181 Fremont

San Francisco, CA

Revised Proposal

1/16/2015

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Structural Option

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Executive Summary

181 Fremont is a 54 story high-rise in the South of Market neighborhood in San Francisco, California. Its construction is a part of the San Francisco Transit Center District Plan – a redevelopment plan that allows for greater building heights within that area of the city. As such, the building rises to 700 feet, the maximum height allowed per the limitations on the site.

In response to the high seismic loading brought about by the site location, the structure expresses a unique and complicated design solution. A mega-frame system, expressed on the exterior of the building, acts as the primary lateral system of the structure into which all other lateral forces are carried. Interior buckling restrained brace frames are located in the upper residential stories of the structure, while exterior moment frames frame the exterior of the office levels. Other contributors to the lateral system include collectors at each floor and viscous dampers in the exterior braces of the structure.

Because the mega-frame system is not defined in ASEC 7-05, an in depth seismic analysis was completed for the existing design that conforms to the San Francisco Department of Building Inspection Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures (SF AB-083, 2010) and the PEER Guidelines for Performance-based Seismic Design of Tall Buildings (PEER TBI, 2010).

The proposed thesis is intended to explore a structural design that results through prescriptive techniques of analysis of 181 Fremont in order that the design results may be compared to the existing performance based design of the structure.

In order to accomplish this, a more conventional outrigger system is proposed for the redesign. Using ETABS and prescriptive means of analysis, multiple outrigger configurations will be explored. The performance of the most optimal configuration will then be compared to that of the existing framing.

The use of outriggers rather than a mega frame also leads into the impact on the façade. A façade study is therefore proposed in order to investigate the cost, constructability, and daylighting impacts this structural change will have.

Introduction

Purpose

The purpose of this report is to propose an alternative, more traditional approach to the structural design of 181 Fremont. Various existing and proposed systems of the San Francisco high-rise will be explained, including the gravity, lateral, and foundation systems, as well as the codes and analysis procedures guiding the system selections.

Building Summary

Figure 1 | Southwest Elevation View (Courtesy of Heller Manus)

181 Fremont, as seen in Figure 1, is a mixed-use high-rise that is located in the South of Market/Transbay neighborhood of San Francisco, California. It is composed of 54 stories above ground, which includes two penthouse levels, and 5 stories below grade. Rising to a total height of 700’ (802’ with the spire), 181 Fremont will be the second tallest building in The City upon completion.

Approximately 2,000 sq. ft. of retail space, over 400,000 sq. ft. of office space (Figure 2) and over 160,000 sq. ft. of residential space are provided in the layout. Offices comprise the first 36 stories of the tower, while the top 15 stories consist of 68 condominiums. Separating the two uses are an amenity floor on level 37 and a mechanical floor on level 38. Additional features include a 78-stall bike barn, valet parking in the underground garage, and a direct connection to the City Park rooftop of the neighboring Transbay Transit Center at the fifth floor.

Construction of 181 Fremont is a contributor to The San Francisco Transit Center District Plan – a redevelopment plan for the area surrounding the previous Transbay Terminal and the future Transbay Transit Center (Figure 3). As part of the plan, height increases will allow for the construction of multiple new skyscrapers. Originally, the height of 181 Fremont was set to be 900 feet tall and consist of 66 floors, but became reduced to 700 feet due to a maximum height limit imposed on the site. In the building’s exterior, structural function merges with architectural design. The exposed primary lateral force-resisting system, composed of mega beams, columns, and braces, provides functional transparency and adds to 181 Fremont’s aesthetic imprint on The City’s skyline.

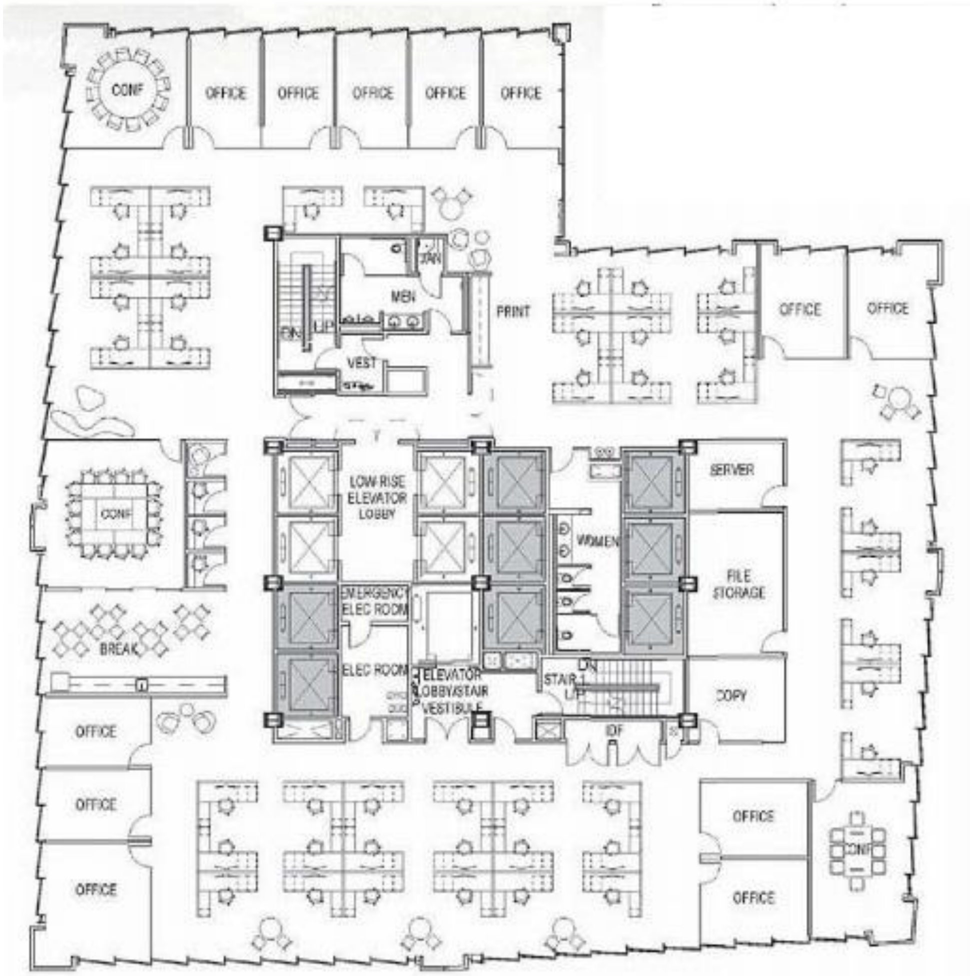


Figure 2 | Typical Office Layout (Courtesy of Heller Manus)

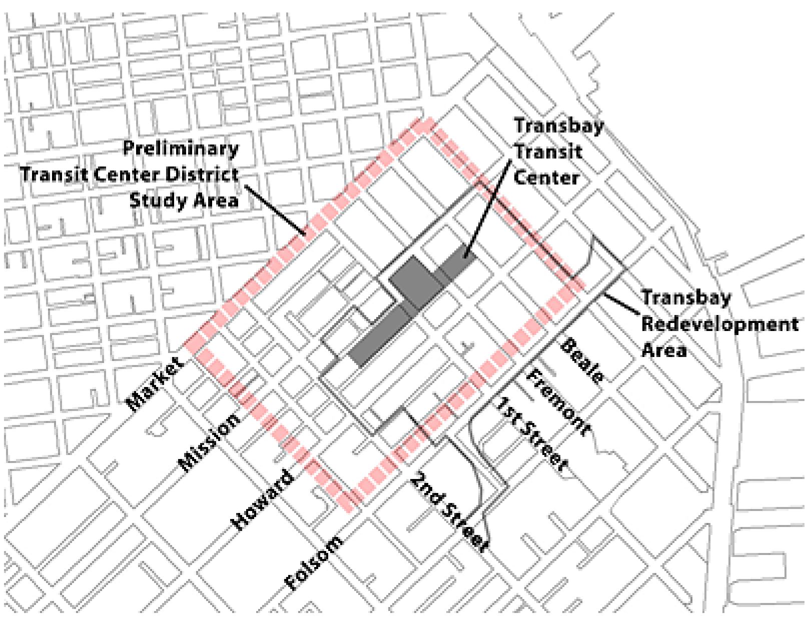


Figure 3 | San Francisco Transbay Redevelopment Area (Courtesy of Heller Manus)

Design Codes and Standards

Relevant design codes and standards used in the structural design are listed below, as well as the exceptions in code usage. The basis of analysis and design of 181 Fremont stems from these codes and standards, as well as from other testing methods in some cases.

Building Codes and Referenced Standards Utilized

- 2010 California Building Code (CBC, 2010)
- 2010 San Francisco Building Code (SFBC, 2010)
- ASCE 7-05
- ASCE 7-10

Seismic References

- SF AB-083, 2010
- PEER Tall Building Initiative
- ATC-72
- ASCE 41-06 for the acceptance criteria of moment frame beams
- Arup's in-house REDi Rating System

Material References

- AISC 360-05
- AISC 341-10
- ACI 318-08

Exceptions

181 Fremont utilizes a mega-brace system, as described in the building summary, that is not in table 12.2-1 of ASCE 7-05. Equivalence to a system that is listed in the code is allowed through other analysis and testing means per ASCE 7-05. To accomplish this, the San Francisco Department of Building Inspection Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures (SF AB-083, 2010) and the PEER Guidelines for Performance-based Seismic Design of Tall Buildings (PEER TBI, 2010) were utilized.

Design Approach

The following section is intended to outline the methods by which the building was designed. Various approaches guided system selection and the determination of an efficient design.

Dead and Live Loads

Due to the high seismic activity in San Francisco, different loads for the same occupancy or use were used depending on what was being investigated. For example, mechanical equipment is considered a live load for gravity design, but a superimposed dead load for seismic design. A summary of the loads used in the project are listed in table 1 below. Loads specific to earthquake design are designated with an "(E)" and loads designated with a "(G)" are specific to gravity design.

Occupancy/Use	Live Load (psf)	Superimposed Dead Load (psf)
Garage	40	10
Office	50	11
Residential	40	20
Mechanical Equipment	Actual Weight (G)	Actual Weight (E)
Stairs/Exits	100	n/a
Office corridors above 1st floor	80	n/a
Residential corridors	40	n/a
Mechanical	125	36
Partitions in Offices	15 (G)	10 (E)
Partitions in Residential	15 (G)	10 (E)
Roof		75
Storage	125 (G), 250 (E)	n/a

Table 1 | Summary of Building Loads

Wind

Although seismic is the controlling lateral force, the structural designers wanted to ensure occupant comfort on a daily basis due to wind loads as well. To achieve this, strength design conforming to ASCE 7-10 and utilizing wind tunnel testing modal output for 4% damping was performed. The analysis utilized a 700 year wind speed of 100 mph for a 3 second gust at 10 meters based on a site-specific climate study, and resulted in wind force equal to 138.2 kip at the 54th story. In order to meet the ISO 10137 residential acceleration criteria, dynamic forces and accelerations determined through wind tunnel testing under a one-year return period wind speed were used to design a supplementary damping system.

Seismic

Multiple methods of seismic analysis were used to account for various performance objectives, including a service level evaluation, Arup's REDi Gold evaluation criteria, and a Maximum Credible Earthquake (MCE) level evaluation. The service evaluation was done with Arup's in-house finite element analysis software, GSA assuming elastic behavior of the structure. The REDi Gold evaluation consisted of an elastic response spectrum analysis to determine the preliminary design, and a non-linear response history analysis (NLRHA) for final load determination in components. LS-DYNA was the software of choice for this evaluation due to its ability to capture non-linear geometry and material. The ground motion development approach also employed LS-DYNA for the same reasons.

Structural Design

The following section is intended to explain the overall structural design by breaking it into components and looking at the specific structural systems.

Overview of Structural Framing

181 Fremont utilizes steel as the only framing material for the lateral systems. Due to the high seismic zone that 181 Fremont is located in, seismic design was the controlling lateral force for the structural design. Wind considerations too, however, were also considered to ensure occupant comfort. Additional measures to mitigate wind effects include an increased number of collector beams in the floor framing as story levels increase. Viscous dampers in exterior braced frames reduce the vibrations caused by wind as well.

A mega-frame exterior acts as the primary lateral seismic-resisting system. Large scale beams, columns, and diagonal bracing members provide most of the structure's stiffness, and are supplemented by exterior moment frames and some interior braced frames.

Depending on the floor level, the gravity system consists of either lightweight or normal weight slab on deck atop steel beams and girders. The foundations are composed of concrete walls and 8'-0" think drilled shaft caps that sit on 5' and 6' diameter caissons.

Floor Framing and Structural Slabs

As the building rises, the exterior inclines inward and the area of the floor plates decrease. A typical lower story floor is just over 12,000 sq. ft., whereas a typical upper story floor is just over 9,000 ft. To mitigate vibrations and for acoustical purposes, the residential floor slabs are normal weight concrete on metal deck. The office floors that comprise the lower portion of the building, however, utilize lightweight concrete.

Figure 4 | Location of Composite Beams (S108)

A typical lower story floor framing plan consist of 5 ¼" light weight concrete on 18 gauge metal deck. The majority of deck is puddle welded to the supporting beams, with the exception of a few locations where studs are utilized (Figure 4). 24'-5" span W24 girders support 18'-9" long W14 beams at the core of the floor plan. The steel girders frame into six columns at the core and into the four mega columns at the corners of the building as well as standard exterior wide flange columns (Figure 5).

Level five framing is consistent with the typical lower floor framing except at the 33' wide connection to the Transbay Transit Center Roof. The connection is centered on the north elevation and is composed of cantilevered W21's spaced at just over 5'. Additional lateral-force collectors diagonal to the regular floor framing are added as well (figure 5).

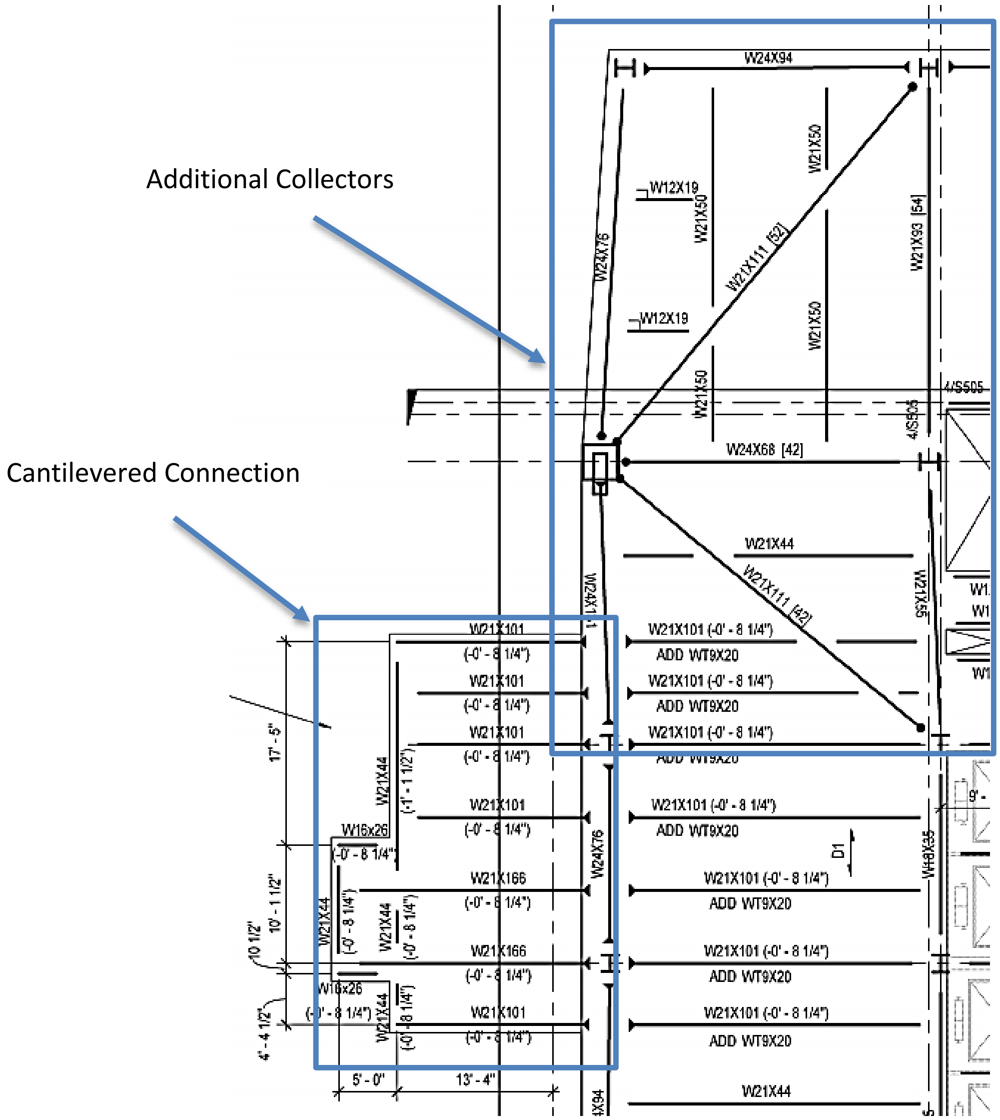


Figure 5 | Cantilevered Connection and Additional Collectors at 5th Floor (S105)

Typical upper story floors have 5 ¼" concrete on 18 gauge metal deck as well, but utilize normal weight concrete rather than lightweight. Other differences include the larger number of collectors to account for the greater seismic loads and a higher proportion of diagonally laid out beams (Figure 6).

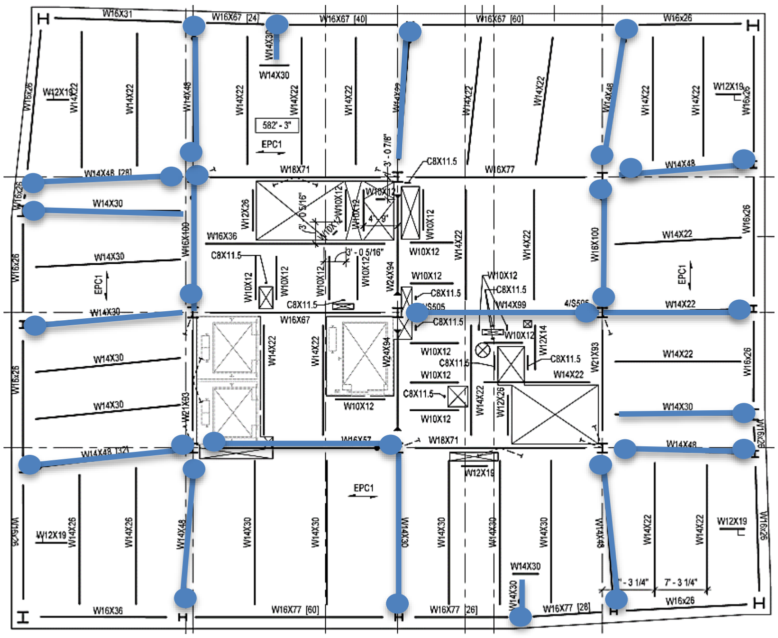


Figure 6 | Upper Story Collectors (S146)

Roof framing includes lightweight concrete over foam at the center of the plan and framing for the rooftop mechanical equipment. Diagonal members surround the gridded core, and each diagonal provides collectors (Figure 7).

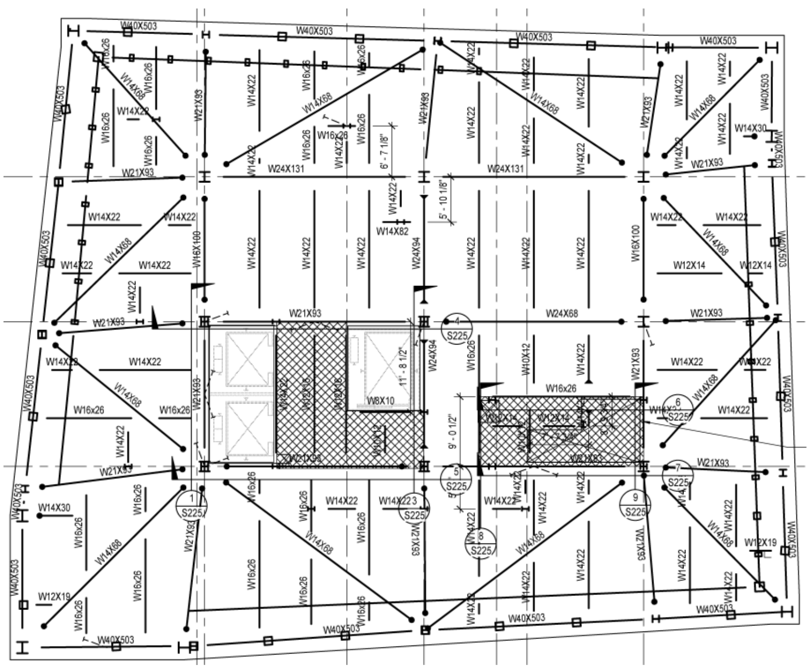


Figure 7 | Roof Framing With Diagonal Collector Members (S156)

Lateral System

A visible primary lateral system on the building's exterior is supplemented by an exterior secondary lateral system at the office levels and an interior secondary lateral system at the core of the residential levels.

Four mega-columns sit at the edges of the building, into which mega beams and steel braces frame. Together, the members form the primary lateral system (Figure 8). Various diagonal members contain viscous dampers as well to mitigate wind vibrations. This provides the additional benefit of decreasing seismic inertial forces.

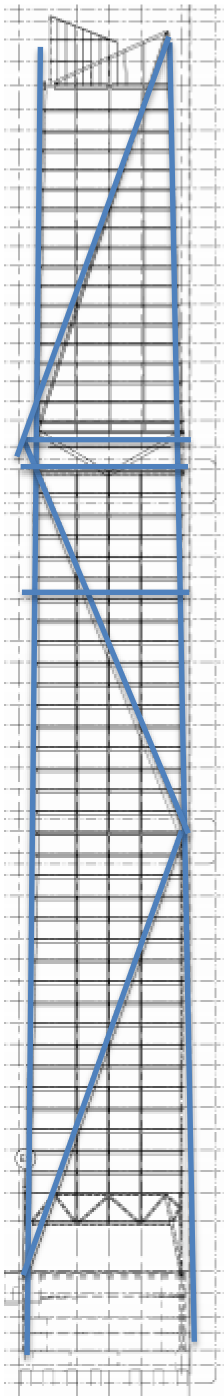


Figure 8 | South Elevation Of Primary Lateral System (1-S201)

Secondary Lateral Systems

At the office levels, exterior moment frames provide additional lateral force resistance while still maintaining the load path to the mega frame. At the residential levels, buckling restrained brace frames (BRBs) provide extra resistance at the core (Figure 9). The design of these BRBs require the braces to have a round HSS casing and pin connection and is contracted out and provided by Star Seismic. Additionally, secondary BRBs require an Fymin = 42.0 ksi and Fymax = 46 ksi.

The exterior trusses, like the one seen in figure 8, transfer gravity loads over the open lobby and transfer seismic loads back to the mega frame as well. All secondary lateral systems are designed to bring loads to the mega frame, as is discussed later in the "Load Path" section.

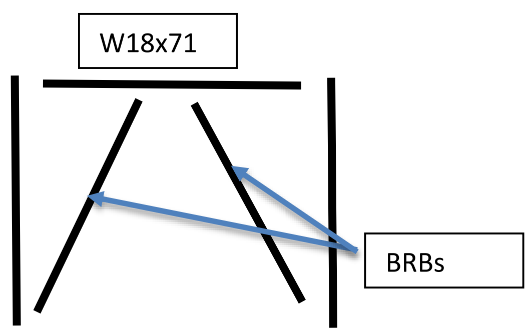


Figure 9 | Buckling Restrained Braces at Upper Levels

Mega Columns

The mega-columns, as mentioned briefly in the Lateral System section above, consist of cruciform steel starter columns encased by a concrete column. Studs on the flanges of the steel cruciform help it act compositely with the concrete, and weld ties are made from the concrete to steel (Figure 10).

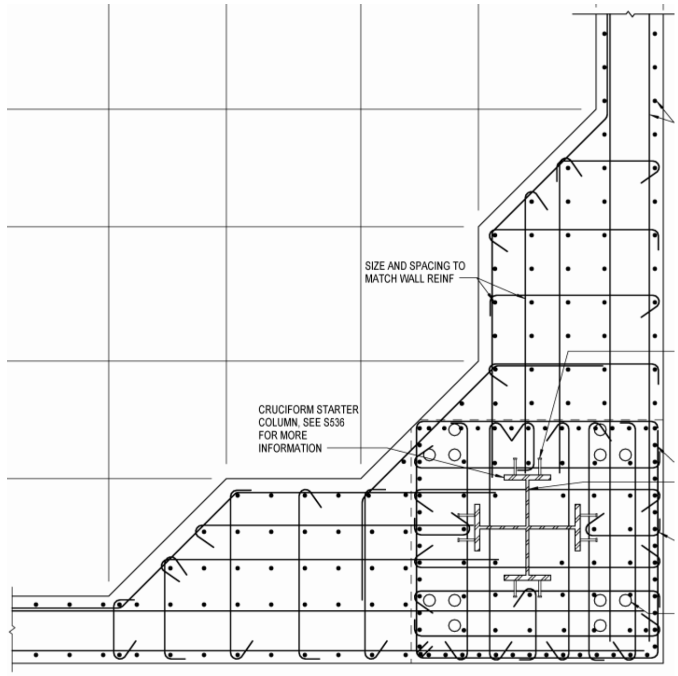


Figure 10 | Typical Mega Column (Detail 1-S332)

Foundation System

The foundation diaphragm consists of a reinforced mat slab with 43 drilled shafts that are 5' and 6' in diameter extending into the bedrock to support the tower. Exterior caissons are spaced at 13'-9" while interior caissons are spaced farther apart (Figure 12). An 8'-0" thick drilled shaft cap sits atop the caissons and extends from the bottom of the excavation up to basement level B4, reaching a height of 12'-0". Enclosing the basement are 2'-6" thick reinforced concrete walls that span between floors from level B4 up to level 1 (Figure 11).

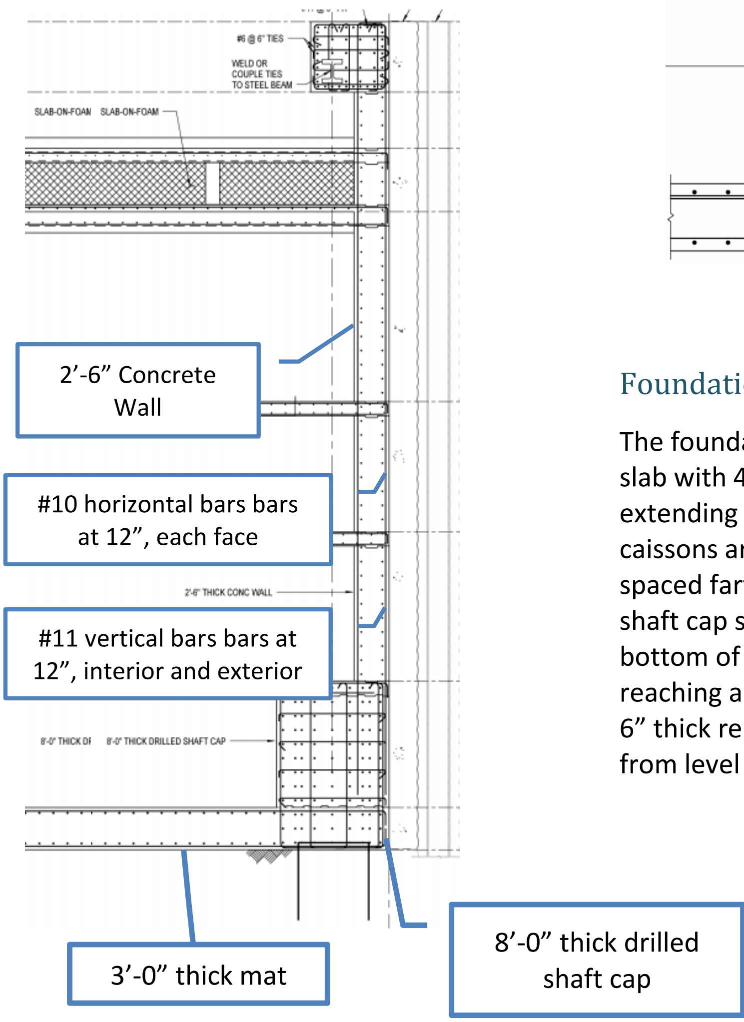


Figure 11 | Section Cut at Foundation (Detail 2-S320)

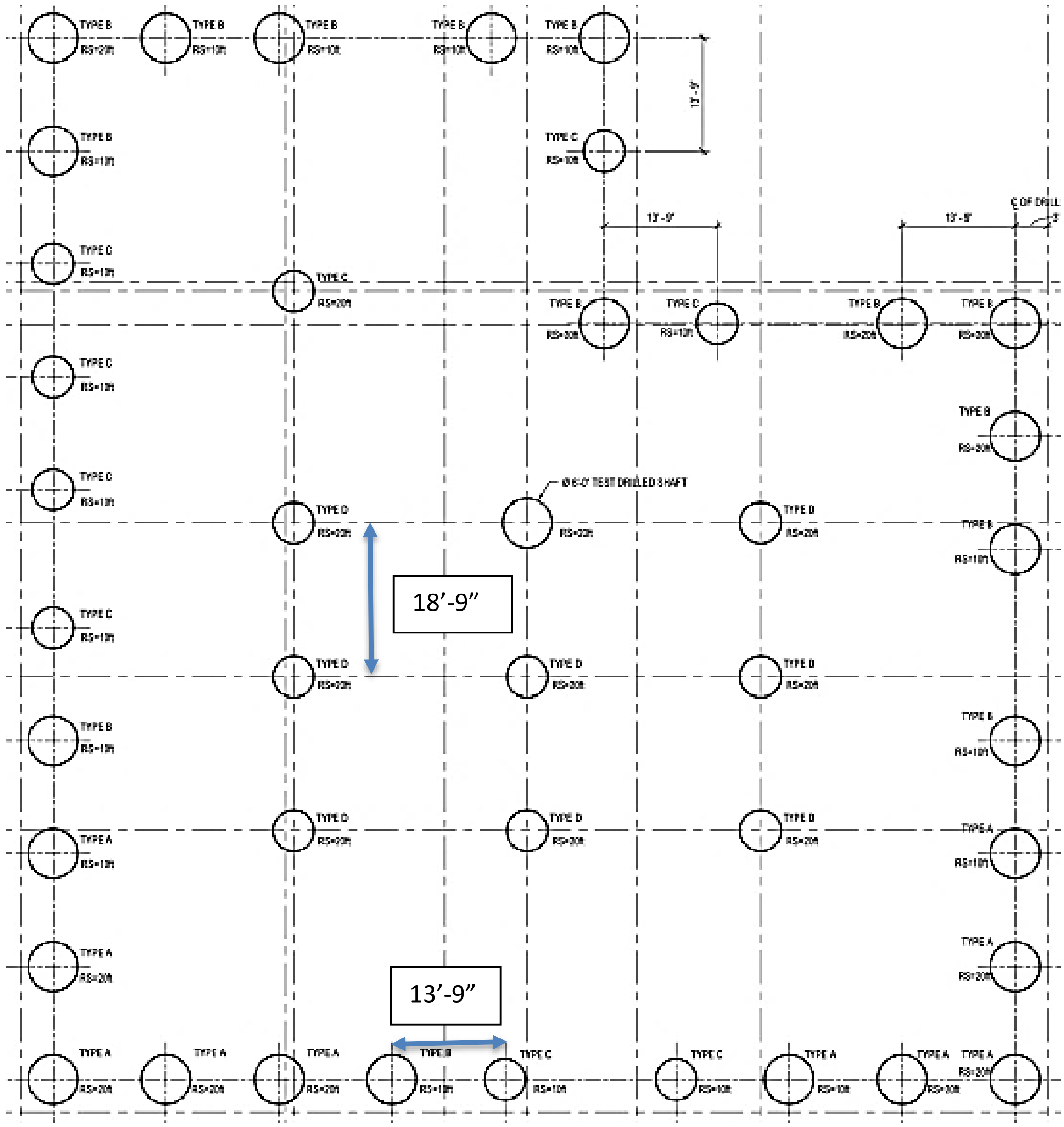


Figure 12 | Foundation Layout (S100F)

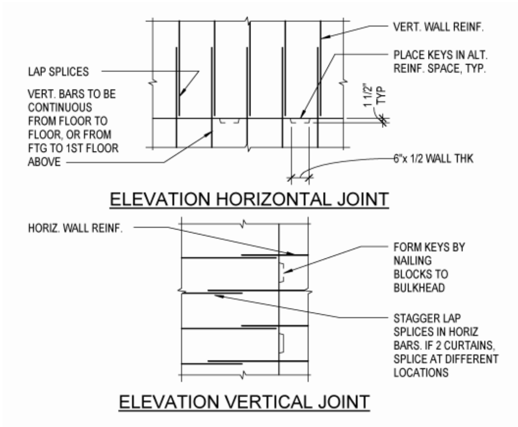


Figure 13 | Horizontal and Vertical Wall Joints (Detail 9-S304)

Joint Detailing

Joint details are critical both for successful construction and for adequate load transfer between members. This section outlines both concrete construction joints, as well as the various steel connections detailed.

Concrete Joints

For horizontal concrete wall joints, every other space between vertical rebar contains a key. Rebar must be continuous from floor to floor. Likewise, vertical joints require keys in every other rebar spacing, and in addition to having continuous rebar across the joint, the rebar laps must be staggered as well (Figure 13). Slab construction joints must be done at mid span and require additional #5 bar reinforcing spaced at 12".

Steel Moment Connection

Moment Frame beam-to-column connections utilize bolted shear plates with the same thickness as the beam webs (Figure 14). The shear plates are factory welded to the column along their full height. Additional stiffness is provided by doubler plates on each side of the column web. To prevent the flange failure characterized by pre-Northridge Earthquake connections, welds are specified to be complete

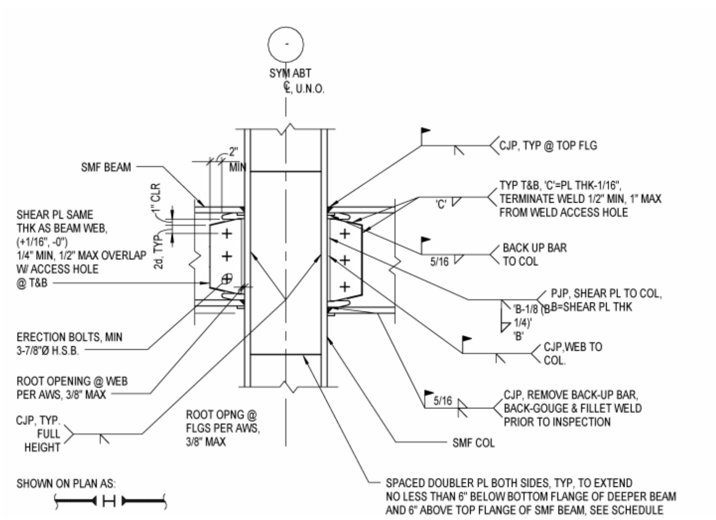


Figure 14 | Two-Sided Moment Frame Connection (Detail 1-S510)

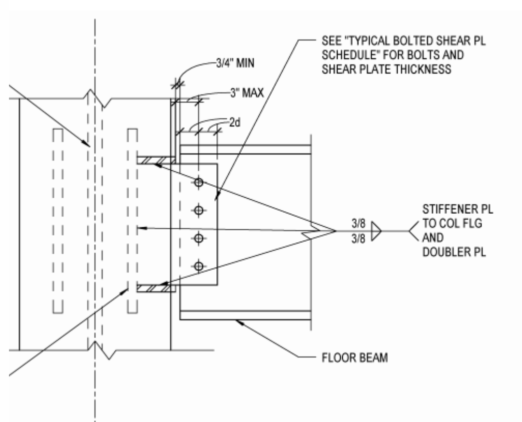


Figure 15| Floor Beam to Moment Frame Column (Detail 4-S510)

joint penetration welds, with back-up bars subsequently removed and the weld back-gouged, followed by the addition of a reinforcing fillet weld. Welds that connect the shear tab to the beam web complete the moment connection.

A typical floor beam framing into an exterior moment frame column has a bolted shear connection from the web of the beam to the web of the column. In addition to the shear plate, there are stiffener plates connecting the beam web to column flange (Figure 15).

Mega Column Connections

Continuity in load transfer to and within the mega-frame is critical. Continuity plates, as seen in figure 16, provide an uninterrupted load path from steel moment frames.

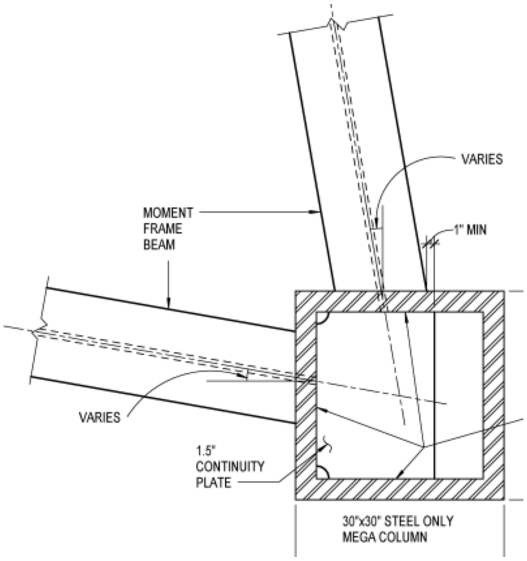


Figure 16 | Continuity Plate Through Mega Column (Detail 6-S511)

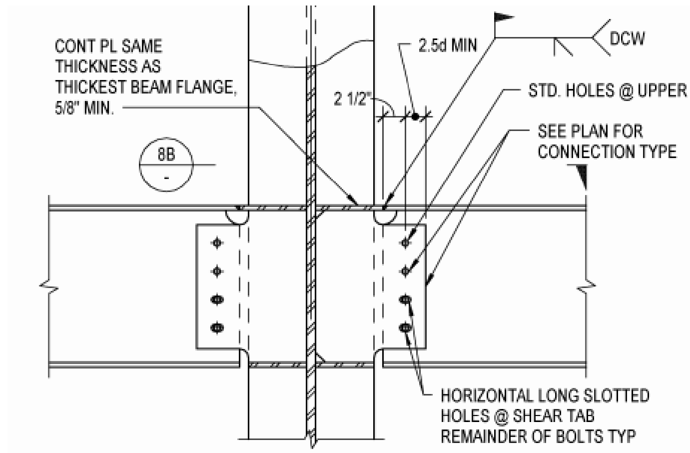


Figure 17 | Beam-to-Column Web Collector Connection (Detail 5-S512)

Collector Details

Collectors play a critical role in transferring lateral load from the floor diaphragms to the primary lateral system. Each level has at least one collector. Beam-to-column web collectors consist of continuous plates attached with demand critical welds and shear tabs consisting of two standard bolt holes at the top of the beam and two long slotted holes at the bottom (Figure 17). Beam-to-column flange connections are similar, but require continuity plates only at the top flange.

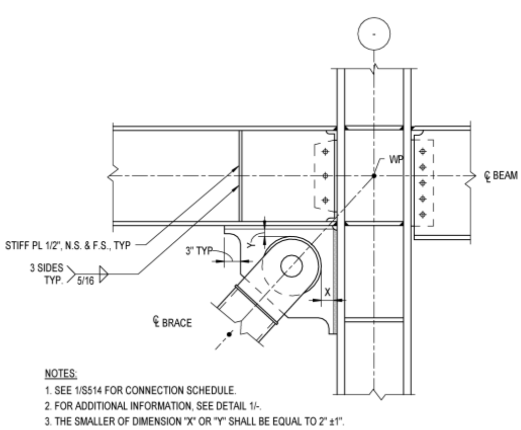


Figure 18 | Buckling Restrained Brace Pin Connection (Detail 2-S515)

Brace Frame Connections

All brace frames achieve pinned-end connections in the braces in a similar manner (Figure 18). The buckling restrained braces attach to the gusset plate through a pin hole. Circular reinforcing plates are shop welded onto each side of the gusset plate. Holes through the gusset and reinforcing plates align with a hole in the buckling restrained brace, through which a pin holds the mechanism together.

Load Path

The lateral load path, beginning from the diaphragm, consists of collectors extending from the core of the building that transfer load from floor and roof diaphragms to the mega-frame. Secondary lateral systems are designed to feed into the mega-frame, which then in turn carries the loads into the foundation. Because the mega-frame resists all lateral load eventually, that load is then distributed through the basement wall to the exterior caissons of level B5. For this reason, the perimeter caissons are spaced closer together than the interior caissons.

Unlike the lateral system, the gravity system does not transfer all loads directly to the mega-frame. Only exterior gravity loads eventually feed into the mega-frame when column loads are transferred into the exterior trusses, but interior gravity loads are carried straight down to the foundation through interior columns that sit on interior caissons at the foundation level.

Proposal

The external mega-frame of 181 Fremont provides an interesting architectural aesthetic, but also adds great cost to the detailing and construction of the façade. This proposal aims to suggest an alternative, more traditional structural framing system design that will provide possible insight into why the mega-frame design was used.

Structural Design Alternative

In lieu of the mega-frame, the proposed thesis will investigate a more conventional method of stiffening tall buildings. This method will aim to resist the loading under the seismic base shear determined in section 12.8.1 of ASCE 7-10. Additionally, the re-designed system will have a maximum total drift of about 7.5' and maximum story drifts of 1% of the story height per table 12.12-1 in ASCE 7-10.

The existing gravity framing is to be kept in place, with modifications made only to accommodate the new lateral system. Using prescriptive methods, the thesis will aim to investigate the use of a concrete core with the addition of supplementary lateral systems as necessary. First, internal shear walls will be sized within a reasonable thickness, then exterior moment and braced frames will be added where needed. The addition of steel-truss outriggers at the mechanical level on floor 37 will be investigated as well. Iteration will be done using all of these systems in order to determine the most efficient design.

Completing this design will allow for comparison to the mega-frame design and help to understand the cost savings associated with each, as well as the possible performance deficiencies that may result from a prescriptive design. Additionally, converting to a more conventional system will significantly reduce the impact the structure has on the façade, and therefore reduce the cost of detailing and façade construction. This gives way to a study of the façade to determine the costs saved.

Research and Analysis Methods

The design approach will first make use of approximate methods for sizing the core shear walls by using equations in the book "Tall Building Structures: Analysis and Design" that approximate shear and moment in the walls. Once the approximate loading is determined, the shear wall will be modeled in spWall to determine possible layouts and wall sizes. Finally, the shear walls will be modeled in the structural software SAP 2000 and conformance with ACI 318-11 will be checked.

Approximate analysis methods completed through the use of equations in the previously mentioned book will be used in developing trial sizes for the moment and braced frames as well. Once trial sizes are found, they will be added to the SAP model.

The design of the steel outrigger framing will be based on the 14th edition of the AISC Steel Construction Manual, AISC 360-10 and AISC 341-10. Initial trial sizes will be determined using RAM Elements and subsequently input into the SAP model with seismic loading determined through ASCE 7-10 applied.

Outrigger design will be investigated at the roof and at the mechanical level at the 37th floor since those levels already provide convenient outrigger truss locations.

Tasks and Tools

I. Task 1: Concrete Core
   a. Establish a trial size
      i. Determine feasible core layouts considering the architectural layout
      ii. Determine approximate loading and trial thickness for the concrete core based on the "Tall Building Structures" text
   b. Modeling
      i. Analyze possible shear wall designs in spWall
      ii. Model the chosen shear wall layout in SAP 2000

II. Task two: Moment Frames
   a. Establish a trial size
      i. Select trial sizes base on the "Tall Building" text
      ii. Add into SAP model

III. Task 3: Braced Frames
   a. Establish a trial size
      i. Select trial sizes base on the "Tall Building" text
      ii. Add into SAP model

IV. Task 4: Outrigger Investigation
   a. Investigate impact of adding outriggers
      i. Add outrigger trusses to the 37th floor
      ii. Add outrigger trusses to the 37th floor and roof
   b. Evaluate performance benefits vs. cost of adding outriggers

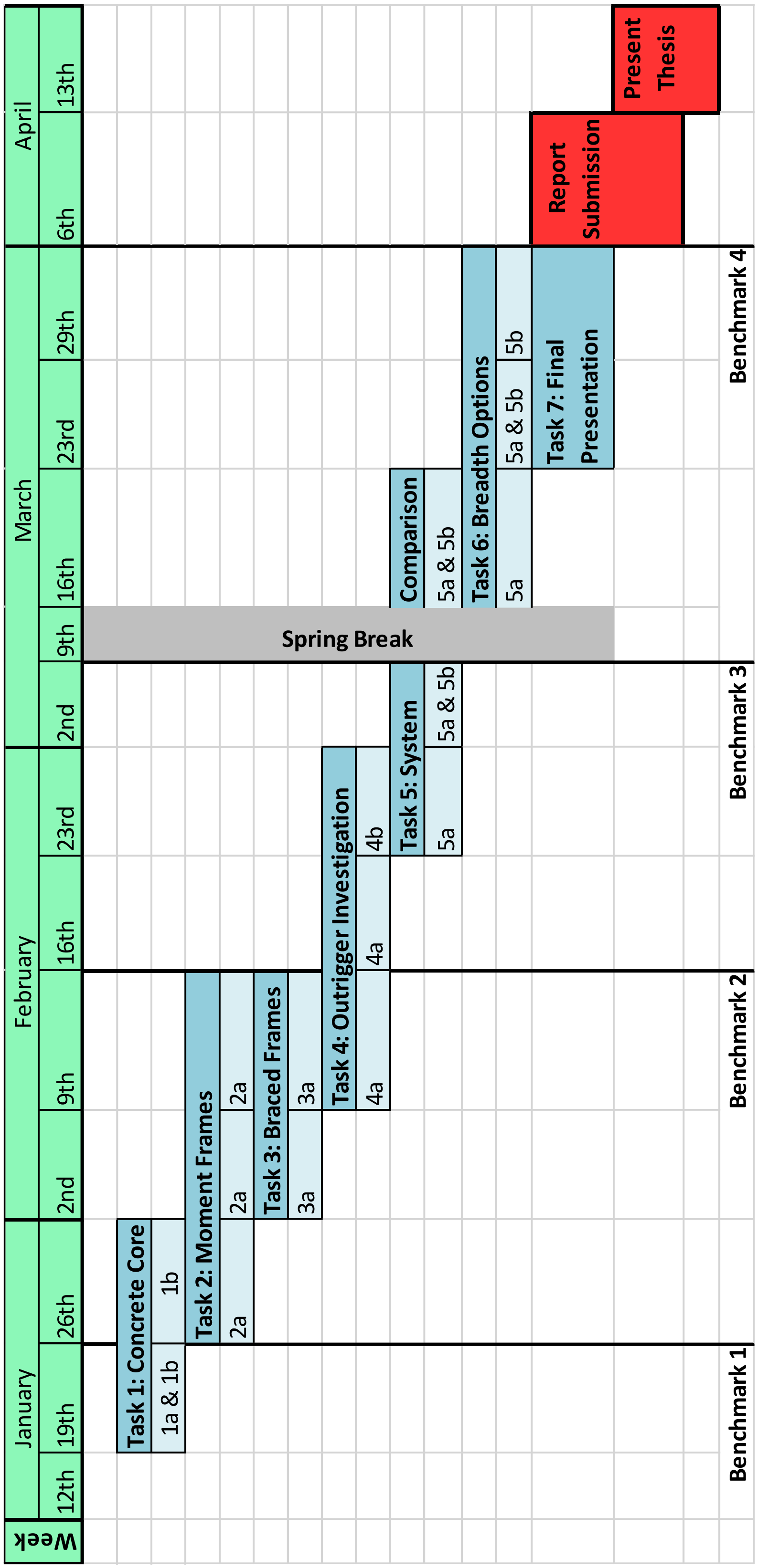
V. Task 5: System Comparison
   a. Compare the following components of each systems costs
      i. Labor
      ii. Material
      iii. Shoring
   b. Evaluate the performance in relation to the impact to architecture
      i. Detraction from floor space
      ii. Impact on floor plan
      iii. Visual impact
   c. Structural Performance
      i. Story Drift
      ii. Stress concentrations
   d. Loads
      i. Determine system self-weight
      ii. Determine seismic loads bases on the 2010 California Building Code
   e. Modeling

        i. Apply calculated loading to SAP model and perform design iterations until an effective system results

VI. Task 6: Breadth Options
   a. Construction
      i. Constructability
      ii. Cost
   b. Façade Study
      i. Thermal Performance

VII. Task 7: Final Presentation

Schedule


Week
January
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April
12th
19th
26th
2nd
9th
16th
23rd
2nd
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23rd
29th
6th
13th
Task 1: Concrete Core
1a & 1b
1b
Task 2: Moment Frames
2a
2a
2a
Task 3: Braced Frames
3a
3a
3a
Task 4: Outrigger Investigation
4a
4a
4b
Task 5: System
5a
5a & 5b
Spring Break
Comparison
5a & 5b
Task 6: Breadth Options
5a
5a & 5b
5b
Task 7: Final Presentation
Report Submission
Present Thesis
Benchmark 1
Benchmark 2
Benchmark 3
Benchmark 4

Breadth Topics

Two breadth topics relating to the façade will be proposed. The first will be a construction breadth aimed to determine the cost and constructability of the current façade system, and the second will be a mechanical breadth to evaluate the façade performance and determine areas for improvement.

Construction Breadth

The construction breadth will first involve analyzing the constructability issues that come with the mega-frame's complex connections and impact on the façade. This breadth will then look at what aspects of constructability are improved in response to the conversion to an outrigger system, and what constructability issues are raise. Finally, the cost of the current façade will be estimated and compared to the cost of a façade with no mega-frame.

Façade Study

The second breadth topic aims to evaluate the tilted glass pane concept behind the façade design. An analysis of the façades effectiveness in improving thermal performance and creating energy savings will be performed. This analysis will then be used to consider additional ways of improving the facades performance.

MAE Requirements

Graduate level coursework will be employed primarily through the use of computer modeling learned in AE 530. ETABS is being used to design and evaluate the lateral system options. Additionally, knowledge gained from the Building Enclosures class will be used in the breadth façade study.

Conclusion

The proposed thesis aims to explore the advantages behind performance based design through utilizing a prescriptive design approach instead. It also intends to explore the impact on the building's façade when there is no mega-brace used.

In redesigning the structure, an outrigger system will be explored, and consideration will be given to the optimal core framing. Through computer analysis in ETABS, evaluation of whether a steel brace frame core or concrete shear wall core will be made.