Corner

Cons:

- Very Long Cantilever Spans
- 'Basket Weave' Framing in Corners
- 5 ft. Span to First Infill Beam
- Decking Direction Changes Frequently

Final Layout



- Typical Infill Beam size of W12x14
- 29' Cantilever Span
- Un-obstructed Views and Architectural Accentuation
- Normalized Framing Layout for Ease of Construction
- Typical Beam Sizes and Lengths to Reduce Waste
- Maximized Plenum Space for MEP Coordination
- Connection and Vibration Considerations
- Designated Floor Cut-Out Area
- Decking Direction is Same for Majority of Floor

Figure E.1: Preliminary Floor Beam Layouts and Final Layout with Pros/Cons

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span 1 SPAN	2 SPAN	3 SPAN	5'-0	5'-6	6'-0	6'-6											
3.50 (t=2.00) 26 PSF	1.5VL22	6'-4	8'-5	8'-6	278	247	222	185											
	1.5VL20	7'-8	9'-7	9'-11	305	271	243	220											
	1.5VL19	8'-8	10'-7	11'-0	329	292	262	237											
	1.5VL18	9'-6	11'-4	11'-9	350	311	279	252											
	1.5VL16	9'-8	11'-5	11'-10	352	312	280	253											
4.00 (t=2.50) 30 PSF	1.5VL22	6'-0	8'-1	8'-1	324	288	258	215											
	1.5VL20	7'-3	9'-7	9'-9	355	315	283	256											
	1.5VL19	8'-2	10'-7	10'-11	382	339	304	275											
	1.5VL18	8'-11	11'-4	11'-5	400	360	323	292											
	1.5VL16	9'-1	11'-4	11'-8	400	360	323	292											
4.50 (t=3.00) 35 PSF	1.5VL22	5'-9	7'-8	7'-8	372	330	275	246											
	1.5VL20	6'-11	9'-2	9'-4	400	361	324	293											
	1.5VL19	7'-9	10'-1	10'-5	400	388	348	315	287	264	221	203	188	174	162	151	140	122	107
	1.5VL18	8'-6	10'-10	11'-0	400	400	369	334	305	279	258	239	200	186	173	161	147	129	114
	1.5VL16	8'-7	10'-10	11'-2	400	400	369	334	304	279	257	239	199	185	172	160	150	140	126
4.75 (t=3.25) 37 PSF	1.5VL22	5'-7	7'-7	7'-7	396	352	293	263	237	216	197	181	167	154	143	133	124	115	108
	1.5VL20	6'-9	9'-0	9'-1	400	385	345	312	262	238	218	200	184	171	159	148	138	129	118
	1.5VL19	7'-7	9'-11	10'-3	400	400	371	336	306	281	235	216	200	185	172	160	150	140	126
	1.5VL18	8'-3	10'-7	10'-9	400	400	393	356	324	298	274	231	213	198	184	171	160	150	133
	1.5VL16	8'-5	10'-7	11'-0	400	400	392	355	324	297	274	230	212	197	183	171	159	149	140
5.00	1.5VL22	5'-6	7'-5	7'-5	400	374	311	279	252	229	209	192	177	164	152	141	131	123	115
	1.5VL20	6'-7	8'-10	8'-11	400	400	367	332	278	253	231	212	196	181	168	157	146	137	128

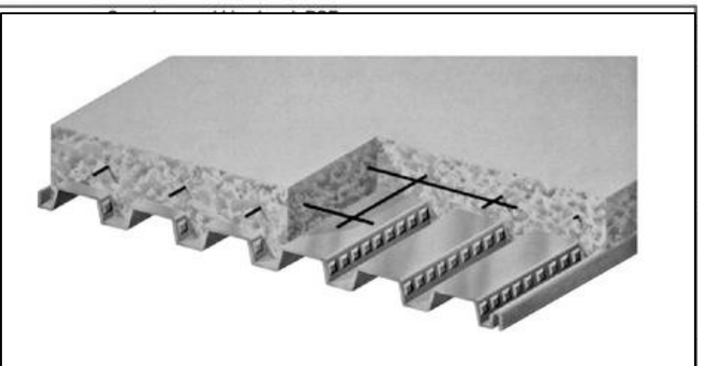


Figure Provided by Vulcraft Steel Deck Catalog

Decking Design:

- Composite Deck with Lightweight Concrete Topping (1.5" deck + 3.25" topping)
- 19 Gauge Decking
- 2-hr Fire Rating Between Floors
- 10'-3" Un-shored Construction Clear Span
- At 10' Beam Span – 172 psf Live Load Capacity

Figure E.2: Decking Design Summary and Table Selection (Vulcraft Steel Deck Catalog)

Steel Joist Check:

Max Span: 42 ft.

Max Trib. Width: 9 ft.

Max Load: (37 + 20 + 5) + (100) = 162psf*(9) = **1,458 plf**

Joist Design:

- **Vulcraft Longspan Steel Joist, LH-Series**:
  - 40LH15
  - Depth = 40 inches
  - Capacity = 1511 plf

* Therefore steel joists were ruled out as a potential gravity system

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists Only)	Depth in inches	Max Load (plf) < 48	SAFELOAD* in Lbs. Between 48-59	60-65	SPAN IN FEET 66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
40LH15	36	40	1511	72510	72510	1101	1068	1036	1006	978	949	924	898	874	850	828	807	786	766	747
						427	408	390	373	357	342	328	315	302	290	279	268	258	248	239

Figure E.3: Steel Joist Design Summary and Table Selection (Vulcraft Steel Joist Catalog)

Composite Beam Validation: RAM Steel Beam Output: W12x14, M_n=151k-ft, b_{eff}=74.75in

$$b_{eff} = min\begin{cases} span/_8 \\ 1/_2\,clear \end{cases} = \begin{cases} \frac{(24.92ft)\left(\frac{12in}{ft}\right)}{8} = \boxed{37.38\ in} \\ \left(\frac{1}{2}\right)(8.42ft)\left(\frac{12in}{ft}\right) = 50.52\ in \end{cases}$$

$$b_{eff} = 2*37.38 = 74.76\ in$$

Locate PNA:

$$V_{c,max} = 0.85f'_c b_{eff} t = (.85)(4)(37.38*2)(3) = 762.45\ kips$$

$$V_{s,max} = F_y A_s = (50)(4.16) = 208\ kips$$ → Case 1

$$V_{s,max} = 208 = 0.85f'_c b_{eff} a = (.85)(4)(74.76)a$$

$$a = 0.818\ in$$

$$M_n = V_{s,max}\left(\frac{d}{2}+t-\frac{a}{2}\right)\left(\frac{1ft}{12in}\right) = 208\left(\frac{11.9}{2}+3-\frac{0.818}{2}\right)\left(\frac{1}{12}\right)$$

$$= 148.04\ k-ft$$

Figure E.4: Composite Beam Design Validation Calculations

Column Design:

Once a floor slab system was chosen and the beam layout was finalized; the structural team moved into gravity column design. To improve efficiency in design and construction, the structural team formed some constraints to column member sizing, as noted below. The gravity column design was completed in RAM Structural System (see image below), however some members were not designed due to the extreme unbraced lengths in the lobby area. The team verified the software with spreadsheets (shown below) and designed a custom built-up column section to handle the 54-foot unbraced lobby height.

Column Design Considerations:

- Column sections were limited to only W14's for ease of construction
- Column section sizes range from W14x43 to W14x730
- Columns are spliced every two stories (26'-4") for ease of construction and transport
- Columns are optimized to achieve a Demand/Capacity interaction ratio between 75-90%
- Ram model shows colored interaction ranges on column designs

Software Quality Control: The Structural Team developed Excel Spreadsheets to quickly check the interaction results of select columns sections. These spreadsheets were helpful in quickly double checking the software design accuracy. The hand calculations verifying the spreadsheets are described on the right.

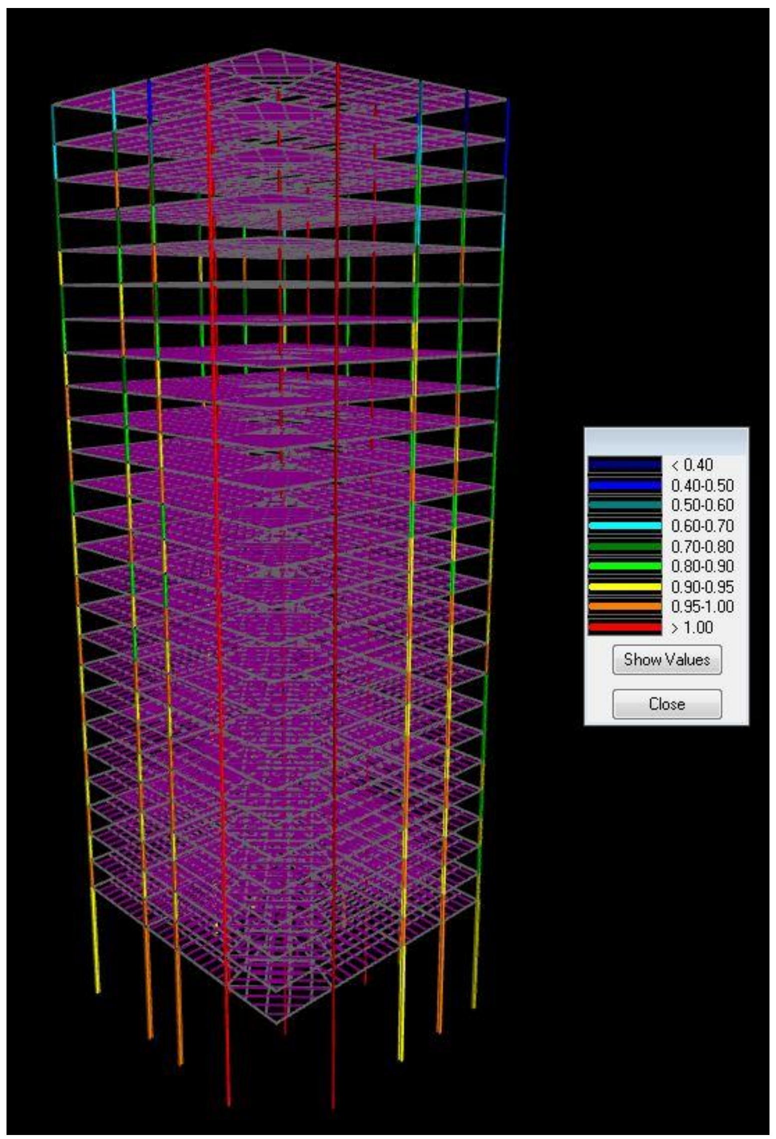


Figure E.5: Full Gravity System Design in RAM Structural System

W14x398

Input		
Ag	117	in^2
k	1.2	
L	158	in2
ry	4.31	in
Pu	2900	k

Output	
kL/(ry)	43.99072
Fe	147.9025 ksi
4.71(E/Fy)^.5	113.4318
Fcr	43.40296
ΦPn	4570.332
Pu<ΦPn?	YES

Regular Section:

Input		
Trial Size	W12x65	
Pu	1644	k
Mu	3456	ftk
K	1.2	
L	13.1667	ft
KL	15.80004	ft
p	0.281	$x10^3$
b	0.155	$x10^3$

Output	
Interaction	0.997644
Ok?	YES

Figure E.6: Excel Spreadsheets Verifying the RAM Column Design

Input: The input values are all manually entered into the graph and found in the AISC Steel Construction Manual.

Output: $kL\ /\ ry = \ ((1.2) * (158))\ /\ 4.31 = 43.99in$

$$Fe = (\pi^2 * E)\ /\ (kL\ /\ ry)^2 = ((\pi^2 * 29000))\ /\ 43.99 = 147.9\ ksi$$

$$4.71\sqrt{(E\ /\ (Fy\ ))} = 4.71\sqrt{(29000/50)} = 113.43$$

$$Fcr = \ [0.658]\ ^\wedge(Fy\ /\ Fe) * Fy = \ [0.658]\ ^\wedge(50\ /\ 147.9) * 50 = 43.4ksi$$

$$\varphi Pn = 0.9 * Ag * Fcr = 0.9 * 117 * 43.4 = 4570.33\ kip$$

Built-Up Column Design: The columns supporting the cantilever beams required section sizes beyond what is found in the AISC Steel Construction Manual. To meet the high loads resulting in an unsatisfactory interaction ratio, the structural team developed a custom column section using ETABS. The custom column consists of:

- W14x730 A992 Steel Column Section
- (2) 1-inch A36 steel plates welded spanning from flange to flange
- Increases Gross Area from 215 in^2 to 259.5 in^2
- Increases Strong Axis Moment of Inertia from 14,300 in^4 to 16,191 in^4

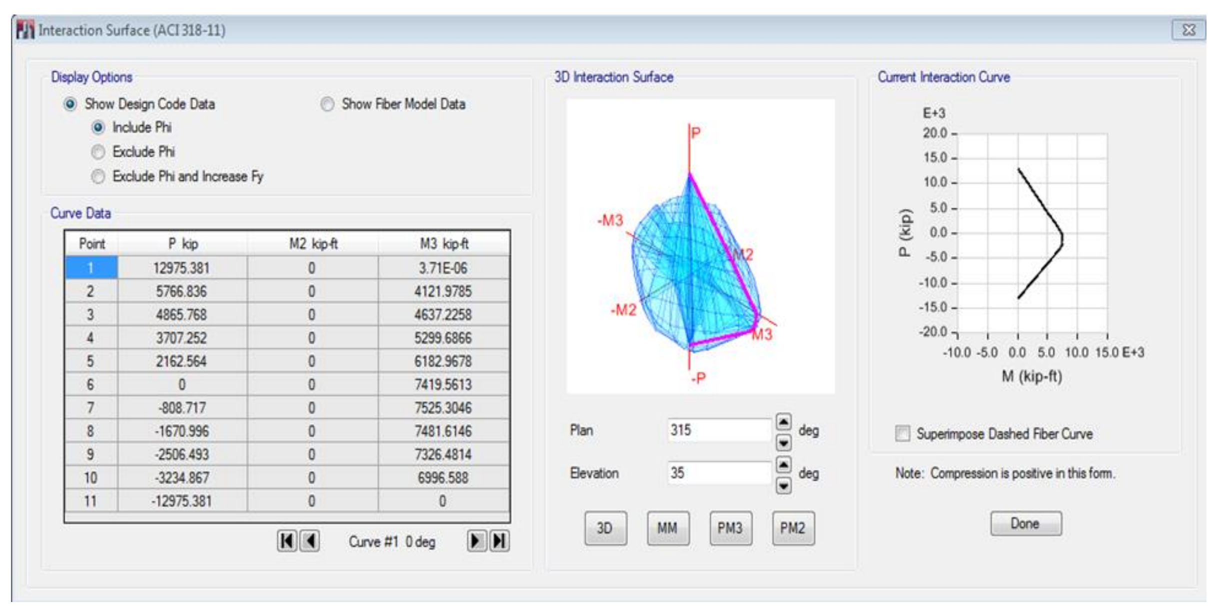


Figure E.7: ETABS Interaction Summary for Built-Up Column Section

Custom W14x730

Area = 259.5 in^2

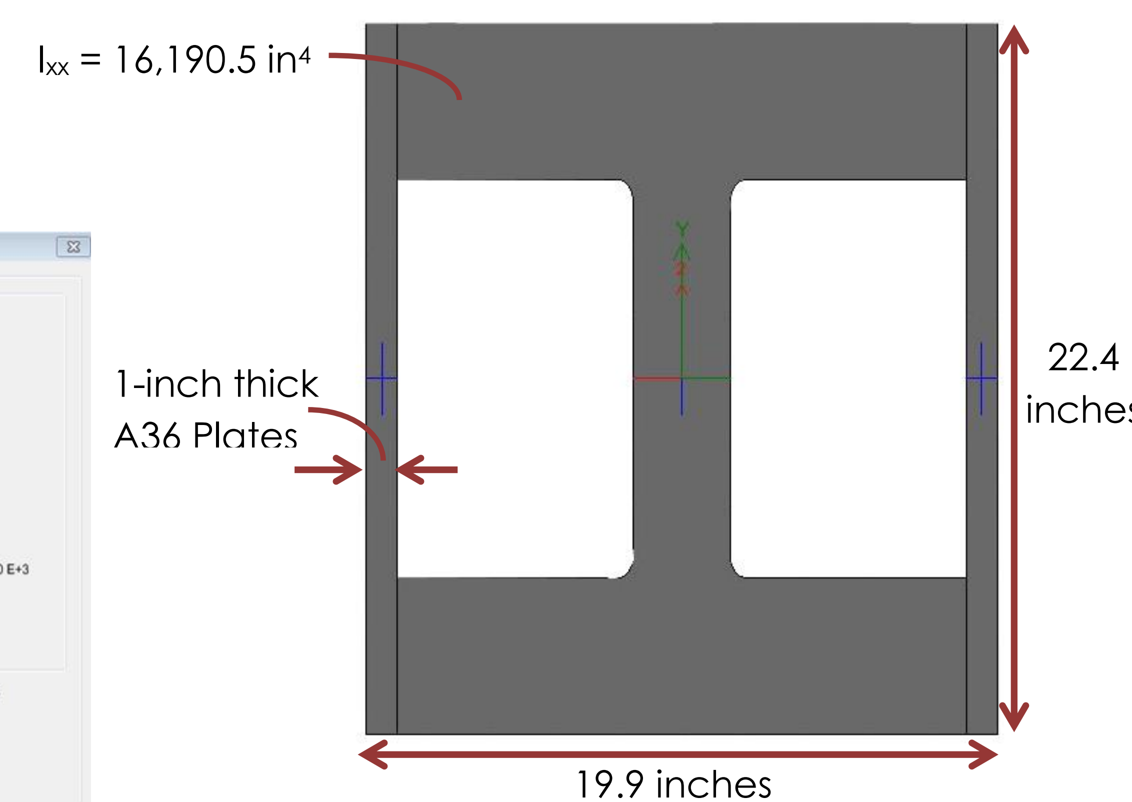


Figure E.8: Section of Built-Up Column with Properties

Cantilever Study: The Structural Team explored strategies for relieving loads on the cantilevered corner, while preserving the architectural features and effect. Two strategies the team looked at were adding moment connections in adjacent bays on the same beam line; or the addition of a tension hanger which would eliminate cantilevered action. The team's main goals were to create a system which did not impose on the architectural significance of the space; and provided a comfortable space for the tenant with no vibration or deflection issues. The two studies exploring additional moment connections and a tension hanger addition are seen below.

Moment Connection Study: The Structural Team found that adding just (1) additional moment connection immediately adjacent the cantilever beam reduced the acting moment forces the greatest. The above images of analysis models created in RISA 2D show these findings. By using this design, the moment force from the cantilever beam was reduced from 424 kip-ft to 288 kip-ft.

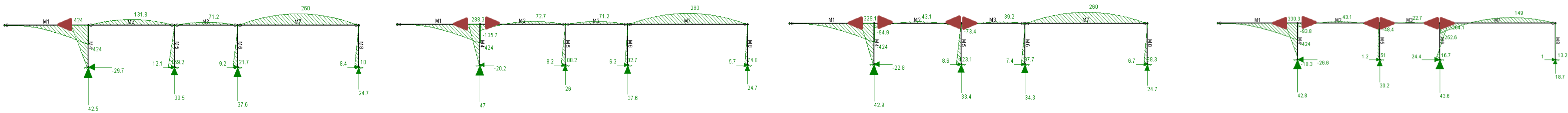


Figure E.9: RISA 2D Cantilever Study Showing Additional Moment Connections

Tension Hanger Study: The Structural Team also analyzed the addition of a tension hanger, as positioned in the RISA 2D model on the right, to eliminate cantilever action at the Southwest corner. A 1-inch diameter steel rod was used with an area of 0.785 in^2. While the tension rod did reduce moment forces from the cantilever from 424 kip-ft to 243 kip-ft; the team decided that the additional connections and visual disturbance of the tension hanger were not worth it.

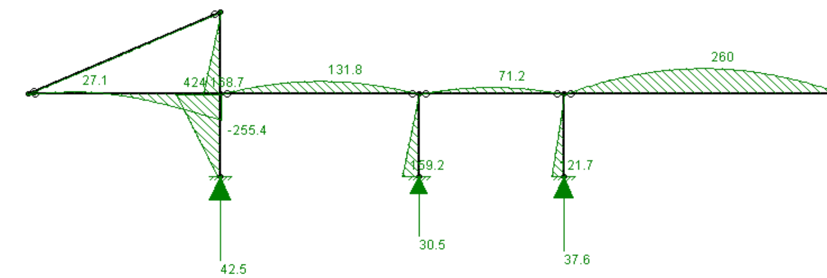


Figure E.10: RISA 2D Cantilever Study Showing Tension Hanger

Connection Design: The structural team designed select gravity connections for a typical floor. Two column locations were identified where numerous beams and girders connected at the same level and these locations were designed to handle gravity loads with shear connections. An example of some hand calculations is shown below. Typical details and drawings for the connections design can be found on Drawing S9.

Calculations: The following calculations are representative of the design process for the gravity shear tab connections described in the narrative and detailed in the drawings.

Beam: W24x62 A992 Steel	**Assume:**
d = 23.7 in	Bolt Spacing = 3 in
t_f = 0.59 in	Edge Dist. Vertical = 1.25 in
b_f = 7.04 in	Edge Dist. Horizontal = 1.5 in
t_w = 0.43 in	Standard Bolt Holes
d_{cope} = 2 in	A36 Plates
c = 4 in	Use (3) ¾" dia. A325-N bolts
V_u = 62 kips	on beam

** Section properties, tables values and equations are from AISC

<u>Bolt Shear:</u> Double Shear → Table 7-1: $\varphi r_n = (35.8\,k)(3\,bolts) = 107.4\,kips > 62\,kips$

<u>Shear Yielding:</u> $h_o = 23.7 - 2(2'') = 19.7\,in$

$\varphi R_n = (1.0)(0.6)F_y t_w h_o = (1.0)(0.6)(50)(0.43)(19.7) = 254.13\,kips > 62\,kips$

<u>Shear Rupture:</u> $A_n = [h_o - 3(3/4 + 1/16 + 1/16)]t_w = [19.7 - 3(7/8)](0.43) = 7.34\,in^2$

$\varphi R_n = (0.75)(0.6)F_u A_n = (0.75)(0.6)(65)(7.34) = 214.7\,kips > 62\,kips$

<u>Coped Beam Flexural Strength:</u> assume pin located at face of support

$$S_{net} = \frac{bh^2}{6} = \frac{t_w {h_o}^2}{6} = \frac{(0.43)*(19.7)^2}{6} = 27.81\,in^3$$

<u>Double Cope:</u> $c \le 2d,\ d_c \le 0.2d, d_{cb} = d_{ct} \rightarrow c = 4\,in \le 2(23.7) = 47.4\,in$

$d_c = 2\,in \le 0.2(23.7) = 4.74\,in$

$$f_d = 3.5 - 7.5\left({d_c}/{d}\right) = 3.5 - 7.5\left({2}/{23.7}\right) = 2.867$$

$$F_{bc} = 56{,}490\left[{{t_w}^2}/{c h_o}\right] f_d \le F_y = 56{,}490\left[{0.43^2}/{4*19.7}\right]*2.867 = 380\,ksi$$

$M_u = R_u e \le \varphi M_n \quad e = 4 + 1/2\,inch\ setback = 4.5\,in$

$\varphi Mn = (0.9)(50)(27.81) = 1251.45\,in-k \ge (62)(4.5) = 279\,in-k$

<u>Block Shear-Coped Beam:</u> Assume L_{eh} = 1.5" – cut tol. = 1.25 inches

$\varphi Rn = \varphi(0.6)F_u A_{nv} + \varphi U_{bs} F_u A_{nt} \le \varphi(0.6)F_y A_{gv} + \varphi U_{bs} F_u A_{nt}$

<u>Table 9.3a</u> → 39.6 k/in <u>Table 9.3b</u> → 163 k/in <u>Table 9.3c</u> → 148 k/in

$\varphi R_n = (39.6 + 148)t_w = (39.6 + 148)(0.43) = 80.67\,kips \ge R_u = 62\,kips$

<u>Bearing/Tear-Out Coped Beam:</u> $\varphi R_n \ge 62^k \le \varphi r_n t_w \le \varphi r_n(0.43) \quad \varphi r_n \ge 144.2\,{}^k/_{in}$

Interior: Table 7-4 → $\varphi r_n = 87.8(.43) = 37.75\,k$

Edge: Table 7-5 → $\varphi r_n = 49.4(.43) = 21.24\,k$

$\varphi R_n = (21.24 + 35.8 + 35.8) = 93\,kips \ge 62\,kips$

<u>Shear Yield-Plates:</u> $\varphi R_n = \varphi(0.6)F_y(2L_p t_p)\ \varphi R_n \ge R_u = 62\,kips$

$62\,kips \le (1.0)(0.6)(36)(2)(8.5)t_p \quad t_p \ge 0.17\,in\ so\ use\ 1/4\,inch\ plates$

$(1.0)(0.6)(36)(2)(8.5)(1/4) \quad \varphi R_n = 91.8\,kips \ge 62\,kips$

<u>Shear Rupture-Plates:</u> $d'_h = 3/4 + 1/16 + 1/16 = 7/8\,in$

$A_n = \left(8.5 - 3(7/8)\right)(1/4)(2) = 2.94\,in^2$

$\varphi R_n = \varphi(0.6)F_u A_n = (0.75)(0.6)(58)(2.94) = 76.73\,kips \ge 62\,kips$

<u>Block Shear-Plates:</u> Assume L_{eh}=1.5" and L_{ev}=1.25"

<u>Table 9.3a</u>→46.2 k/in <u>Table 9.3b</u>→117 k/in <u>Table 9.3c</u>→132 k/in

$\varphi R_n = (46.2 + 117)t_p = (46.2 + 117)(1/4)(2\,plates) = 81.6\,kips \ge 62\,kips$

<u>Bearing/Tear-Out Plates:</u> Interior: Table 7-4→$\varphi r_n = 78.3(.25) = 19.58\,kips$

Edge: Table 7-5→$\varphi r_n = 44.0(.25) = 11\,kips$

$\varphi R_n = (2\,plates)(11 + 17.9 + 17.9) = 93.6\,kips \ge 62\,kips$

Bearing/Tear-Out Column: Interior: Table 7-4→$\varphi r_n = 78.3(1.0) = 78.3\,kips$

$\varphi R_n = (17.9 + 17.9 + 17.9)(2) = 107.4\,kips \ge 62\,kips$

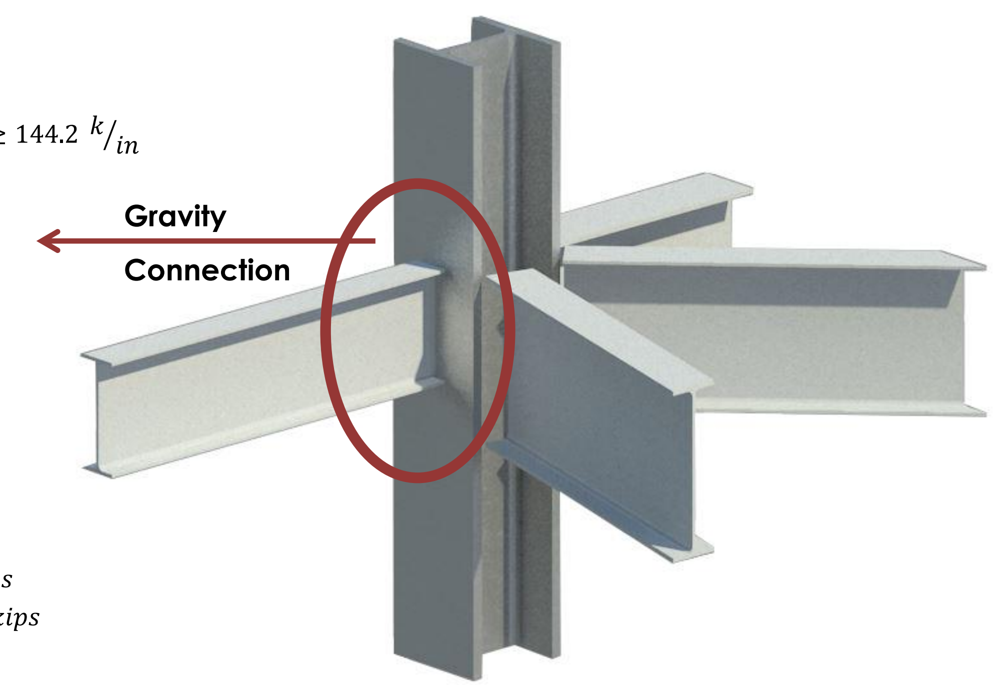


Figure E.11: 3D Render Showing Sample Gravity Connection Location

Appendix F: Lateral System Methodology, Calculations, and Description

System Choice: With extensive precedent research done on various high performing lateral systems as outlined above in Appendix D, the structural team identified possible pairings of the best solutions. These parings are presented in Table F.1 below. Through extensive conversations with the other project team members, the structural team identified braced frames as the proper selection for their core. The symmetric layout of the core allowed the structural team to quickly lay out a bracing scheme shown below and analyze it in ETABS. MRS analysis was run and design iterations completed using the design module in ETABS. The basic results are shown below.

	Concrete Shear Wall with Post-Tension Slab	Base Isolation and Steel Plate Shear Walls	Braced Frame Core with Viscous Dampers
Pros	- Contractor Familiarity - Ease of Construction - Cost Effective - Slab Thickness	- Durable and Reliable -Reduced Overturning Moment - Decreased Drift - Speedy Construction - Decreased Building Weight, - Increased Square Footage	- Reduced Seismic Weight - Steel Floor – Tenant Benefits - Coordination with MEP - Overturning Moment - Passive System Possibilities - Decreased Repair
Cons	- Slab Weight and Drift - Overturning Moment - Tenant Limitations - Slow Construction - Post-Event Occupancy	- Buffer Zone Required - Flexible Utility Entries - Unjustifiable Over-Design - MEP Coordination - Fabricator Issues	- Increased Initial Cost - Special Connections - Architectural Clashes

Table F.1: Potential Lateral System Pairings with Pros/Cons

Chosen System

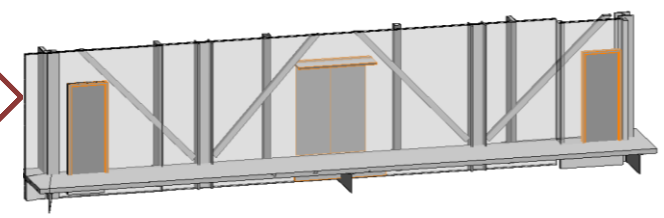


Figure F.1: South Side of Core Isometric: Structure-Architectural Clash Detection: The symmetric layout was very conducive to accommodating all architectural openings.

Braced Frame Core	
Structural	Symmetric layout of core is conducive to braced frames
Mechanical	Allows for simple duct system coordination
Electrical	Allows for simple utilities coordination
Construction	Simple construction and modularization ability
Architecture	Avoids clashes with openings into the core

Table F.2: Braced Frame Core Pros for Each Discipline

Analysis and Design

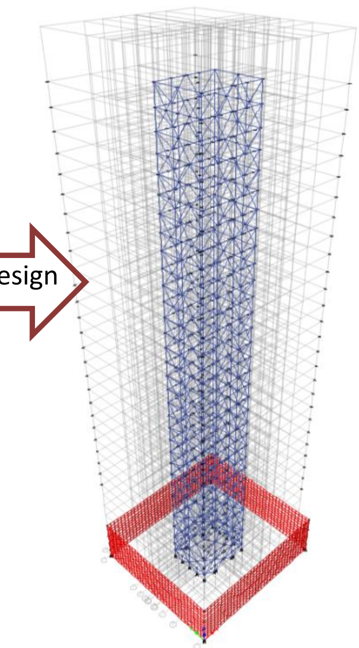


Figure F.2: Core Only ETABS Model

T_y=3.863s
T_x=2.339s
T_x=2.433s

Smallest Brace: <u>W12x96</u>
Largest Brace: <u>W36x652</u>
<u>UNNACEPTABLE</u>

Re-examine Goals and Preliminary Analysis:

The design iterations run in ETABS indeed showed the design was capable of meeting the team's high performance drift requirement. However it resulted in braces that ranged from very large and uneconomical sizes of W12x96 to W36x652. In re-examining their goals for the lateral system the structural team noted that though they could meet their high performance drift requirement, at this point they had not added any value to the structure. The core was thicker than the original shear wall in some locations and it offered no opportunity for architectural enhancement. Therefore the structural team looked at the mobilizing the full aspect ratio of the building in lateral resistance. The three methods considered were an outrigger at the top of the building, a perimeter moment frame, and mega diagonal bracing on the exterior. The structural team quickly disregarded the idea of an outrigger at the top story because it was not conducive to their gravity layout as shown in Figure F.3. The structural team made two additional models: core and perimeter moment frames and core with external diagonal braces. Quick MRS analysis was run. Each system's story drifts were plotted as shown. The structural team noted that the exterior mega brace system presented the greatest potential for controlling drift. Furthermore the perimeter moment frame system required spandrel members up to 36" of depth. This was a direct result of the long perimeter spans. At this stage the Electrical team was unsure of a daylighting scheme around the perimeter and wanted to preserve the tall perimeter views. Also the structural team felt that the perimeter mega brace system presented the most unique opportunity for architectural enhancement in accentuating the south west connection to the Transbay Terminal and creating a unique identity for the building.

<u>Decision:</u> Pursue an exterior mega brace system to alleviate demands on core and enhance architecture.

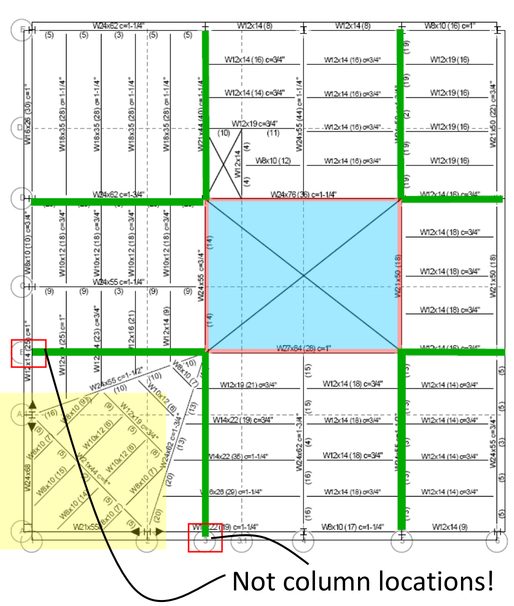


Figure F.3: Potential Outrigger Layout Showing

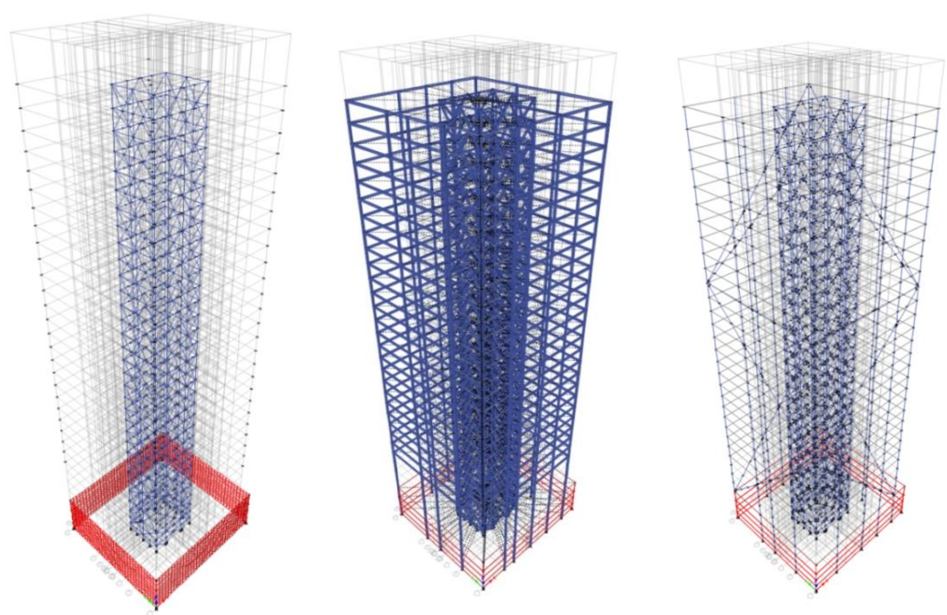


Figure F.4: ETABS Models for All 3 Considered Lateral Systems

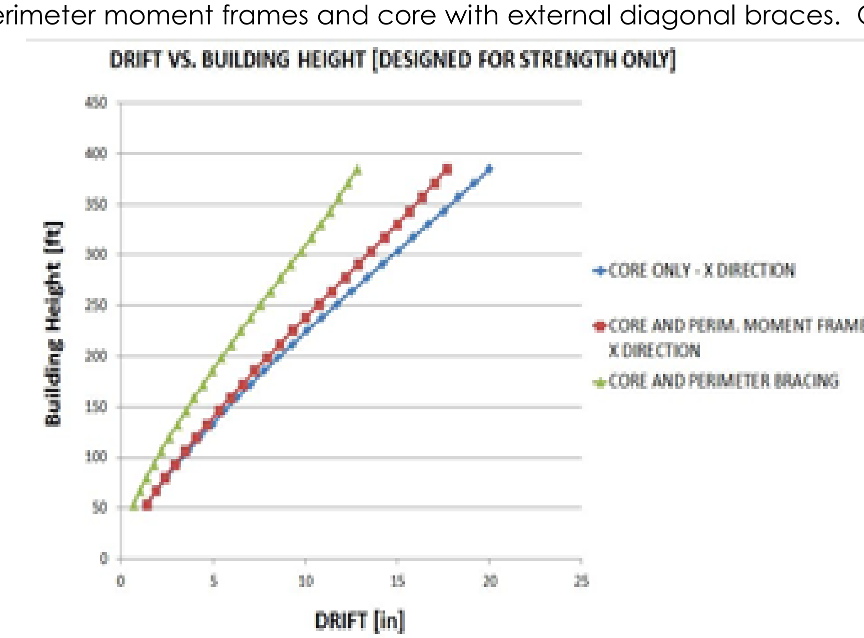
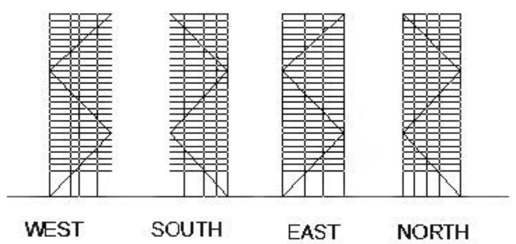


Figure F.5: 3 Potential Lateral Systems Drift Comparison

Layout and Load Path Determination:

Layout:
The structural team proceeded to lay out their external mega brace system. As part of their architectural goals for the project the structural team wished to provide an opportunity for architectural enhancement. The layout of the mega brace system was intended to be as elegant and slender as possible and still articulate the buildings high seismic performance. The system consists of a single line of exterior mega braces on each façade as depicted below. From the ground floor they slope upwards in the south west direction to further highlight 350 Mission's important urban connection to the Transbay Terminal. They also leave the southwest corner entrance to the lobby and cantilever unhindered and open, helping to create an engaging environment with the public street scape.



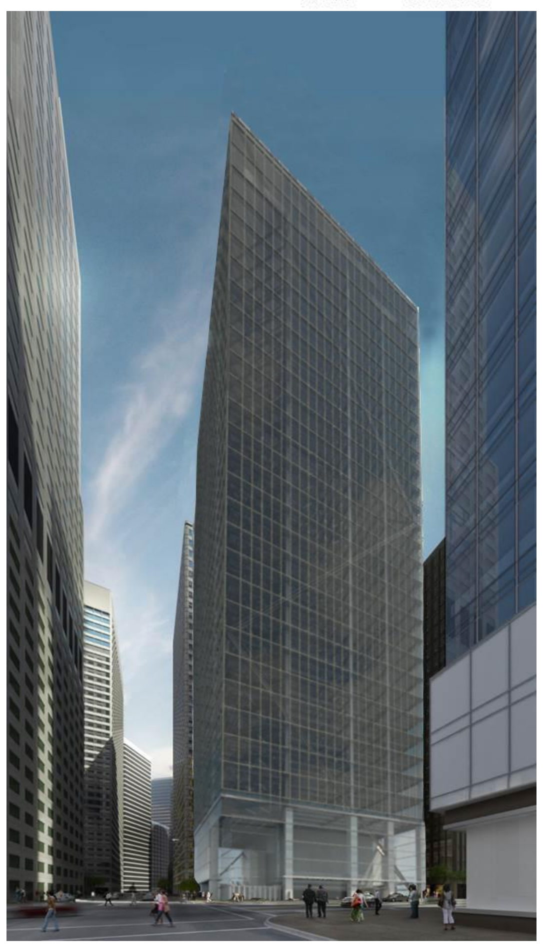
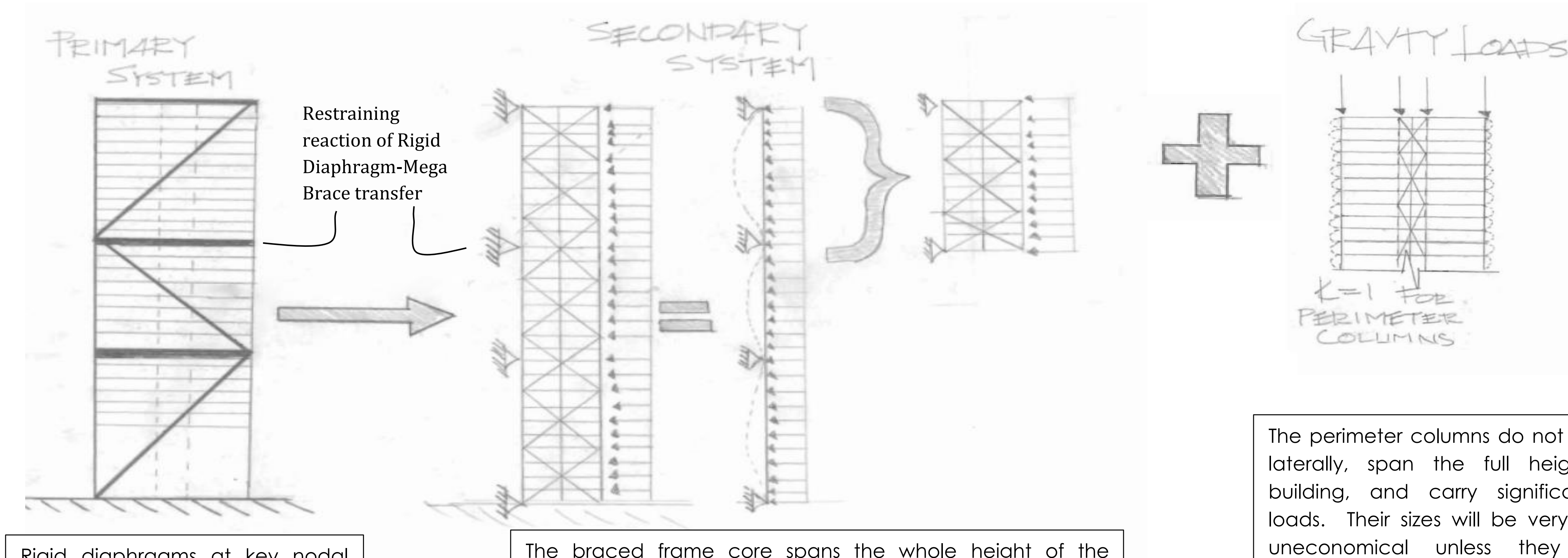


Figure F.6: Exterior Bracing Elevations and Rendering w/ Facade

Explanation of Load Path:

1. Inertial lateral loads are excited in all of the floor diaphragms or wind lateral loads are transferred there via the façade connection.
2. Lateral loads are first resisted by the braced frame core which spans the full height of the building.
3. The braced frame core is restrained at "key nodal levels" by the exterior mega brace system. These key nodal levels transfer the lateral load from the core out to the exterior mega braces through rigid diaphragms at those levels.



Rigid diaphragms at key nodal levels restrain the core and allow lateral load to be "kicked" out of the core and transferred to the mega braces at those stories. The lateral load in these mega braces is then transferred into the foundation walls and mat slab. For a full explanation please see foundation Appendix G.

The braced frame core spans the whole height of the building and is restrained at key nodal levels by a rigid diaphragm transfer to the exterior mega braces. Conceptually the core is like a vertically oriented deep beam spanning between key nodal levels. Preliminary sizing can then be done on this basis of 9-11 story "core modules. " The reactions from this deep beam represent the lateral load transferred out to the exterior mega braces. Preliminary sizing can then be done on this basis of taking reactions from the core modules restraining reactions.

The perimeter columns do not participate laterally, span the full height of the building, and carry significant gravity loads. Their sizes will be very large and uneconomical unless they can be designed with an effective length factor,K, equal to 1. That is to say either the "secondary" core is stiff enough to effectively brace the perimeter columns via the floor diaphragms or it is not and the columns are only effectively braced by the "primary" mega brace system at key nodal levels. This was checked with an iterative P-Delta analysis as explained on Page 12 of the Appendices.

With a load path fully determined, the structural team proceeded to preliminarily size the lateral force resisting system's components. With preliminarily sized components the structural team assembled them together into a complete lateral force resisting system model and preformed MRS analysis. They then made necessary adjustments. They also checked the perimeter columns capacity to withstand second order effects with an iterative P-Delta analysis. This process is outlined below.

1. Preliminary Size Braced Frame Core→Core Design Modules
2. Preliminary Size Exterior Braces→ Core Design Module Reactions
3. Assemble Full Model and Analyze→ Adjust as necessary
4. Verify K=1 for Perimeter Columns →Iterative P-Delta
5. Connection Detailing

1-Preliminary Braced Frame Core Sizing→Core Design Modules:

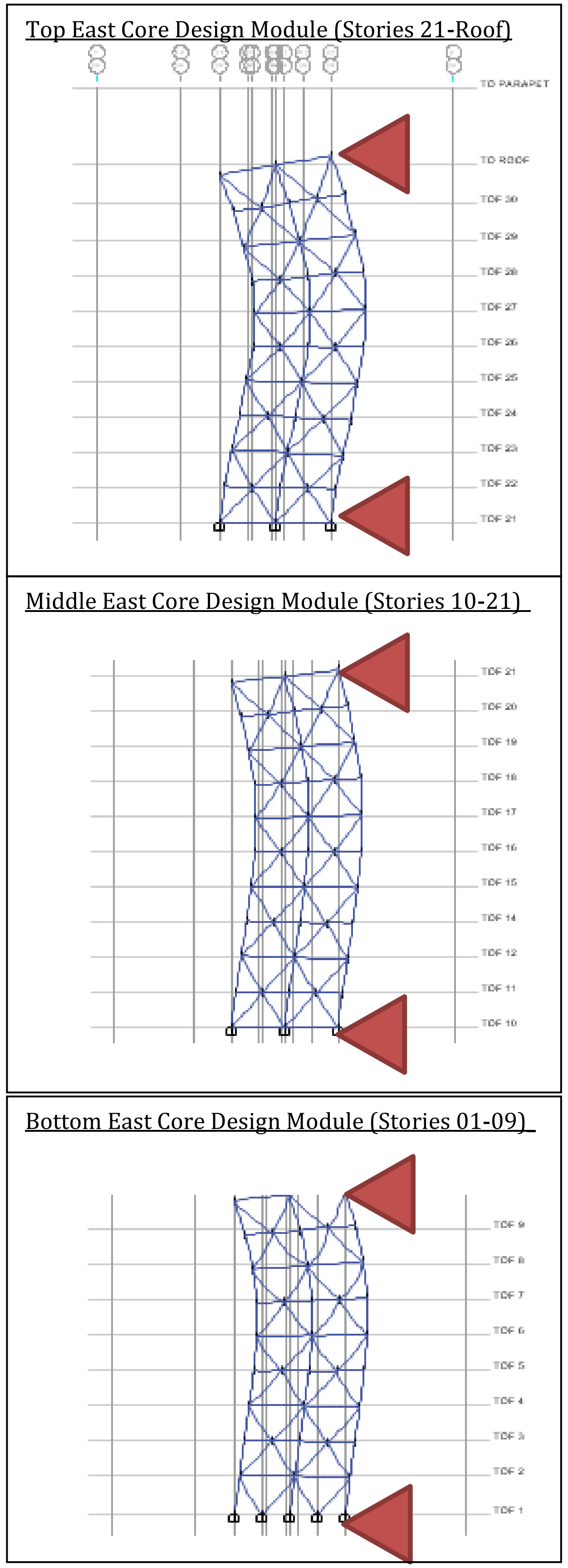


Figure F.7: Core Modules in ETABS for Entire East Elevation

Preliminary sizes for the core were obtained by building "Core Design Modules" in ETABS. These core modules spanned the same number of stories as their respective restraining mega brace. The core design modules were pinned from lateral movement at their top and bottom stories. These reactions represent the lateral load being "kicked out" and transferred to the exterior mega braces. Design iterations were run on these modules.

Lateral loads were input directly as shown below. These lateral loads were taken from the story force results of a past MRS analysis and applied directly to the frames. The core is symmetrical so the structural team distributed story forces by applying 50% of the story forces to each frame because of their same stiffness values.

A total of 12 core design modules were completed, 3 for each face for the core.

East Core Face Story Forces	
Top Core Module	
Story	Force (k)
Roof	145.829
30	139.695
29	132.065
28	125.615
27	118.388
26	110.821
25	103.003
24	95.506
23	88.298
22	81.37
21	74.741

Table F.3: Seismic Story Force Input for Core Module Analysis

2- Preliminary Exterior Mega Brace Sizing→Core Design Module Reactions:

Now the restraining reactions of the core design modules (shown to the left) were taken and combined as applicable. These forces were resolved into the reactions of the mega braces. Spreadsheets were then set up to size the braces in both tension and compression. It was assumed that the floor diaphragms and the spandrel beams that the exterior mega braces cross at each story will effectively brace it for compression. The slight slab overhang also led the structural team to assume that the braces were braced against out of plane buckling by the floor diaphragms.

A sample of the spreadsheets set up is shown below along with the verification of calculations.

Tension:

Input		
ϕ_y=	0.9	
ϕ_r=	0.75	
F_y=	50	ksi
F_u=	65	ksi
P_u=	5057.27	kips
A_g=	147	in^2
A_e=	110	in^2
Output		
Rupture:	ϕP_n=	5362.5
Yield:	ϕP_n=	6615
	Use:	W14X500

$$Try\ W14x500$$
$$\emptyset_y = .9$$
$$\emptyset_r = .75$$
$$F_y = 50ksi$$
$$F_u = 65ksi$$
$$A_g = 147in^2$$
$$A_e = 110in^2$$
$$P_u = 5057.27k$$

$$\emptyset P_{nRupture} = \emptyset_R F_u A_e = 5362.5k \geq P_u = 5057k$$
$$\emptyset P_{nYeild} = \emptyset_y F_y A_g = 6615k \geq P_u = 5057k$$

Compression:

Input		
Ag=	134	in2
k=	1	
L=	209.64	in
ry=	4.38	in
Pu=	5057.27	kips
Output		
kL/(ry)=	47.863	
Fe=	124.939	ksi
4.71(E/Fy)	113.432	
Fcr=	42.2887	ksi
ϕPn=	5100.02	kips
Pu<ϕPn?	YES	

$$Try\ W14x455$$
$$k = 1$$
$$L = 209.64in$$
$$r_y = 4.38in$$
$$P_u = 5057.27k$$

$$F_e = \frac{\pi^2 E}{\frac{KL}{r_y}^2} = 125\ ksi$$

$$\frac{KL}{r_y} = \frac{(1)(209.64)}{4.38} = 47.863 \leq 4.71\sqrt{\frac{E}{F_y}} = 113.43$$

$$\therefore$$

$$F_{cr} = .658^{F_y/F_e} F_y = 42.3k$$
$$\emptyset P_n = \emptyset A_g F_{cr} = 5100k \geq P_u = 5057.2$$

Figure F.8: Mega-Bracing Design Spreadsheets and Validating Calculations

3-Whole System Analysis & Results

With the total lateral system preliminarily sized the structural team assembled a complete lateral force resisting system ETABS model, MRS analysis was performed using the MCE acceleration spectra values provided by the Geotechnical Report. Analysis was also performed in accordance with ASCE7-10 Ch. 12 ∮7 "Modeling Criteria", taking into account cracked section properties for the foundation walls as well as proper fixity of the base. About 50% of the members were overstressed; these were increased as necessary for a final design

One area of interest was the three perimeter columns. These columns which were sized in RAM SS for gravity loading were in fact taking a significant amount of tension introduced at the key nodal stories from the mega braces as shown in Figure F.9. The controlling load combination was .9D+1.0E; this makes sense as the reduction of the dead load meant less counteraction of the tension induced from the mega braces. These columns were increased in size as necessary a sample is shown of the North framing elevation.

The structural team verified their model with extensive study of the shear, moment, and axial force diagrams of both the core and the mega braces. Also by performing an ELFP on their building the structural team verified their analysis was on the right order of magnitude.

The structural team now had a complete design of a concentrically braced system both with and without an exterior mega brace scheme. They then were able to compare the two in order to fully verify their system choice. The structural was thrilled to achieve all the goals as shown in Figure F.10 below. Achieving these values served as great verification of system choice.

As a final system check the structural team also documented their story drifts to verify it was meeting the high performance ½ Code Allowable Drift limit as shown in **Table XX**. It came as no surprise that they were significantly under the drift limit. This led the structural team to postulate in hindsight that there is potential that a moment framed core instead of a concentrically braced frame core could provide sufficient stiffness to span between the primary mega brace system.

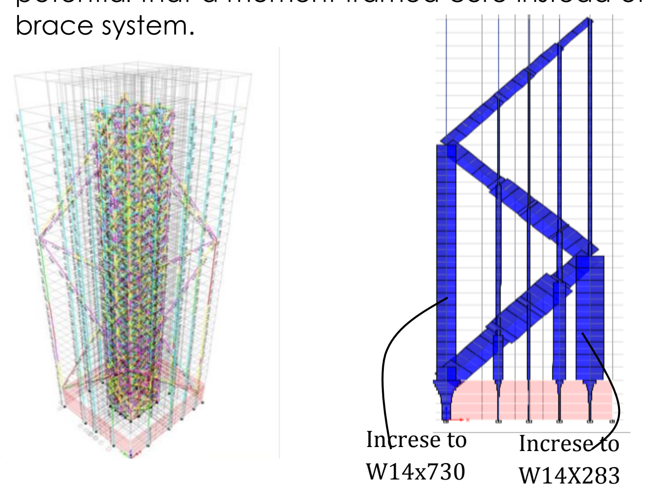


Figure F.9: Full Lateral System ETABS Model and Model Showcasing Tension in Perimeter Columns

	Material Usage Comparison	
	Core Only System	Core and Mega Braces
Total Amount of Steel [tons]	1420	745.1
Cost of Steel [$]	x	x
Average Core Thichness [in]	33	9

48% Reduction in Steel Weight

$8.6 million Savings

24" Reduction in Core Thickness

9100ft² Increase in Rentable Space

Figure F.10: Various Savings Values Due to the Structural System Design

Seismic Importance Factor, I_e=1.0	NOTE: Design and analysis completed on Performance Based Design basis. Use R=1 because system does not have an assigned R value and because no desire for inelastic defformations over lifetime of the building
Response Modification Facotr, R=1.0	
Deflection Amplification Factor, C_d=1.(	

Level	Story Height (ft)	Code Allowable Story Drift (.02hsx) [in]	Code Allowable High Performance Story Drift =1/2(Code Allable)=$\partial_{allowable}$ [in]	MRSA Story Drift=∂	$\partial<\partial_{allowable}$
Roof	14.17	3.40	1.70	0.69	OK
30	13.17	3.16	1.58	0.90	OK
29	13.17	3.16	1.58	1.08	OK
28	13.17	3.16	1.58	0.83	OK
27	13.17	3.16	1.58	1.06	OK
26	13.17	3.16	1.58	1.28	OK
25	13.17	3.16	1.58	1.27	OK
24	13.17	3.16	1.58	1.30	OK
23	13.17	3.16	1.58	1.17	OK
22	13.17	3.16	1.58	1.17	OK
21	13.17	3.16	1.58	0.94	OK
20	13.17	3.16	1.58	0.98	OK
19	13.17	3.16	1.58	1.16	OK
18	13.17	3.16	1.58	0.78	OK
17	13.17	3.16	1.58	0.94	OK
16	13.17	3.16	1.58	0.93	OK
15	13.17	3.16	1.58	0.97	OK
14	13.17	3.16	1.58	1.13	OK
12	13.17	3.16	1.58	1.10	OK
11	13.17	3.16	1.58	1.18	OK
10	13.17	3.16	1.58	1.20	OK
9	13.17	3.16	1.58	1.17	OK
8	13.17	3.16	1.58	1.19	OK
7	13.17	3.16	1.58	1.19	OK
6	13.17	3.16	1.58	1.50	OK
5	54.00	12.96	6.48	6.20	OK
Total	384.17	92.20	46.10	33.32	OK

Table F.4: Story Drifts vs. ½ Code Allowable High Performance Requirement

4- Iterative P Delta Analysis

The perimeter gravity columns as well as the core will carry a significant amount of gravity load. These large gravity forces will weaken the lateral stiffness of both the core and perimeter columns and will induce a high propensity to buckle both globally and locally. A check on this is necessary to ensure that second order effects are not disrupting global stability. It is also necessary because the perimeter columns were designed under the assumption of being effectively braced at each floor. Therefore the core must be stiff enough to effectively brace them at each floor through the floor diaphragms. The structural team checked all these concerns through an initial iterative P-Delta Analysis in ETABS. In the iterative P-Delta analysis a convergence tolerance is set and the structure is analyzed. If either global buckling of the whole structure (core and mega braces) or local buckling (perimeter columns) is induced by the compressive gravity loads, or if deflections do not converge the analysis is terminated as a failure. The preset convergence tolerance in ETABS was used and adequate stiffness of the core was established. The input for the analysis is shown in Figure F.11.

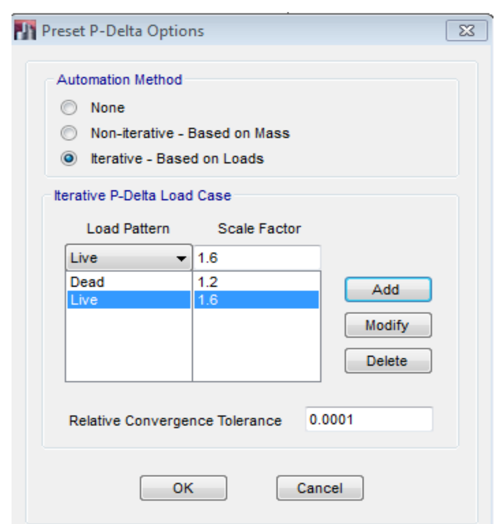


Figure F.11: Iterative P-Delta Input

5- Connection Detailing

Lateral connections in the core were detailed according to the AISC Seismic Provisions and Standards of the Seismic Design Manual. These connections were modeled to fabrication level detail in Tekla Structures. Examples of drafted details of these connections can be found on Drawing S9 along with 3 Dimensional images for coordination purposes. Accurate detailing and modeling of these connections is not only important from a seismic standpoint but also from a coordination standpoint as the MEP vertical distribution occurs in the core. An example of the connection detail is shown to the right in Figure F.12.

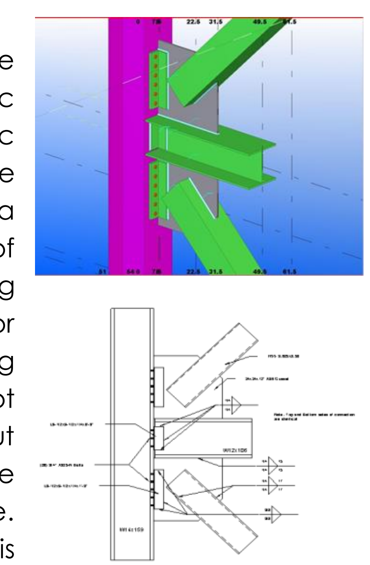


Figure F.12: Lateral Connection Details

Additional Consideration:

The structural team also recognized the opportunity and necessity for collaboration with the Construction Team to accurately analyze the mega brace system for the gravity loads they will see during the construction process. Though these mega braces are lateral elements they do connect to the floor diaphragms and cross both spandrel beams and perimeter gravity columns. The mega frame should be analyzed for gravity loads sequentially with the construction schedule to give an accurate depiction on how loads will be distributed in the construction process.

Appendix G: Foundation System Methodology, Calculations and Description

Mat Slab Reduction:

In collaboration with the mechanical and construction teams, a desire to reduce the thickness of the recommended mat slab was born. A reduced mat slab thickness allows for significant savings in the construction schedule. Additionally, a reduced mat slab thickness allowed for the proper housing of the anaerobic digestion facility the Mechanical team wished to implement. Details on these beneficial impacts can be viewed in the Construction and Mechanical reports, a detailed narrative of the integrated decision to pursue a thinner mat slab can also be found in the Integration report. Examples of the structural team's mat slab calculations and process is depicted below.

1. Check if loads are suitable for soil bearing → 2. Size Mat Slab for punching shear → 3. Reinforcement → 4. Global Stability Checks

1-Preliminary Check on Bearing:

$Pdead = (Total\ Framing\ Weight\ from\ ETABS\&RAM\ Model) + (Substructure\ Weight\ calculated\ by\ Hand)$

$Pdead = 18286k + 11581.5$

$\underline{Pdead = 29867k}$

$Plive = (26\ above\ graade\ office\ loading) + (4\ sub\ grade\ parking)$

$$Plive = \left(\frac{100lb}{ft^2} * 128.33ft * 120.83ft\right) + \left(\frac{40lb}{ft^2} * 128.33ft * 120.83ft\right)$$

$\underline{Plive = 42798k}$

$Ptotal = Pdead + Plive$

$Ptotal = 29867k + 42798k$

$\underline{Ptotal = 72665k}$

$$q = \frac{Ptotal}{A} \le qa = 9\frac{k}{ft^2}$$

$$\frac{72665k}{120.833ft * 128.33ft} \le 9\frac{k}{ft^2}$$

$$q = 4.7\frac{k}{ft^2} \le qa = 9\frac{k}{ft^2}$$

OK, PROCEED WITH MAT DESIGN

2-Size Mat for Punching Shear of Critical Column:

$Pu = 158k$ From Ram SS

$Atrib = 665.4\ ft^2$

$$q_u = \frac{Pu}{Atrib}$$

$$q_u = \frac{158k}{665.4ft^2}$$

$q_u = 1.65\ psi$

$$v_c = lesser\ of \left\{\emptyset\left(2 + \frac{4}{\beta}\right) * \sqrt{f'c}\right\}; \left\{\emptyset\left(\alpha * \frac{d}{b_o} + 2\right) * \sqrt{f'c}\right\}; \left\{\emptyset 4 * \sqrt{f'c}\right\}$$

$\beta = 1$ therefore first equation does not control

assume lat equation controls (succesfully verified; not shown)

assume $f'c = 3000psi$

$\emptyset = .75$

$v_c = 164psi$

$$d^2(4v_c + q_u) + d(2v_c + q_u)(b + c) = q_u(BL - cb)$$

$d = 65$ → H=d+3"clear cover

$\underline{H = 68"}$ → **H = 6ft thick mat slab**

**Mechanical team desired an additional 3-10ft, therefore successful design.

3-Rough Design of Reinforcement

The structural team came up with minimum amount reinforcement necessary for the mat slab based on continuity steel to meet shrinkage and temperature requirements. It is noted that a more in depth analysis and reinforcement of the mat between column locations should be completed to ensure adequate shear and flexural capacities.

$\rho_{s\&t} = .0018$

$$As_{s\&t} = \frac{.0018}{bH}$$

$$As_{s\&t} = \frac{.0018}{(12" * 72")}$$

$$As_{s\&t} = 1.56\ \frac{in^2}{ft}$$

For shrinkage and temperature use **(2) Layers of #8 bars spaced every 12"** horizontally

→Plenty of other options.

4-Global Stability Checks:

a) Check Bearing:

$$e = \frac{Mtot}{Ptot}$$

$$e = \frac{1032573ftk}{86621k}$$

$e = 11.9ft$

$$q_a = \frac{P}{BL} + \frac{Pe6}{BL^2}$$

$$9 = \frac{86621}{B128.33} + \frac{86621(11.9)(6)}{B(128.33^2)}$$

$$\frac{B}{6} = 19.5ft \ge e = 11.9ft$$

OK, in Kern should be ok for OTM

$$q_{max} = \frac{P}{BL} + \frac{Mtot6}{BL^2}$$

$$q_{max} = \frac{86621}{120.833 * 128.33} + \frac{1032573 * 6}{120.833 * 128.33^2}$$

$$q_{max} = 8\frac{k}{ft^2} \le q_a = 9\frac{k}{ft^2}$$

∴ OK FOR BEARING

b) Check Overturning:

$M_{ovt} = 1032573\ kft$

$$M_{resist} = P\left(\frac{B}{2}\right)$$

$M_{resist} = 3586323\ kft$

$$FS_{ovt} = \frac{M_{resist}}{M_{ovt}}$$

$$FS_{ovt} = \frac{3586323}{1032573}$$

$FS_{ovt} = 3.5 \ge 3to5$

OK FOR OVERTURNING

c) Check Sliding:

$$C_a = \frac{1}{3}C = \frac{1}{3} * 229 = 76\frac{lb}{ft^2}$$

$\sum F_v = 75040k$

$\delta = .2$ From Geotech Report

$V_{tot} = 4323.2k$

$$Resistance = R = \left(\sum F_v\right)\tan\delta + BC_a$$

$R = 75040\tan(.2) + 120.833(76)$

$R = 9445k$

$$FS_{Sliding} = \frac{R}{V_{Tot}}$$

$$FS_{Sliding} = \frac{9445}{4323.2}$$

$FS_{Sliding} = 2.2 \ge 1.5\ \therefore$

OK FOR SLIDING

Note: All stability checks shown for the E-W Directions, N-S completed but not shown. Total shear and moment taken from ETABS MRSA model. Conservative assumptions for sliding check: Will not consider surcharge loads against sliding. Cohesion of Colma Sand layer is the common saturated value of 229psf for silty sand.

Figure G.1: Base Shear, Total Loading, and Overturning Moment

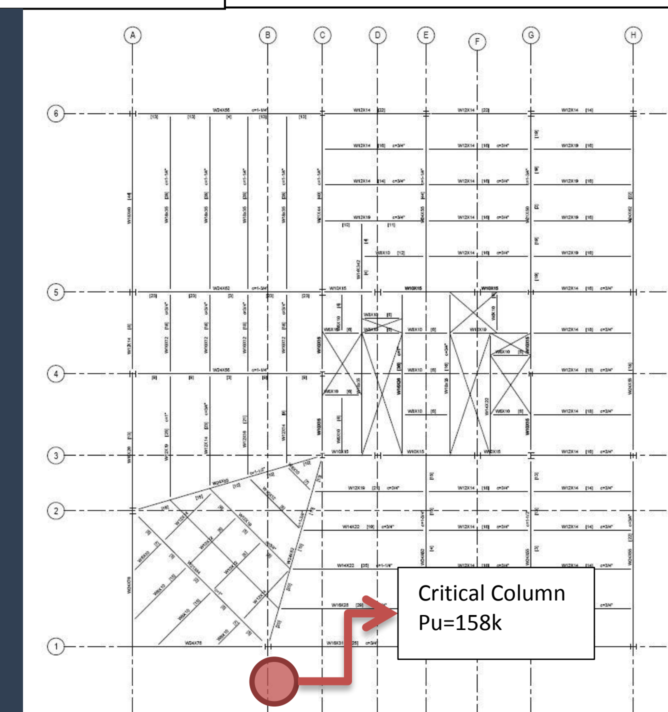


Figure G.2: Key Plan Showing Column Checked for Punching Shear

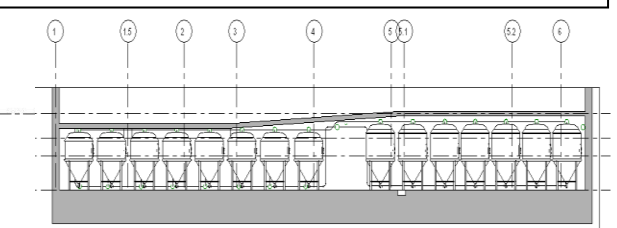


Figure G.3: Section of foundation lowest basement level only showing how the reduced mat slab allowed for a larger floor to ceiling height in the lowest basement level for the housing of the anaerobic digestion system.

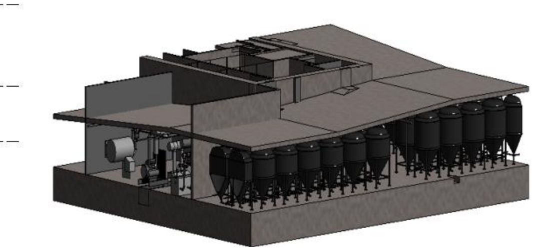


Figure G.4: Isometric image showing how the reduced mat slab allowed for a larger floor to ceiling height in the lowest basement level for the housing of the anaerobic digestion system.

Foundation Wall Loading Condition:

In designing the foundation perimeter walls the structural team also had to consider how the walls not only act to retain the subgrade hydrostatic soil and surcharge loading but also to transfer shear and axial force from the mega braces into the mat slab. Effectively these perimeter foundation walls need to be designed as both retaining structures and as shear walls. This load path is depicted below.

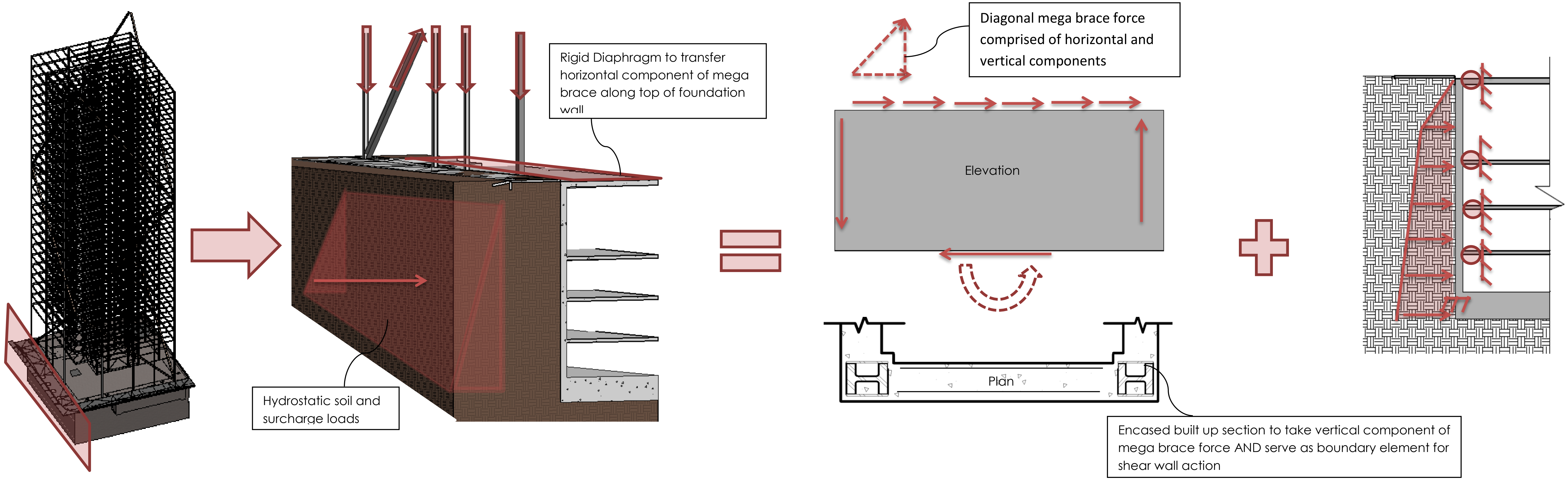


Figure G.5: Schematic Foundation System Load Path

In order to complete the lateral load transfer into the mat slab foundation the structural team needed to transfer the large tensile/compressive forces from the mega braces into the perimeter foundation walls. The mega brace forces are so large that it necessitated a built up section, the same built up section described in the Gravity narrative and appendix (Apppendix E) of this report. These large diagonal forces have both a horizontal and vertical component in the plane of the perimeter wall. The same condition is present on all four sides of the building. This horizontal shear component is intended to be transferred along the top of the perimeter wall via the lobby floor diaphragm acting as a rigid diaphragm. This floor diaphragm was thickened anyway in order to safely house the radiant cooling/heating system. Therefore the structural team settled on the idea of further thickening and reinforcing the lobby floor diaphragm in order to accomplish this transfer of shear force. The vertical component of the mega bracing system is meant to be taken by these foundation shear wall's boundary element: an encased built up section. Additionally these perimeter foundation walls must be able to resist the hydrostatic soil and surcharge loads they are retaining. Figure G.6 shows a one foot strip in plan view of a foundation wall and the identified forces acting on it.

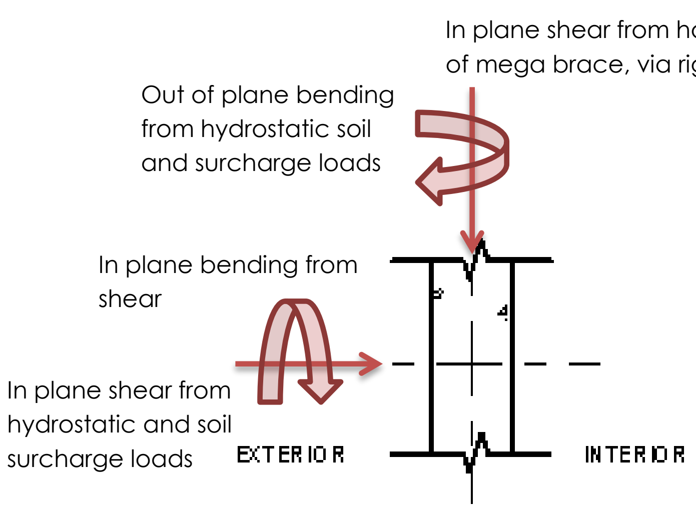
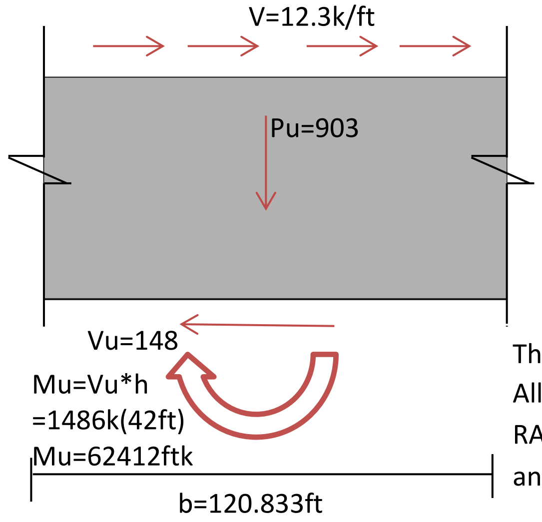


Figure G.6: Foundation Wall Unit with Applicable Forces

With these forces identified the structural team began by designing and detailing the foundation wall for the in plane shear and bending alone as a special structural shear wall according to Chapter 21 of ACI 318-11. They then proceeded to design and detail the foundation wall at its critical section for out of plane shear and bending for the hydrostatic soil and surcharge loads. Then using the principle of superposition combined the two results. These calculations are depicted below. The structural team noted that a more economical solution could be developed considering the full interaction of these forces as well as considering the ability to discontinue certain reinforcing bars at certain locations. While the designs of the retaining wall presented in this submittal are code based prescriptive designs using ACI 318-11 the structural team recognizes that due to the high water table (3' below grade) that these perimeter foundation walls should also be designed as tank structures complying with ACI 350.

Shear Wall Design:



Thickness=t=22in
All values taken from RAM SS gravity model and ETABS MRSA model

Figure G.7: Shear Wall Design Schematic

1-Boundary Elements

a) Calculate Potential Boundary Element Axial Force

$P_{uBE} = \frac{P_U}{2} + \frac{M_u}{d}$

$P_{uBE} = 1937.5k$

$P_{uT} \simeq P_{uc} = 1635k$ (from ETABS MRSA)

$P_{uBEtotal} = P_{uBE} + P_u = 3572k$ *either T or C*

b) Check Need for Boundary Elements ∮21.9.6.3

$A_g = \frac{22}{12} * 120.833 = 221.5ft^2$

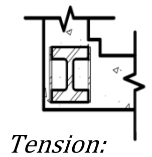
$I_g = t * \frac{b^3}{12} = 269487ft^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{b}{2}}{I_g}$

$f_c = 1.62\ ksi > .2 * f'_c = .2 * 8 = 1.6ksi$

∴ Need Boundary Element; Needed it anayway to take vertical component of mega brace

c) Check Boundary Element Capacity



$A_g = 735.6in^2$

$A_{st} = 259.52in^2$

Tension:

$\emptyset P_n \geq P_u$

$.9F_yA_g \geq P_u$

$11678k \geq 3572k$

OK For Tension

Compression:

$\emptyset P_n \geq P_u$

$.8\emptyset[.85f'_c(A_g - A_{st}) + F_yA_{st} \geq P_u$

$8431k \geq 3572k$

OK For Compression

2-Reinforcement

a) Check 1 or 2 Curtains ∮21.9.22

$V_u \geq 2A_{cv}\sqrt{f'_c}$ *then 2 curtains*

$1486k > 5706k$

Only need 1 curtain, but will need both positive and negative reinforcement for flexure and because below water table ∮14.3 says need 2 ∴ split needed amount into two curtains

b) Vertical Reinforcement Based on Min. *Unit Strip Method*

$\rho_{vmin} = \frac{A_{smin}}{bh}$ ∮21.9.22

$A_{smin} = .396\frac{in^2}{ft}$

∴Provide .198 in²/ft in each curtain

c) Horizontal Reinforcement Based on Min. *Unit Strip Method*

$\rho_{hmin} = \frac{A_{smin}}{bh}$ ∮21.9.22

$A_{smin} = .66\frac{in^2}{ft}$

∴Provide .33 in²/ft in each curtain

d) Check Capacity of Trial Minimum ∮21.9.4.1

$\frac{h_w}{l_w} \leq 1.5 \therefore \alpha_c = 3$

$\rho_h = \frac{.33}{(12 * 22)} = .00125$

$\emptyset V_n \geq V_u$

$\emptyset A_{cv}\left(\alpha_s\lambda\sqrt{f'_c} + \rho_hF_y\right) \geq V_u$

$8214k \geq 1486k$

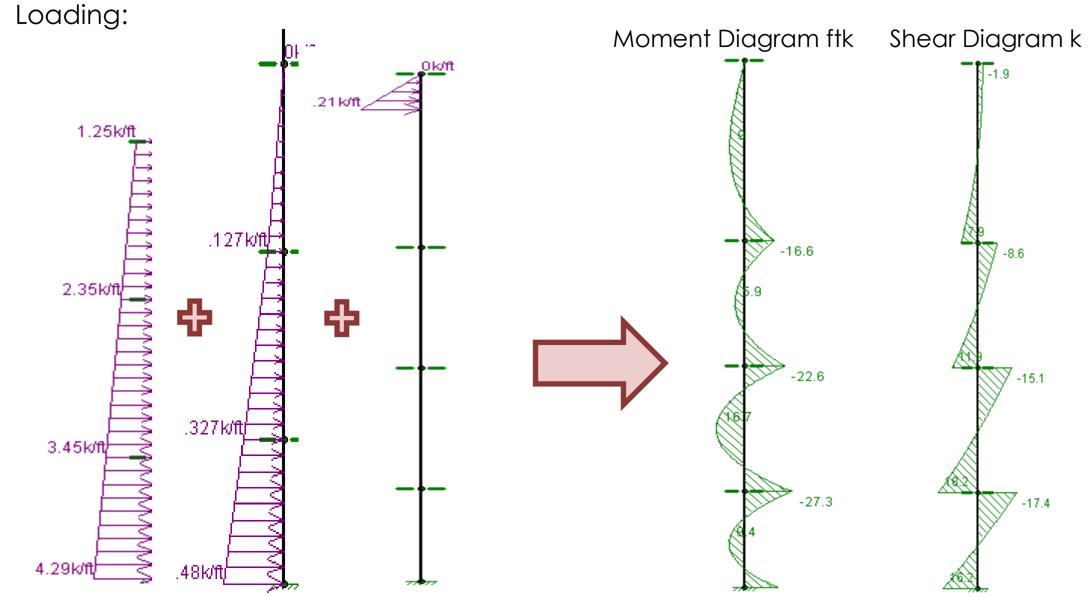
Min. OK therefore superposition of flexural reinforcement OK too

NOTE: f'c=8000psi for the foundation perimeter walls. f'c=3000psi for all other concrete in building. The design was originally completed using 3000psi concrete, and then again with 4000psi concrete, however both wall thicknesses were in excess of 36 inches. In collaboration with the Construction team a desired thickness of 22 inches was established and the need for high strength concrete in the foundation walls.

Retaining Wall Design:

Calculations are shown for the negative reinforcing at the critical section. The same process was carried out for the positive reinforcement as well.

Loading:



Pressure	Source (Per Geotech Report)
20H=20psf(24ft)=.48k/ft	Surcharge from 50 Beale St. @ -18ft
100 lb/ft	Traffic Surcharge (not applicable to direction shown)
(40+30psf)3ft=.210k/ft	Above Water Table Seismic Pressure
(80+30psf)39ft=4.29k/ft	Below Water Table Seismic Pressure

Load Combination=1.6H

$M_u^+ = 1.6(16.7) = 26.7ftk$

$M_u^- = 1.6(27.3) = 43.7ftk$

$V_u = 1.6(17.4) = 27.8k$

Reinforcement

a) Check Shear

$\emptyset V_c = \emptyset 2\sqrt{f'_c}bd$

$.75(2)\sqrt{8000(12)}d_{req} = V_u = 27800$

$d_{req} = 17.2 \leq d = 22" - clear\ cover - .375$

OK → Assumed 3"clear cover and #6 bar

b) Check Flexure (Vertical Reinforcement)

$a = \frac{A_sF_y}{.85f'_cb}$

$a = .735A_s$

$M_u \leq \emptyset M_n = \emptyset A_sF_y\left(d - \frac{a}{2}\right)$

$524.4 = 1005.75A_s - 19.845A_s^2$

$A_{sreq} = .53$

OK Provide .53 in²/ft vertically

Check Ductility

$A_s = vertical\ reinf.from\ shear\ wall + .53$

$c = \frac{1.96A_s}{.85}$

$c = 1.68"$

$\varepsilon_s = \frac{.003}{c}(d - c)$

$\varepsilon_s = .03 > .005$

∴ OK for Ø =.9

c) Shrinkage and Temperature (Horizontal Reinforcement)

$\rho_{min} = .0025 = \frac{A_{smin}}{bh}$

$A_{smin} = .66$

OK Provide .33 in²/ft in each curtain

d)Shear (Out of Plane)

Provided by curtains, provide minimum as 135̊ ties to bind the two curtains together as a cage to aid in constructability

$A_{vmin} = \max of\{\frac{.75\sqrt{f'_c}bs_{max}}{F_y}; or; \frac{50bs_{max}}{F_y}\}$

$A_{vmin} = .12\frac{in^2}{ft}$

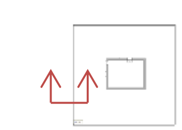
OK Provide #4 135̊ ties at max spacing = 24" ∮11.4.5.1

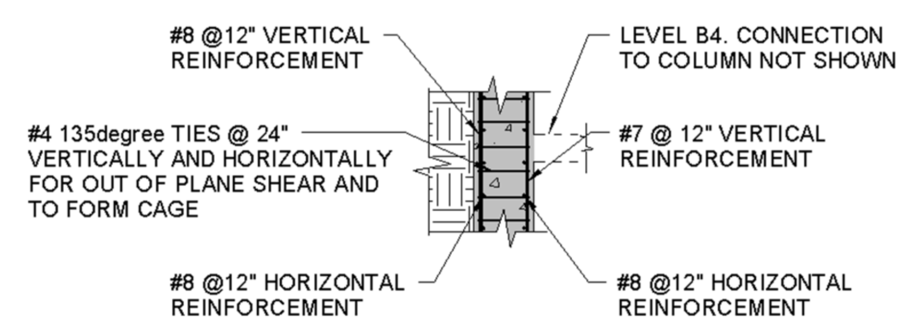
Note: above calculations completed for negative reinforcing at critical section; same process carried out for positive reinforcement as well. Results summarized to the right.

Summary

	Inner Curtain		Outer Curtain	
	Vertical Reinforcing	Horizontal Reinforcing	Vertical Reinforcing	Horizontal Reinforcing
Shear wall As_{req}	.198 in²/ft	.33 in²/ft	.198 in²/ft	.33 in²/ft
Retaining wall As_{req}	.53 in²/ft	.33 in²/ft	.32 in²/ft	.33 in²/ft
Total As_{req}	.73 in²/ft	.66 in²/ft	.52 in²/ft	.66 in²/ft
Design: $As_{provided}$	#8 bars at 12"	#8 bars at 12"	#7 bars at 12"	#8 bars at 12"

Detail shown below:





Appendix H: Building Enclosure Methodology, Calculations, and Description

In order to meet their responsibilities regarding the façade design, the structural team began by garnering an understanding of typical geometries and sizes of high rise curtain wall elements. The structural team did this through preliminary analysis of the MWFRS study (Appendix C) to 350 Mission's specific geometry. The structural team then proceeded to identify specific structural requirements that the chosen façade must meet. This was accomplished through the preparation of a Movements and Tolerances Report. Next the structural team conducted extensive Product Research and Verification (in accordance with ASCE7-10) to identify a product to meet the movements and tolerance criteria as well as the other disciplines' criteria. Finally the structural team conducted an Anchoring Design to detail the chosen system and verify that it could be sufficiently supported. An outline of this process is shown below.

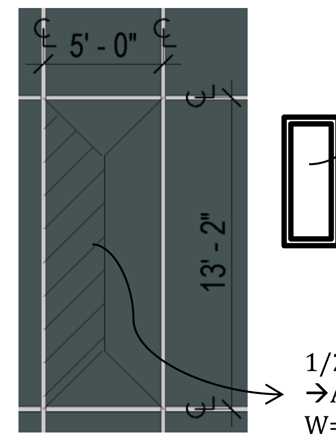
1. Preliminary Sizing and Research → 2. Movements and Tolerance Report → 3. Product Research → 4. Product Selection and Drift Verification → 5. Anchoring Design and Summary

1-Preliminary Sizing and Research

The structural team decided in collaboration with the other project team members to pursue a Unitized system design over a stick built system. These being the two predominant curtain wall systems available for high rise construction, a unitized system offers more inter story movement allowance, greater quality control as units are prefabricated in factory settings, and have very low labor intensity (Wausau).

Assumed dimensions of the façade system shown below were based off the original design, precedent research, and desired mullion spacing of Electrical team for accommodation of their potential shading layout. Preliminary mullion sizing was assumed simply supported at each floor level. Though this is more costly in material compared to some systems which span multiple floors, it results in conservative estimates of sizes. Mullion widths were also assumed as 3 inches: based off product research.

Potential Manufacturers: Kawneer, BISEM, Tubelite, Wausau, Sota Glazing, others.



Assumed mullion section: inner void accounts for 70%of the gross section properties

1/2A$_{trib}$
→A$_{trib}$=(2.5)(13.167+8.167)=53.34sf
W=37lb/ft^2(53.3)/13.167ft=.15k/ft

Aluminium:
$E = 10{,}000\ ksi$
$F_y = 16\ ksi\ (approximate)$

$$M_u = \frac{wl^2}{8}$$
$$M_u = 1.6 * \frac{(.15)(13.167^2)}{8}$$
$$M_u = 5.6\frac{k}{ft}$$

$$M_u \le \emptyset M_p$$
$$M_u \le \emptyset(Z_p - .7Z_p)F_y$$
$$M_u \le \emptyset\left(\frac{bh^2}{4} - \frac{.7bh^2}{4}\right)F_y$$
$$5.6(12) \le .9\left(\frac{.3(3)h^2}{4}\right)16$$
$$h \ge 5.5''$$

OK → fits with precedent research; information relayed to mechanical and electrical teams for their glazing research

Figure H.1: Mullion Spacing Schematic and Calculations

2-Movements and Tolerances Report

Vertical Movement: Horizontal Panel Joint

The horizontal joint between cladding panels must allow for the differential movement between vertically adjacent floors. The horizontal panel joint must be able to accommodate all such movements otherwise individual panels could fail in the resulting tension/compression. The structural team investigated the potential contributors to this motion and their magnitude so that a proper system could be selected:

- Live Load Deflection of spandrel member
- Thermal movement of cladding panel's framing element (vertical mullion)
- Installation and/or manufacturer tolerance

Live Load Deflection of spandrel member:
The structural team revisited their gravity analysis and identified the south east spandrel girder to be the controlling deflection with **1.18"** of live load deflection.

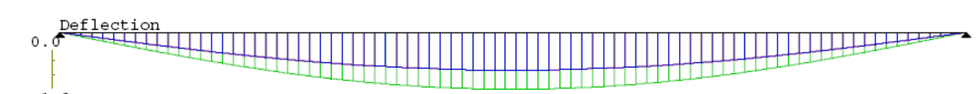


Figure H.2: Live Load Deflection Diagram of Spandrel Member

Thermal movement of framing elements:
After the framing elements of the cladding have been manufactured in a controlled environment assumed to be 68F, they will expand/contract along their length. This movement of the vertical mullions must be allowed for in the horizontal panel joint. In coordination with the Mechanical team a range of worst case temperatures was identified as: *exterior: 38F-95F; interior: 68F-85F*. Aluminum expands/contracts by the formula: $\Delta l = 12.3 * 10^{-3}(l)(\Delta T)$ Thus the structural team was able to calculate an expected expansion and compression of the horizontal panel joint of **.052"** and **.058"** respectively.

Installation/Manufacturer Tolerances:
Even though the American Architectural Manufacturer's Association (AAMA) does not publish any industry-wide standard of tolerances for curtain wall systems, certain independent publishers do. The Glazing Manual as well as The Handbook of Construction Tolerances recommends a clearance of **.25"**.

	Expansion	Contraction
Live Load Deflection	1.183"	1.183"
Thermal Movement	.052"	.058"
Installation/Manufacturer	.25"	.25"
Total	**1.485"**	**1.491"**

Table H.1: Façade Vertical Movement Considerations

This section (Movements and Tolerances Report) continued on next page)

Horizontal Movement: Vertical Panel Joint

The structural team next considered how the cladding panels would move under horizontal loading. The vertical panel joint must accommodate all such movements otherwise individual panels could fail in the resulting tension/compression. The structural team investigated the potential contributors to this motion and their magnitude so that a proper system could be selected:

- Building Sway (seismic drift or wind)
- Thermal movement of cladding panel's framing element (horizontal mullion)
- Installation and/or manufacturer tolerance

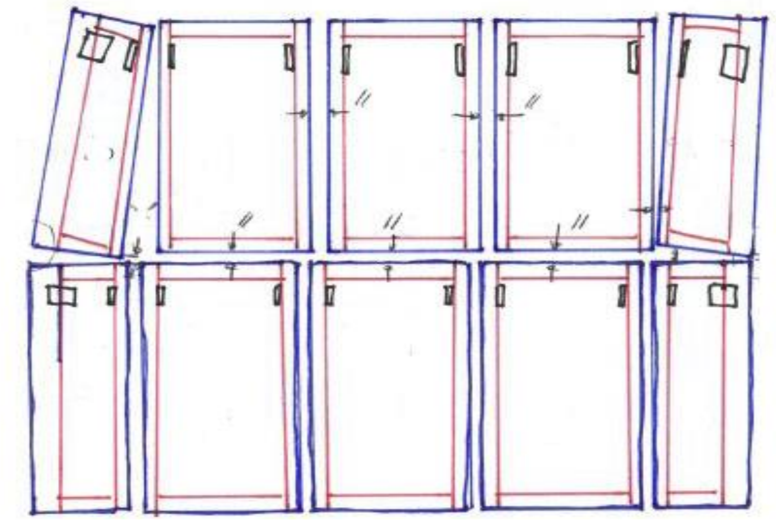


Figure H.3: Unitized System Drift Behavior (Provided by Scheldebouw B.V.)

Building Sway (inter-story drift)

Under inter-story drift a unitized system will drift together along the length of the floor diaphragm. This will occur until the corner panel is restrained by its interlocking panel perpendicular to the direction of drift. Thus the vertical panel joints must be able to handle this compression. Figure XX is representative of how a unitized system moves under drift. It has already been established that seismic forces are controlling the drift requirement:

$Seismic\ Drift$:

$.02h_{sx}\left(\frac{1}{52}\right) \rightarrow ASCE7\ allowable\ interstory\ drift\ with\ 50\%\ high\ performance\ requirement$

$.02(13.167)(12)(.5)$

$1.58"\ \mathrm{max}\ allowed$

$1.15"max\ interstory\ drift\ from\ MRSA<1.58"\mathrm{max}\ allowed$

Therefore the max possible compression for the vertical joint under seismic loading is **1.15".** The structural team chose to use this drift value (from their MRSA) as opposed to the maximum allowable of 1.58" from ASCE7 because evidence has been shown that the ASCE7 value for allowable seismic interstory drift is overly conservative in regards to curtain wall panel movement (Gowda, et. al.).

Thermal Movement of framing elements:

Again with the assumption of manufacturing taking place in a controlled environment of 68F, the horizontal panel framing elements (horizontal mullion) will experience exterior and interior temperature variations of 38F-95F and 68F-85F respectively. Aluminum expands/contracts by the formula: $\Delta l = 12.3 * 10^{-3}(l)(\Delta T)$ Thus the structural team was able to calculate an expected expansion and compression of the horizontal panel joint of **.02"** and **.022"** respectively.

Installation/Manufacturer Tolerance:

Similarly as explained above in the Vertical Movement section, a clearance of **.25"** is recommended for the installation and manufacturer processes.

	Expansion	Contraction
Live Load Deflection	1.183"	1.183"
Thermal Movement	.052"	.058"
Installation/Manufacturer	.25"	.25"
Total	**1.485"**	**1.491"**

Table H.2: Façade Horizontal Movement Considerations

3-Final Product Research

Now with a full understanding of the general size of façade components, their expected movements/deflections, and type of general system desired; the structural team began looking at specific products to meet these criteria. In researching potential products the structural team further refined their specification of a high performing product. Recent tests conducted by Memari, et. al. have shown via AAMA 501.6 testing results that the use of Rounded Corner Glass (RCG) lites as opposed to standard rectangular glass lites can reduce stress concentrations and thereby sufficiently increase a curtain wall system's cracking and fallout drift capacities by as much as 50%-90% depending on type of glass and finish. An image of RCG can be seen in Figure H.4. By specifying a simple, and often economical, annealed insulated glass lite with the applicable unitized system, the façade's drift capacity can increase significantly (Memari, et. al.). While the current code philosophy permits cracking of glass and minor damage to framing elements under inelastic drifts, the structural team wished to keep the lifecycle costs and performance of the building in prime importance. The use of RCG will help accomplish the high performing qualities desired.

Further product research led the structural team to specify a 4 Sided Structural Sealant Glazed System (4SSG) over the commonly used 2 Sided Structural Sealant Glazed System (2SSG) or Gasketed Systems. Recent AAMA 501.6 racking tests conducted by Memari, et. al. show an increased cracking drift capacity of 4SSG curtain wall systems over dry glazed and 2SSG systems of 146% and 56% respectively. An example of a 4SSG can be seen in Figure H.5. Now the structural team has further detailed their specification requirements for a high performing façade to include both RCG and a 4SSG system.

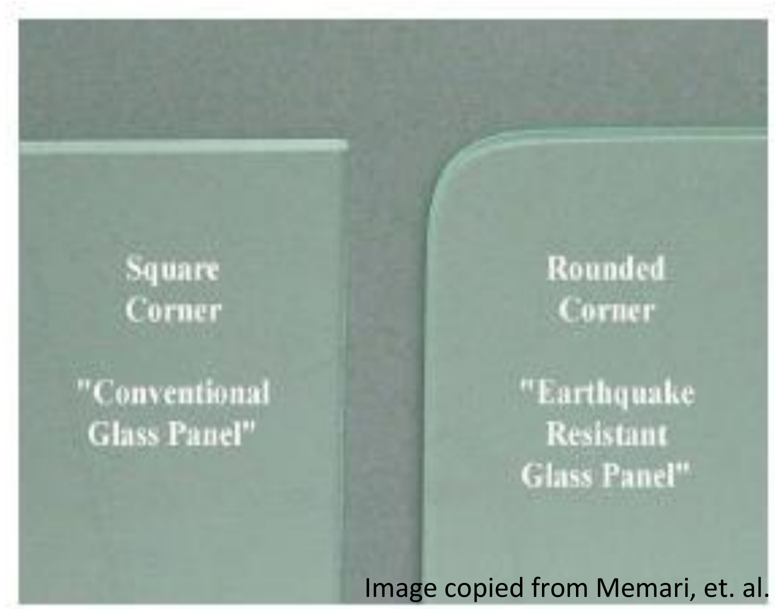


Figure H.4: Schematic of Rounded Corner Glass (Provided by Memari, 2006)

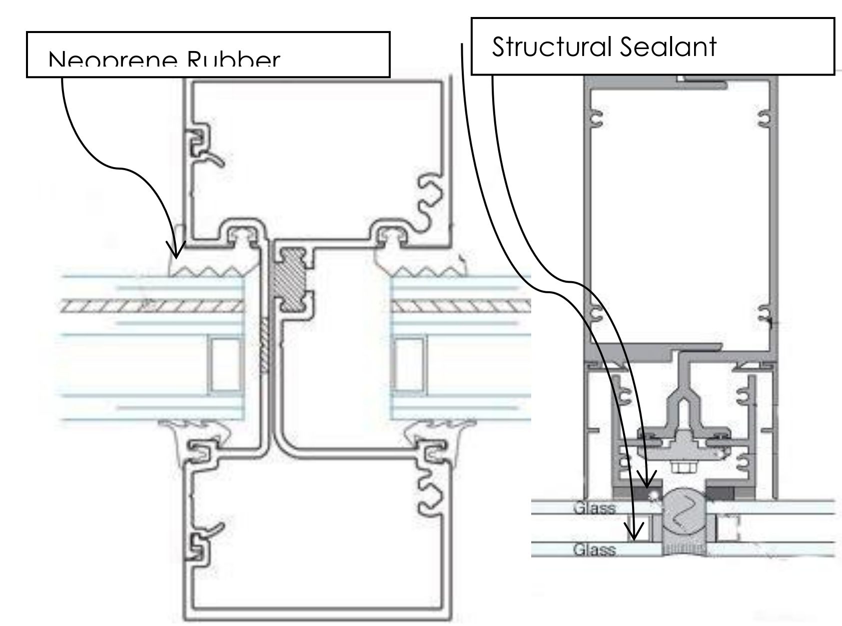
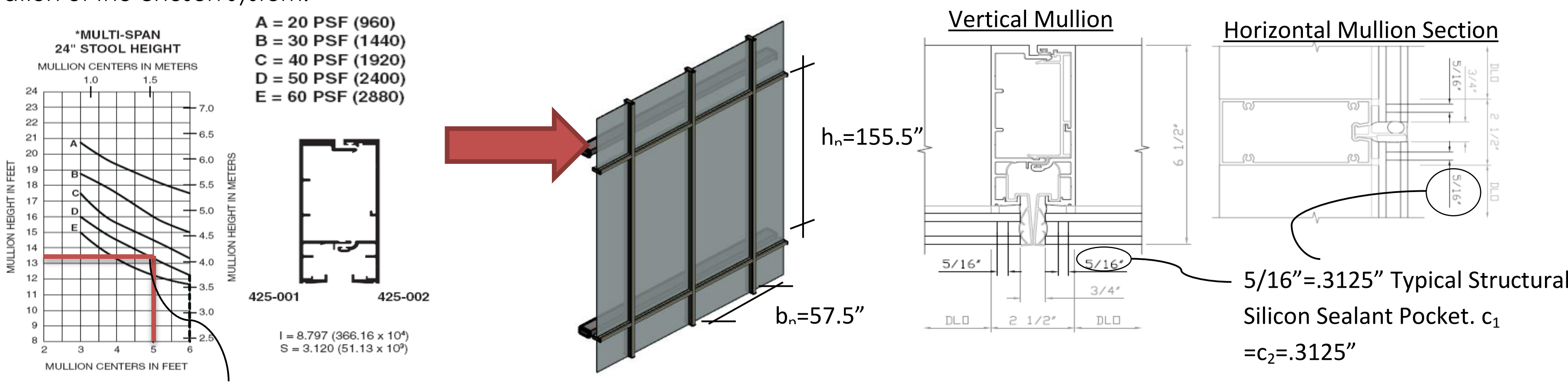


Figure H.5: Detail of Gasketed vs. Structural Sealant Glazed Systems (Provided by Wausau)

4- Product Selection and Drift Verification

With all this product knowledge documented along with the other disciplines, the structural team was able to identify a unitized, 4SSG, RCG system compatible with our buildings loads and geometry: The Kawneer 2500 PG Unitwall, Verification of selection was then completed using the specific product's cut sheet wind load charts (provided by Kawneer).

Next because the chosen system (like most curtain wall systems) does not have published AAMA 501.6 racking test data, the structural team had to verify its drift capacity according to ASCE7-10 section 13.5.9. This section permits curtain wall drift capacities to be checked via modified Bouwkamp Equation when test data is not available. The Bouwkamp equation requires curtain wall systems to have a drift capacity (D_{clear}) at least 25% larger than the minimum necessary glass fallout drift capacity as determined by analysis (D_p). This minimum necessary value was previously identified by the Movements and Tolerances Report as the total horizontal expected drift. The following calculations show the verification of the chosen system.



E=60psf>MWFRS

$I_p = 1.0$
$D_p = min.\ necessary\ drift\ from\ Movements\ and\ Tolerances\ Reprot\ section = 1.42"$
$h_p = 155.5"$
$b_p = 57.5"$
$c_1 = c_2 = .3125"$

$D_{clear} > 1.25 I_p D_p$

$$2c_1\left(1+\frac{h_p c_2}{b_p c_1}\right) > 1.25 I_p D_p$$

$$2(.375)\left(1+\frac{155.5(.375)}{57.5(.375)}\right) > 1.25(1.0)(1.42)$$

2.31">1.8"

OK, chosen Kawneer 2500 PG Unitwall verified for drift capacity

Figure H.5: Façade System Data, Schematic and Details (All Images from Kawneer)

5- Anchoring Design and Summary

The curtain wall manufacturer (Kawneer) was able to provide fastener design in the form of bi-directionally adjustable single angles. These single angle fasteners are specific to this unitized system and require less installation time and effort than many of the double angle mullion fasteners that are often used. This also reduces the chances of fit-up error and their associated built in stresses. These single angle fasteners are bolted just below the surface of the slab on metal deck after curing and then grouted over for finishes. The bolts holding this system to the slab were checked according to ASCE 7-10 Ch. 13 ∮13.5.1 for their ability withstand seismic loads. This system is shown in Figure H.6 below as provided by the manufacturer, Kawneer.

Additionally the structural team was tasked with the responsibility of designing the anchoring of this system to the façade. The structural team designed the slab overhang in order to induce no torsion into the spandrel beams. This was accomplished by designing the slab overhanging in accordance with the ***AISC Design Guide 22 – Façade Attachments to Steel Frame Buildings***. The structural team chose the economical choice of resolving the moment induced by the façade by using a properly reinforced slab overhang and light gauge metal pour stop. The method, calculations, and detail are shown below.

Development Length of Hook

ACI 318-11 ∮12.2

$\varphi_e = 1.0$
$\varphi_t = 1.0$
$\varphi_s = .8$
$f_y = 60000psi$
$d_b = .375"$
$c_b = 2$
$k_{tr} = 0$
$f'_c = 4000$

#3 hook in tension

$$l_{dh} = \left(\frac{.2 f_y \varphi_e}{\lambda\sqrt{f'_c}}\right) d_b \ge 8d_b\, or\, 6"$$

$l_{dh} = 9.5 < 14"available$

#3 bar backspan

$$l_{dh} = \frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\varphi_t\varphi_e\varphi_s}{\frac{c_b+k_{tr}}{d_b}}(d_b) \ge 12"$$

$l_{dh} = 12"$

Design Reinforcement and Pour Stop:

$M_u = M_{slabsw} + M_{ufacade}$

$$M_u = 1.4\left[\frac{wh_{slab} l_{oh}^2}{2} + P_{facade}(l_{oh} + 2")\right]$$

$M_u = 6.8kin$

OK → per AISC DESIGN GUIDE 22 AND SDI TABLES 5 – 1 AND 5 – 2 (Figure XX)
USE #3BARS AT 12" WITH TYPE 12 LIGHT GAUGE METAL POUR STOP

Check Shear Strength:

concerned about 1way wide beam shear in one overhanging bay $\rightarrow A = b_w l = 49.4ft^2$

$$q_u = 1.2\left(150\frac{lb}{ft^3}\right).4ft + 1.6(100) = .232\frac{k}{ft^2}$$

$V_u = q_u A = .22(49.4) = 11.5k$

$$\emptyset V_n = V_c = \emptyset 2\lambda\sqrt{f'_c b_w l}$$

$\emptyset V_n = 123k \ge V_u = 11.5k$

OK for shear

Check on Fastener Bolts for Seismic

ASCE7-10 ∮13.5.1

$a_p = 1.25$
$R_p = 1.0$
$I_p = 1.0$
$S_{DS} = 1g$

$$w_p = 10\frac{lb}{ft^2}(5ft)(13.167ft) = .66k$$

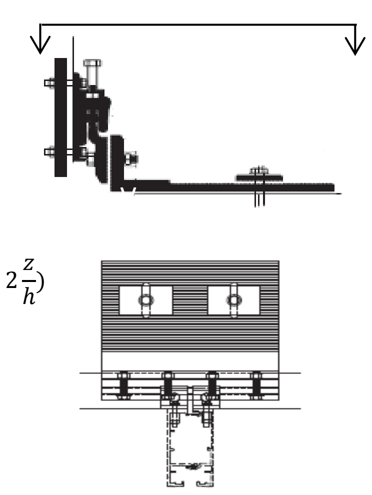
$z = h = 384.167'$

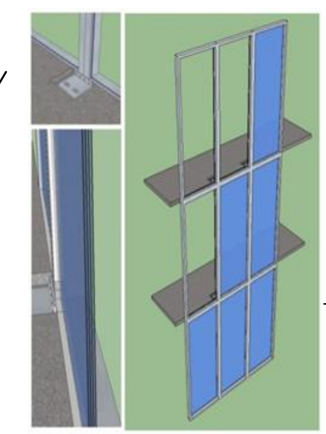
$$.3S_{ds}I_p w_p \le F_p = [.4a_p S_{ds} w_p/(\frac{R_p}{I_p})](1+2\frac{z}{h})$$

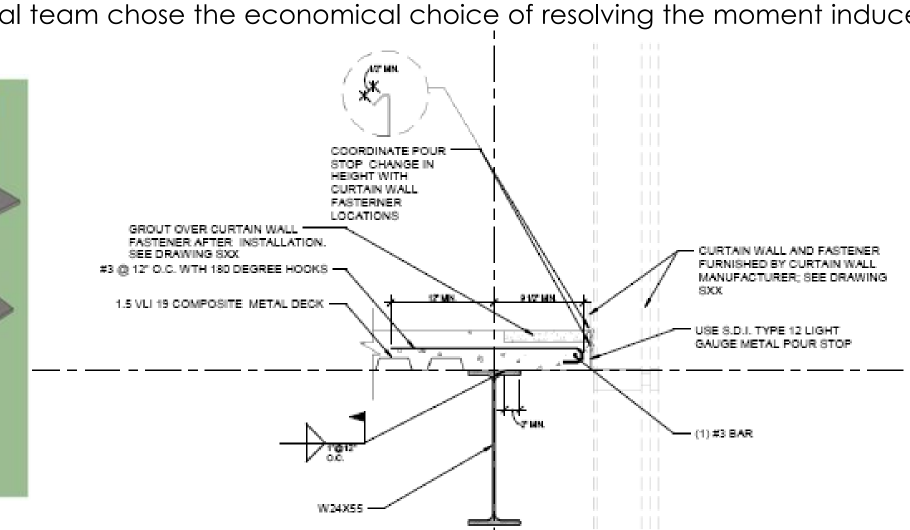
$F_p = 1k$

1/2" bolts ok by inspecition

See Section Below







SLAB DEPTH (INCHES)	OVERHANG (INCHES) 0	1	2	3	4	5	6 POUR STOP TYPES	7	8	9	10	11	12
4.00	20	20	20	20	18	18	16	14	12	12	12	10	10
4.25	20	20	20	18	18	16	16	14	12	12	12	10	10
4.50	20	20	20	18	18	16	16	14	12	12	12	10	10
4.75	20	20	18	18	16	16	14	14	12	12	10	10	10

Table 5-2. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength $f'_c = 3{,}000$ psi
2-in. Composite Floor Deck Parallel to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.									
Bars	in.²/ft	4	4½	5	5¼ [6]	5½	6	6³⁄₁₆ [8]	6½ [7]	7	7¼ [9]
#3@18 [5]	0.0733	2.92	4.89	6.86	7.85	8.83	10.8	11.5	12.8	14.7	15.7
#3@16 [5]	0.0825	3.28	5.52	7.76	8.88	10.0	12.2	13.1	14.5	16.7	17.8
#3@12	0.110	4.18	7.15	10.1	11.6	13.1	16.1	17.2	19.0	22.0	23.5

Figure H.6: Slab overhang Design Tables (AISC Design Guide 22) and Façade Attachment Details (Provided by Kawneer)

Appendix I: Sustainability, LEED and Environmental Analysis

Sustainability and LEED:

The structural team wanted to make sure that their high performing system was not only effective from a structural stability standpoint but from an environmental lifecycle perspective as well. Early on in the design process steel was established as being the structural material with the greatest potential for forming a lightweight yet stiff structure. This coupled with steel lateral systems' ability for such possibilities as targeted yielding and various passive methods of energy helped lead the structural team towards choosing steel as the primary structural material. Thus they began researching ways in which to minimize the environmental impact when buildings and designing with steel. Listed below are items pertaining to the design and construction of the structural system where the project team is achieving LEED points. Please see the Construction Report for a more detailed explanation of some of these items and how they relate to construction activities.

- Regional Materials-[2 Points]: Use building materials located within 500 miles of the building site.
  - Steel Fabricators
    - Prefabricated Core Braced Frames (See Construction Report for a detailed explanation of the benefits of prefabrication)
    - Prefabricated Rebar Cages (See Construction Report for a detailed explanation of the benefits of prefabrication)
  - Concrete Suppliers
  - Aggregate and CMU's from Local Suppliers
  - Formwork from Local Lumber Yards

Please see the below table, a piece of the subcontractor log maintained by the Construction Team, estimating the environmental impact of using structurally applicable locally available materials. Also please see the figure to the right, a map depicting the research done on the locations of these materials/services in order to ensure that pursuing these sorts of environmental savings are feasible. The entire table and figure can be found in the Construction Report.

SUBCONTRACTOR/SERVICE	LOCATION	DISTANCE	COST OF FUEL*	GAL	EQUIVALENT GRAMS OF CO_2
CO 3.0 – CONCRETE INSTALLATION	Burlingame, CA 94010	19 MILES	$7.62	2.05	20870
CO 3.1 – ADDT'L CONCRETE MAT'L	San Francisco, CA 94124	4 MILES	$2.43	0.65	6617
CO 4.0 – MASONRY	San Francisco, CA 94121	7 MILES	$4.77	1.28	13030
CO 5.0 – STEEL ERECTOR	San Francisco, CA 94124	5 MILES	$3.23	0.87	8857
CO 5.1 – STEEL FABRICATOR	Salinas, CA 93912	108 MILES	$40.22	10.81	110046
CO 5.2 – STEEL FABRICATOR	Richmond, CA 94804	15 MILES	$6.15	1.65	16797

Table I.1: Potential Suppliers and Manufacturers in San Francisco Area

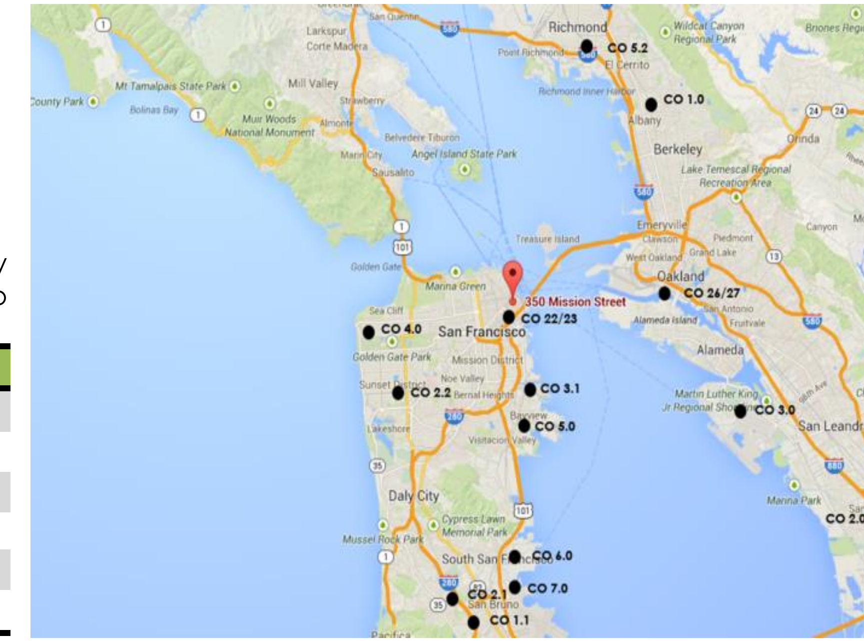
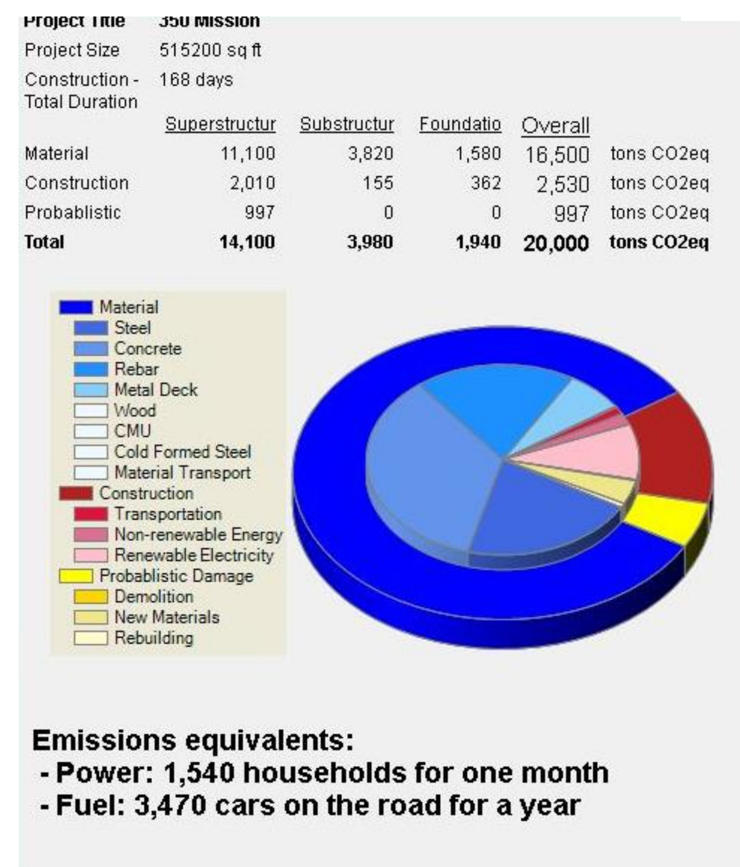


Figure I.1: Location of Potential Manufacturers and Suppliers in San Francisco

Environmental Analysis:

The structural team wished to reduce the amount of carbon embodied in their system as much as possible. Thus taking a Performance Based Design standpoint and performing all lateral system design with an R=1 fit well with our intended behavior for the building: no inelastic deformations occurring over its lifetime. This meant that additional embodied carbon would not be introduced into the buildings structural system in the form of structural repairs after every major seismic event. In order to gauge the environmental impact of their designed system the structural team made use of SOM's Environmental Analysis Tool™. This tool enabled the structural team in coordination with the Construction team to estimate the equivalent carbon dioxide emissions for the structural system with consideration of initial construction, service life, repair, and deconstruction. The construction team also made use of SOM's Environmental Analysis Tool™ concrete building of the similar size and location to 350 Mission for comparison purposes. The results of the analysis are presented to the right. For full explanation of this analysis please see Integration Report and Construction Report.



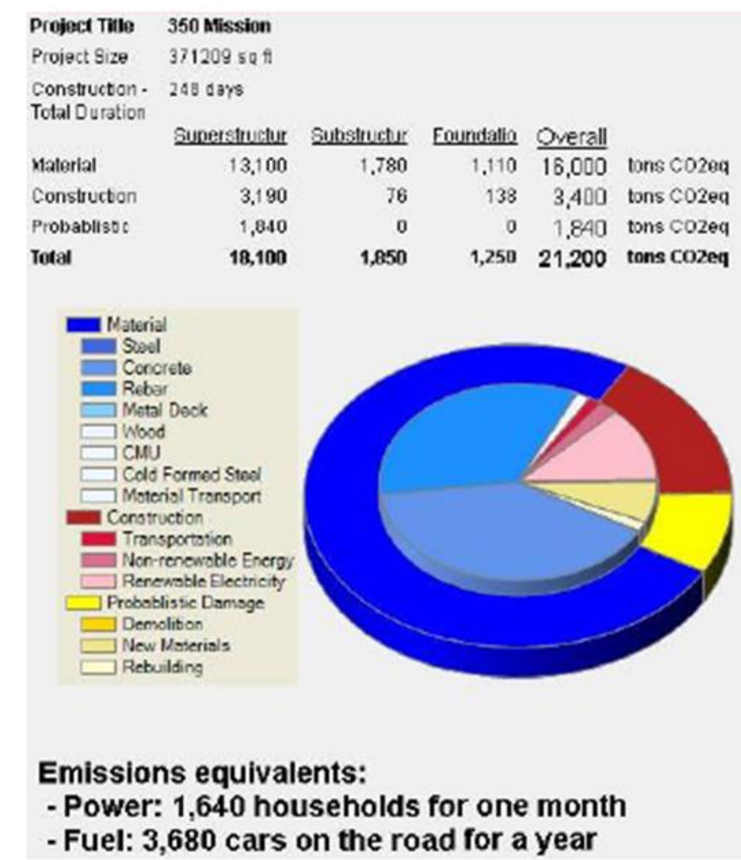


Figure I.2: SOM Environmental Analysis Tool Output

Appendix J: References

Below please find a numbered list of all references used in this report.

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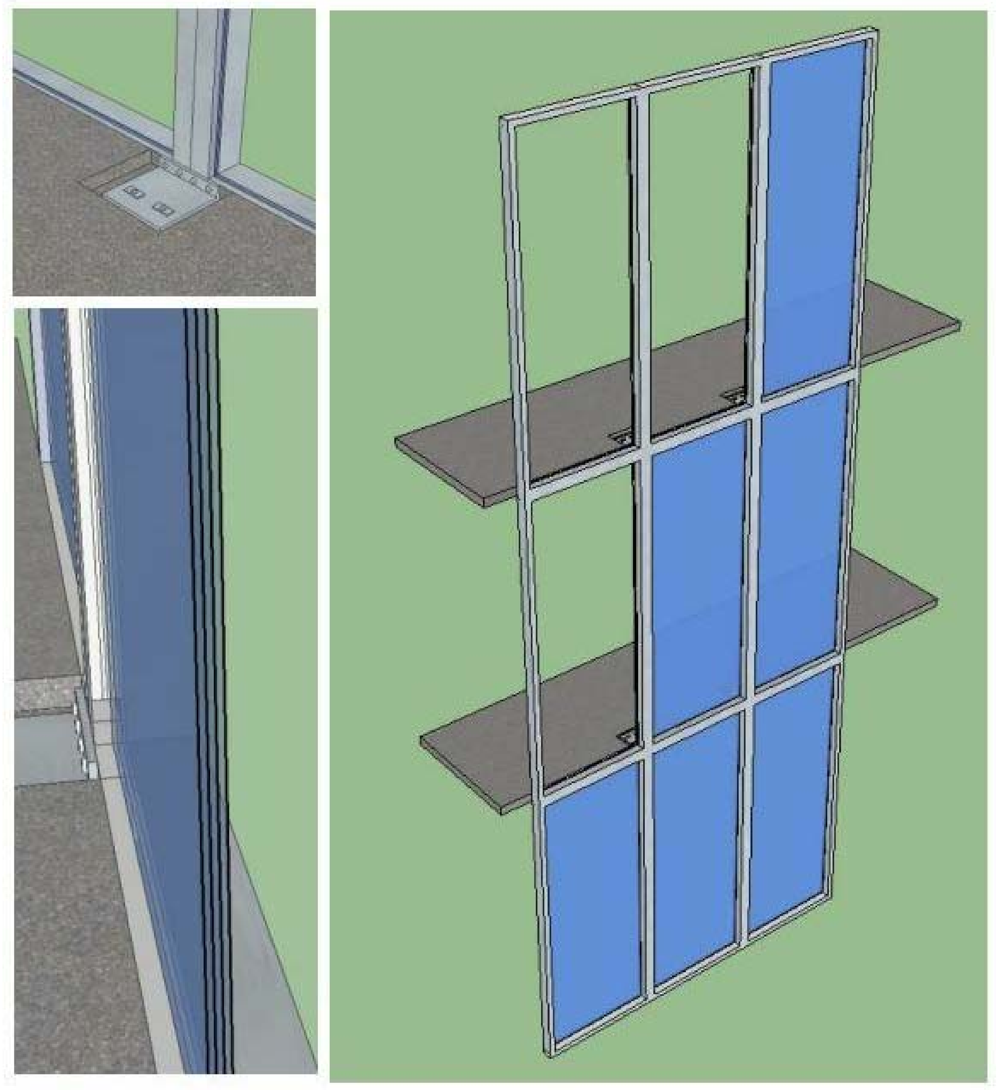
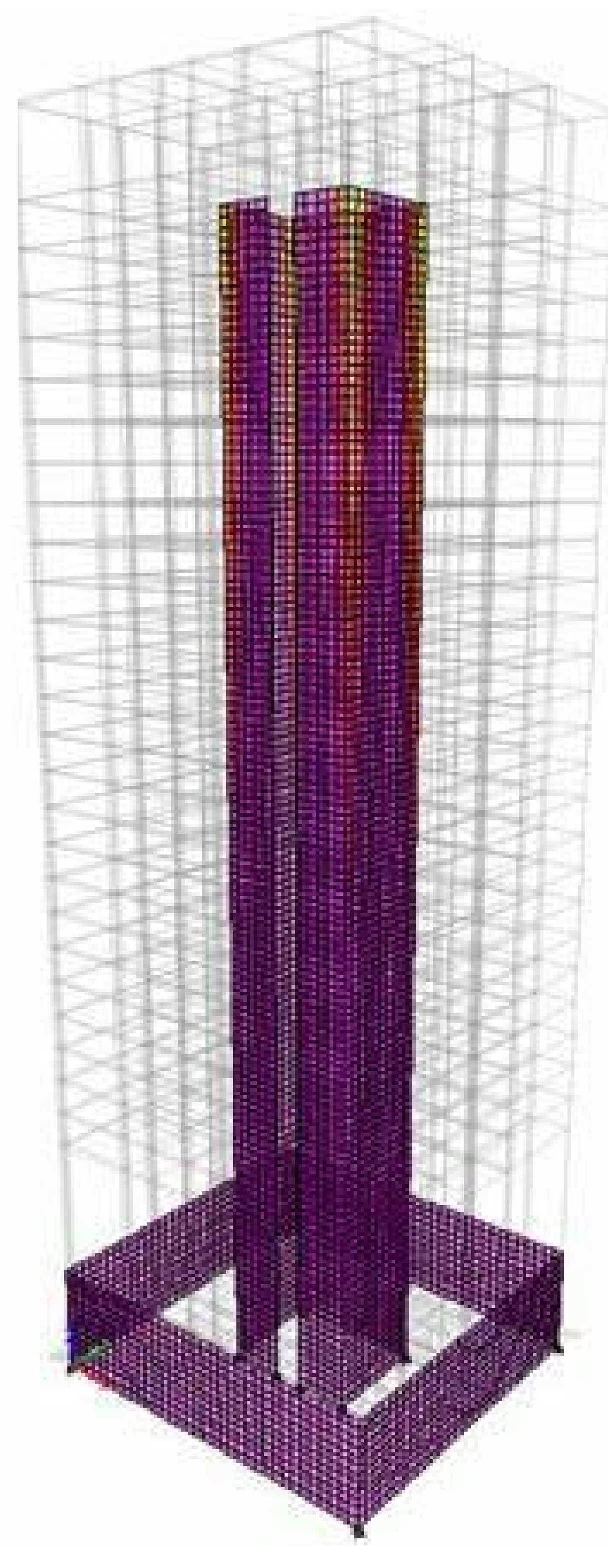
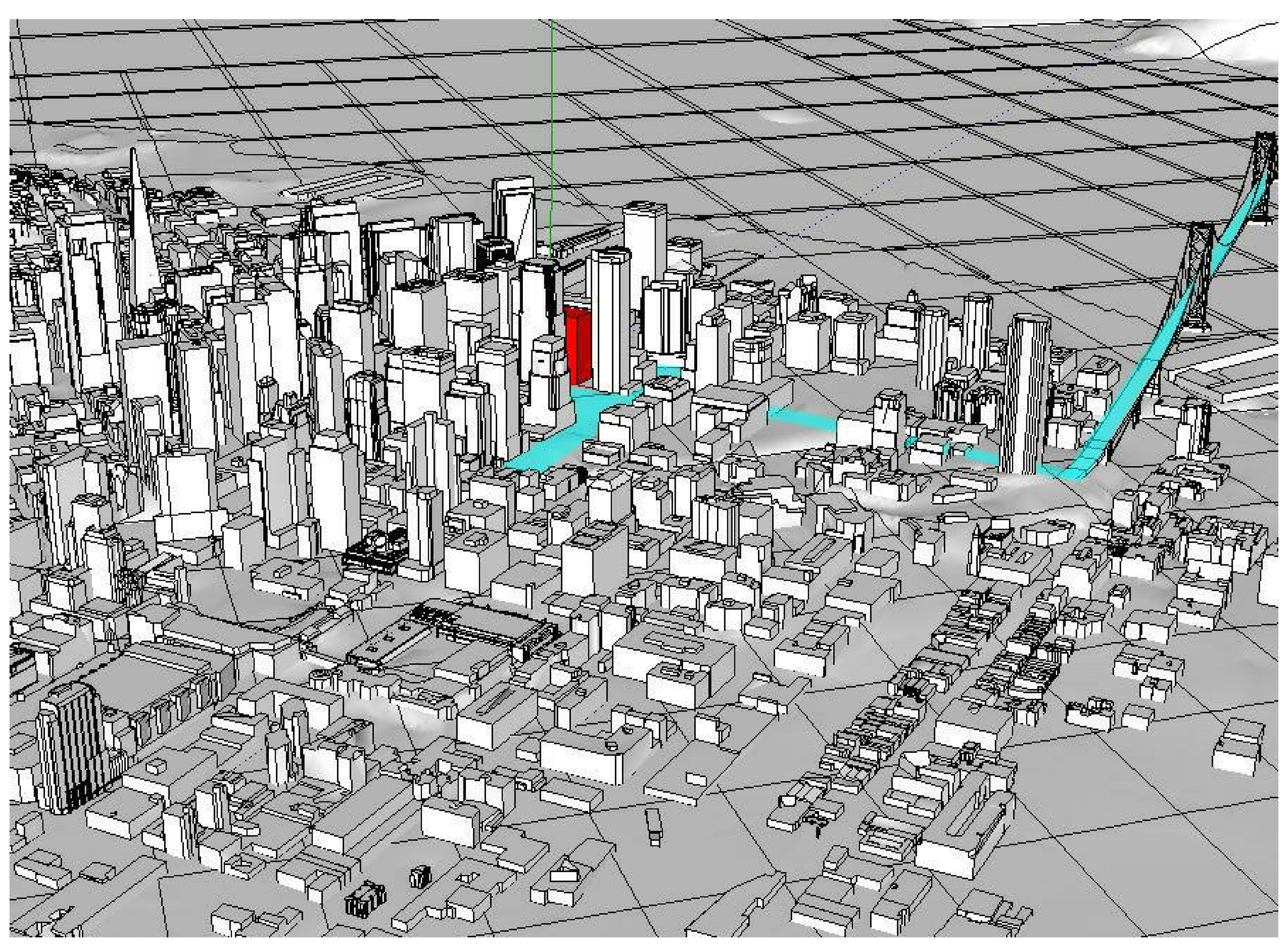
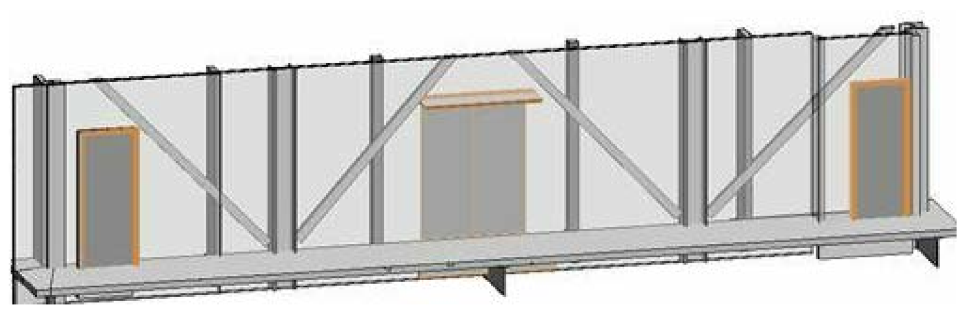
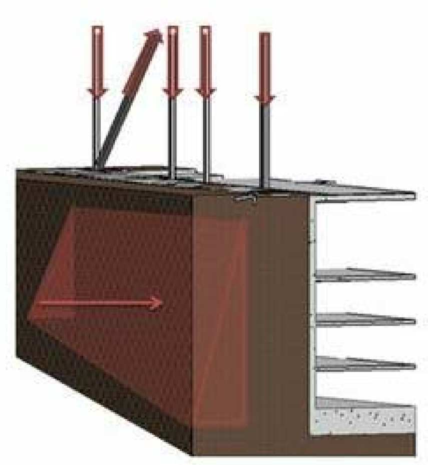










350 Mission

San Francisco, California

AEI Team 2 - 2014

Structural Engineering - Building Overview

Foundation: 350 Mission is built on a concrete mat slab foundation located 42 feet below grade. The mat slab is 6 feet thick which the structural team was able to reduce from an original design of 10feet. The extra floor to ceiling space resulting from teh reduced thickness is used to house the anaerobic digestion plant on the lowest sub-grade level. The foundation was designed to handle all potential loading situations and effeciently handles shear and overturning moment forces. 22" thick retaining walls are designed for the foundation perimeter and are detailed to act as both retaining structures and shear walls helping in transfering the lateral superstructure loads into the mat slab.

Gravity System: The superstructure design for 350 Mission consists of a steel composite beam and composite slab-on-metal deck floor framing system. Steel gravity columns both in the core and along the perimeter carry all loads to the mat slab below grade.

Lateral System: The lateral force resisting system consists of a concentrically braced frame core and perimeter diagonal mega-braces. The diagonal mega braces alleviate the lateral demand on the core and help to thin it out. They also creat a unique opportunity for architectural enhancement. The lateral force resisting system handles all seismic wind and gravity loads efficiently; and creates a safe, confortable environment for the tenant.

Facade: The facade of 350 Mission was designed to create a comfortable and inspiring work space in addition to connecting the building with the surrounding urban fabric. The structural team influenced the design by conducting movement and tolerance studies, analyzing behavior due to seismic activity, and considering the facade attachment detials/supports. The facade was a key area of collaboration throughout the entire project team

Images (Clockwise from top-left):

1. AEI Team 2's 350 Mission Design as seen from Freemont St. adjacent to the future Transbay Tower.
2. View of open office space on typical floor of 350 Mission. Note the open views and natural lighting conditions created by the facade.
3. Google SketchUp model of facade design. Overview, slab attachments and glazing cross section shown.
4. ETABS model of existing concrete shear wall 350 Mission. This model was used as a preliminary baseline model.
5. Google SketchUp model of entire San Francisco bay area showing some key urban feature take-aways from the project teams site analysis shown in light blue.
6. Clash detection model showing the concentrically braced frame core successfully accomodating architectural openings.
7. Isometric image of the loading condition created on the one of the foundation perimeter walls. Showing soil retaining loads, gravity loads from supper structure, as well as lateral loads from diagonal mega brace.

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

ASCE | STUDENT COMPETITION

AEI 2 - 2014

DRAWING TITLE

Structural
Overview

SCALE:

JOB NUMBER: 350 MISSION

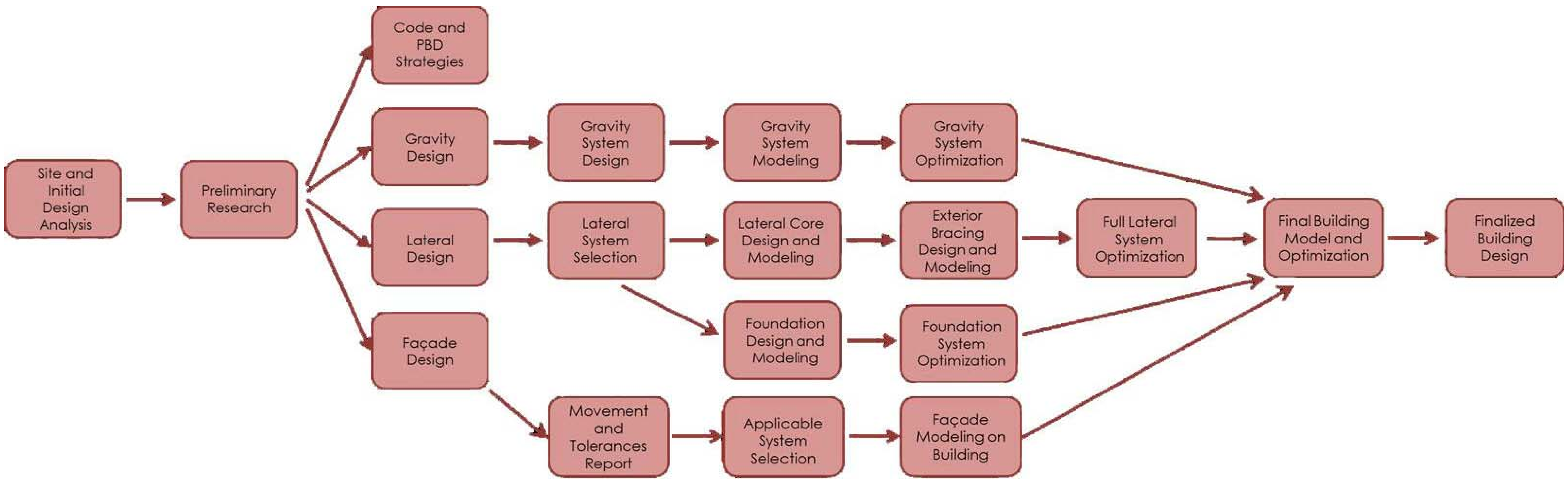
DATE: 02/17/2014

DRAWN BY: AEI 2 - 2014

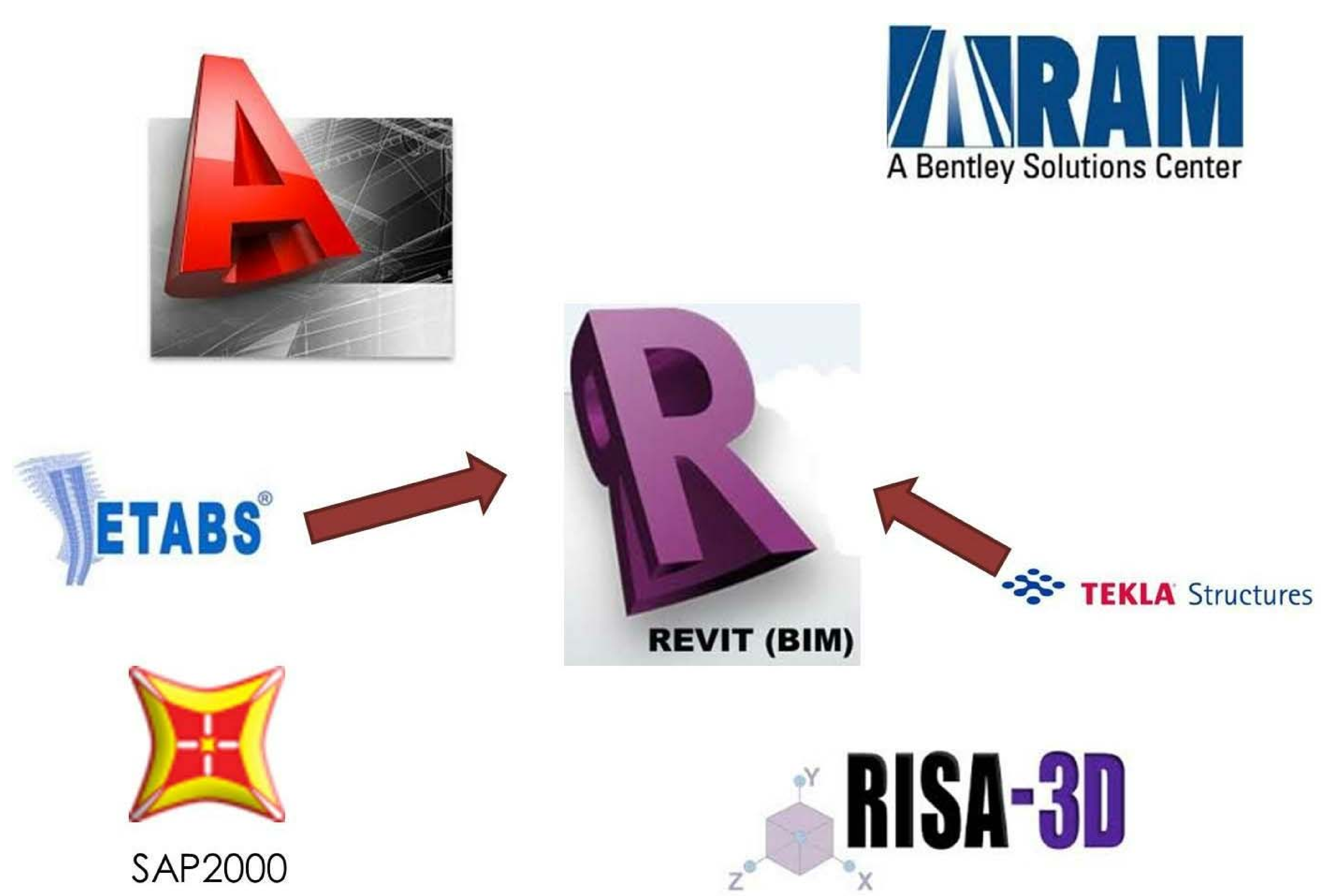
SHEET NUMBER:

S1

Structural Design Flowchart



Software Interaction



Revit: Primary modeling software used to form complete structural model and integrate with other discipline models.

Autocad: Used for drafting details and doing empirical spatial analyses.

RAM Structural System: Analysis and Design of gravity system. Un-able to link to Revit 2014.

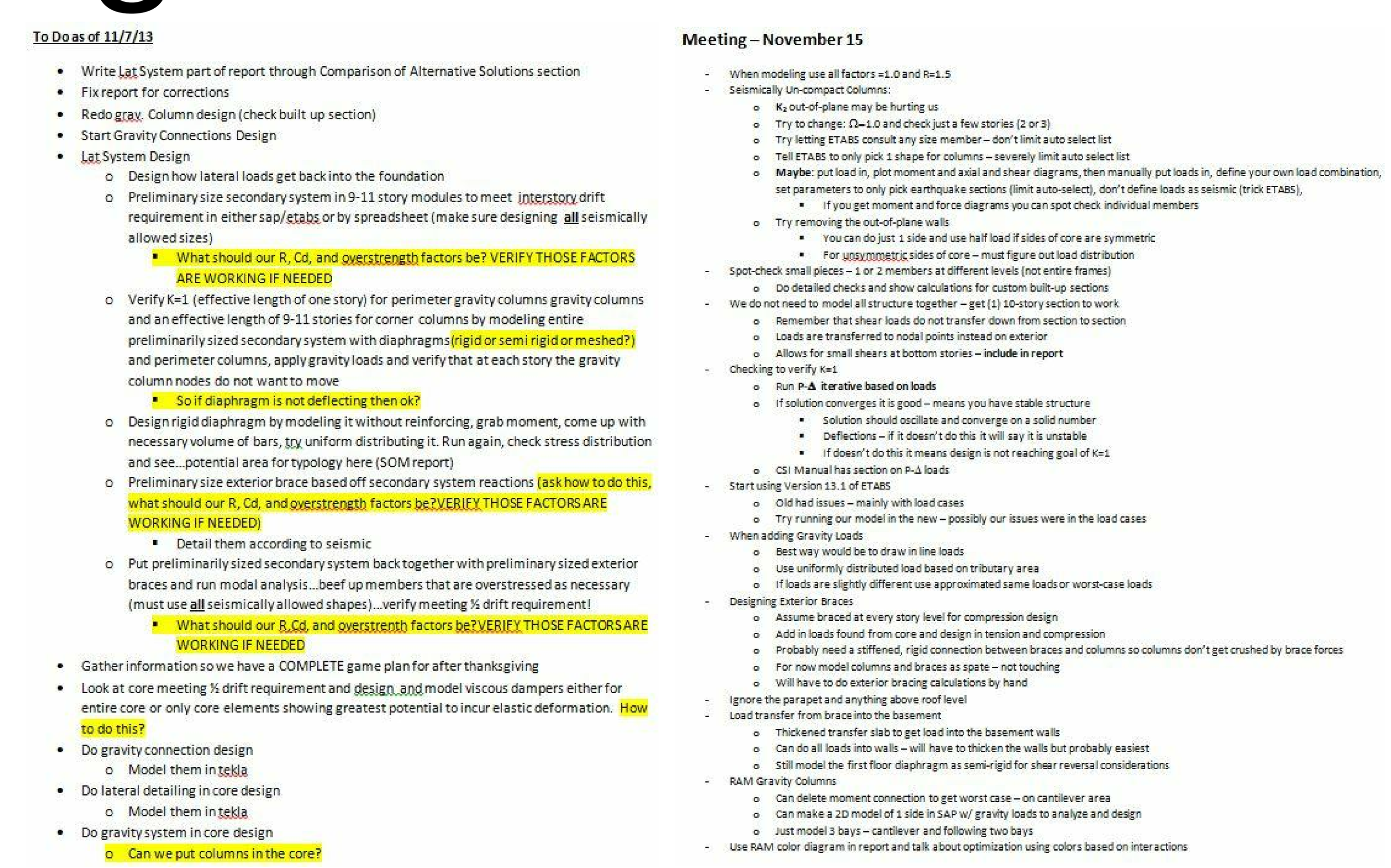
RISA: Used for quick model verifications and small member and frame analyses.

ETABS: Main Analysis and Design software used for full lateral design and total structure optimization. Able to link to Revit.

SAP: Used for analysis of cantilever forces on supporting columns.

Tekla Structures: Used to model connection details and generate descriptive 3-D images of connections. Able to link to Revit.

Organization Tactics



To Do as of 11/7/13

- Write Lat System part of report through Comparison of Alternative Solutions section
- Fix report for corrections
- Redo grav. Column design (check built up section)
- Start Gravity Connections Design
- Lat System Design
  - Design how lateral loads get back into the foundation
  - Preliminary size secondary system in 9-11 story modules to meet interstory drift requirement in either sap/etabs or by spreadsheet (make sure designing **all** seismically allowed sizes)
    - What should our R, Cd, and overstrength factors be? VERIFY THOSE FACTORS ARE WORKING IF NEEDED
  - Verify K=1 (effective length of one story) for perimeter gravity columns gravity columns and an effective length of 9-11 stories for corner columns by modeling entire preliminarily sized secondary system with diaphragms (rigid or semi rigid or meshed?) and perimeter columns, apply gravity loads and verify that at each story the gravity column nodes do not want to move
    - So if diaphragm is not deflecting then ok?
  - Design rigid diaphragm by modeling it without reinforcing, grab moment, come up with necessary volume of bars, try uniform distributing it. Run again, check stress distribution and see...potential area for typology here (SOM report)
  - Preliminary size exterior brace based off secondary system reactions (ask how to do this, what should our R, Cd, and overstrength factors be? VERIFY THOSE FACTORS ARE WORKING IF NEEDED)
    - Detail them according to seismic
  - Put preliminarily sized secondary system back together with preliminary sized exterior braces and run modal analysis...beef up members that are overstressed as necessary (must use **all** seismically allowed shapes)...verify meeting ½ drift requirement!
    - What should our R,Cd, and overstrenth factors be? VERIFY THOSE FACTORS ARE WORKING IF NEEDED
- Gather information so we have a COMPLETE game plan for after thanksgiving
- Look at core meeting ½ drift requirement and design and model viscous dampers either for entire core or only core elements showing greatest potential to incur elastic deformation. How to do this?
- Do gravity connection design
  - Model them in tekla
- Do lateral detailing in core design
  - Model them in tekla
- Do gravity system in core design
  - Can we put columns in the core?

Meeting – November 15

- When modeling use all factors =1.0 and R=1.5
- Seismically Un-compact Columns:
  - K_2 out-of-plane may be hurting us
  - Try to change: Ω=1.0 and check just a few stories (2 or 3)
  - Try letting ETABS consult any size member – don't limit auto select list
  - Tell ETABS to only pick 1 shape for columns – severely limit auto select list
  - **Maybe**: put load in, plot moment and axial and shear diagrams, then manually put loads in, define your own load combination, set parameters to only pick earthquake sections (limit auto-select), don't define loads as seismic (trick ETABS),
    - If you get moment and force diagrams you can spot check individual members
  - Try removing the out-of-plane walls
    - You can do just 1 side and use half load if sides of core are symmetric
    - For unsymmetric sides of core – must figure out load distribution
- Spot-check small pieces – 1 or 2 members at different levels (not entire frames)
  - Do detailed checks and show calculations for custom built-up sections
- We do not need to model all structure together – get (1) 10-story section to work
  - Remember that shear loads do not transfer down from section to section
  - Loads are transferred to nodal points instead on exterior
  - Allows for small shears at bottom stories – **include in report**
- Checking to verify K=1
  - Run **P-Δ iterative based on loads**
  - If solution converges it is good – means you have stable structure
    - Solution should oscillate and converge on a solid number
    - Deflections – if it doesn't do this it will say it is unstable
    - If doesn't do this it means design is not reaching goal of K=1
  - CSI Manual has section on P-Δ loads
- Start using Version 13.1 of ETABS
  - Old had issues – mainly with load cases
  - Try running our model in the new – possibly our issues were in the load cases
- When adding Gravity Loads
  - Best way would be to draw in line loads
  - Use uniformly distributed load based on tributary area
  - If loads are slightly different use approximated same loads or worst-case loads
- Designing Exterior Braces
  - Assume braced at every story level for compression design
  - Add in loads found from core and design in tension and compression
  - Probably need a stiffened, rigid connection between braces and columns so columns don't get crushed by brace forces
  - For now model columns and braces as spate – not touching
  - Will have to do exterior bracing calculations by hand
- Ignore the parapet and anything above roof level
- Load transfer from brace into the basement
  - Thickened transfer slab to get load into the basement walls
  - Can do all loads into walls – will have to thicken the walls but probably easiest
  - Still model the first floor diaphragm as semi-rigid for shear reversal considerations
- RAM Gravity Columns
  - Can delete moment connection to get worst case – on cantilever area
  - Can make a 2D model of 1 side in SAP w/ gravity loads to analyze and design
  - Just model 3 bays – cantilever and following two bays
- Use RAM color diagram in report and talk about optimization using colors based on interactions

The structural team stayed organized and on task by taking detailed notes when meeting with advisors or industry professionals, and periodically creating To-Do lists as they planned ahead. This extra groundwork kept the team focused and on schedule, and proved to increase efficiency throughout the design process.

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

ASCE | STUDENT COMPETITION

AEI 2 - 2014

DRAWING TITLE

WORKFLOW AND SOFTWARE

SCALE:

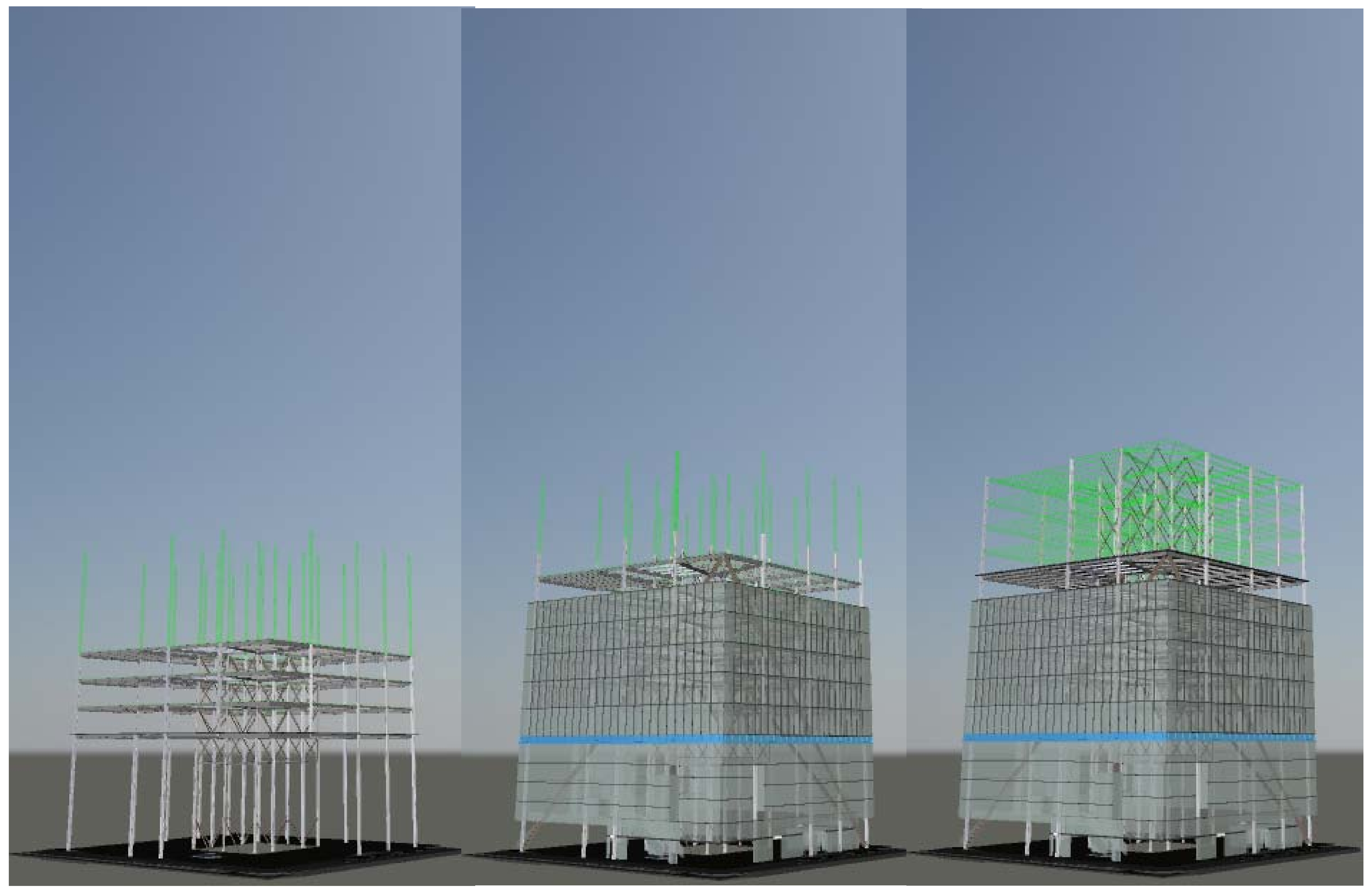
JOB NUMBER: 350 MISSION

DATE: 02/17/2014

DRAWN BY: AEI 2 - 2014

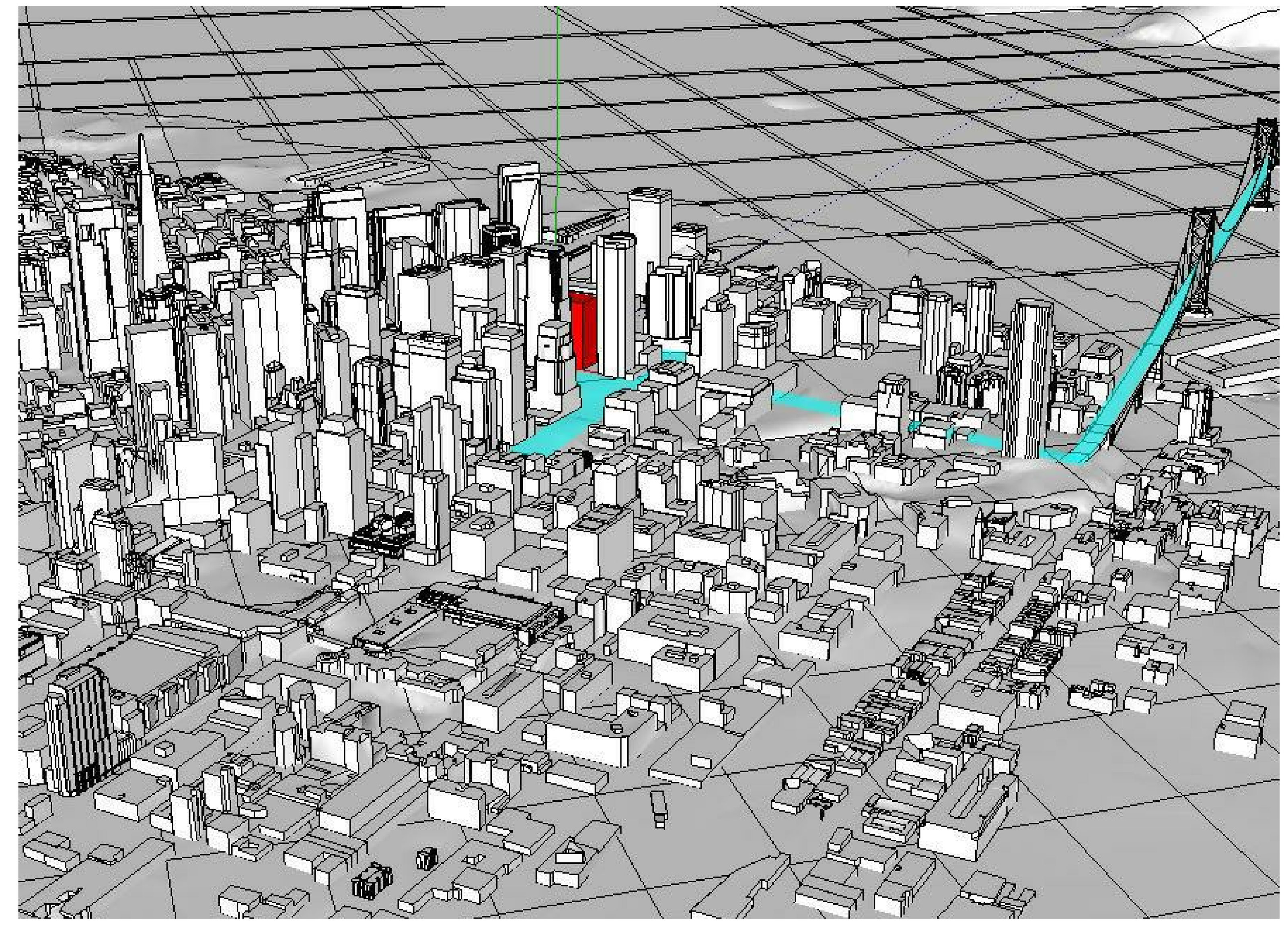
SHEET NUMBER:

S2



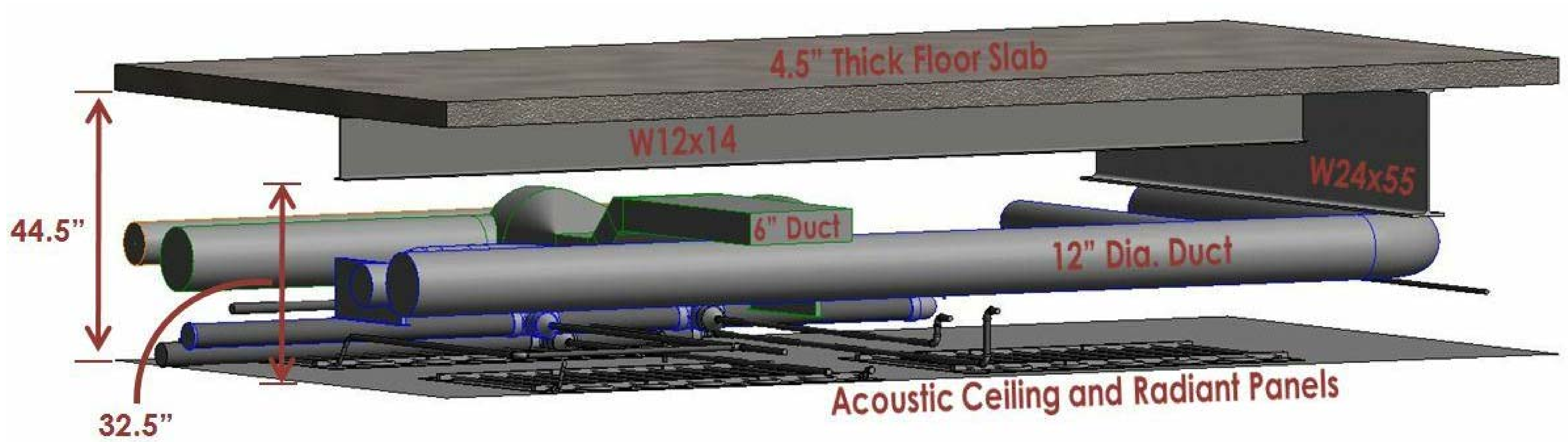
1 S3 4D STRUCTURAL SYSTEM ERECTION MODEL IMAGES
N.T.S.

The structural team also recognized the opportunity and necessity for collaboration with the construction team to accurately analyze the mega brace system for the gravity loads they will see during the erection process. Though these mega braces are lateral elements, they do connect to the floor diaphragms and cross both spandrel beams and perimeter gravity columns. Therefore the mega braces should be designed and analyzed for gravity loads sequentially applied in accordance with the construction schedule in order to get an accurate understanding of how loads will be distributed during the construction process. The structural team recognized this necessity but noted it as outside the competition scope.



2 S3 SKETCHUP MODEL OF SANFRACISCO BAY AREA - KEY URBAN FEATURES FOR ARCHITECTURAL ENHANCEMENT
N.T.S.

Coordinating with the key architectural features of 350 Mission was an important goal of the structural team. Therefore early on in the design process they engaged in extensive site context analysis in order to better understand the opportunities available to them to provide architectural enhancement per the competition guidelines. The above SketchUp model of the entire San Fracisco Bay Area shows 350 Mission in its greater urban setting. Notable features are the importance of the Southwest cantilever due to its engagement of Freemont Street, which directly connects 350 Mission with the Financial District and the iconic Bay Bridge. Also of note is the nearby site of the future Transbay Terminal and Tower also in the key Southwest direction. Site context analysis such as this allowed the team to stategicly lay out both their gravity and lateral systems in a way such that the architecture connecting 350 Mission with these key urban features is not impacted negatively.



3 S3 COORDINATION OF CEILING PLENUM
N.T.S.

Keeping as high a floor to ceiling space as possible was an important goal for all project team members in order to preserve occupant comfort and impressive architectural views. Therefore tight coordination was necessary to reduce the ceiling plenum as much as possible. The above image depicts the tight integration by the structural and MEP teams in order to minimize the ceiling plenum depth.

System	Impacts/Input from other discipline					
	Construction Management		Mechanical		Lighting	
	Pros	Cons	Pros	Cons	Pros	Cons
Outriggers	Fairly common system; Potential to prefab trusses?	Connections are difficult to ensure quality control on	Large outriggers can complicate mechanical system placement	MEP must work around the large trusses	-	-
Belt Trusses	Fairly common system; Potential to prefab trusses?	Connections are difficult to ensure quality control on	The moving of trusses to the exterior of the building alleviates potential clashing with MEP	-		Large trusses mess up the look of the façade Large belt trusses affect the daylighting of those floors
Steel Plate Shear Walls	decresed cost decreased schedule increase ease of construction increase square footage	Very little out of plane stiffness during construction		Deffinitely no penetrations through them makes it difficult for coordination		Deffinitely no penetrations through them makes it difficult for coordination

4 S3 COORDINATION INPUT SPREADSHEET FOR POTENTIAL LATERAL SYSTEMS
N.T.S.

When researching potential lateral systems, the structural team created a spreadsheet which the other disciplines could add to explaining how the proposed systems affected their goals and systems. Above is shown a small portion of the comprehensive spreadsheet.

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

ASCE STUDENT COMPETITION
AEI 2 - 2014

DRAWING TITLE

COORDINATION

SCALE:

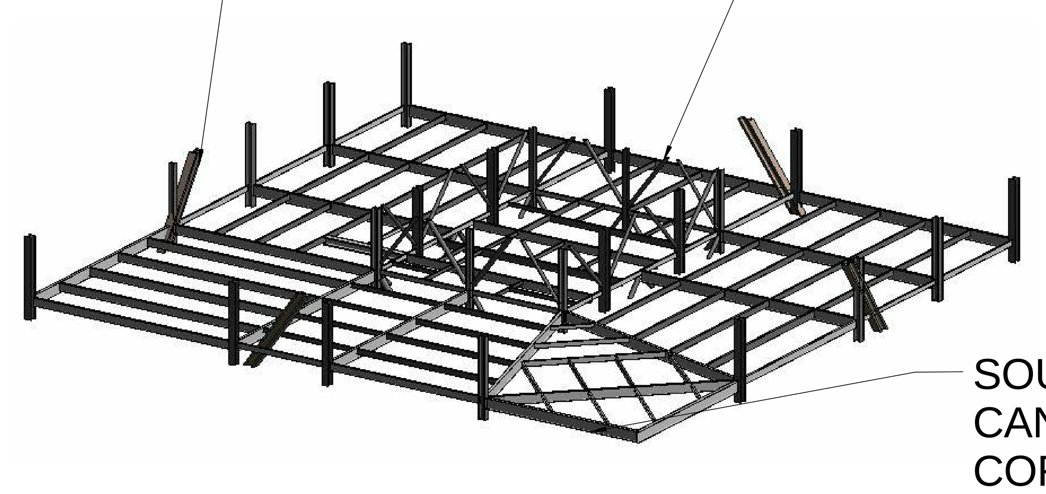
JOB NUMBER: 350 MISSION

DATE: 02/17/2014

DRAWN BY: AEI 2 - 2014

SHEET NUMBER:

S3



2 / S4 3D IMAGE OF TYPICAL FLOOR
N.T.S.

T.O. FIN FLR. 26	W14X193	W14X193	W12X96	W10X88	W12X65	W12X96	W14X193	T.O. FIN FLR. 26
317' - 4" T.O. FIN FLR. 25								317' - 4" T.O. FIN FLR. 25
304' - 2"								304' - 2"
Column Locations	A-2, B-1	A-4, A-5, A-6, C-5, D-3, D-5, E-1, E-6, F-3, F-5, G-1, G-3, G-6	C-3	C-4, G-4	C-6	G-5	H-1, H-3, H-5, H-6	

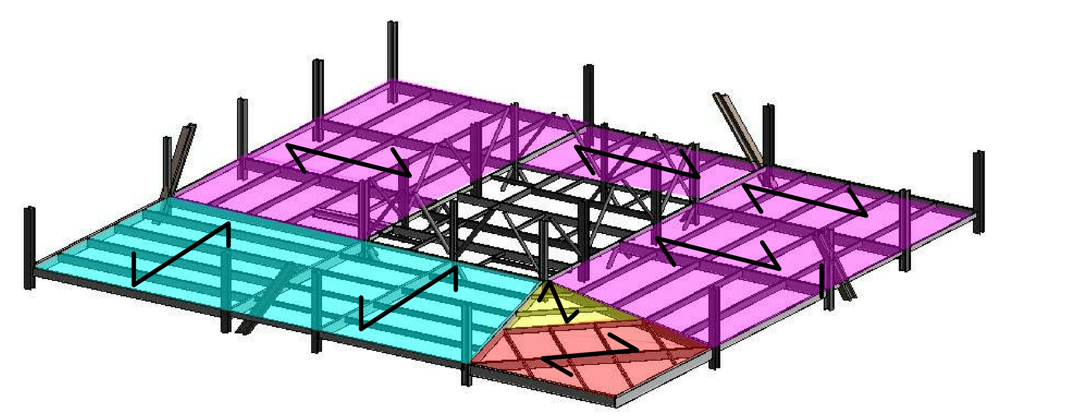
4 / S3 SAMPLE COLUMN SCHEDULE FOR TYPICAL FLOOR
N.T.S.

Typical Floor Notes:

1. FLOOR TO FLOOR HEIGHT IS 13'-2".
2. THE BUILDING IS 30 STORIES TALL WITH 26 OCCUPIABLE FLOORS.
3. LW CONC (f'c=4000psi @ 28 days)
4. 1.5" 19 GAGE GALVANIZED COMPOSITE FLOOR DECK (Vulcraft or Approved Equivalent)
5. TYPICAL SLAB THICKNESS = 4.75"
6. MAX UNSHORED SINGLE SPAN: 7'-7"
   MAX UNSHORED 2-SPAN: 9'-11"
   MAX UNSHORED 3-SPAN: 10'-3"
7. ALL STEEL = ASTM A992 GRADE 50
8. REINFORCING STEEL = ASTM A615 GRADE 60
9. SHEAR STUDS: 3/4" DIA., 4" LONG
10. DECK DIRECTION ALWAYS PERPENDICULAR TO INFILL BEAMS

PROJECT
AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE
ASCE STUDENT COMPETITION
AEI 2 - 2014

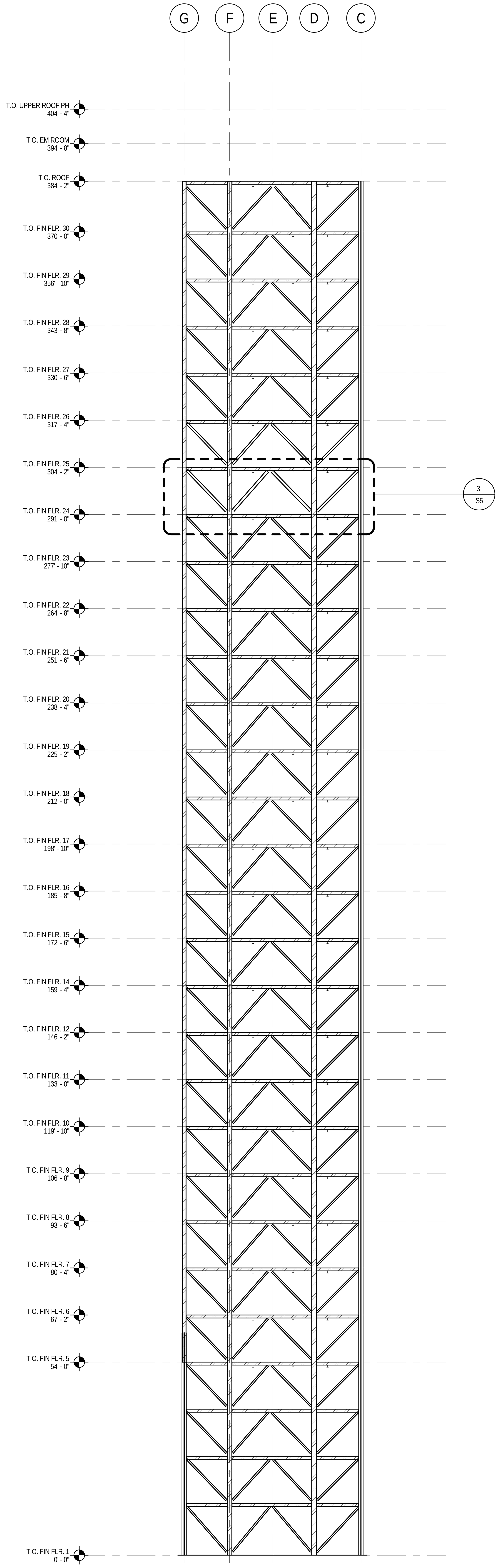


3 / S4 3D IMAGE OF TYPICAL FLOOR SHOWING DECK SPAN
N.T.S.

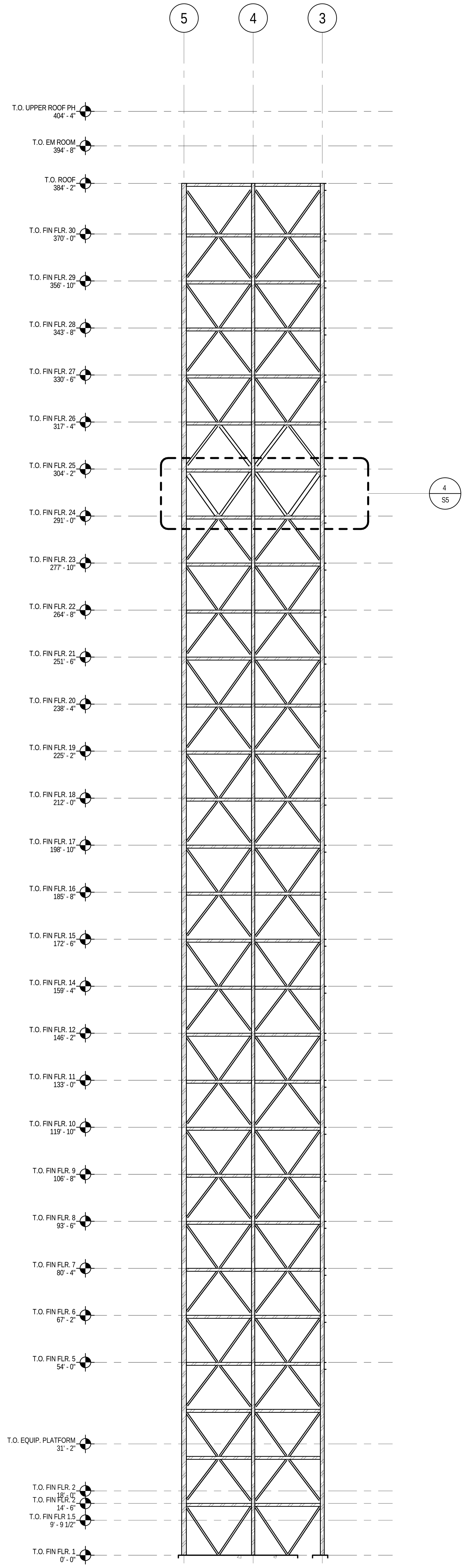
1 / S4 TYPICAL FLOOR PLAN
1/8" = 1'-0"

DRAWING TITLE
Typical Floor

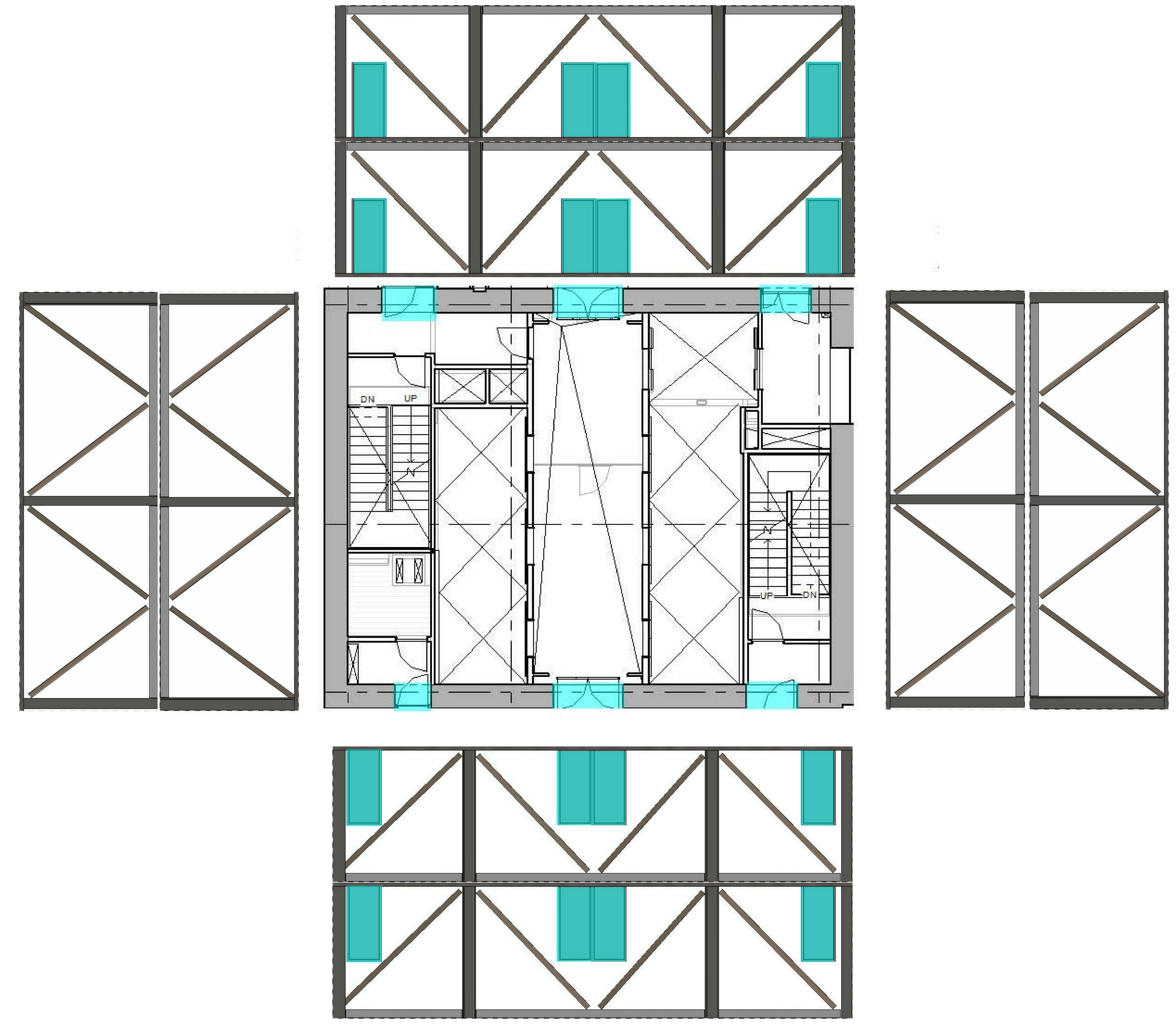
SCALE: 1/8" = 1'-0"
JOB NUMBER: 350 MISSION
DATE: 02/17/2014
DRAWN BY: AEI 2 - 2014
SHEET NUMBER:
S4



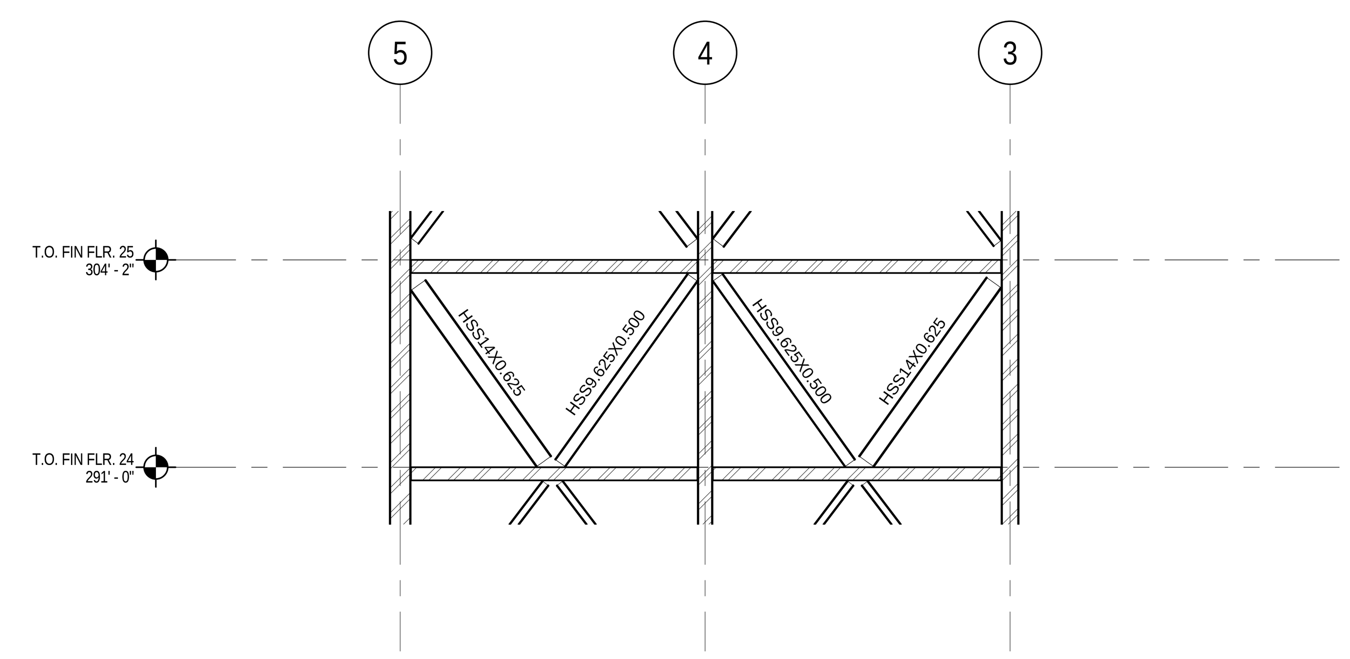
1 S5 NORTH AND SOUTH CORE FRAMING ELEVATION
1/16" = 1'-0"



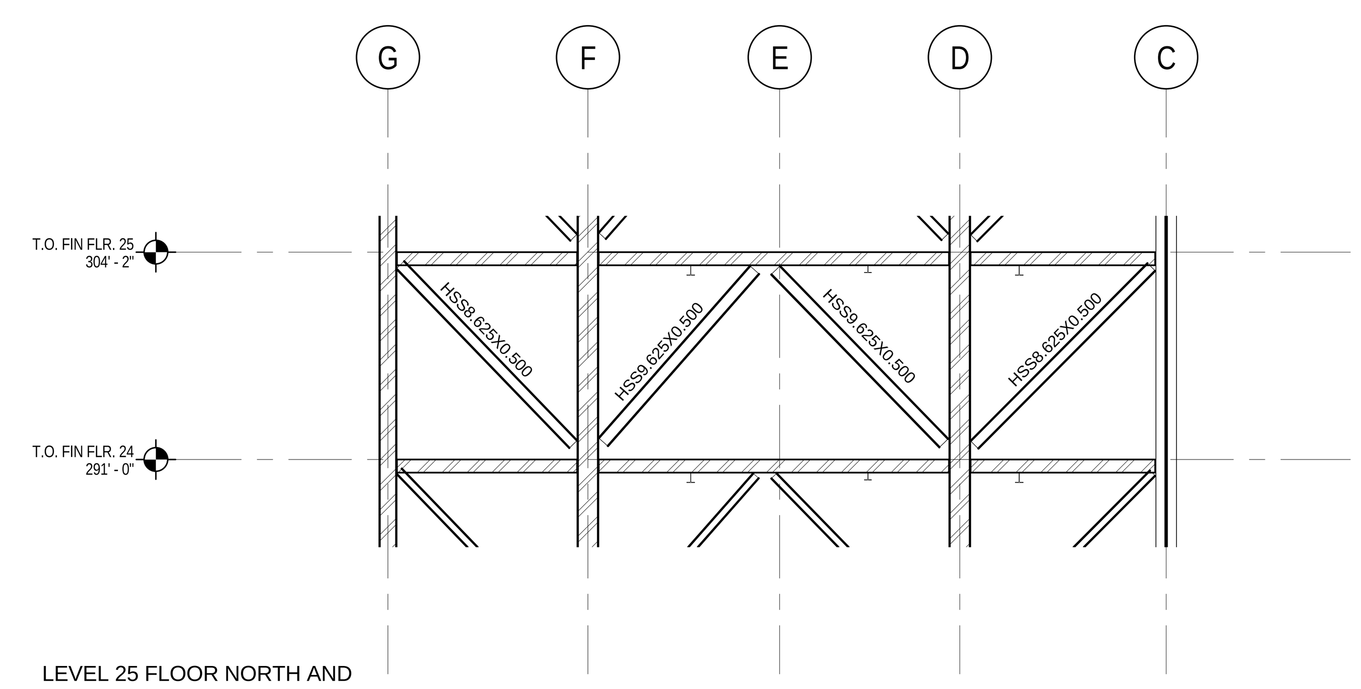
2 S5 EAST AND WEST CORE FRAMING ELEVATION
1/16" = 1'-0"



2 S4 UNFOLDED CORE ELEVATIONS SHOWING ARCHITECTURAL OPENING ACCOMADATION
N.T.S.



4 S5 LEVEL 25 EAST AND WEST CORE ELEVATION BRACE SIZES
1/8" = 1'-0"



3 S5 LEVEL 25 FLOOR NORTH AND SOUTH CORE ELEVATION BRACE SIZES
1/8" = 1'-0"

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

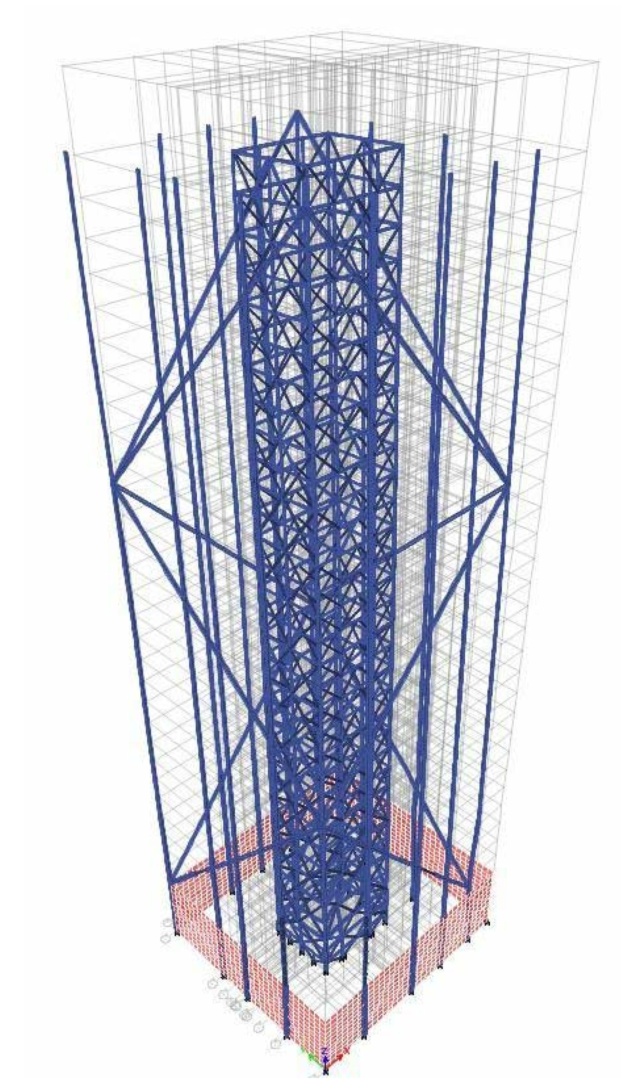
ASCE | STUDENT COMPETITION

AEI 2 - 2014

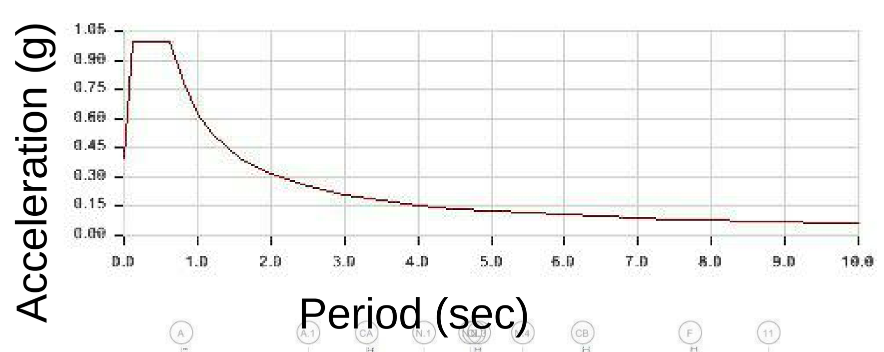
DRAWING TITLE

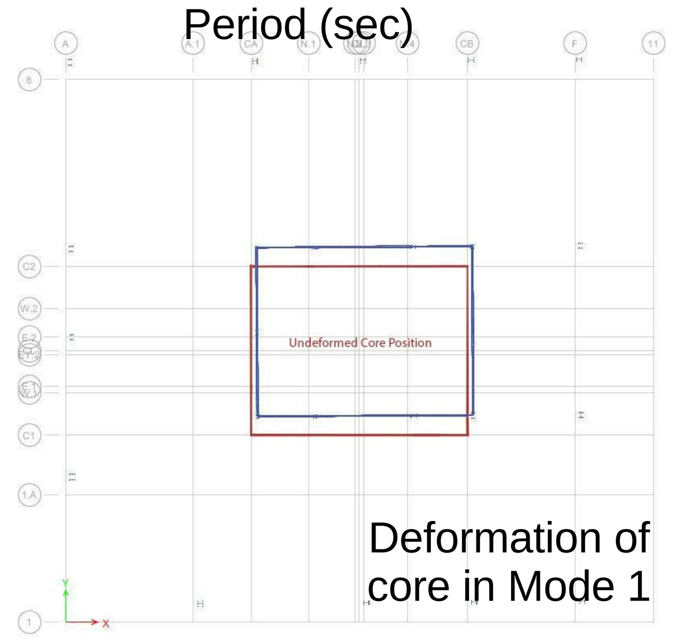
Core Elevations

SCALE:	As indicated
JOB NUMBER:	350 MISSION
DATE:	02/17/2014
DRAWN BY:	AEI 2 - 2014
SHEET NUMBER:	S5



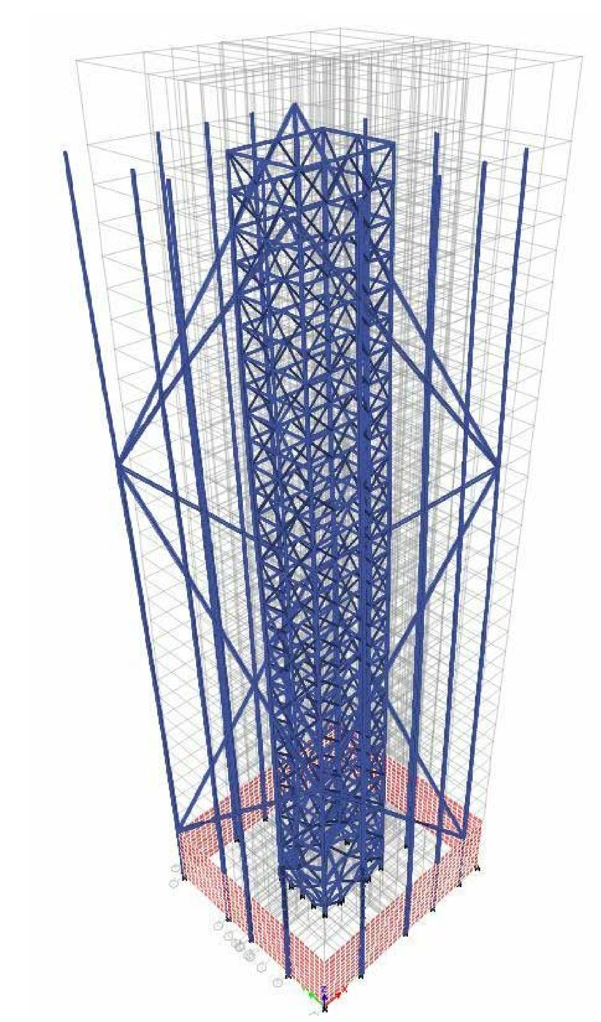
Function Graph

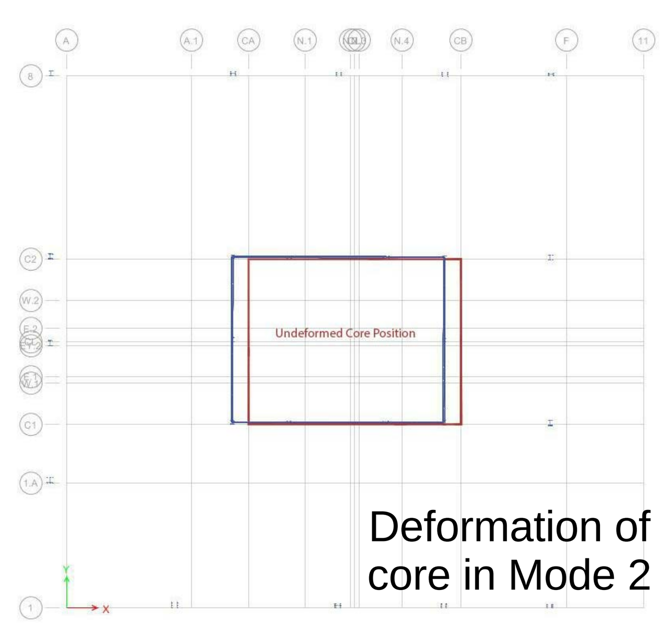




Deformation of core in Mode 1

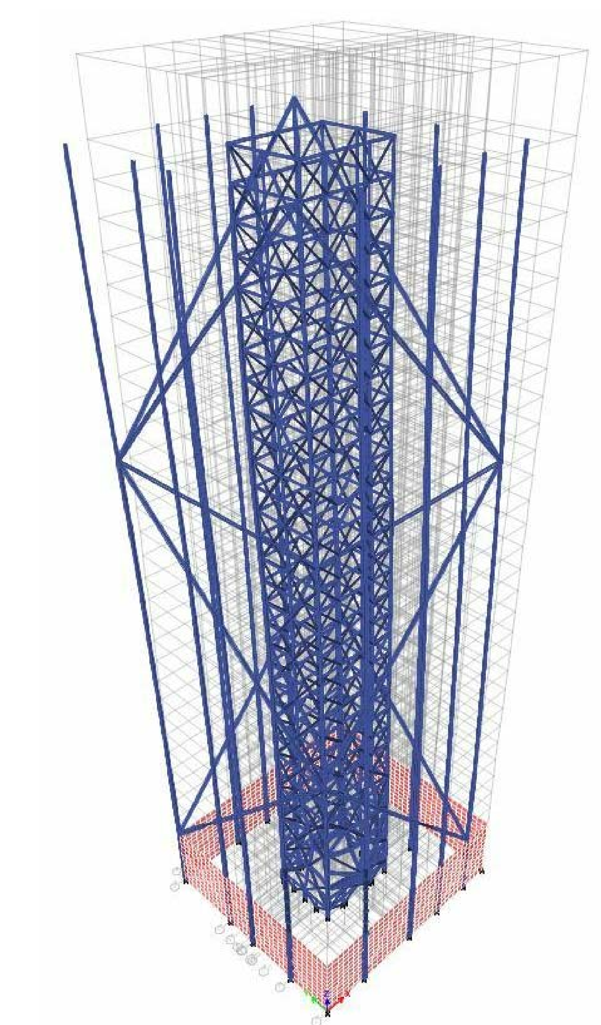
Mode 1: Multi-Directional - Period = 2.99 sec

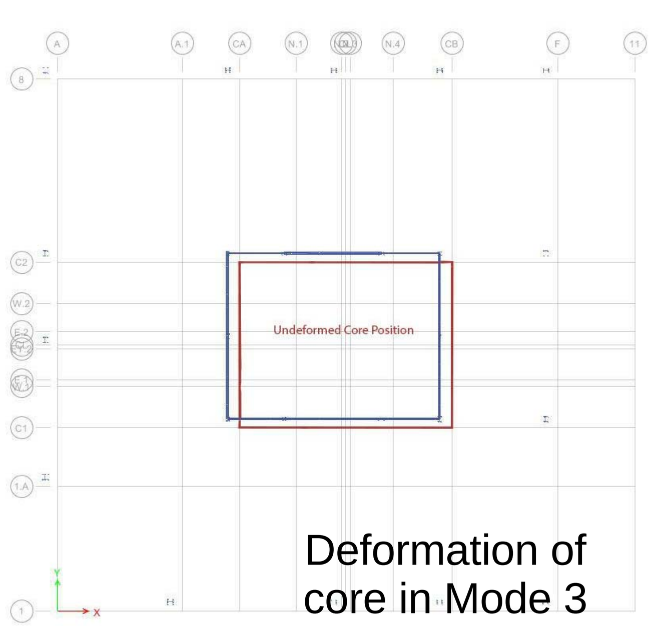




Deformation of core in Mode 2

Mode 2: X-Direction - Period = 2.818 sec

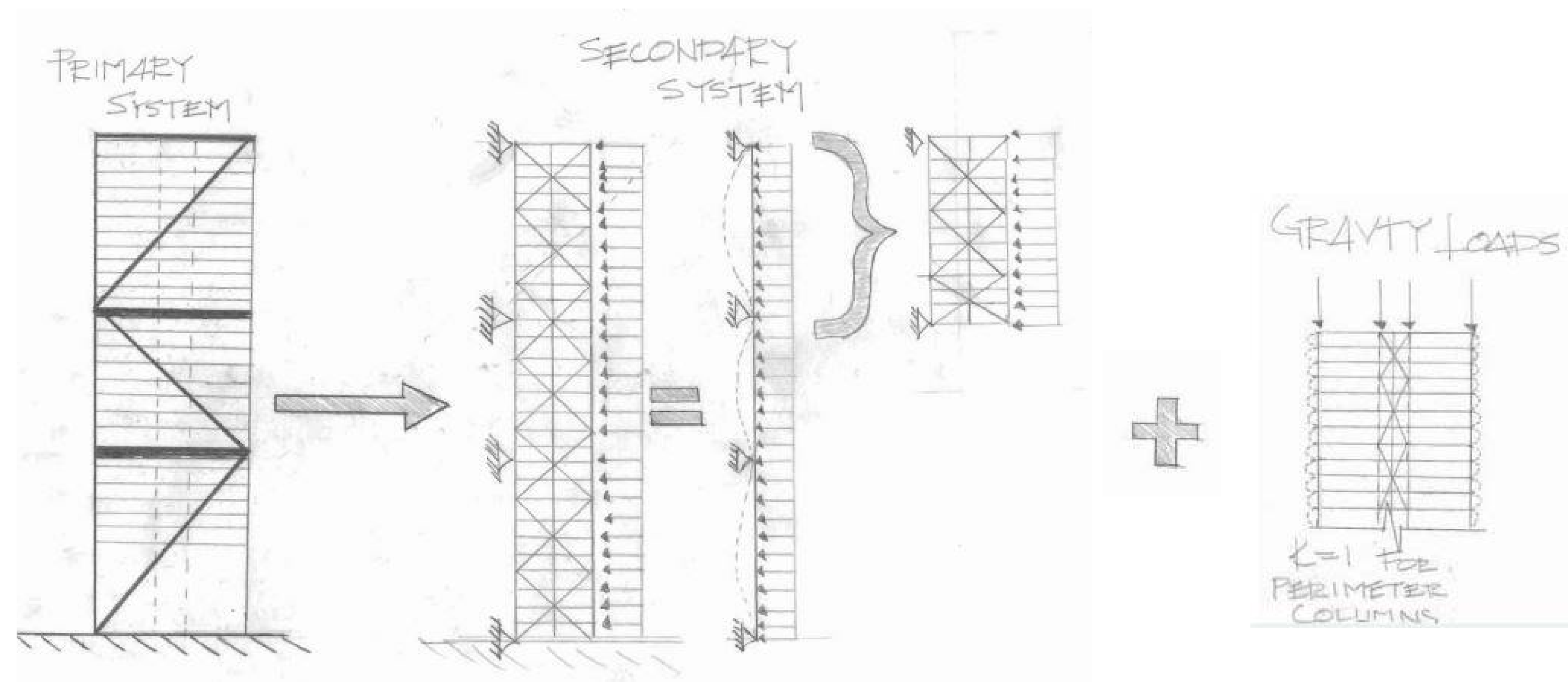




Deformation of core in Mode 3

Mode 3: Multi-Directional - Period = 2.80 sec

3 S6 MODAL RESPONSE OF LATERAL SYSTEM - ETABS MODEL
N.T.S.

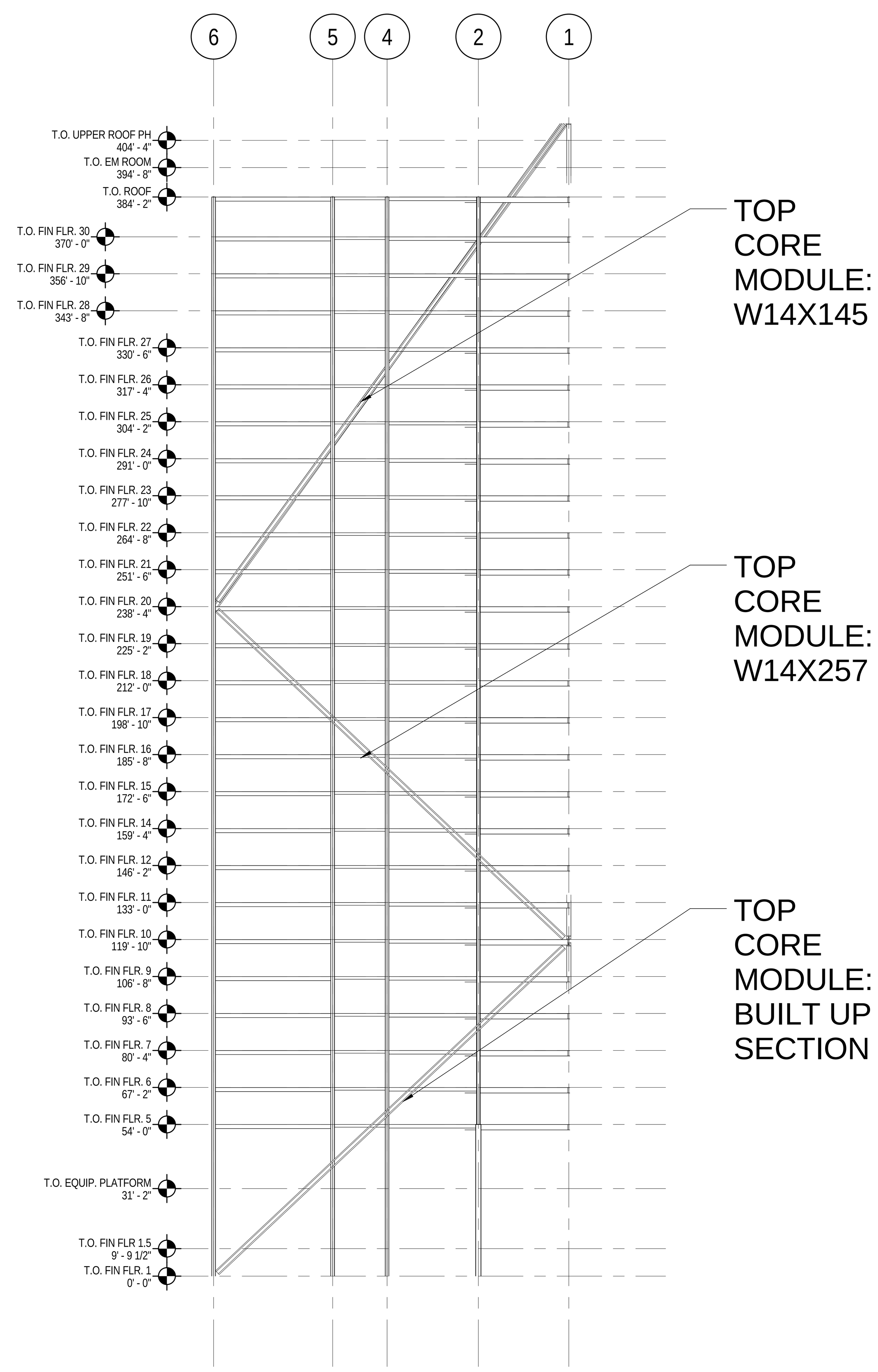


4 S6 LOAD PATH DEPICTION
N.T.S.

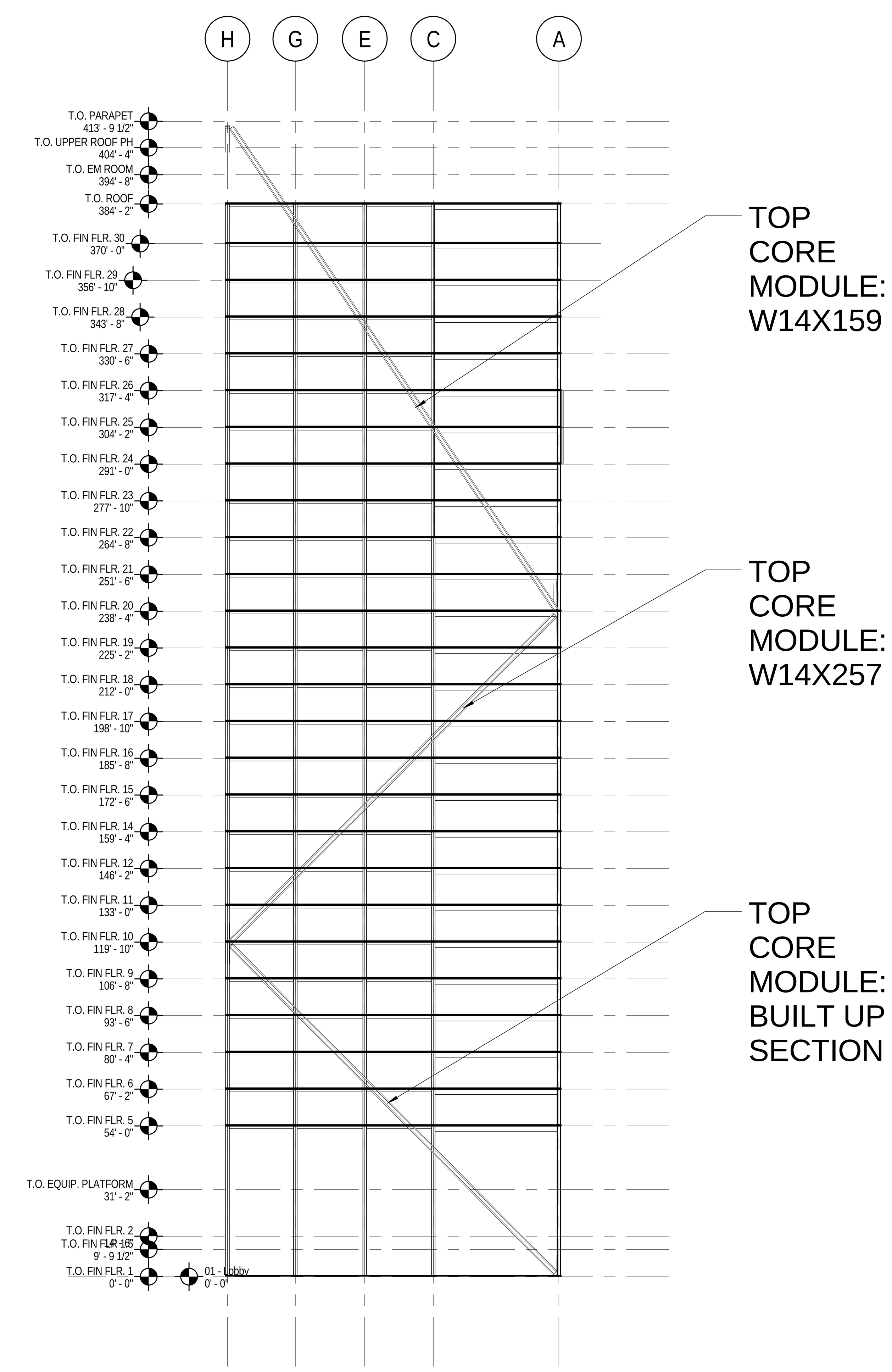
Structural Notes:

1. MODAL RESPONSE SPECTRUM ANALYSIS COMPLETED FOR THE SITE SPECIFIC GROUND ACCELERATION SPECRA VALUES FOR MAXIMUM CONSIDERED EARTHQUAKE PER GEOTECHNICAL REPORT TABLE D-4.
2. DESIGN COMPLETED ON PERFORMANCE BASED DESIGN STANDPOINT OF R=1, OVERSTRENGTH=1, DEFLECTION AMPLIFICATION=1

Load Path: The core spans the full height of the building and recieves lateral loads via the diaphragms. This "secondary" core system is further restrained by the "primary" mega brace system at the key nodal levels. Lateral load at these key nodal levels is "kicked out" of the core and transfered to the mega braces by a rigid diaphragm. Conceptually the core is like a deep beam spanning between the mega brace key levels as the supports. Preliminary core design was done on these restrained ten story core modules by applied story forces from a previous MRS analysis. Preliminary mega brace design was then completed using the core modules' reactions. A complete model was assembled and increased in size as necessary. Gravity loads on the core and perimeter columns were run with an initial iterative P-Delta analysis to ensure the core had adequate stiffness to effectively brace the perimeter columns at each level.



1 S6 EAST FRAMING ELEVATION (WEST SIMILAR)
1/32" = 1'-0"



2 S6 NORTH FRAMING ELEVATION (SOUTH SIMILAR)
1/32" = 1'-0"

PROJECT
AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

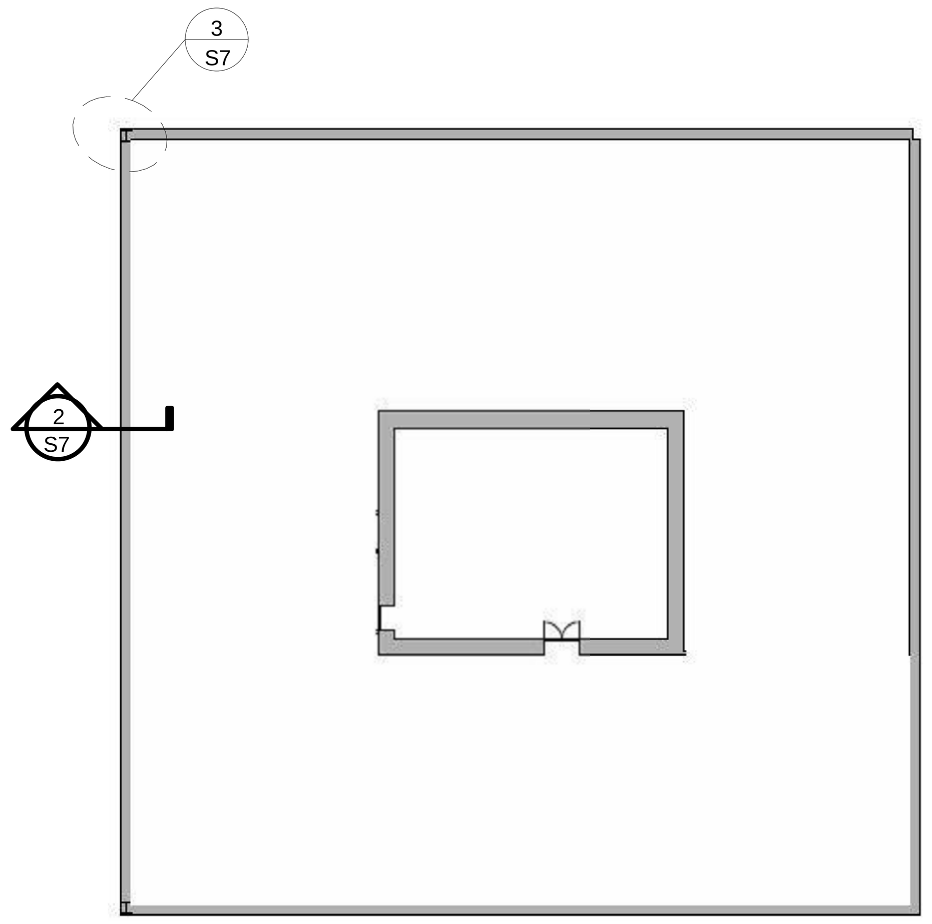
PURPOSE
ASCE STUDENT COMPETITION
AEI 2 - 2014

DRAWING TITLE
Lateral System Design

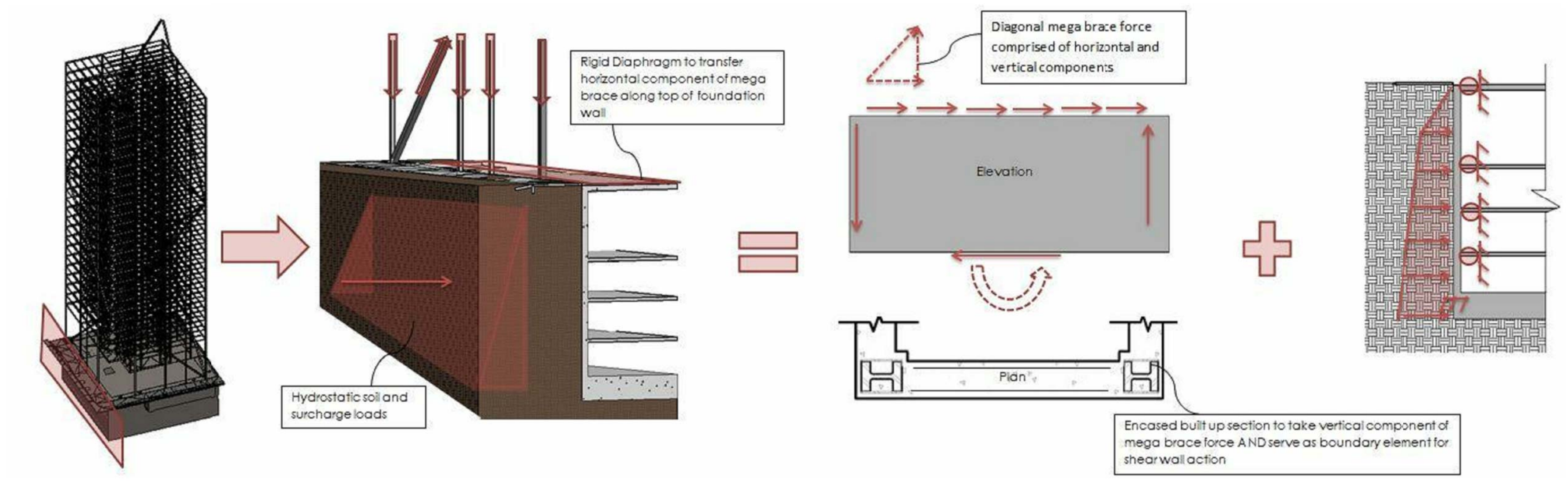
SCALE: 1/32" = 1'-0"
JOB NUMBER: 350 MISSION
DATE: 02/17/2014
DRAWN BY: AEI 2 - 2014
SHEET NUMBER:
S6

FOUNDATION NOTES:

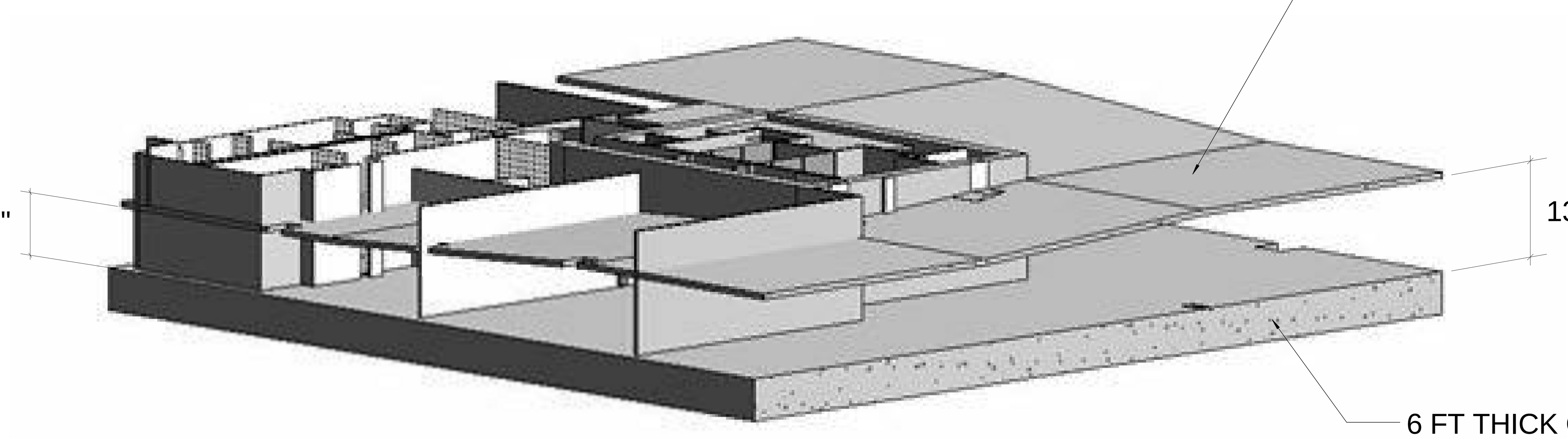
1. SITE SOIL CLASSIFICATION: SITE CLASS D
2. f'c=8000psi FOR FOUNDATION PERIMETER WALLS
3. f'c=3000psi FOR MAT SLAB
4. REINFORCING STEEL = ASTM A615 GRADE 60
5. ALLOWABLE SOIL BEARING PRESSURE= 8000-10000psf



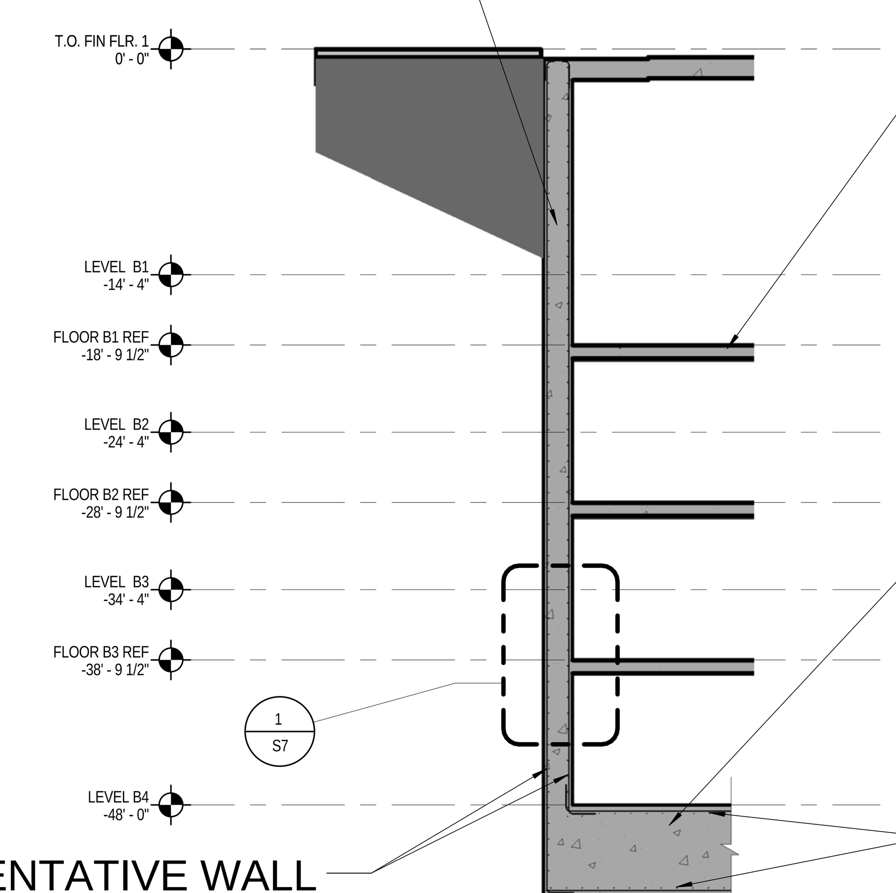
4 FOUNDATION KEY PLAN
S7 N.T.S.



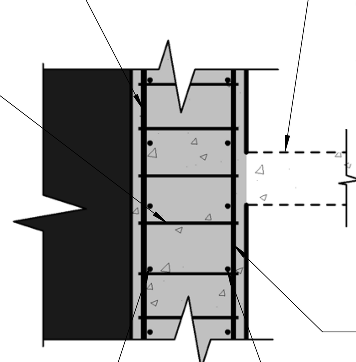
5 TYPICAL FOUNDATION PERIMETER WALL LOADING CONDITION
S7 N.T.S.



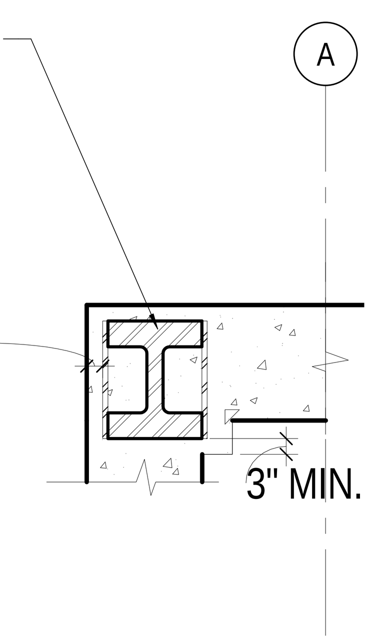
2 3D ISOMETRIC SECTION OF FOUNDATION LEVELS
S4 N.T.S.



2 TYPICAL FOUNDATION WALL SECTION
S7 1/8" = 1'-0"



1 PERIMETER WALL REINFORCING DETAIL
S7 1/2" = 1'-0"



3 ENCASED COLUMN IN PERIMETER FOUNDATION WALL- SHEAR WALL BOUNDARY ELEMENT
S7 1/2" = 1'-0"

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

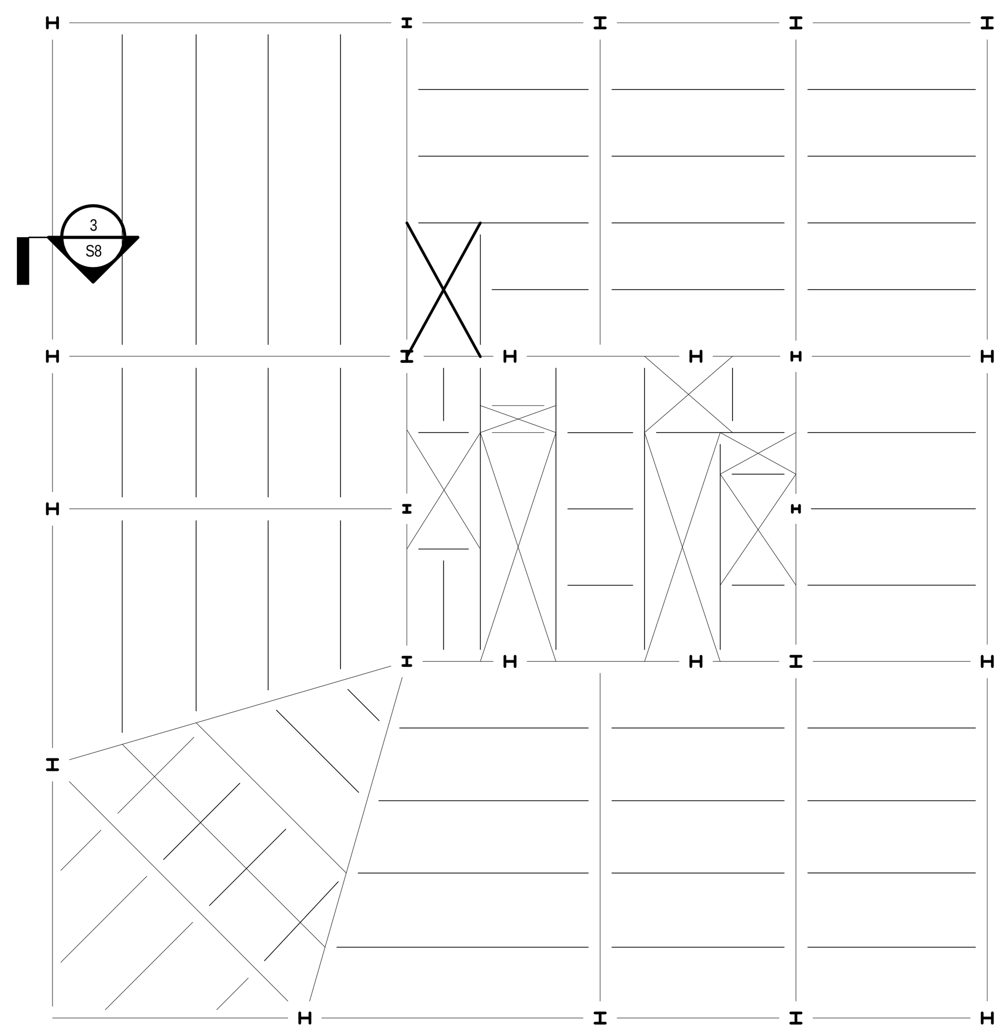
ASCE STUDENT COMPETITION
AEI 2 - 2014

DRAWING TITLE

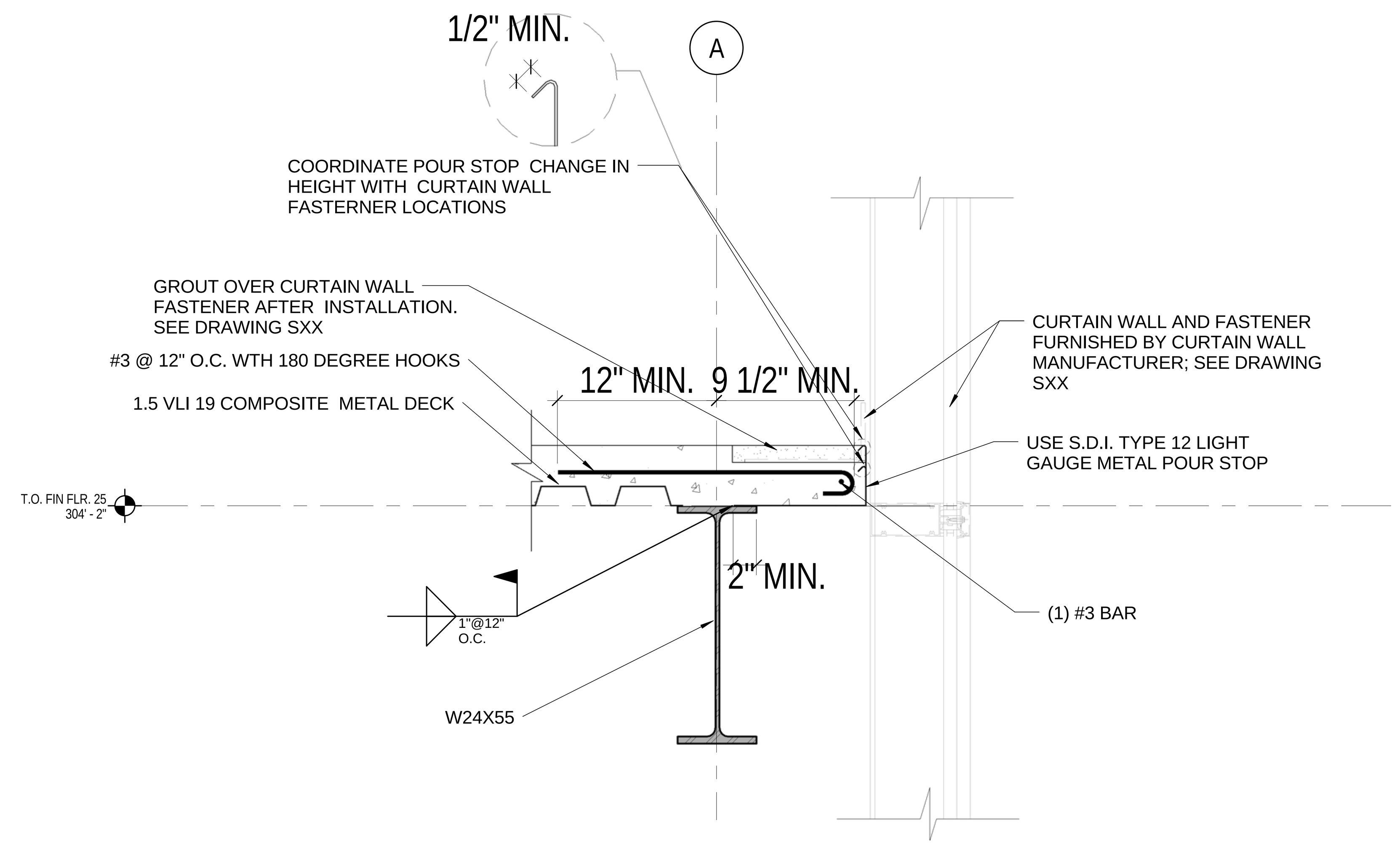
Foundation Plan and Details

SCALE:	As indicated
JOB NUMBER:	350 MISSION
DATE:	02/17/2014
DRAWN BY:	AEI 2 - 2014
SHEET NUMBER:	

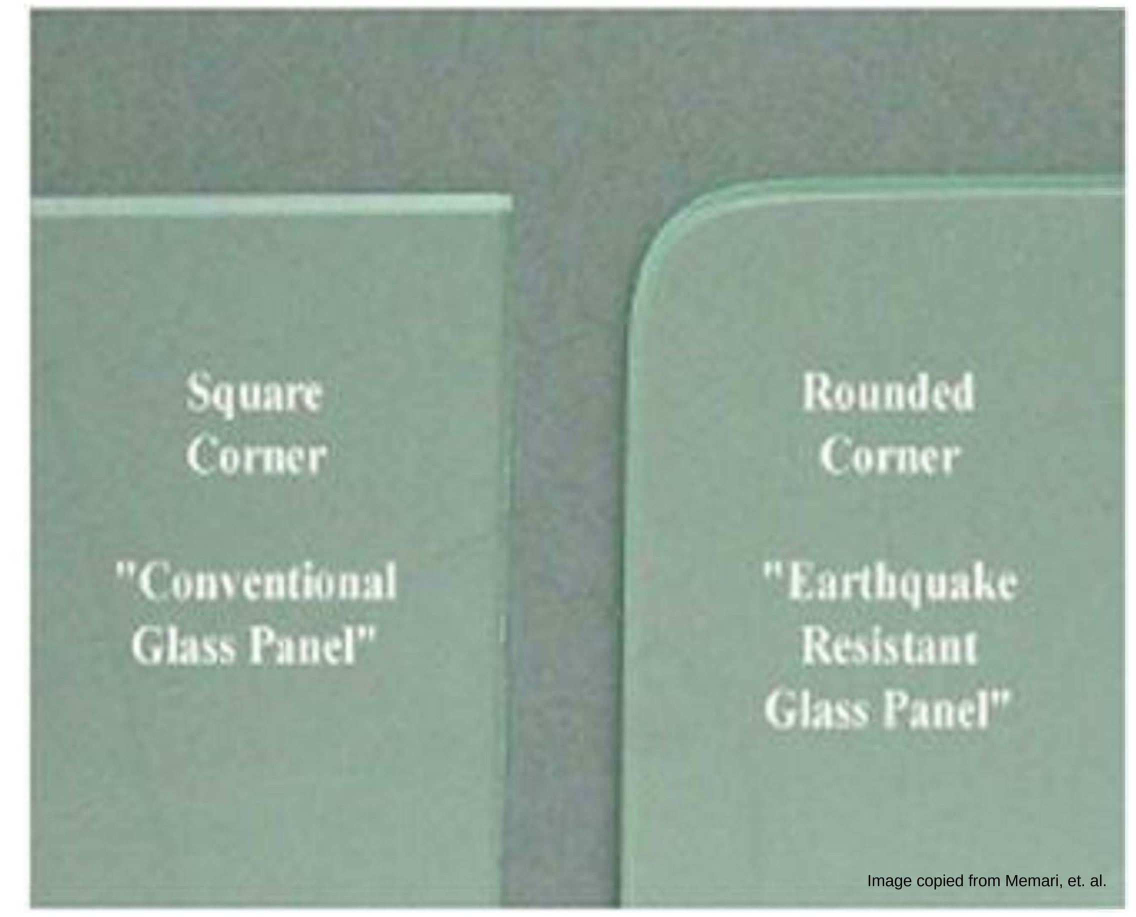
S7



1 BUILDING ENCLOSURE KEY PLAN
S8 1/16" = 1'-0"

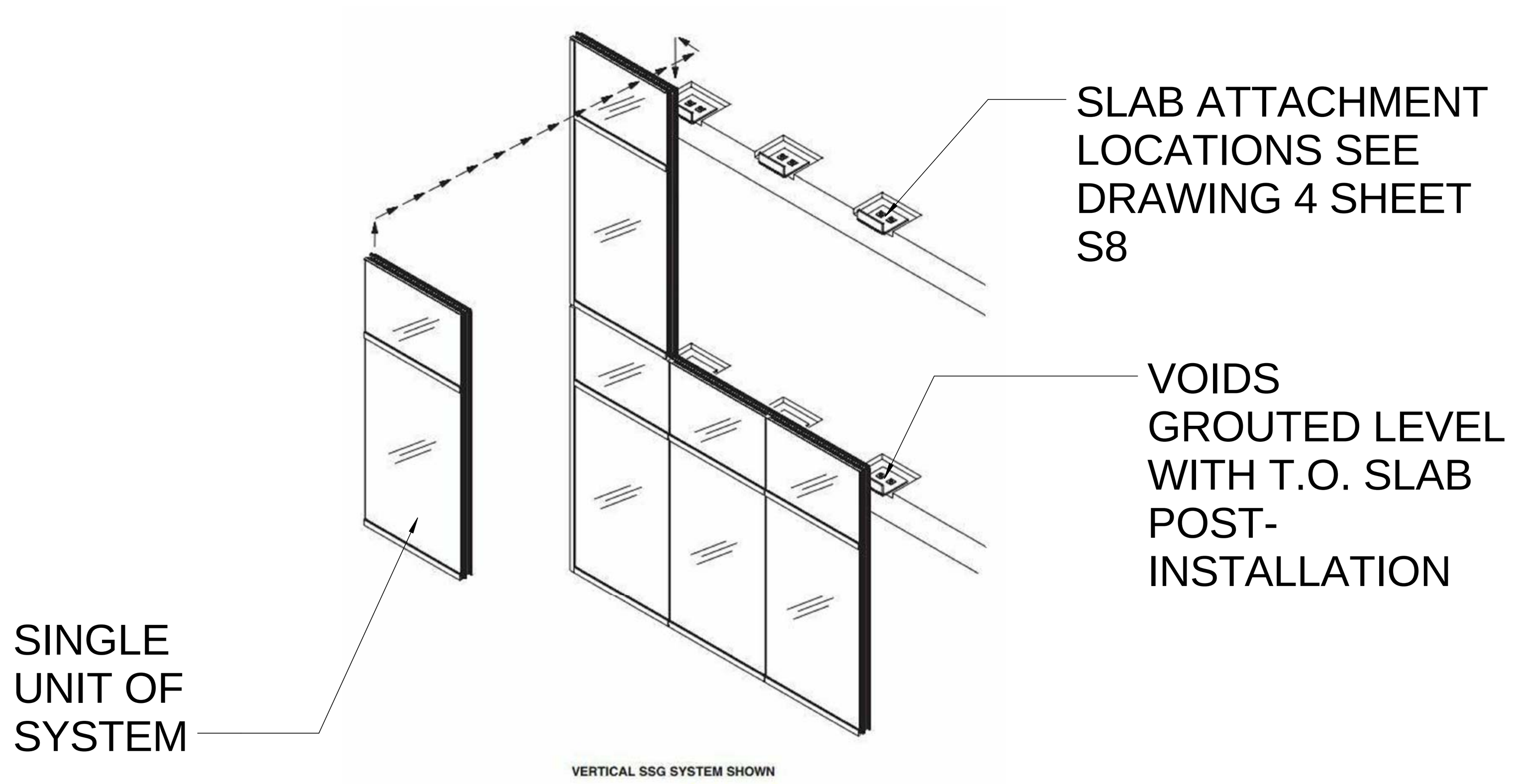


3 Facade Anchorage Detail
S8 1 1/2" = 1'-0"

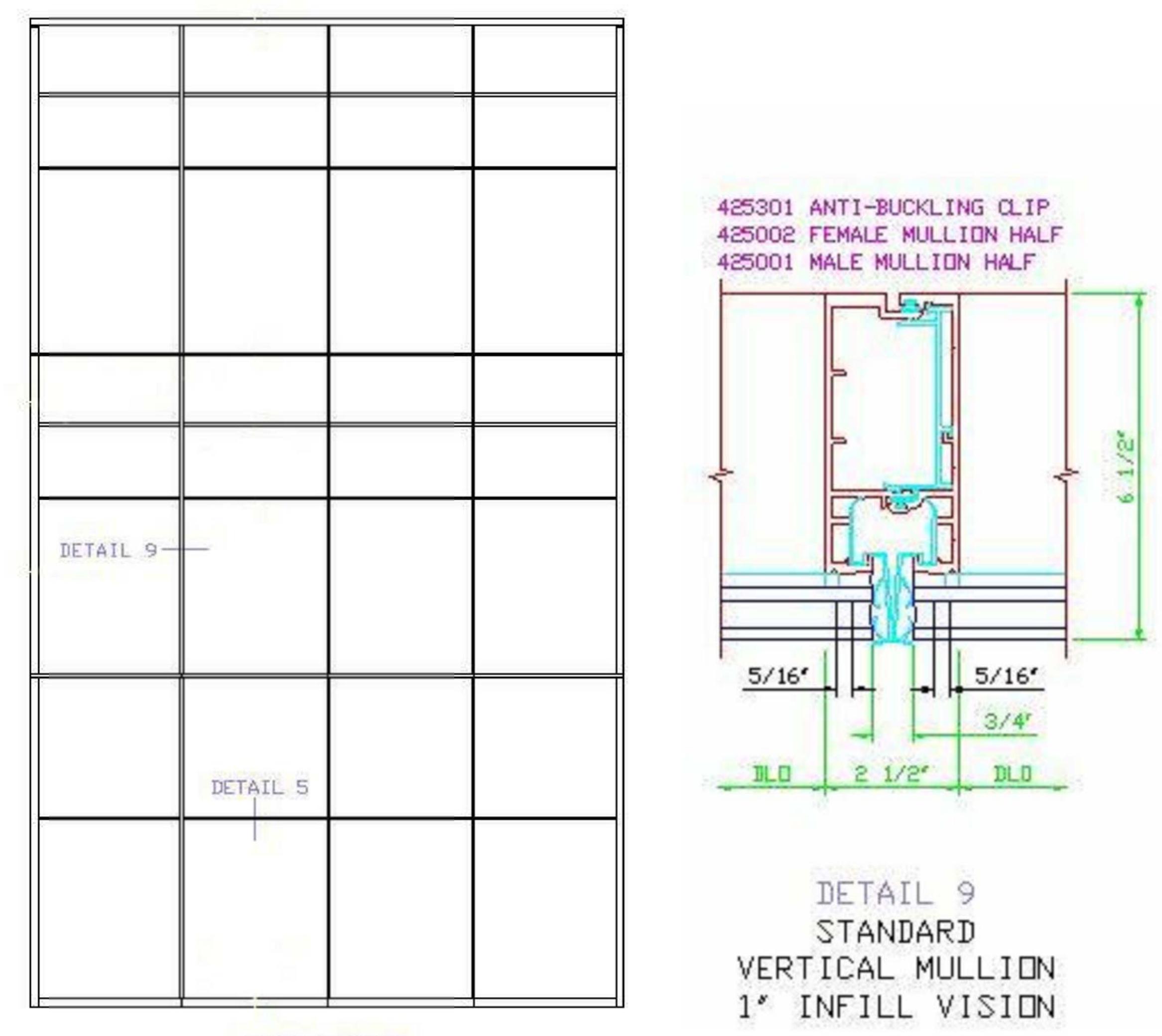


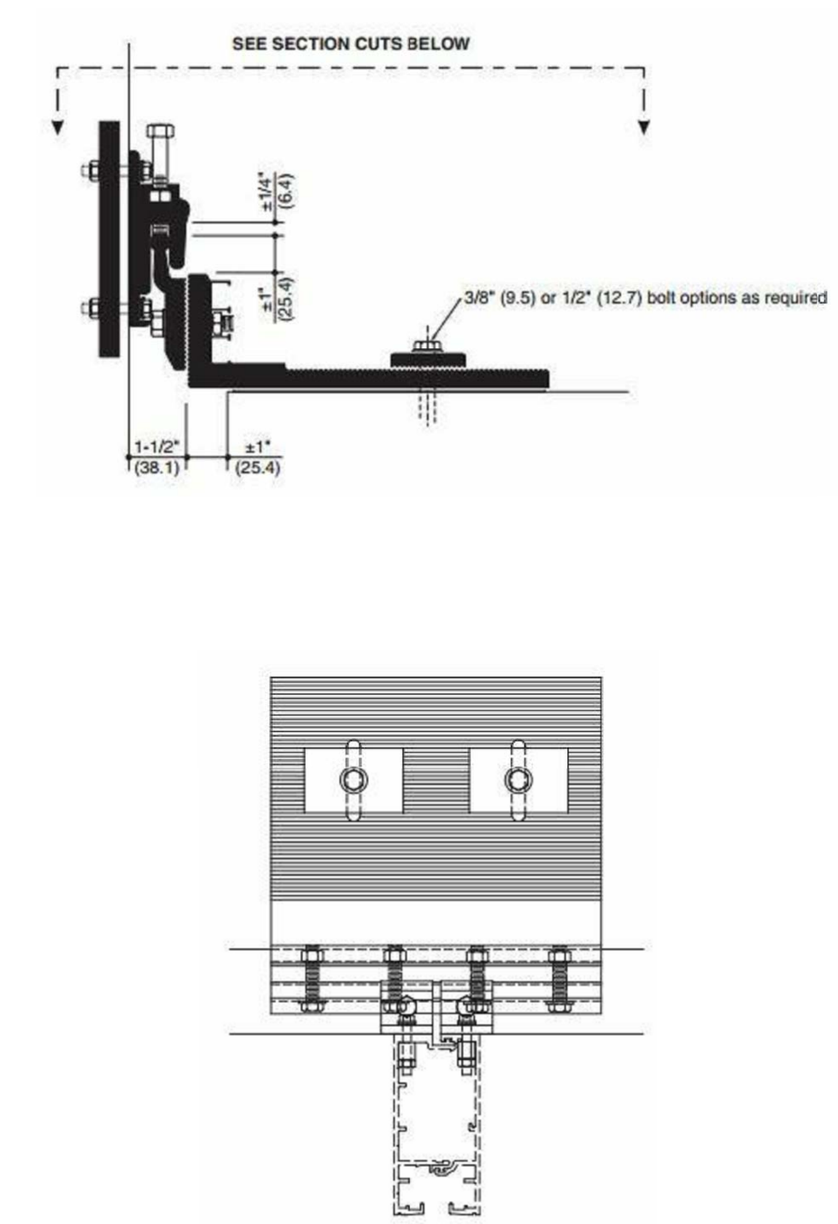
8 Square Corner vs. Rounded Corner Glass
S8 N.T.S.

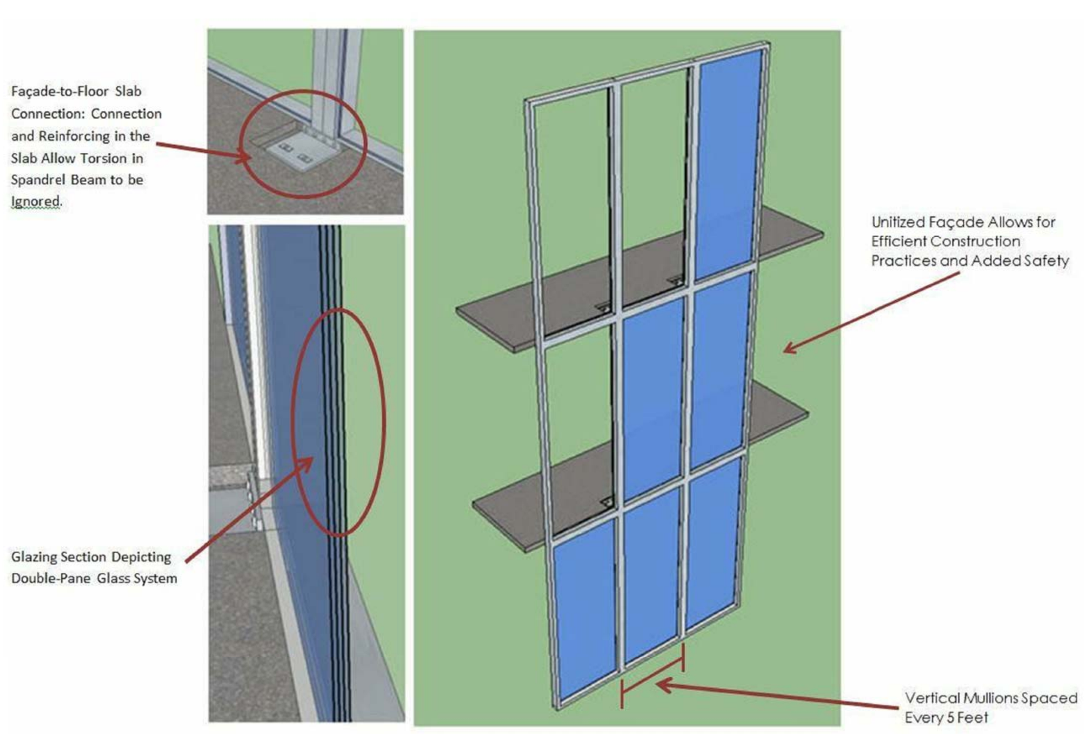
According to Memari, 2006; under seismic loading, rounded corner glass panels are as much as 50-90% more resistant against cracking and fallout issues due to drift.



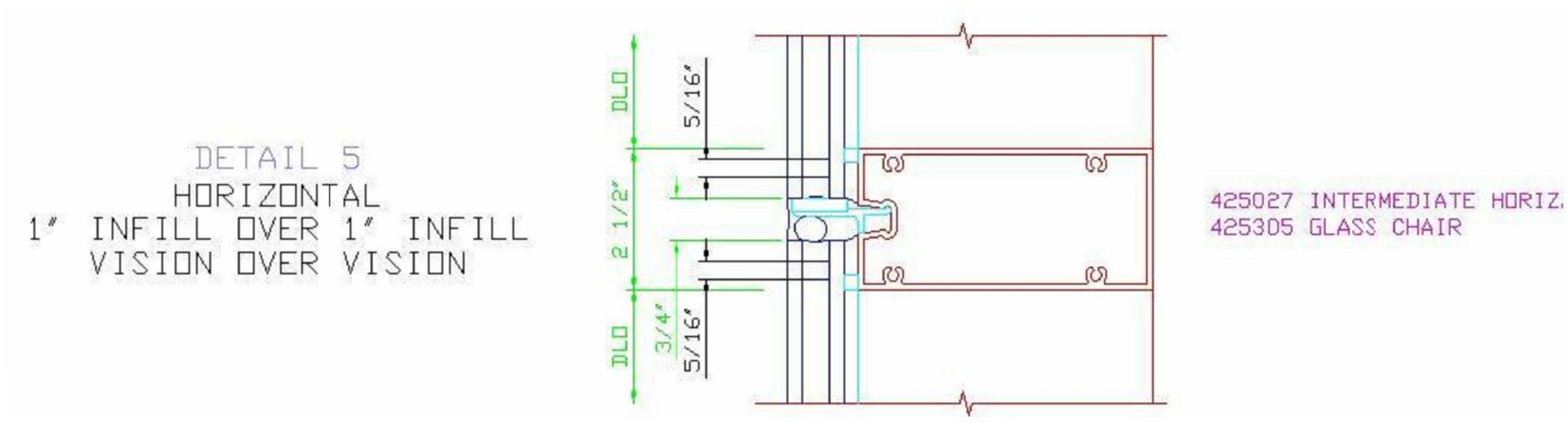
5 UNITIZED FACADE ATTACHMENT VIEW PROVIDED BY MANUFACTURER
S8 N.T.S.







6 GOOGLE SKETCHUP FACADE DETAIL AND GLAZING SECTION
S8 N.T.S.



7 KAWNEER FACADE ATTACHMENT DETAILS VERTICAL AND HORIZONTAL
S8 N.T.S.

4 FACADE ATTACHMENT DETAILS - KAWNEER
S8 N.T.S.

PROJECT
AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

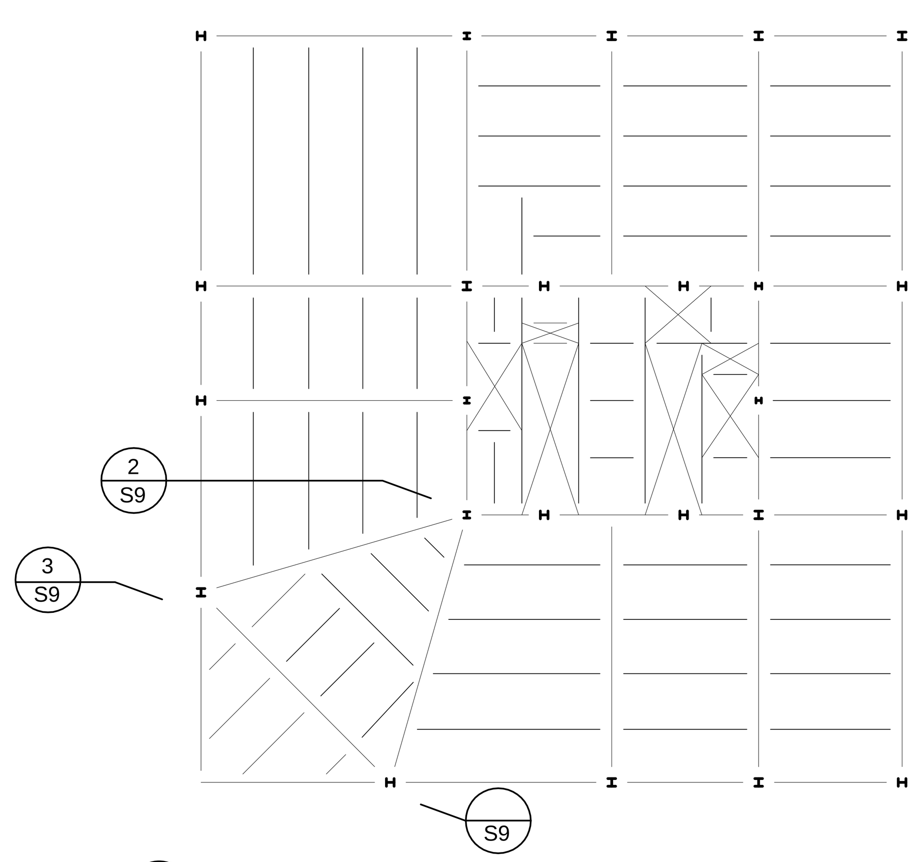
PURPOSE
ASCE STUDENT COMPETITION
AEI 2 - 2014

DRAWING TITLE
Building
Enclosure Details

SCALE:	As indicated
JOB NUMBER:	350 MISSION
DATE:	02/17/2014
DRAWN BY:	AEI 2 - 2014
SHEET NUMBER:	

S8

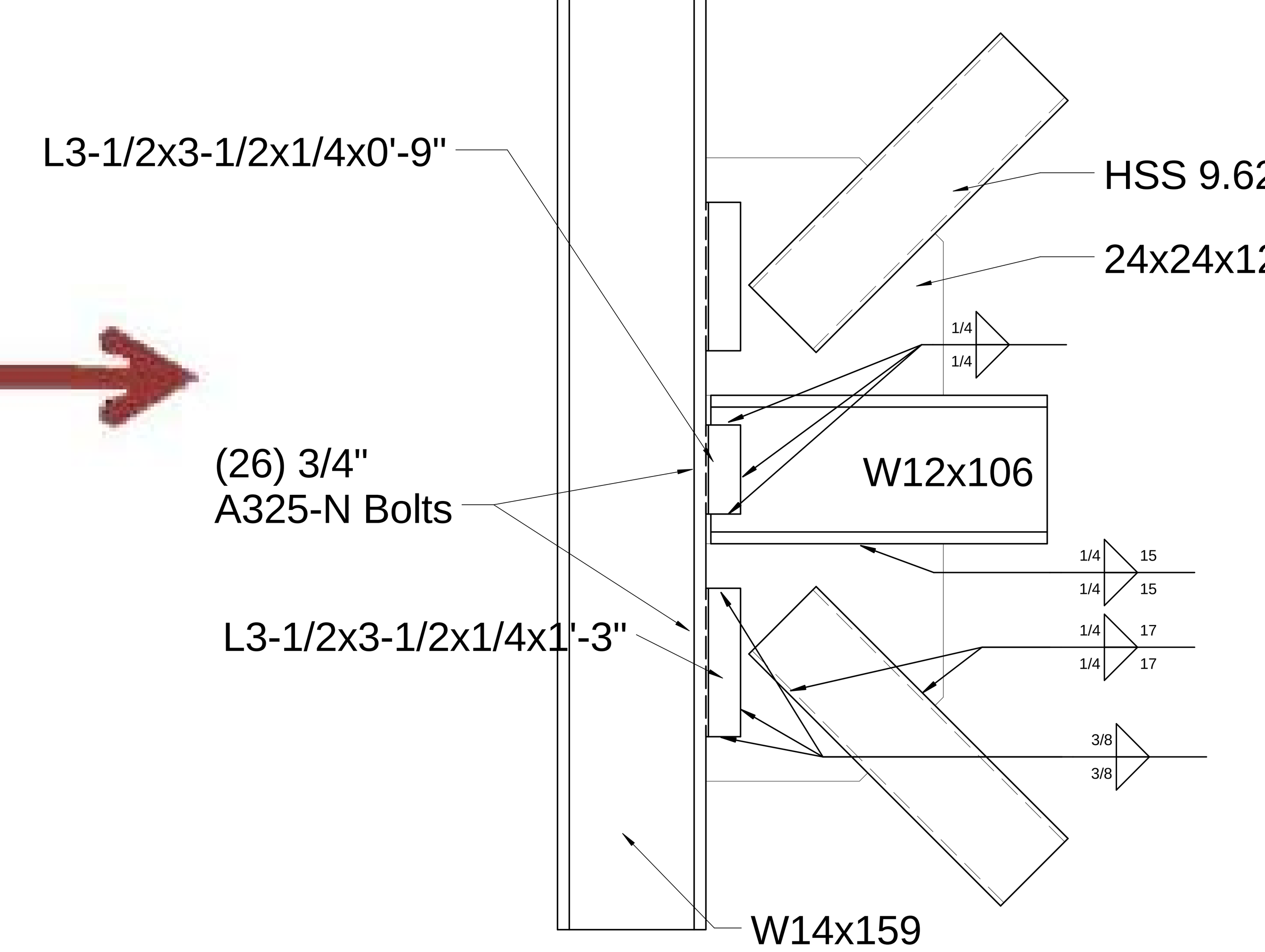
Notes:
1. All Plates, Angles, Channels use A36 Steel
2. All W-Shapes use A992 Steel
3. All welds use E70xx weld strength
4. All bolts use 3/4" DIA A325-N Bolts
5. All seismic considerations included in design



4 S9 Connection Key Plan
3/64" = 1'-0"

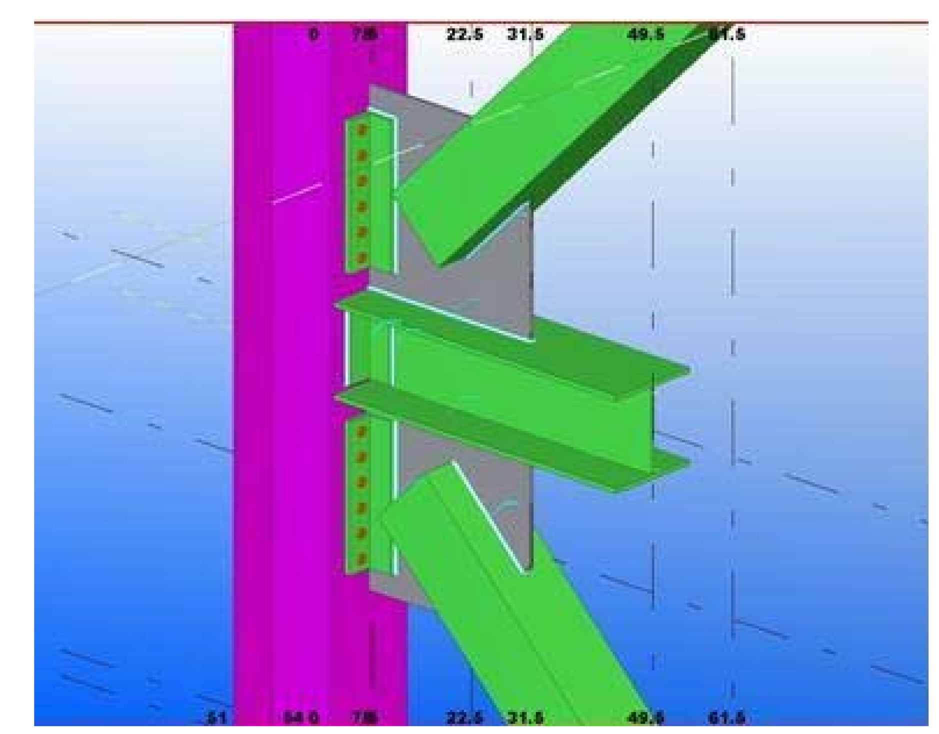
5 S9 3D LATERAL GUSSET CONNECTION LOCATION RENDER
N.T.S.

Note: Top and Bottom sides
of connection are identical

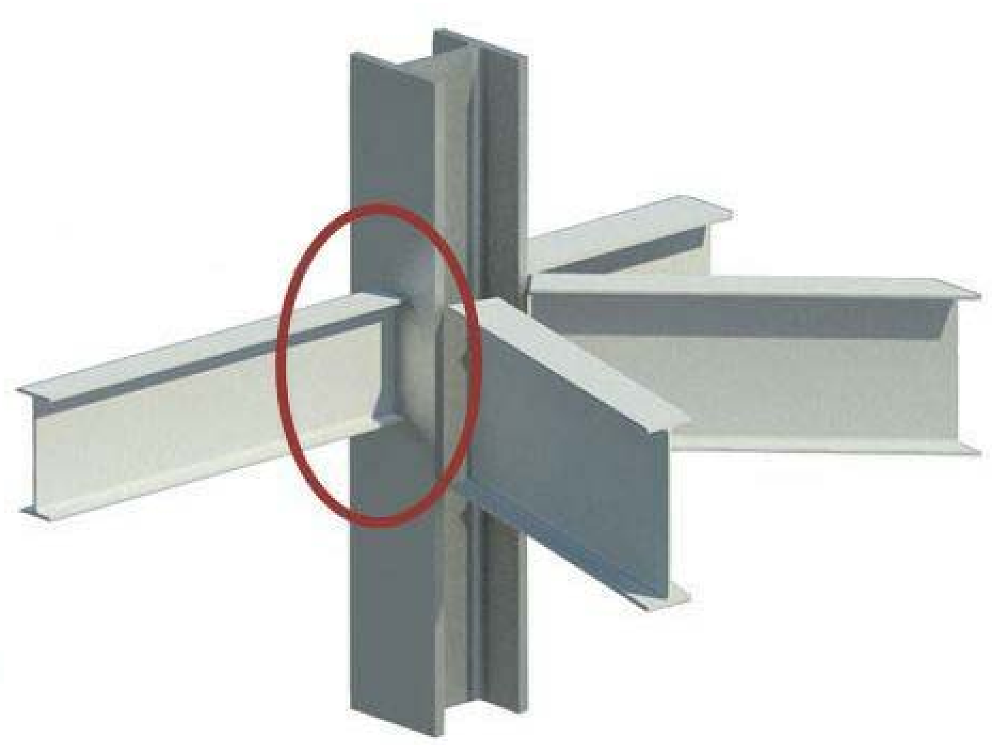


2 S9 Gusset-Beam-Column Connection
1" = 1'-0"

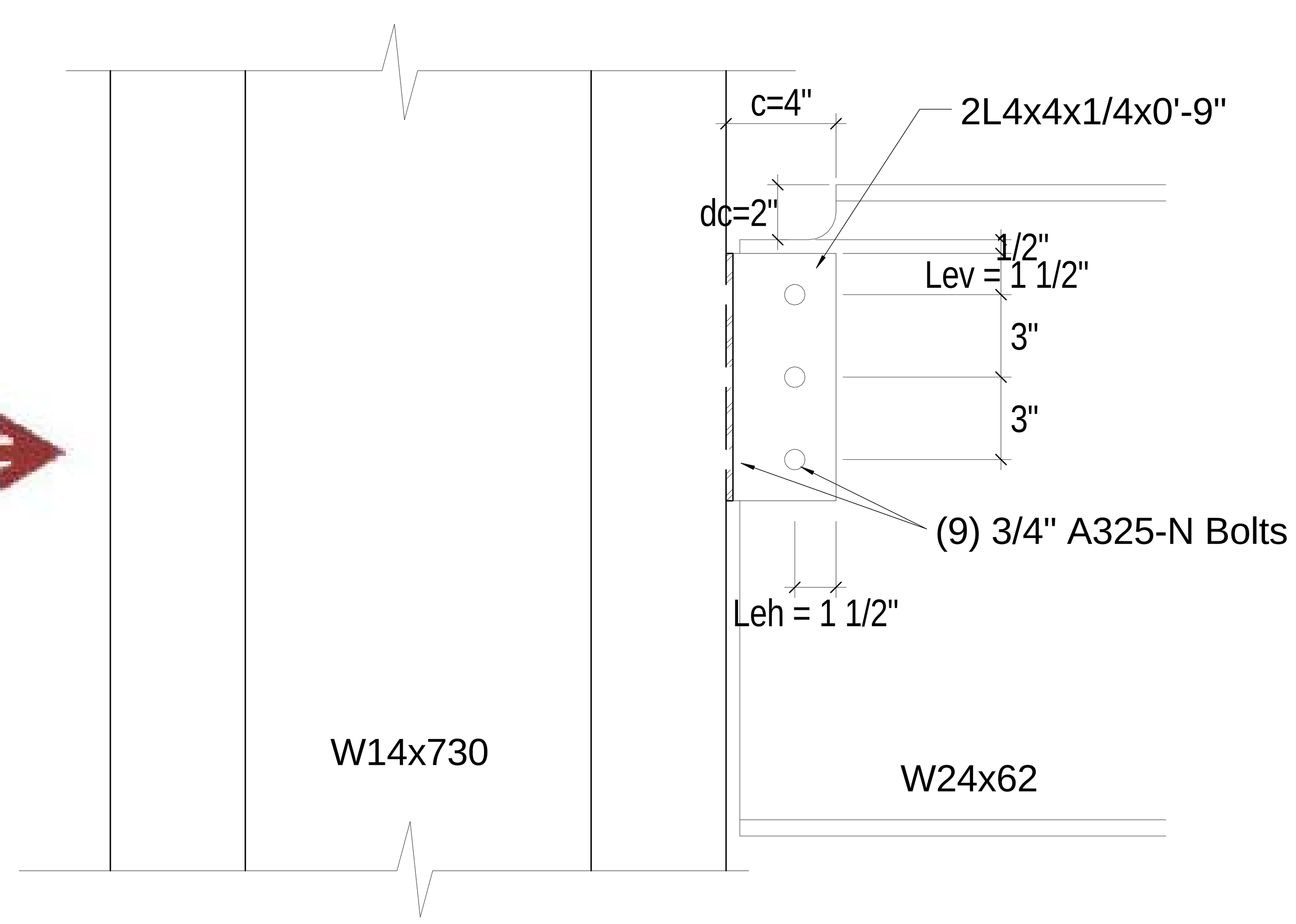
Lateral Bracing Connection Tekla Model



4 S9 LATERAL GUSSET CONNECTION TEKLA MODEL
N.T.S.

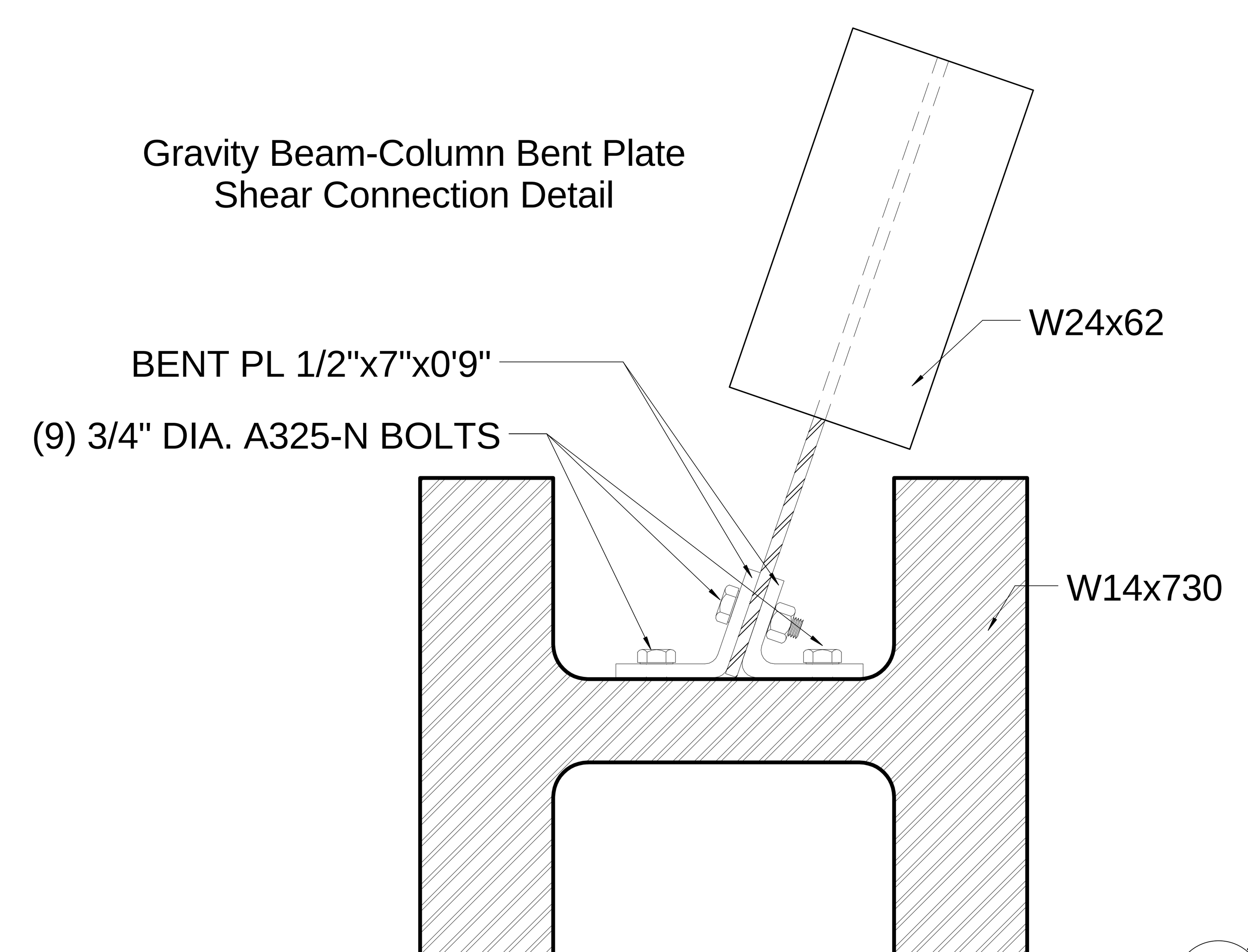


6 S9 3D GRAVITY BEAM-COLUMN CONNECTION LOCATION RENDER
N.T.S.



1 S9 Double-Angle Shear Connection
3" = 1'-0"

Gravity Beam-Column Bent Plate
Shear Connection Detail



3 S9 Bent Plate Shear Connection
3" = 1'-0"

PROJECT
AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE
ASCE STUDENT COMPETITION
AEI 2 - 2014

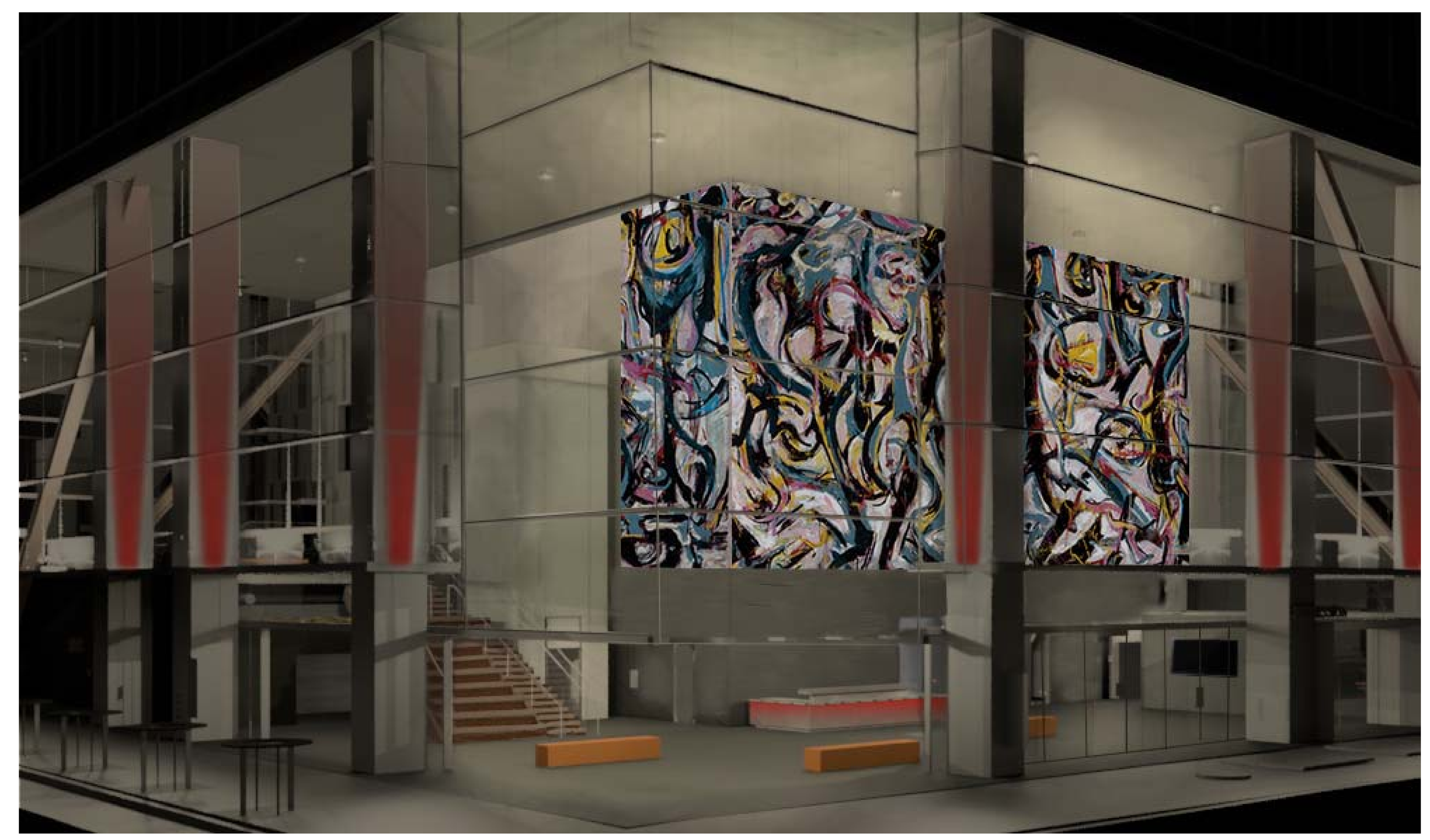
DRAWING TITLE
Connection
Details

SCALE: As indicated
JOB NUMBER: 350 MISSION
DATE: 02/17/2014
DRAWN BY: AEI 2 - 2014
SHEET NUMBER:
S9

1 STRUCTURAL FRAMING ONLY RENDER
S10 N.T.S.

2 FULL BUILDING DESIGN RENDER
S10 N.T.S.

3 TYPICAL OFFICE FLOOR SPACE
S10 N.T.S.



4 PUBLIC LOBBY SPACE AT NIGHT
S10 N.T.S.

PROJECT

AEI STUDENT COMPETITION
350 Mission Street, San Fransisco, CA

PURPOSE

AEI 2 - 2014

DRAWING TITLE

3D Renderings
and Images

SCALE:

JOB NUMBER: 350 MISSION

DATE: 02/17/2014

DRAWN BY: AEI 2 - 2014

SHEET NUMBER:

S10