181 Fremont

San Francisco, CA

Final Report

4/8/2015

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Structural Option

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181 Fremont

San Francisco, California

General Information

Dates of Construction | Nov 2013 - 2016
Project Delivery Method | Design-Bid-Build
Occupancy| Mixed-use Office and Residential
Cost | $375 Million
Number of Stories | 54 Stories
Height | 700 ft.
Size | 411,000 sq. ft.

Project Team

General Contractor | Level 10 Construction
Construction Manager | Jay Paul Company
Owner | Jay Paul Company
Architect | Heller Manus
Structural Engineer | Arup
MEP Engineer | Arup

Structural Systems

The structure rests atop a mat foundation, below which roughly 60 piles extend 150 feet down to reach bedrock. Various systems such as viscous dampers and steel moment frames provide lateral force resistance, but the primary lateral force resisting system is an exterior steel mega-frame.

Sustainability

In pursuit for LEED Platinum, multiple steps toward sustainability including a curtain wall system that favors natural lighting, a green roof, grey water system, and use of recycled materials are featured.

Architecture

The architectural design features transparency in the structural system by exposing the exterior steel mega-frame, which extends beyond the roofline. A curtain wall system with angular glass units and walls that taper in as the building rises also add to the building's exterior aesthetic expression.

Various amenities are provided for residents, including a two-story open air terrace that wraps around the 36th floor. Also features is a pedestrian bridge on the 5th floor that allows residents to access the Transit Tower's rooftop City Park, as shown in the photos at left and below.

Mechanical Systems

181 Fremont's mechanical system is comprised of a forced-air ventilation system, with air intake and filtration occurring on the mechanical floor on level 37. Air is then transferred to each individual residential unit, where it is again filtered and either heated or cooled by a fan coil unit.

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ADVISOR | DR. THOMAS BOOTHBY

http://www.engr.psu.edu/ae/thesis/portfolios/2015/cjk5258/index.html

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Executive Summary

In order to better understand the purpose and benefits behind completing a performance-based design and utilizing a non-traditional lateral system, a prescriptive approach to the design in accordance with ASCE 7-10 was performed. This approach brought to light the cost benefits in using a prescriptive approach, but also brought to light many of the drawbacks.

Although both systems have their pros and cons, the existing system proves ideal for predicting building behavior in the case of seismic loading as well as for serviceability and occupant comfort. In utilizing a prescriptive approach, the type of nonlinear behavior could be better estimated in the existing system, failure modes were addressed in a more specific manner, and occupant comfort could be ensured. This design exceeds the minimum performance requirements and increases the chance that the building will be quickly re-inhabitable after an extreme earthquake.

The alternative concrete core and outrigger system designed is not able to offer the same performance objectives, but it does make for a more economical solution, as well as a more straight-forward construction process. On top of that, time and money is saved in the design process from eliminating the need for a PEER Review.

Building Summary

Project Background

Due to be completed in early 2016, 181 Fremont is a mixed-use commercial and residential high-rise under construction in San Francisco's South of Market/Transbay neighborhood. The 55 stories the building adds up to are composed of 36 commercial floors and 17 residential floors over the top third of the building height. A recreational floor that serves the apartments and a mechanical floor, located on levels 37 and 38 respectively, are sandwiched between the commercial and residential levels. These levels are also where the exterior truss is located, as seen in Figure 1.

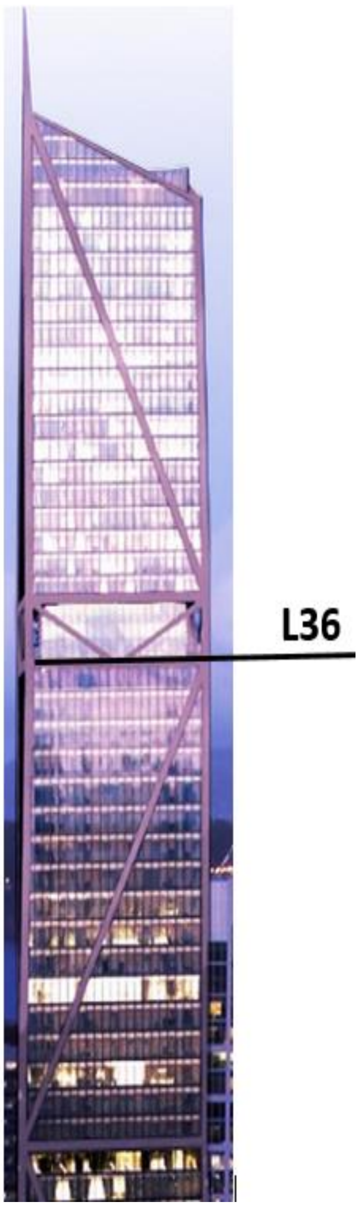


Figure 1|Truss at Levels 36 through 39 (Courtesy of Heller Manus)

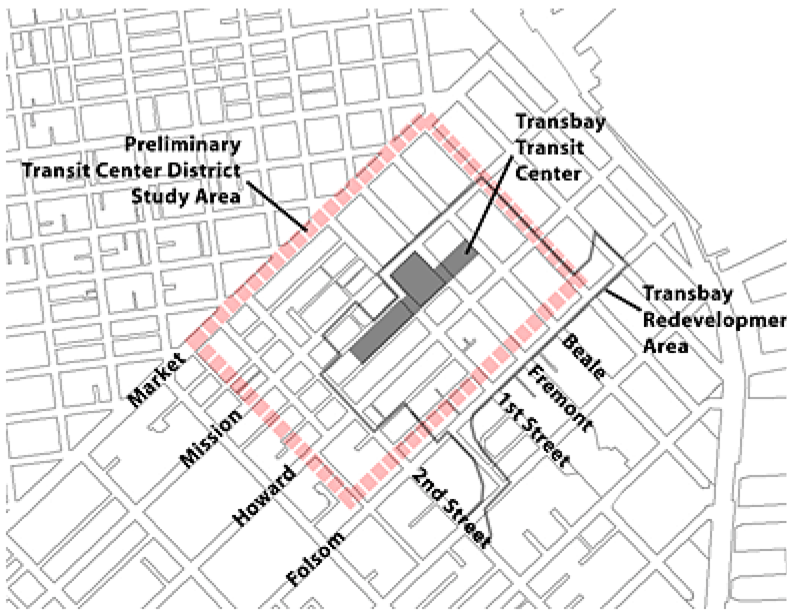


Figure 2|Map of Transbay Redevelopment Plan (Courtesy of Heller Manus)

The project is a part of the Transit Center District Plan – a redevelopment plan for the area surrounding the future Transbay Transit Center, shown in Figure 2. Part of this plan includes height increases which will allow for the construction of multiple new skyscrapers, and which has allowed for 181 Fremont to attain it's 802' height to the top of the spire – thereby qualifying it as the second tallest building in the city until the completion of the Transbay Tower.

Site and Architecture

Situated just a few blocks from the Eastern Bay, 181 Fremont offers views of the city as well as views of the Oakland Bay Bridge from its upper stories, as demonstrated in Figure 3. The site's location adjacent to the future Transbay Transit Center is taken advantage of by providing public access to the center's rooftop city park. A connecting bridge may be accessed from the fifth floor, as shown in Figure 4 and Figure 5. Additional features include an open-air terrace and a common area with a fitness center and lounges serving the residential floors.

Approximately 2,000 ft^2 of retail space, over 400,000 ft^2 of office space, and over 160,000 ft^2 of residential space will be provided. Figure 7 displays the typical office plan. The open floor plate provided not only allows ample daylight into the space, but allows for variability in office layout as well.

Exterior architecture is expressed in a variety of ways: tilting façade, a "sawtooth" curtain wall, and the structural transparency all add to the building's aesthetic. At each elevation, sections of the façade tilt inwards in two dimensions as the tower extends

Figure 3|181 Fremont With the Bay Bridge in the Background and City Park in the Foreground (Courtesy of Heller Manus)

Figure 4|Street Level View of Bridge to City Park (Courtesy of Heller Manus)

Figure 5|Aerial View of City Park (Courtesy of Heller Manus)

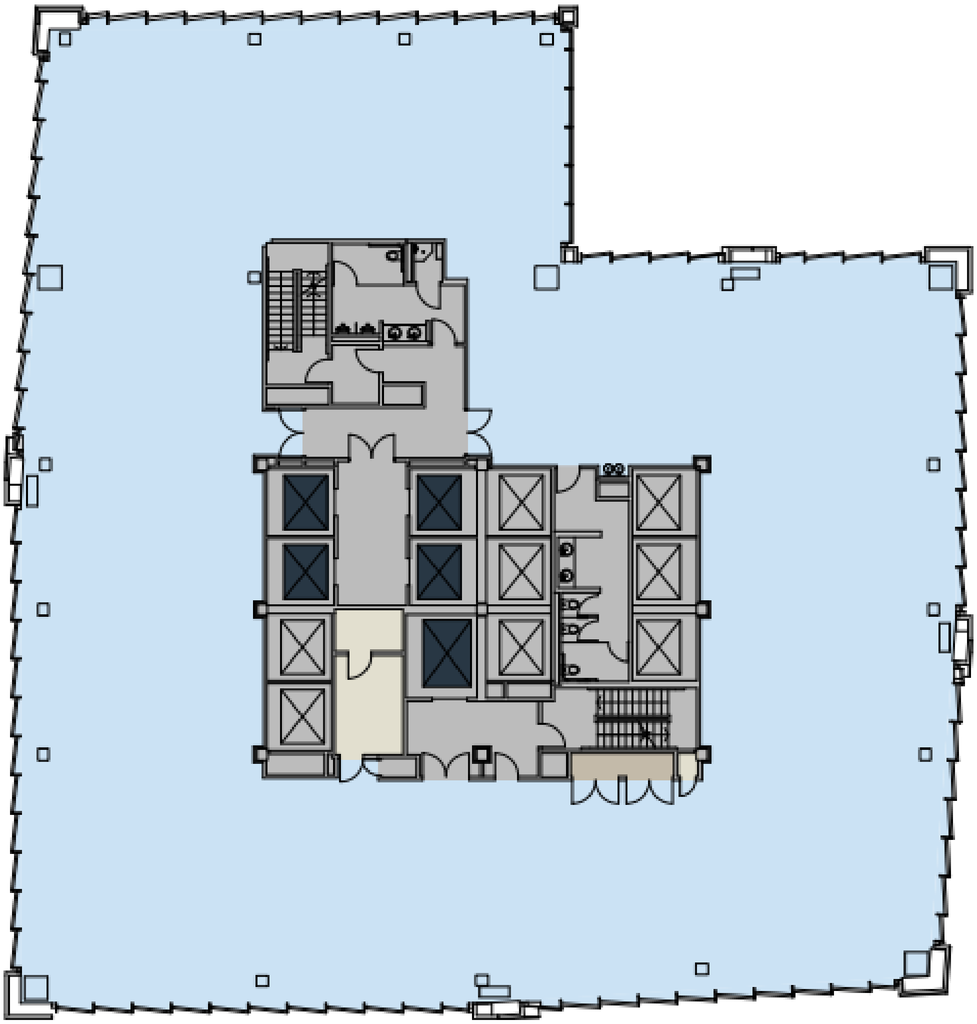


Figure 7|Typical Low-Rise Office Floor Plan (Courtesy of Heller Manus)

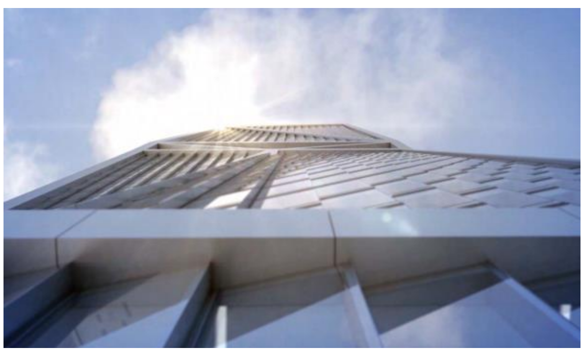


Figure 6|Tilted Facade and Windows (Courtesy of Heller Manus)

upwards, thus improving the view from ground level. The curtain wall adds texture to the enclosure through the use of angled windows, shown in Figure 6.

The structural framing system utilized is another significant aspect in the building's aesthetic. The exterior columns and lateral bracing is emphasized by the contrast its cladding has with the curtain wall, thus accentuating the angular expression.

Existing Structure

Design Approach for Wind Loads

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, wind tunnel testing modal output for 4% damping was performed in accordance with the requirements of The American Society of Civil Engineers' "Minimum Design Loads for Building Structures" reference standard (ASCE 7-10). 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.

Design Approach for Seismic Loads

Due to the buildings location, performance as a seismic design category D structure needed to be evaluated. Multiple methods of seismic analysis were used to account for various performance objectives, including a service level evaluation, Arup's REDi Gold evaluation criteria, a code level analysis in accordance to the 2010 San Francisco Building Code (SFBC 2010), and a Maximum Credible Earthquake (MCE) level evaluation. The service evaluation was done with Arup's in-house finite element analysis software 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.

The REDi Gold evaluation criteria was used in order to achieve higher performance in the lateral system than that required by code. The purpose of the REDi rating system is to enable a resilience-focused design for the lateral system, which is intended to allow owners to quickly resume use of their buildings after a 475 year return period earthquake.

Gravity System

The foundations are composed of concrete walls and 8'-0" thick drilled shaft caps that sit on 5' and 6' diameter caissons. These support the core columns as well as four megacolumns. The megacolumns, which are large box sections below the truss on level 37 and large W14s above the truss, are bridged by a transfer truss at level 2 on each elevation, visible in Figure 8. This allows for an open entryway on each side of the building.

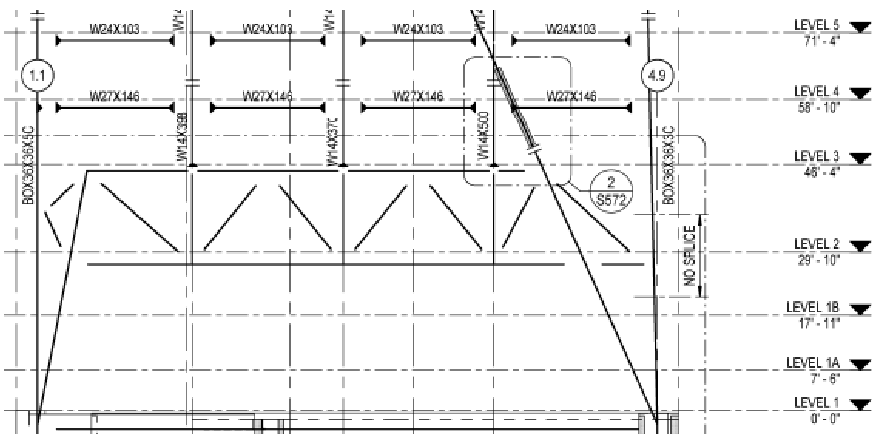


Figure 8|Transfer Truss at Level 2 (Courtesy of Heller Manus)

As the building rises, the exterior inclines inward and the area of the floor plate decreases. A typical lower story floor is just over 12,000 square feet, whereas a typical upper story floor is just over 9,000. Depending on the floor, the gravity system consists of either lightweight or normal weight slab on deck atop steel beams and girders. For acoustic purposes, the normal weight slab on decks are located on the upper floors where the residences are located. A typical lower story floor 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, which is shown in Figure 9.

Figure 9|Location of Composite Beams (Courtesy of Heller Manus)

Lateral System

The primary lateral force resisting system is an exterior megaframe, shown in Figure 10, which is composed of large built up box members below the 37th floor truss and large W-shape members above the truss. As part of the megaframe system, four mega-columns sit at the edges of the building (Figure 11), into which exterior steel mega beams and braces frame. This primary system 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. Various diagonal members contain viscous dampers as well to improve damping under wind loading. This provides the additional benefit of decreasing seismic inertial forces.

Figure 11|Megacolumn Plan Locations (Courtesy of Heller Manus)

The megaframe is designed such that all secondary systems transfer load into it. 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, chevron-shaped buckling restrained brace frames (BRBs) provide extra resistance at the core.

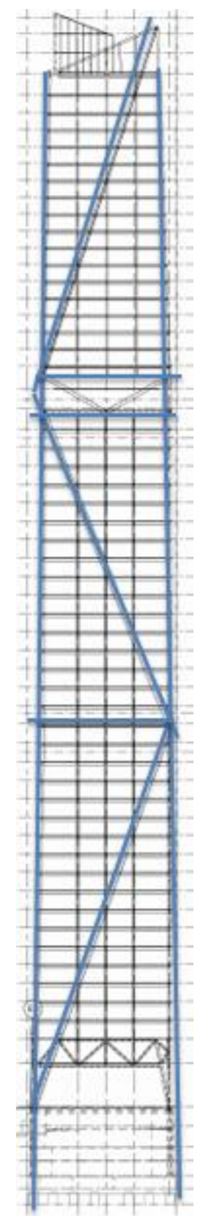


Figure 10|Exterior Megaframe (Courtesy of Heller Manus)

Alternative Solution

Objective

The utilization of a megaframe precludes 181 Fremont from being able to use a prescriptive analysis, as it is not able to be classified in table 12.2-1 of ASCE 7-10. As a result, a performance-based design is required to show code equivalent performance. To provide insight into both the benefits and the drawbacks of the megaframe system and its respective design method, a more traditional lateral system was designed. In doing so, a basis for comparison between performance-based and prescriptive analyses was afforded.

Solution

In lieu of the megaframe, a dual system consisting of a concrete shear wall core with steel truss outriggers and external moment frames was designed. This allowed the structure to be classified in table 12.2-1 of ASCE 7-10 as a dual system with special moment frames resisting at least 25% of prescribed seismic forces. As a result, prescriptive analysis of the resulting seismic forces in accordance with ASCE 7-10 was able to be performed.

The modified design maintained the existing gravity system, with the exception of reduced member sizes at the exterior moment frames and replacement of 7 core columns with shear walls, as demonstrated in Figure 12. Addition of extra bracing was to be investigated along with the moment frames, but the extra stiffness proved unnecessary after the addition of outriggers.

Figure 12|Existing Core Columns and Replacement Shear Wall (Courtesy of Heller Manus)

In addition to the structural redesign, two breadth topics were studied in order to gain insight into other aspects of 181 Fremont's design: a construction breath and a façade study. The construction breadth involved a constructability study of the current façade and the interaction it has with the megaframe. This allowed for comparison with the constructability issues of the new design. Additionally, a cost analysis of the megaframe and façade was conducted to determine the extra expense incurred.

The façade study focused on evaluating the functionality of the enclosure and determining the effectiveness of the curtain wall's tilted window pane concept. Further explanation of the structural design iterations and modeling approach performed, as well as of the breadth studies, is provided in the following sections of this report.

Preliminary Approach

Seismic Code Considerations

Due to the high seismic base shear expected, a lateral design catered to seismic performance was first established. This was done in accordance with ASCE 7-10. From Table 12.6-1 of ASCE, shown in Figure 13, the structure doesn't meet the required period for a height exceeding 160 feet as shown in Equation 1. The Equivalent Lateral Force Analysis (ELF) is therefore not permitted. This qualifies it as "All other structures," and as a result a Modal Response Spectrum Analysis (MRSA) was performed.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

Figure 13|ASCE 7-10 Table 12.6-1, Permitted Analytical Procedures

Required for Equivalent Lateral Force Analysis: $T < 3.5 * T_S$

$$3.5 * T_S = 3.5 * \frac{SD1}{SDS} = 3.5 * \frac{0.6}{1} = 2.1$$

$T \approx 7 \text{ to } 8 \text{ sec} > 2.1$, therefore ELF not permitted

Equation 1|Required Period for ELF

In addition to parameters regarding the analysis method, lateral system type is also limited. Table 12.2-1 of ASCE 7-10 outlines allowable system types. Of these, only a fraction are permissible for 181 Fremont: steel special moment frames, special reinforced concrete moment frames, steel and concrete composite special moment frames, and the majority of dual systems with special moment frames capable of resisting at least 25% of prescribed seismic forces. Out of all these options, only one of the dual systems is practical. For the scope of this thesis, the dual system with special reinforced concrete shear walls is explored.

In designing this system, horizontal and vertical irregularities, as defined in ASCE Tables 12.3-1 and 12.3-2, are considered as well. Per section 12.3.3.1, an extreme weak story irregularity—a vertical irregularity in which a given story's lateral strength is less than 65% of that in the story above it—is not permitted. The other applicable irregularity is a torsional irregularity, which is avoided in the design.

The redundancy factor, ρ, is permitted to be taken equal to 1.0 as long as each story that resists more than 35% of the base shear complies with the ASCE table in Figure 14 (ASCE section 12.3.4.2a). No shear walls have a height-to-length ratio greater than 1.0 at any story, therefore the only requirement to meet is for the moment frames. To account for this, the design of the new lateral system does not allow for any extreme torsional irregularities; it also does not allow for over 33% loss in story strength after moment resistance loss in connections of a single beam.

Table 12.3-3 Requirements for Each Story Resisting More than 35% of the Base Shear

Lateral Force-Resisting Element	Requirement
Braced frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Shear walls or wall piers with a height-to-length ratio greater than 1.0	Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b). The shear wall and wall pier height-to-length ratios are determined as shown in Figure 12.3-2.
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Other	No requirements

Figure 14|ASCE Table 12.3-3 Redundancy Factor Requirements

Model Setup and Assumptions

Due to its better interface for automated load generation, ETABS 2013 was used to construct a new model rather than using the SAP model created as part of Tech 4 in the fall semester. Using this software provided the ability to capture the seismic behavior using a Modal Response Spectrum Analysis.

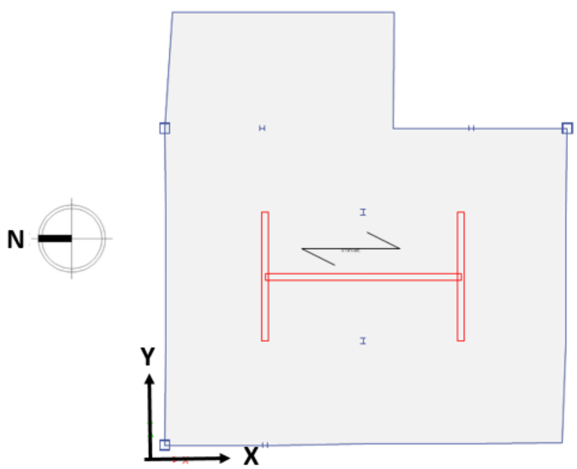


Figure 15|Plan View of Orientation of ETABS Model

Figure 15 shows the orientation of the model—the x-axis corresponds to project North/South and the y-axis to project East-West.

Stiffness modifiers of 0.35*f22 and 0.35*f11 membrane axes are used per the recommendation of ACI 318-11 for cracked shear walls using elastic second-order analysis.

Additionally, the following assumptions are applied:

- Shear Walls modeled as thin shells
- Fixed bases
- Shear wall f'c = 6000
- Seismic weight determined from model self-weight plus superimposed dead load

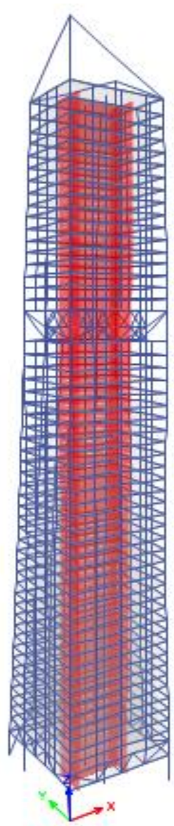


Figure 16|ETABS 3D Model of 181 Fremont

A complete 3-D model of the lateral system—including diaphragms and select gravity members needed to obtain correct model behavior—was assembled as shown in Figure 16.

Modal Response Spectrum Analysis

Performing the MRSA involved applying accelerations in each orthogonal direction with x-a scale factor of Ig/R, or 55.2. The base shear that then results is less than 85% that of the base shear determined using the Equivalent Lateral Force Procedure, and must then be scaled in each direction. An example of the scaling factor calculations for one iteration performed is shown in Equation 2.

$$\text{Scale Factor} = 0.85*(Ig/R)*(V_{ELF}/V_{MRSA})$$

$$\text{x-dir: } 0.85*(1.0*386.4/7)*(3153.554/1359.513) = 108.84$$

$$\text{y-dir: } 0.85*(1.0*386.4/7)*(3153.554/842.187) = 175.69$$

Equation 2|MRSA Scale Factors

The MRSA requires enough modes be defined in order to obtain at least 90% building mass participation in each orthogonal direction; defining 35 modes achieves a mass participation of 91.5% in the x-direction and 96% in the y-direction. Furthermore, an eccentricity of 5% is accounted for in the response spectrum load case, and P-delta effects are considered by specifying their inclusion in "Modal Case" under the Define tab.

The modal combination method used is the square root of the sum of the squares (SRSS), due to it's applicability when periods differ by more than 10%; this is opposed to the complete quadratic combination (CQC) which is best used when periods are closely spaced and there is cross-correlation in mode shapes.

Additionally, other factors and assumptions required for seismic analysis are listed in Figure 17.

R = 7	Ss = 1.5	Site Class D	Fv = 1.5
I_e = 1	S1 = 0.6	Seismic Design Category D	SDS = 1
C_d = 5.5	T_L = 12s	Fa = 1	SD1 = 0.6

Figure 17 | Seismic Analysis Assumptions

Seismic Loading

After final design iteration, the seismic base shear was found to be 2463 kips in the x-direction and 2216 kips in the y-direction. The allowable seismic drift, from ASCE Table 12.12-1, is 20% of the story height—a total of 14 feet. In accordance with section 12.9.2, actual displacement and drift quantities must be multiplied by C_d/I for comparison with the drift limit. This drift limit is satisfied after designing for seismic forces alone in both the x and y-directions. Figure 18 shows the story displacements for each axes—after amplification, displacement in the East-to-West direction is just under the maximum limit.

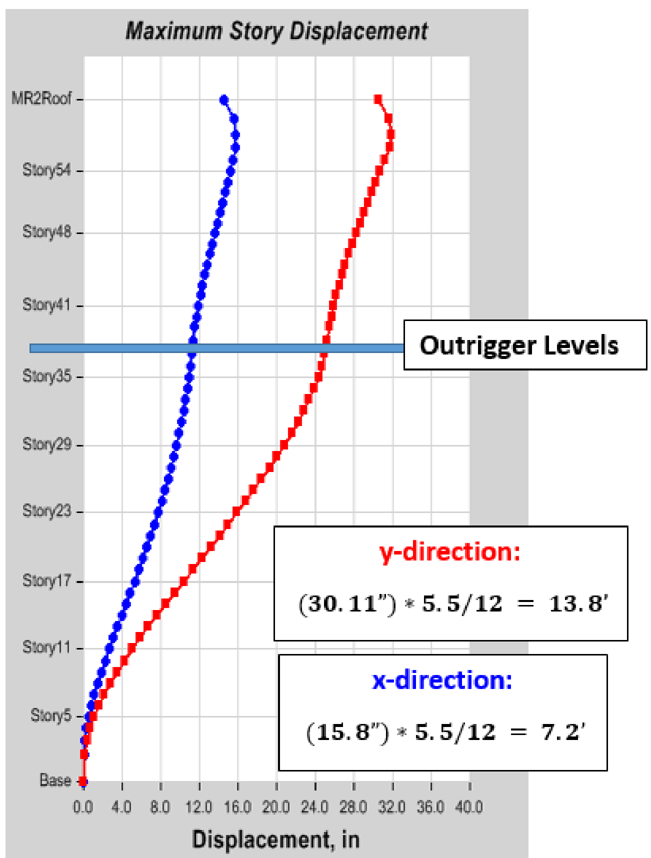


Figure 18 | Maximum Seismic Story Displacements

The displacement curves demonstrate the interplay between moment framing,

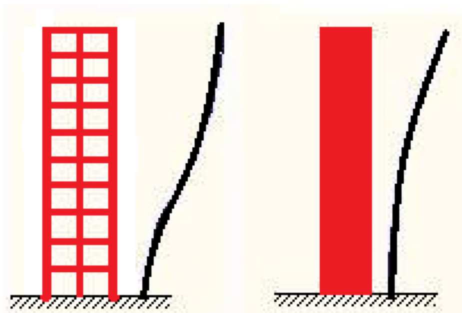


Figure 19|Moment Frame Versus Shear Wall Deformation

shear walls, and outriggers. As shown in Figure 19, moment frames and shear walls differ in the way they deform; In the x-direction, the displacement curve more closely resembles the shape of a deformed shear wall, whereas the y-direction curve resembles moment-frame behavior. Both curves, however, demonstrate reduction in displacements due to outriggers—seen by their reduction in slope where the outriggers are placed.

The story shears plotted in Figure 20 demonstrate an apparent irregularity at the outrigger floors. This reduction in shear is a result of loss of lateral stiffness at the exterior truss on the 37th floor. No further action need be taken to address this, however, as the behavior of this horizontal irregularity type is taken into account by performing the MRSA.

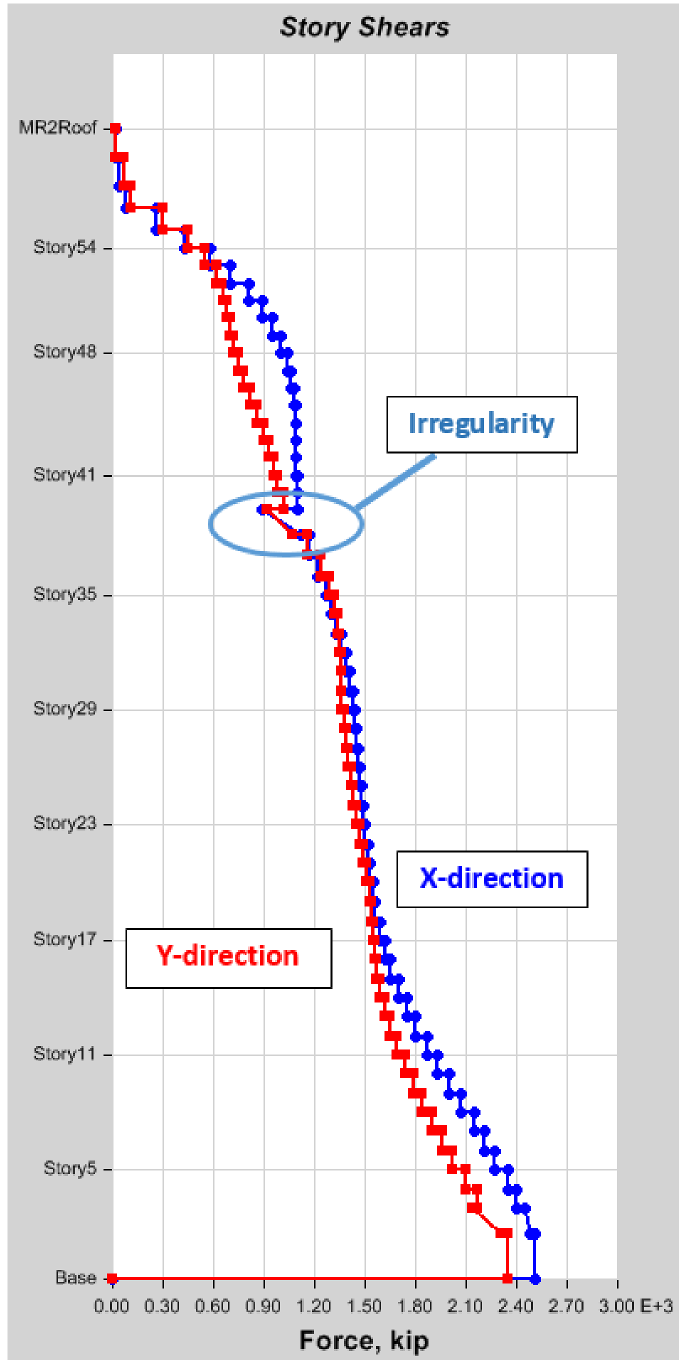


Figure 20|Story Shears

Center of Mass and Minimization of Torsion

The tower's floor plan changes in geometry as it rises in a complex way, as can be seen in the floor plans in Appendix A: Typical Floor Plans. To minimize ill-effects due to torsion, lateral system layouts that were relatively symmetric were analyzed. Adding moment frames at the extrusion on the East Façade helped to maintain a small eccentricity. Detail into the layouts explored will be further discussed in the following sections.

The torsional period is captured in the third mode and has a value of almost 5 seconds. Figure 21 and Figure 22 show the undeformed shape compared to the deflected shape under this mode for each elevation, with the undeformed shape on the left and deformed shape on the right.

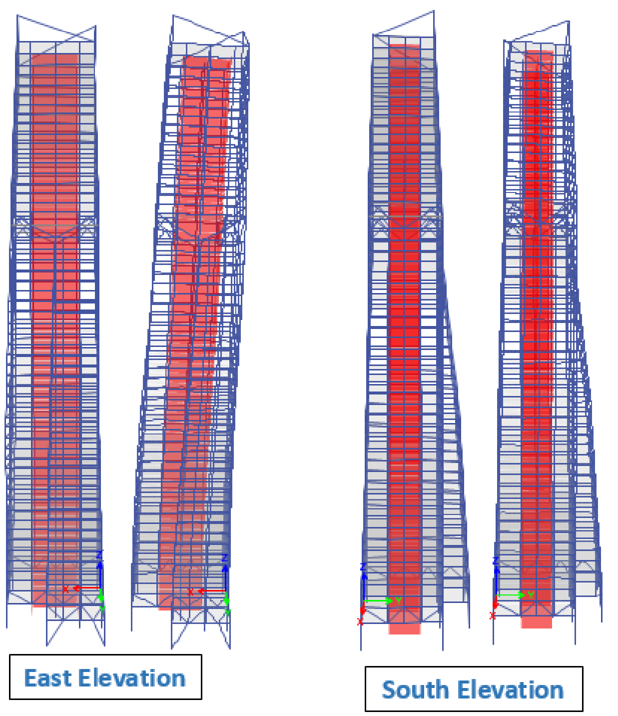


Figure 21|East and South Elevation Under Torsional Deformation

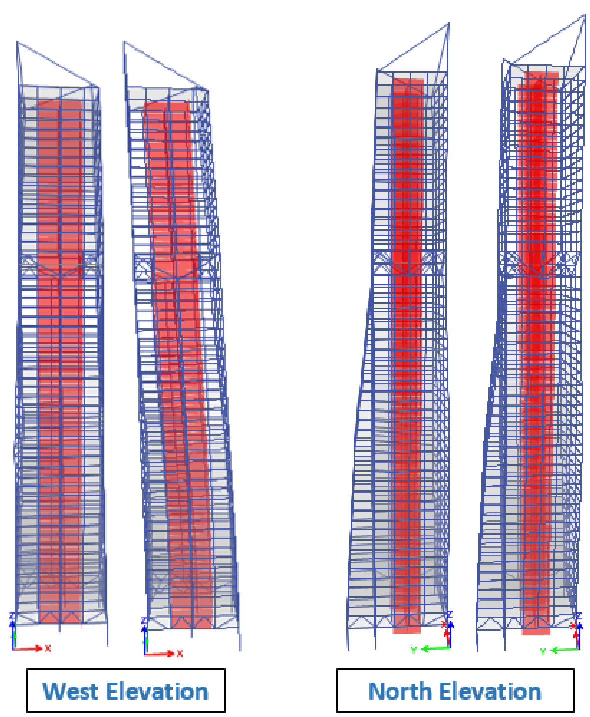


Figure 22|West and North Elevation Under Torsional Deformation

Lateral Design

As is done in the existing system, moment frames are provided at the exterior framing up until Level 37. Once they reach this floor, they are discontinued, as their impact on shear resistance above the outriggers is minimal. Because the same locations for moment frames is kept, their analysis began with using the same sections present in the existing design. From there, the process that followed was selection and design of shear walls, addition of outriggers, and iteration for the most efficient system.

Special Moment Frames

In analyzing the acceptability of the steel moment frames, classification as a seismically compact section in accordance with AISC 341-10 Seismic Provisions was required. Framing iterations were therefore done using Table 1-3 of the provisions, which predetermines what W-shapes are considered seismically compact for certain uses.

ASCE 7-10 requires the moment frames be capable of resisting 25% of prescribed seismic forces; 616^k in the x-direction and 554^k in the y-direction. Conformance with this parameter is demonstrated through determining the total seismic forces resisted by the shear walls in each direction. Summing up the reactions at the base of the shear wall found 1490^k of base shear resistance in the x-direction—about 60% of the total base shear. Moment frames, therefore, are capable of providing at least 40% of the base shear resistance.

The existing member sizes for moment frames were kept through design of the shear wall and outriggers. Before the addition of outriggers, drift limits in the Y-direction were not able to be met. After outriggers were added, however, drift was satisfied and the moment frame members experienced less stress overall.

Design of Shear Walls

Shear wall design first took into account the optimal plan locations. Early stages of development included diagrams of existing locations where shear walls may easily be placed and considered the how the play layout would be affected at higher stories, as shown in Figure 23.

Upon further iterations with the lateral system, however, it was determined that a feasible shear wall solution with no impact on the architectural layout is not practical. The final layout shown in Figure 25 and Figure 26 does, however, provide sufficient stiffness to the lateral system while minimizing the amount of openings and architectural modifications that need to be made.

Shear walls A and C are each 37.5’ long and 24” thick. This thickness was not in order to achieve sufficient strength, but rather to meet drift limits as mentioned earlier. Drift in the x-direction is less critical, which is why shear wall B is 18” thick. A practical, minimally intrusive solution to having two shear walls spanning the North-South direction could not be found. As a result, shear wall B is almost 57’ long.

Detailing of the shear walls is done in accordance with ACI 318-11 Section 21.9 for Special Reinforced Shear Walls. The critical section of the wall, which occurs at the outrigger levels is used to determine reinforcing. Outriggers use coupling action to reduce overturning moments, but this may come at the cost of increased shear in the core, as it did in this case (Figure 24). For detailed reinforcing output, see Appendix D: Shear Wall Detailing.

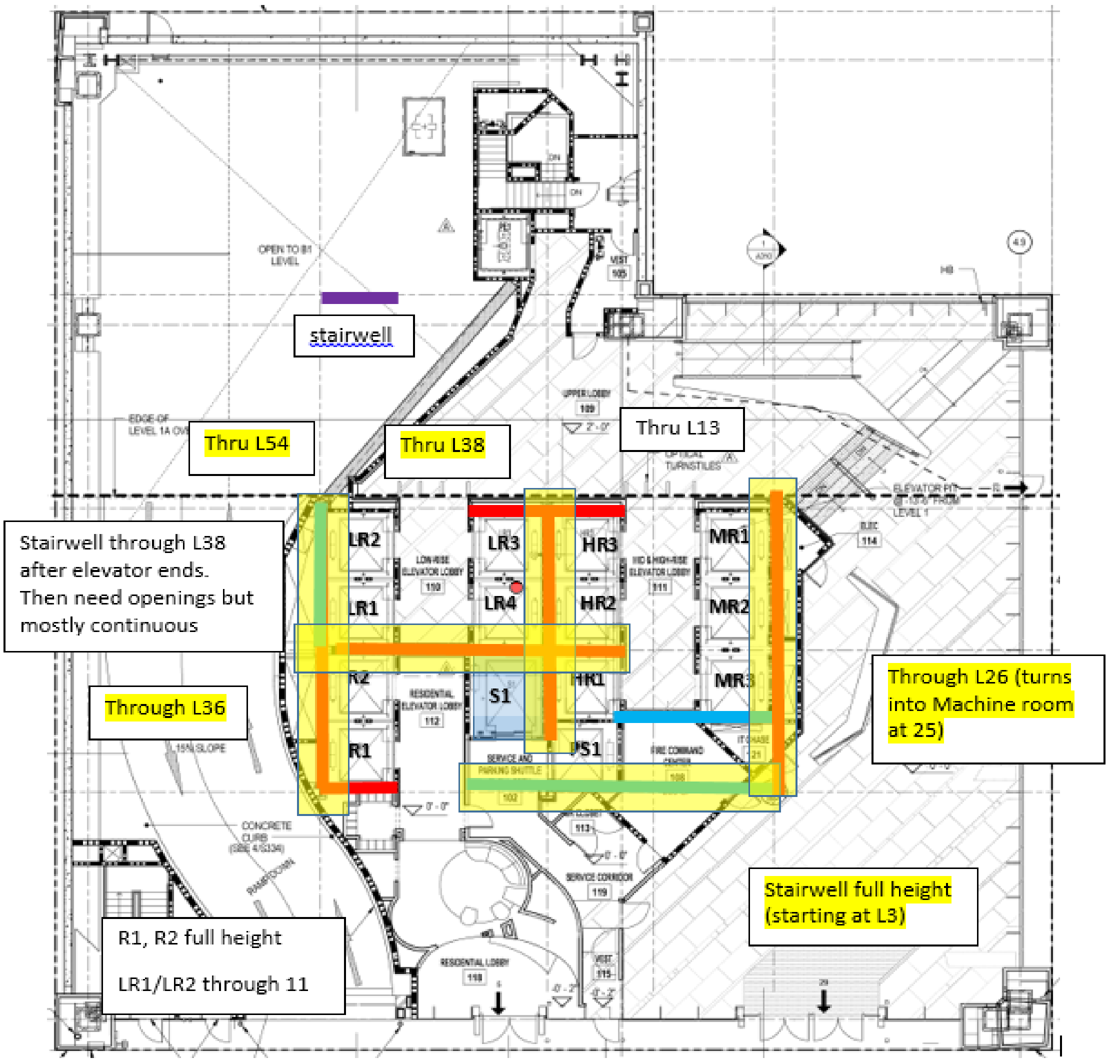


Figure 23|Shear Wall Placement Analysis (Adapted from Heller Manus)

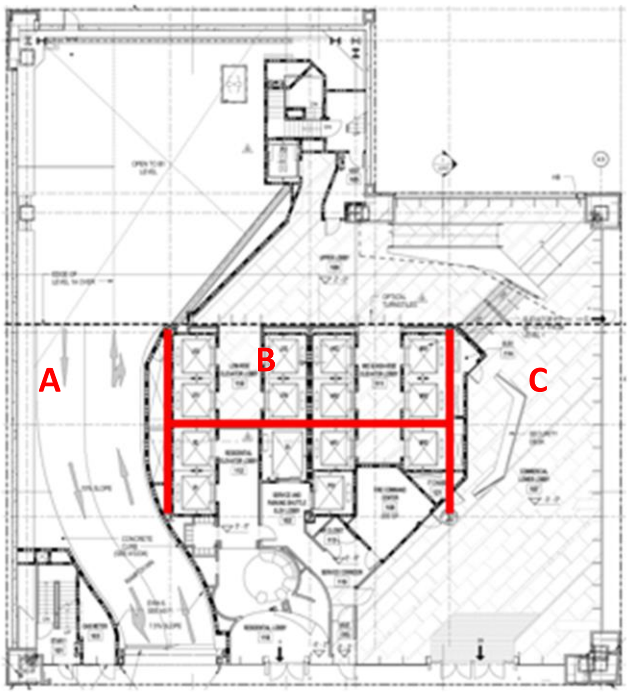


Figure 25|Final Shear Wall Layout (Adapted from Heller Manus)

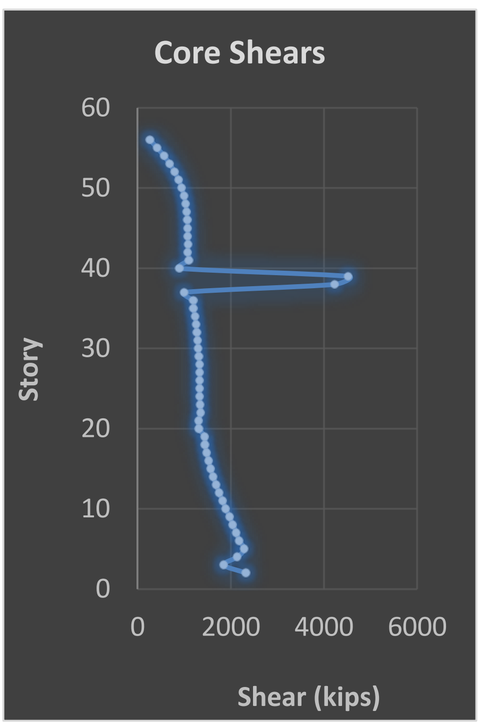


Figure 24|Shear at North-South Wall

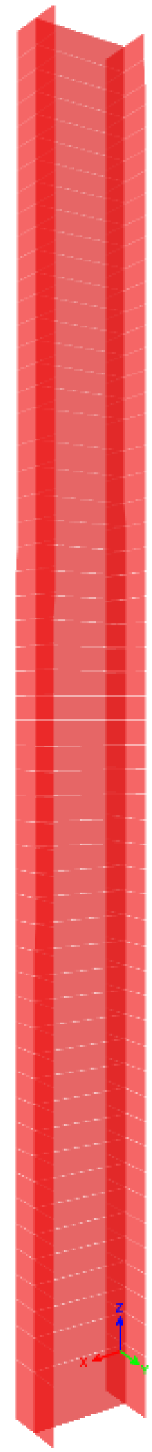


Figure 26|Shear Wall Isometric

Design of Outriggers

The addition of outriggers to the new lateral system has two main benefits: it reduces the overturning moment roughly 80,000 foot-kips in the x-direction, and it allows ASCE seismic drift limits to be met without greatly having to increase the shear wall thicknesses. Typically, outriggers are most beneficial placed where the response under lateral loading differs the most between component systems. They usually perform best at about halfway up the structure as well. Outriggers in the new design were placed between levels 37 and 39, shown in Figure 27, because it is already architecturally feasible and is at a low enough level to still be useful.

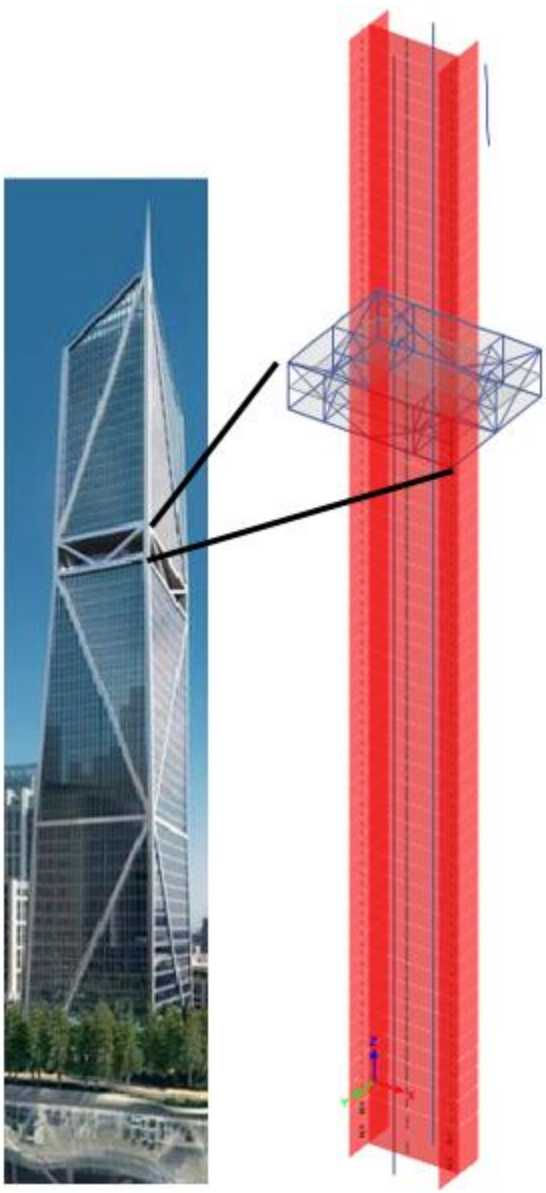
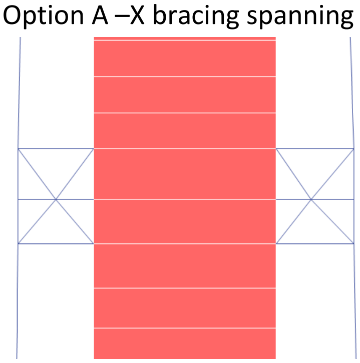


Figure 27|Outrigger Location (Adapted from Heller Manus)

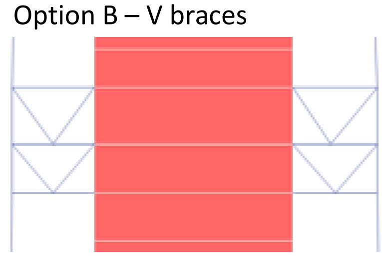
Connecting the outriggers straight into the megacolumns would allow for direct load transfer into the foundations, but would be difficult to construct and would be more harmful to the architectural plan. Utilizing a system more like the belt truss engages the perimeter columns other than megacolumns. Additionally, it makes use of the existing exterior truss, and thereby modifies the architecture less.

Several types of steel truss outriggers were considered as outlined below:

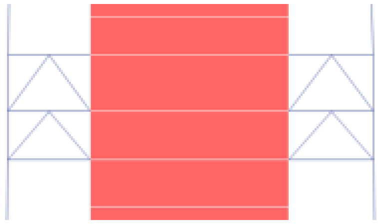
- Option A –X bracing spanning two floors



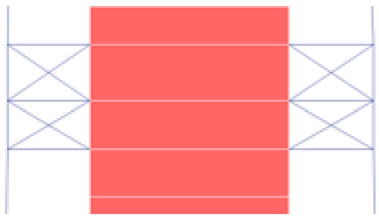
- Option B – V braces



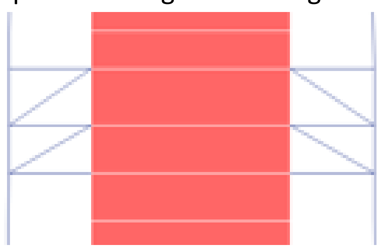
- Option C – Inverted V braces (Chevron)



- Option D – X bracing both stories



- Option E – diagonal bracing



Out of these options, the Inverted V brace was chosen. Not only does it provide a more efficient load path than x-bracing, but it also preferred architecturally for it allows for an opening. An additional bonus is that the existing gravity beams may be maintained—something not possible with the V-brace. Option E provided a nice option architecturally, but was not chosen because of the forces it incurred. More diagrams demonstrating each system's performance may be found in Appendix E: Outrigger Comparison Output.

As mentioned earlier, outriggers improve overturning moment. Figure 27 shows the moment reduction in the x-direction for the system before and after addition of outriggers. Figure 28 shows the improvement in story drift also for before and after the addition of outriggers.

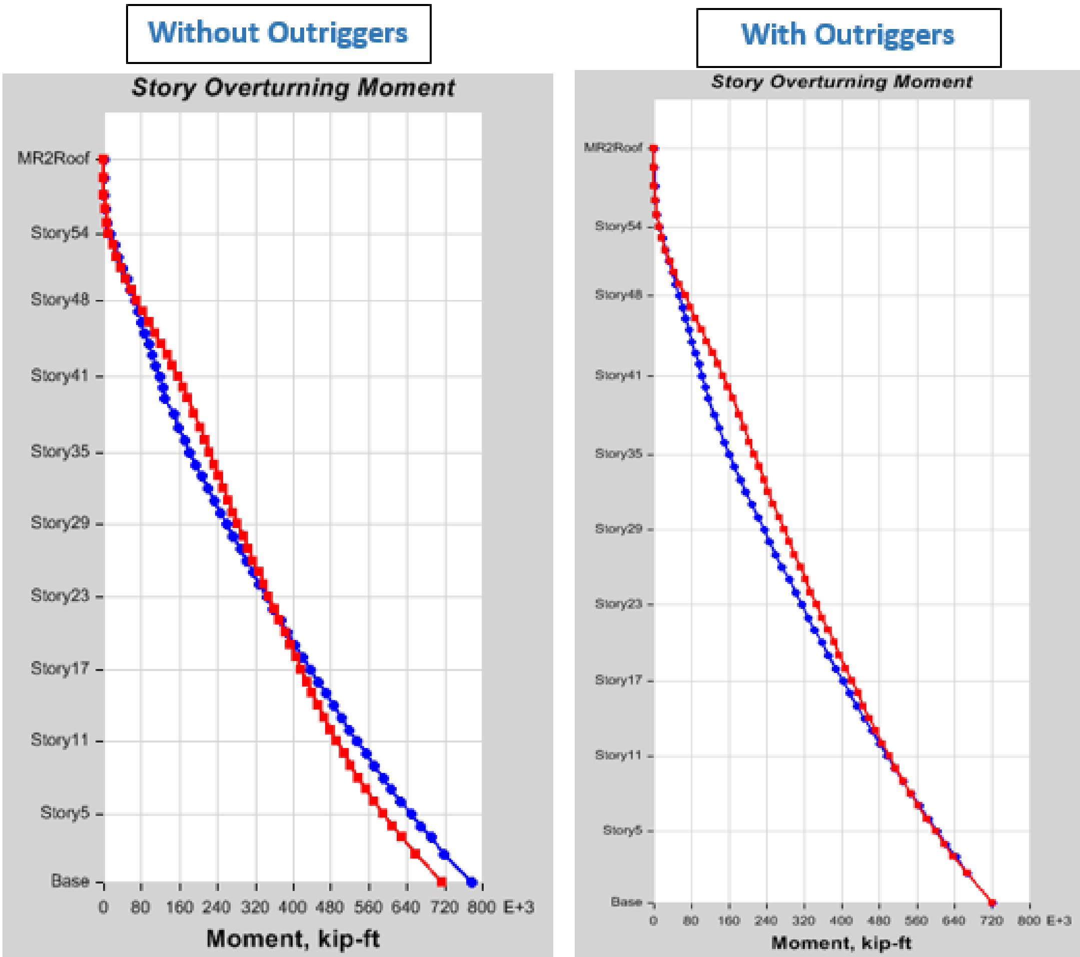


Figure 28|Overturning Moments

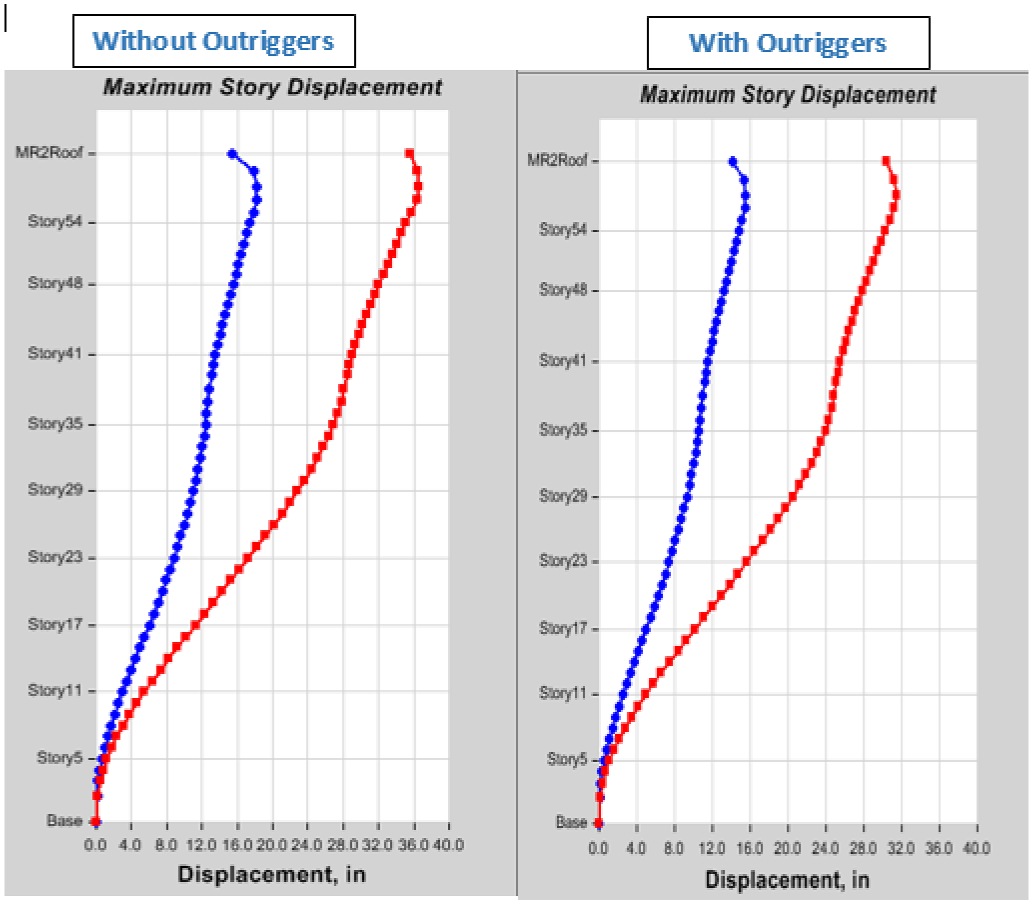


Figure 29|Maximum Story Displacement

Impact on Gravity System

Changes in the gravity system were made to accommodate the new lateral system. This included the replacement of seven core columns with three shear walls for bearing instead, as shown in Figure 12. Besides the elimination of the columns and the beams bracing between them, no modification of floor gravity framing was made, as this did not affect the load carrying ability or placement of the existing framing.

The other gravity systems that were affected, however, were the transfer truss at level two and the truss at level 37. At Level 2, bracing was simply added to make up for the megabrace removal. At level 37, however, vertical members were added in order to provide something for the outriggers to frame into. Although none of the existing sizes needed to be changed as a result, it did affect the aesthetic of the structure, as shown in Figure 30.

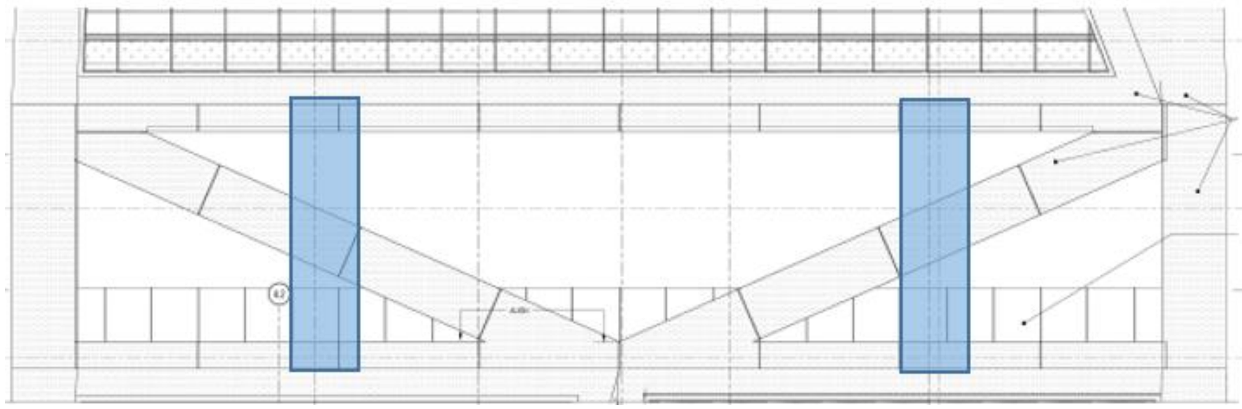


Figure 30|Added Columns At Level 37 Truss (Adapted from Heller Manus)

Comparison With Existing System

Existing System Performance

The existing system uses a high-performance megaframe that not only ensures occupant comfort beyond the standard, it is also expected to require little to no repair after an extreme earthquake event. This is the primary benefit to the existing system. Arup's REDi Gold objectives result in a structure with better overall lateral performance.

This high-performance is not achieved easily, however. This approach not only comes with great expense, it also causes scheduling delay and other constructability issues as described in the next section.

New System Performance

Although not designed for criteria beyond that of the code, the new lateral system proves possible through prescriptive means. It is also much more affordable. As shown in Figure 31, the estimated cost of the shear walls comes just under $7 million, whereas–as outlined in the next section—significant expenses arise in the construction of the megaframe.

Item	Amount	Unit	Material Unit Price	Labor Unit Price	Total Cost	Duration	Rounded	Crew
Formwork	195580	SFCA	$ 0.88	$ 13.30	$ 2,773,324.40	201.6289	202	C-2
Concrete	11122.22	CY	$ 139.00	$ 197.65	$ 3,744,296.11	216.9766	217	
Rebar- #8's	242.97	ton	$ 970.00	$ 560.00	$ 371,744.10	80.99	81	4 Rodmen

Total Cost	$ 6,889,364.61

Figure 31|Cost Estimate of Shear Walls

Breadth One: Construction Breadth

The purpose of the construction breadth was to determine how the use of the mega-frame impacted both the cost of the façade and its constructability. By finding this information, the cost savings that result from removing the exterior bracing, as well as the elimination of certain constructability issues, were able to be determined.

Façade Cost Estimate

Above the existing transfer truss on levels 37 through 39, each elevations' exterior mega-bracing is comprised of W14x342's. Below the truss, the megabracing system consists of a primary buckling restrained brace that is flanked by two secondary buckling restrained braces on each side (Figure 32). These braces are restrained to the structural framing on almost every floor, significantly increasing the cost of labor due to added connections and specialty detailing (Figure 33). Further adding to the cost is a metal cladding system that runs the length of the exterior bracing and columns.

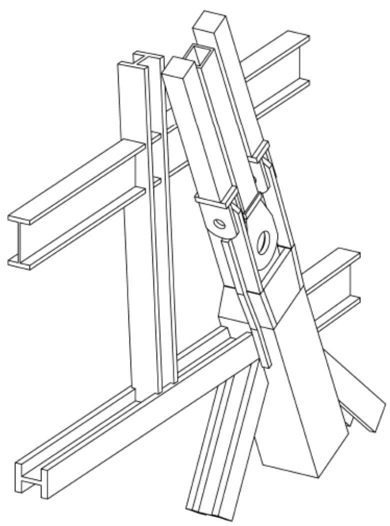


Figure 32 | Isometric of Megabrace Below Level 37 (Courtesy of Arup)

In estimating the façade's total, the cost of the curtainwall glazing and framing, cladding of the megaframe, the megabraces, and special connections between the bracing and structure are considered. The total façade cost comes to about eight percent of the total building cost and is estimated at $29,871,469 – about $2.2 million of which is from the mega-cladding system used on the megaframe, and $2.7 million from the special connections.

Without the mega braces, significant cost savings arise from the elimination of cladding and connection expenses. Excluding the cladding, extra connections, and bracing members results in a façade cost of just over $21 million, reduced by over $8.5 million from the original enclosure cost.

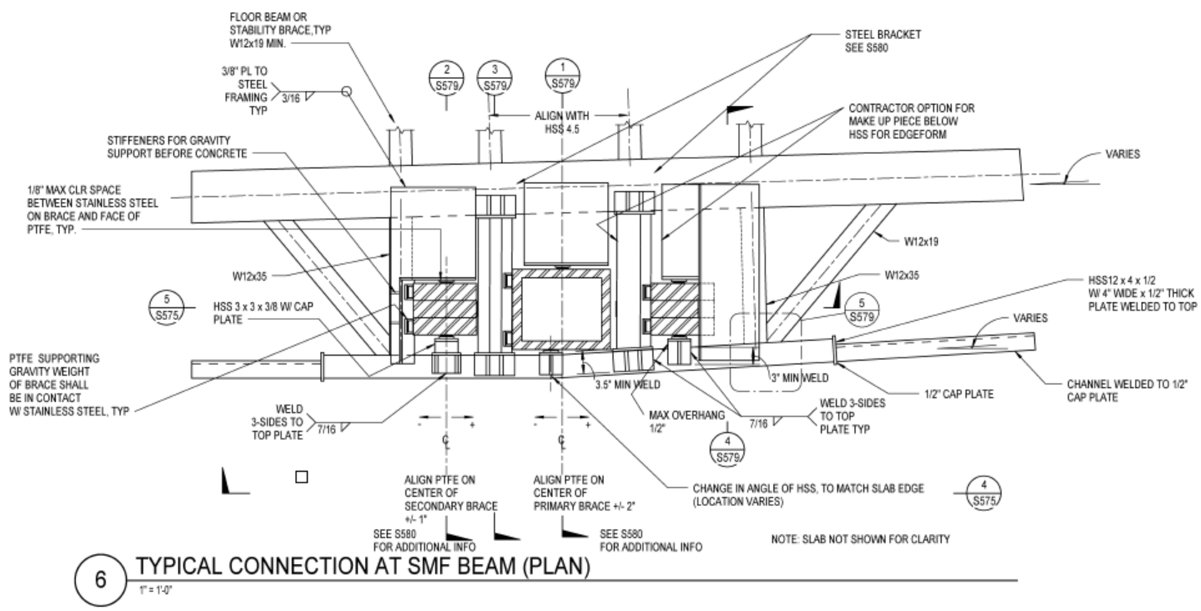


Figure 33 | Typical Megabrace Connection to Structure (Courtesy of Arup)

Enclosure Constructability

Not only does the utilization of a megabracing system come with extra cost, but it also poses challenges in the construction process. Typically steel framing is faster to build than concrete, but with the large tube sections used and the complicated connections that must be completed at each floor, the erection time is more comparable between the two systems.

Additionally, to save time on the shear wall construction, an hydraulic form system could be utilized. This is also beneficial for quality control.

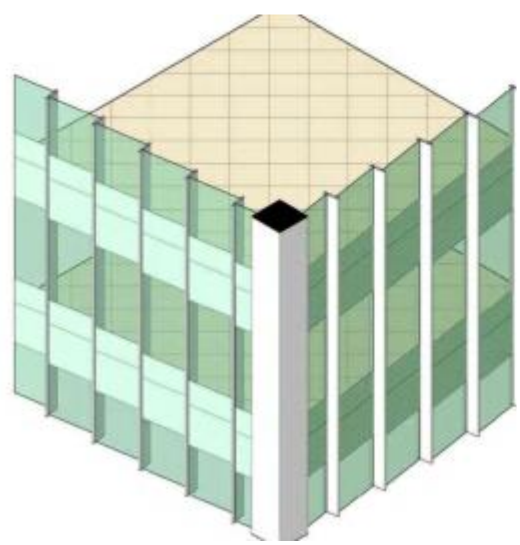


Figure 34 | "Saw-Tooth" Curtain Wall Concept (Courtesy of Heller Manus)

Breadth Two: Façade Study

The façade of 181 Fremont employs a unitized curtain wall in which a "saw-tooth" layout, as seen in Figure 34, is employed. This layout reduces the amount of direct afternoon sun entering by tilting the glass lites horizontally between vertical mullions, thereby increasing mullion surface area exposed to direct sunlight and reducing glass surface area exposed.   Each vertical mullion extrudes approximately 7.5" past the glazing it shades. In order to quantify the system's benefit, as well as the facades performance, an analysis of the thermal performance and sunlight path was performed, as detailed in the next section.

Sun Path

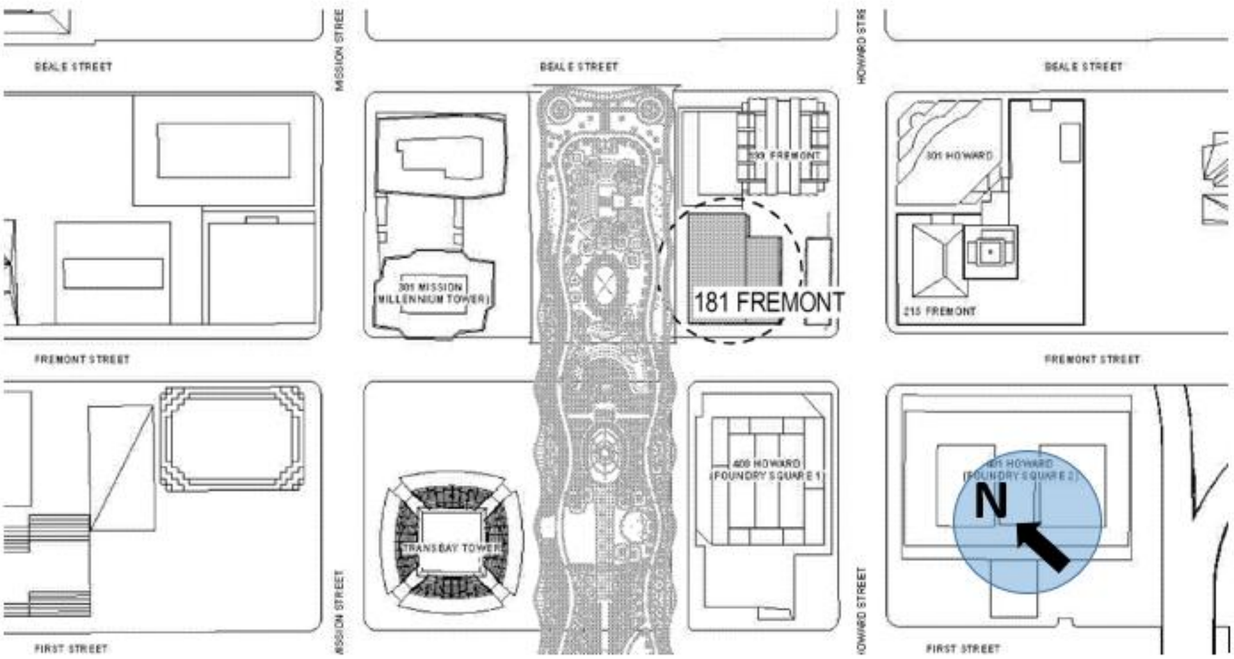


Figure 35 | Site Orientation of 181 Fremont (Courtesy of Heller Manus)

As shown in Figure 35 the project site is orientated with the street-facing elevation at a 135 degree angle from due South. The location of the sun in relation to the building at 9:00 a.m., 10:30 a.m., 12:00 p.m., 1:30 p.m., 3:00 p.m., and 5:00 p.m. is displayed for both winter (December 22) and summer (June 21) in Figure 38 and Figure 37. The tilted panel design is intended to improve cooling demands in summer afternoons. In the summer, after 12 p.m. is when the shading begins to take effect on the southeast elevation. By 3 p.m., however, the vertical mullions cease to provide any significant shading. In the winter, when it would be beneficial to allow more sun in, a portion of direct sunlight is instead blocked for most of the day.

Due to the hilly terrain and location by the bay, San Franciscos temperatures do not very greatly throughout the year. The average high temperature in January is 57 degrees farenheit and the average low is 46 degrees, while the average high temperature in June is 66 degrees and average low is 53 degrees. The annual heating degree days is around 3000, while the annual cooling degree days is under 200. Furthermore, a significant portion of the heating degree days occur during the summer. Because of San Francisco's unique climate, more benefit would come from allowing direct sunlight into the building.

Options for Improved Performance

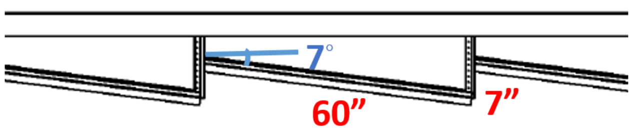


Figure 36|Inclination of Curtainwall (Courtesy of Heller Manus)

The mullions extend out 7" and the glass panes are 60" in width, creating glass lites that extrude out almost 7 degrees from the horizontal plane of each elevation, as shown in Figure 36|Inclination of Curtainwall. This is especially effective in blocking out the sun when it is at an azimuth between -7 and 7 degrees in the summer. This only occurs for a short amount of time during the day, but nonetheless there is some shading afforded by the vertical mullions which reduces cooling demands compared to a flat curtainwall system.

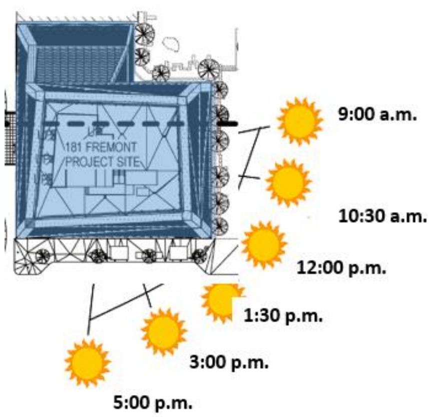


Figure 37 | Winter Azimuth Angles (Adapted from Heller Manus)

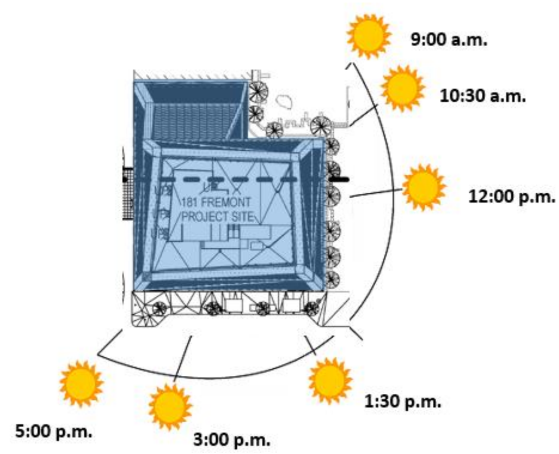


Figure 38|Summer Azimuth Angles (Adapted from Heller Manus)

To increase the amount of sun that enters the building year round, the inclination of the glass units could alternatively be flipped. Because there is a smaller change in sun azimuth during the winter, this would have a greater effect in increasing daylight levels than it does in decreasing it in the other configuration.

Conclusion

After redesigning the lateral system, it was deemed that both systems have their benefits and drawbacks. The existing system is ideal for exceeding minimum performance requirements and increasing the chance that the building will be quickly re-inhabitable after an extreme earthquake. The alternative system, however, may be taken for an economical solution.

Both systems have their drawbacks in constructability as well. While the concrete shear wall's involve added schedule time due to the need to wait for curing, the megaframe adds time to an otherwise efficient building method of using steel.

The original proposal sought to investigate the purpose behind the performance-based design of 181 Fremont by using a prescriptive approach. In doing so, light was brought to the method's ability to better predict serviceability and failure mechanisms through nonlinear analysis. In doing so, the specific issues are able to be designed for. A prescriptive approach, however, simply provides a means of obtaining a conservative design that may incur moderate structural damage at 2/3 the Maximum Considered Earthquake.

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Appendices

Appendix A: Typical Floor Plans

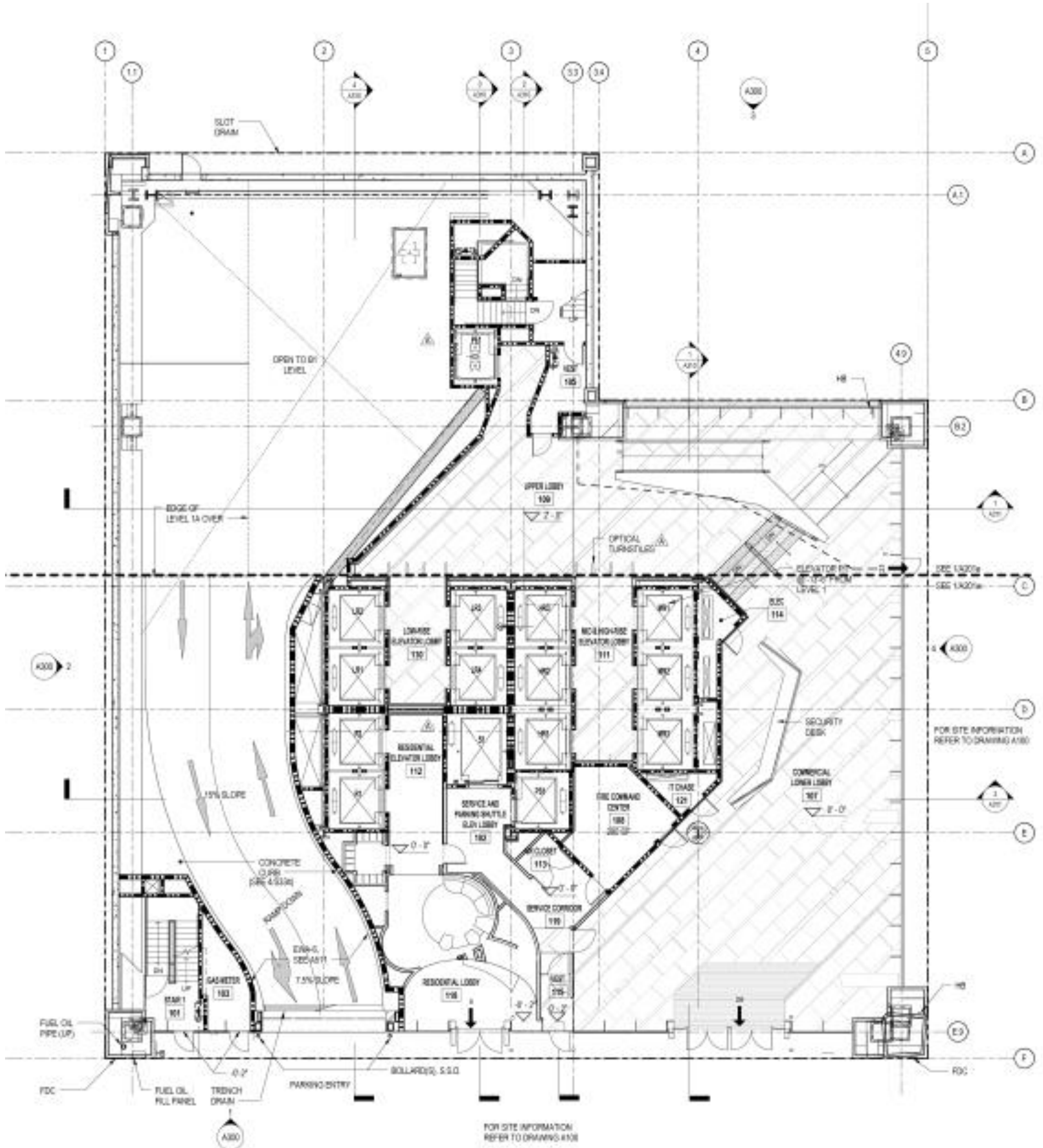


Figure 39|Ground Level (Courtesy of Heller Manus)

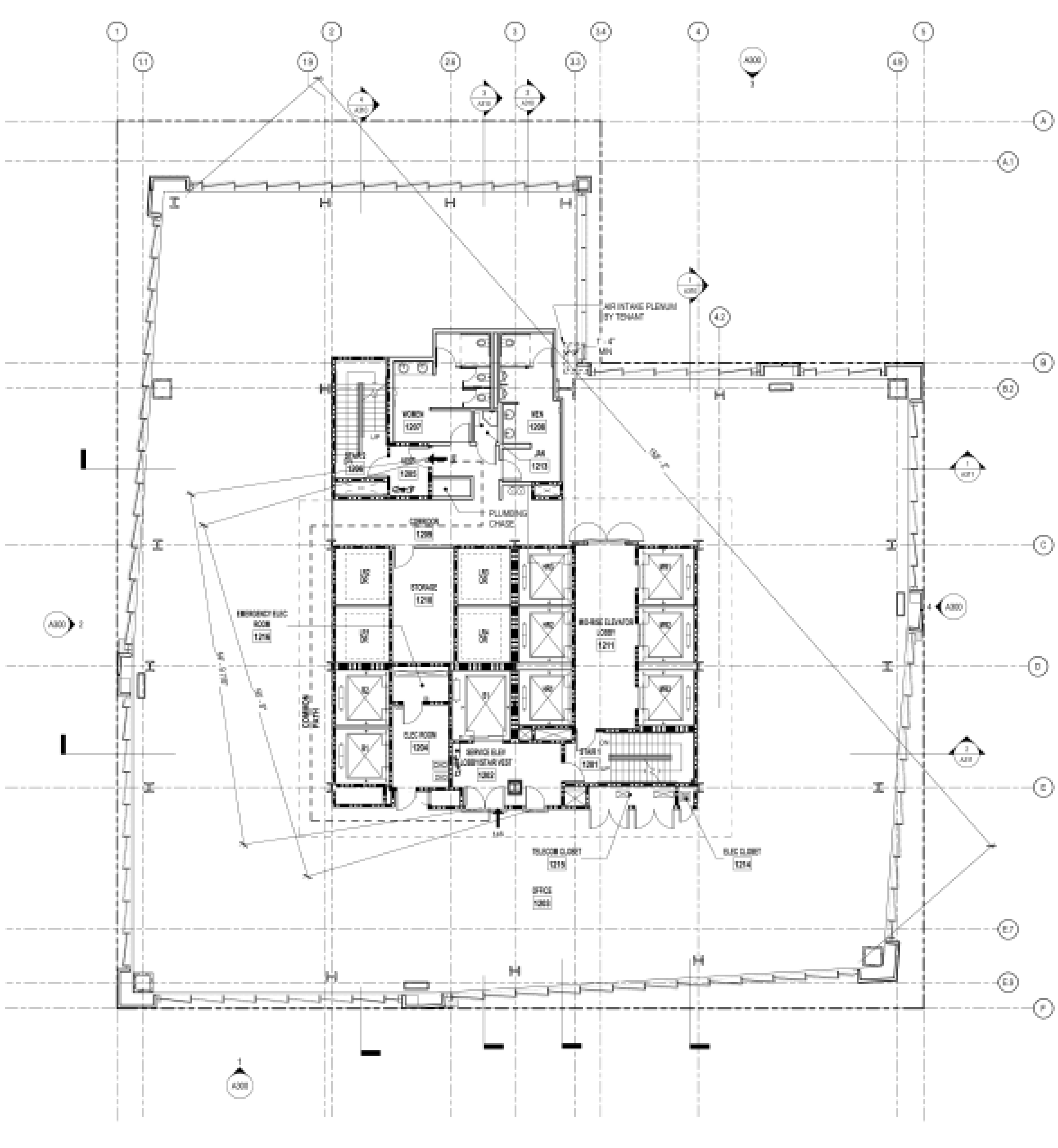


Figure 40|Level 12 (Courtesy of Heller Manus)

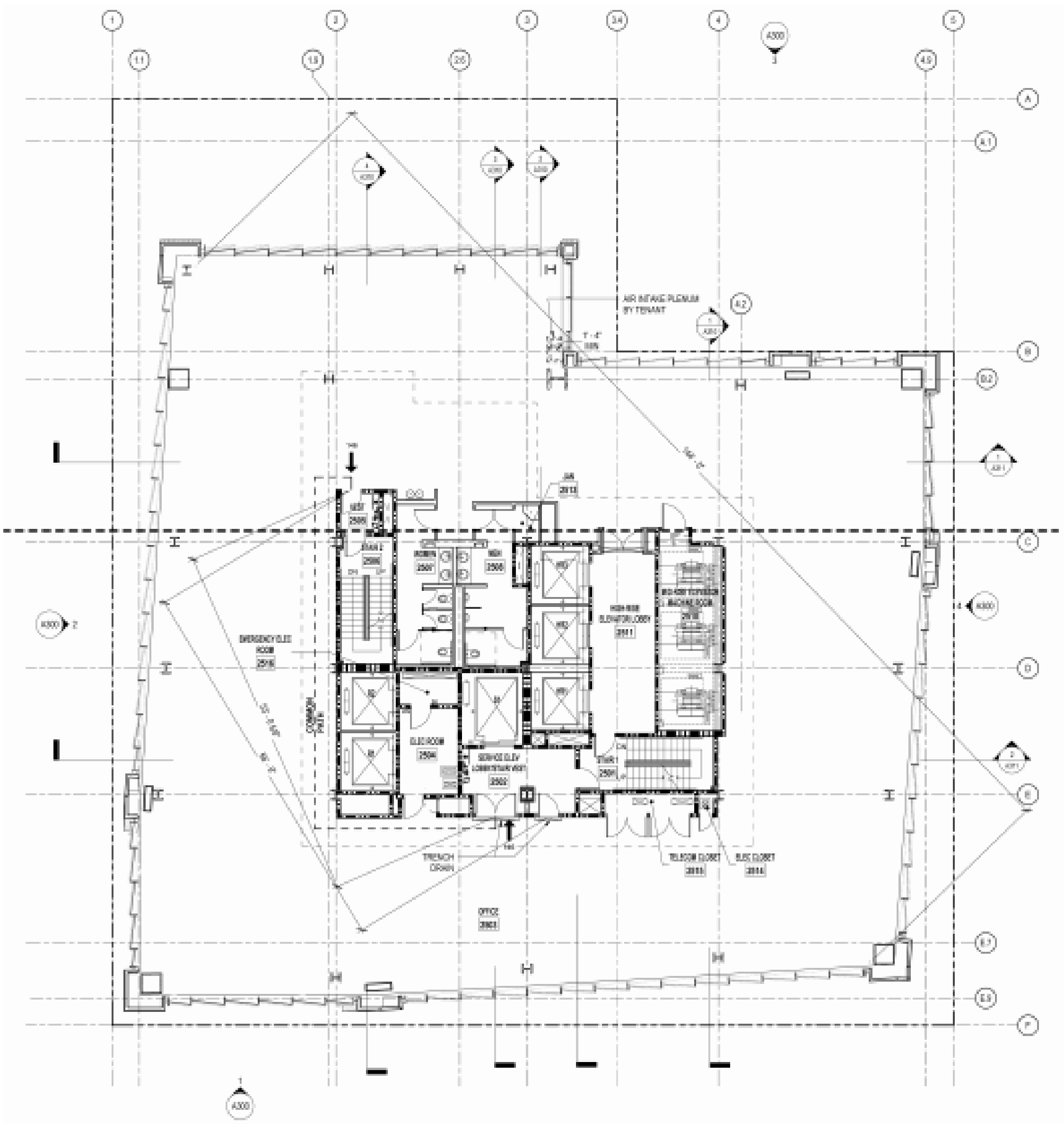


Figure 41|Level 25 (Courtesy of Heller Manus)

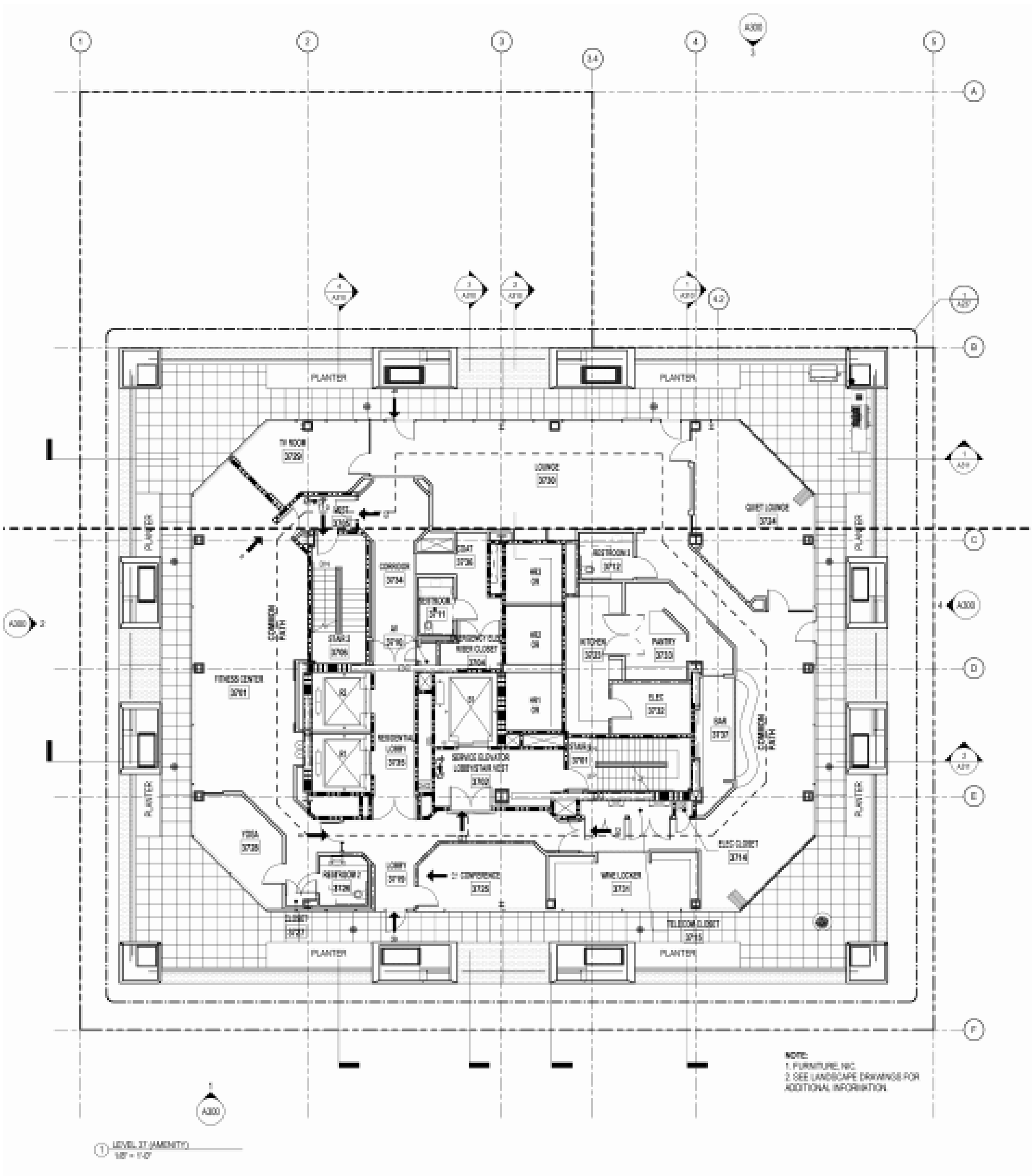


Figure 42|Level 37 (Courtesy of Heller Manus)

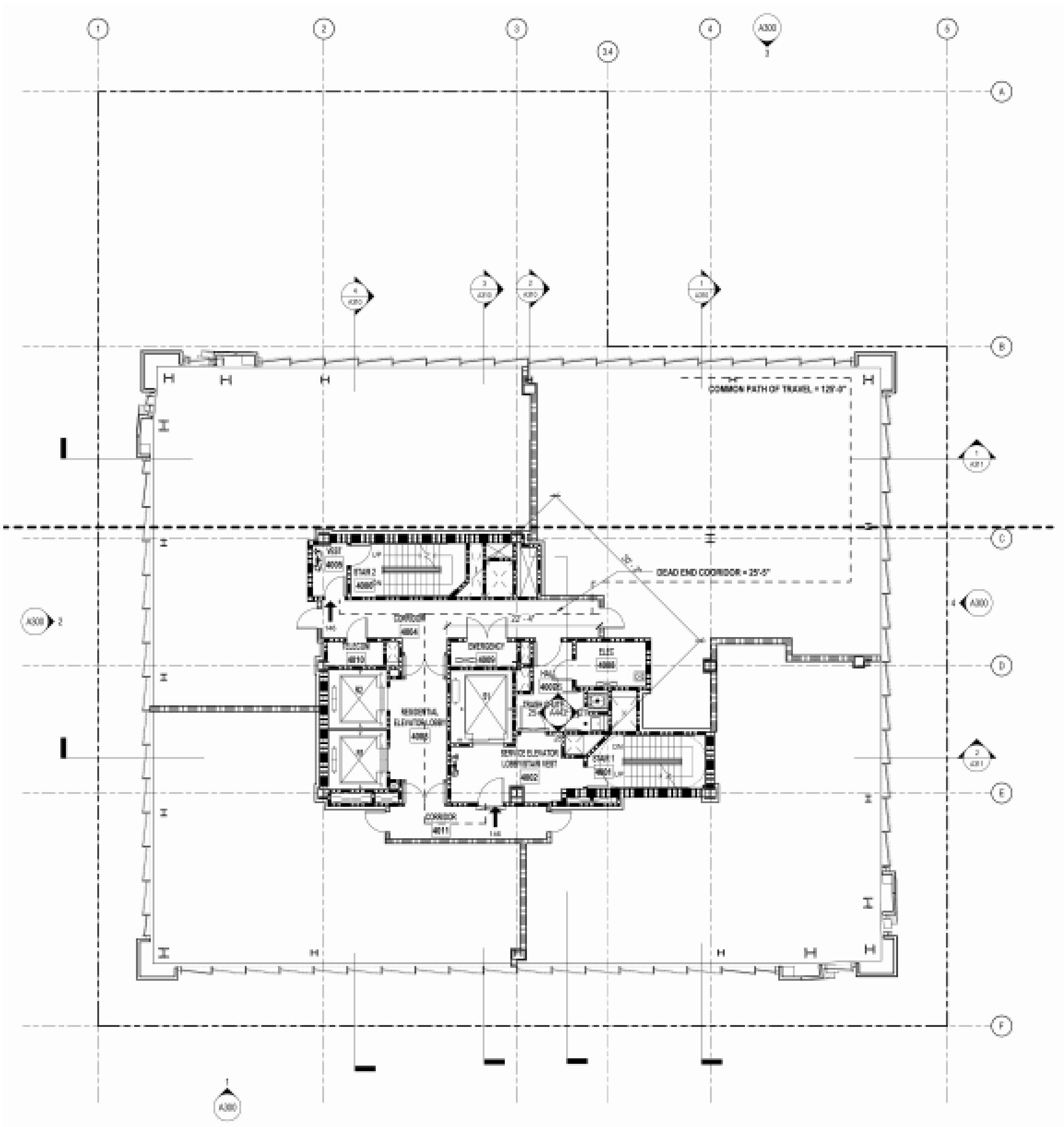


Figure 43|Level 40 (Courtesy of Heller Manus)

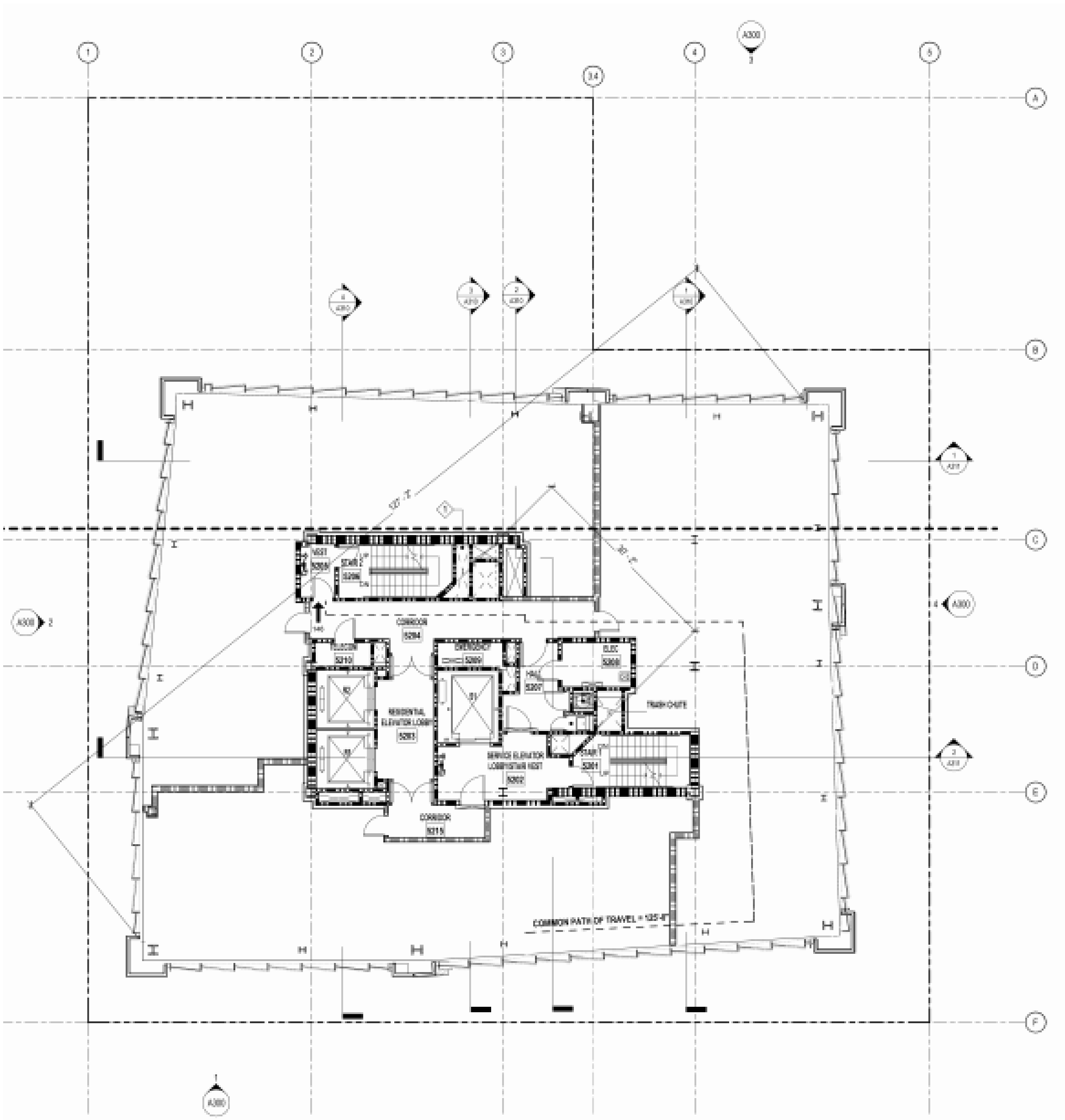


Figure 44|Level 52 (Courtesy of Heller Manus)

Appendix B: Curtainwall Cost Estimate

CSI Division	Item	Pricing Method	Quantity	Material	Installation	Total							Total Cost
B2020 210	Tubular Aluminum Framing												
	thermal break frame	cost/s.f. opening	214563	23.5	15.15	38.65							10175339.2
B2020 220	Curtain Wall Panels												0
1200	1" thick IGU	cost/s.f.	199505	18.5	14.65	33.15							8114875.85
5500	Sandwich Panel	cost/s.f.	45058	13.4	6.35	19.75							1091901.78
08 44	Curtain Wall and Glazed Assemblies												
50	Average, single glazed	H-1	195	0.164	SF	20558	53.5	8		61.5	71.5		1803563.62

Total Curtainwall: 21185680

	Item	Crew	Daily Output	Labor-Hours	Unit	Quantity	Material	Labor	Equipment	Total	Total Incl O&P		Total Cost
Mega-cladding													0
09 22 13	Metal Furring												
0.003	Beams and Columns, 7/8" channels, 12" oc	1 Lath	155	0.052	SF	61954	0.37	2.1		2.47	3.5947		273260.316
07 25	Weather Barriers												0
3000	Building wrap	2 Carp	8000	0.002	SF	61954	0.15	0.09		0.24	0.3502		26621.3488
05 50 13	Column Covers												0
180	24" diameter, aluminum	2Sswk	32	0.5	VLF	5859	61	26.5		87.5	113		812356.209
09 29 10	Gypsum Board Panels												0
3500	On beams, columns, or soffits	2 Carp	675	0.024	SF	61954	0.38	1.11		1.49	2.13		161917.399
07 21 13.13	Foam Board Insulation												0
600	1" thick	1 Carp	680	0.12	SF	61954	0.25	0.55		0.8	1.13		85899.8405
05 41 13.25	Framing, Boxed Headers/Beams												0
2200	Double, 18 ga. X 6" deep	2 carp	220	0.073	LF	61954	5.1	3.33		8.43	10.75		817188.749

Total Mega-Cladding: 2177244

	Item	Crew	Daily Output	Labor-Hours	Unit	Quantity	Material	Labor	Equipment	Total	Total Incl O&P		
Mega-Braces													
05 12 23	Structural Steel for Buildings												
4900	Heavy Sections, Moment connections	E2	7.8	7.179	ton	396	3175	370	194	3739	5002.5		2430674.73
5650	Braces	E2	50	1.12	EA	218	775	58	30	863	1132.75		1077160.29
7450	W14x342	E2	912	0.061	LF	1059	257	3.18	1.66	261.84	664.7		863706.527
Connections													
22 05 48.10	Vibration and Bearing Pads												0
740	Mounts, neoprene	1 Sswk	5	1.6	EA	400	111	78		189	242		118773.6
05 12 23.17	Columns, Structural												0
3300	Structural tubing	E-2	11270	0.005	Lb	240.3333	1.33	0.26	0.13	1.72	4.08		1203.14712
7150	W12x35	E-3	1032	0.054	LF	2400	73	2.81	1.46	77.27	99.475		292933.98
.65 0400	Plates				SF	932.3333	27			27	36.875		42184.0044
05 05 21.90	Welding Steel												0
1800	3 Passes	E-14	30	0.267	LF	3600	1.08	14.55	4.86	20.49	64		282700.8
05 12 23	Structural Steel for Buildings												0
4900	Heavy Sections, Moment connections	E2	7.8	7.179	ton	396	3175	370	194	3739	8700		4227260.4

Mega-Brace Total: 9336597

Overall Total: 32699521.7

Appendix C: Load Combinations

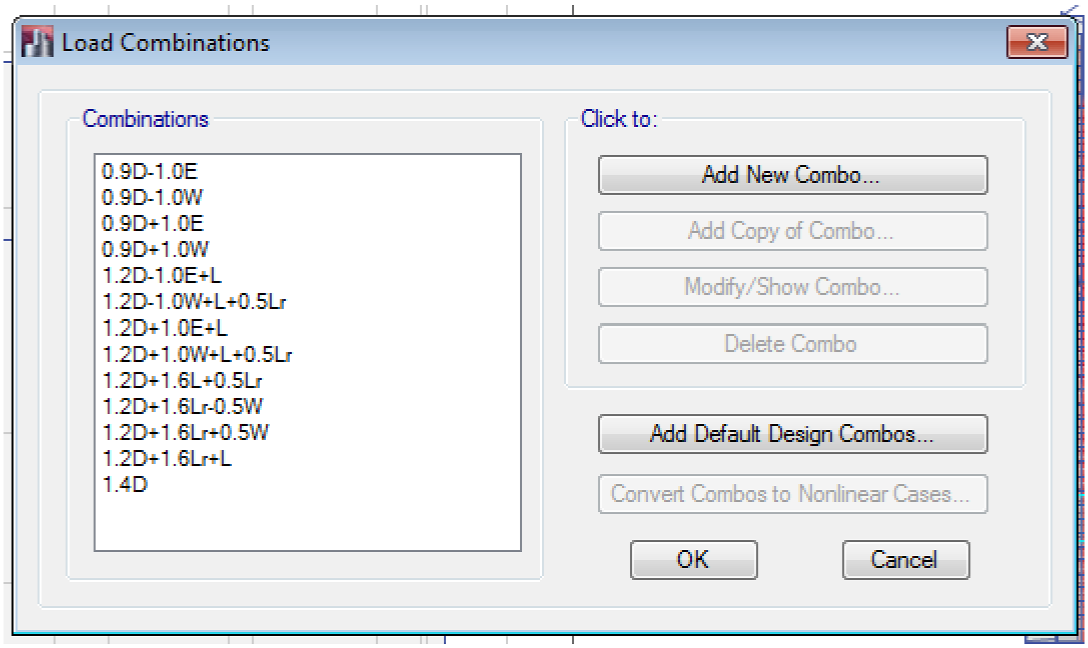


Figure 45|Load Combinations Used

Appendix D: Shear Wall Detailing

ETABS 2013 Shear Wall Design

ACI 318-11 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	LLRF
Story39	Shear Wall 3' $_{Core}$ Y	739.9998	635.7	681.9996	18	0.4

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
4415201	6000	1	60000	60000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in
Top	Leg 1	399	635.7	1080.9996	635.7	681.9996	18
Bottom	Leg 1	399	635.7	1080.9996	635.7	681.9996	18

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (in²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft	Pier A_g in²
Top	30.69	0.0025	0.003	0.9D-1.0E	2070.659	-738.4833	-55530.4469	12275.9934
Bottom	30.69	0.0025	0.003	0.9D-1.0E	2070.659	-423.9505	-28906.1966	12275.9934

Shear Design

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	ΦV_c kip	ΦV_n kip
Top	Leg 1	1.6029	0.9D-1.0E	2031.515	54429.3108	4373.63	1749.998	4373.63
Bottom	Leg 1	1.6029	0.9D-1.0E	2031.515	25483.4691	4373.63	1749.998	4373.63

Boundary Element Check

Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²	C Depth in	C Limit in
Top–Left	Leg 1	0	1.2D+1.0E+L	4096.782	-55843.4546	813.97	1200	Not Required	Not Required
Top–Right	Leg 1	0	1.2D+1.0E+L	4096.782	54116.303	799.12	1200	Not Required	Not Required
Bottom–Left	Leg 1	0	1.2D+1.0E+L	4096.782	-31030.3537	600.58	1200	Not Required	Not Required
Bottom–Right	Leg 1	0	1.2D+1.0E+L	4096.782	23359.3121	534.61	1200	Not Required	Not Required

Figure 46|Shear Wall B Detailing

ETABS 2013 Shear Wall Design

ACI 318-11 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	LLRF
Story38	Shear Wall $3'_N$	399	635.7	450	24	0.4

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
4415201	6000	1	60000	60000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in
Top	Leg 1	399	410.7	399	860.7	450	24
Bottom	Leg 1	399	410.7	399	860.7	450	24

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (in²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft	Pier A_g in²
Top	41.8075	0.0039	0.0023	0.9D-1.0E	42.039	269.9237	-33445.1534	10800
Bottom	71.7755	0.0066	0.0023	0.9D-1.0E	42.039	-795.3673	59672.8794	10800

Shear Design

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	ΦV_c kip	ΦV_n kip
Top	Leg 1	1.9559	1.2D-1.0E+L	1999.131	30013.9738	3677.386	1565.014	3677.386
Bottom	Leg 1	1.9559	1.2D-1.0E+L	1999.131	63831.5334	3677.386	1565.014	3677.386

Boundary Element Check

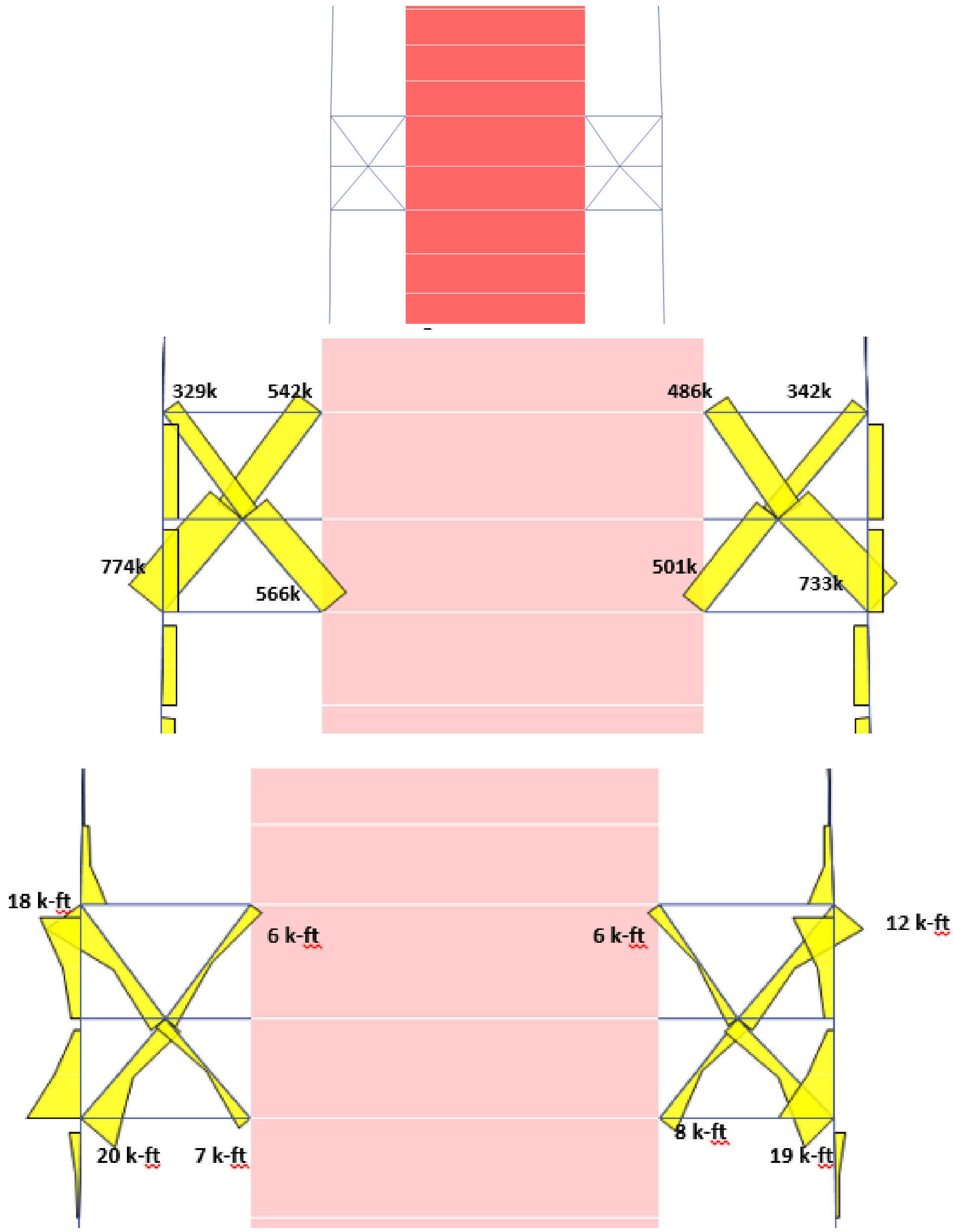
Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²	C Depth in	C Limit in
Top–Left	Leg 1	0	1.2D+1.0E+L	5827.532	-34580.2837	1051.89	1200	81.5223	96.1008
Top–Right	Leg 1	0	1.2D+1.0E+L	5827.532	30013.9738	984.24	1200	81.5223	96.1008
Bottom–Left	Leg 1	0	1.2D+1.0E+L	5827.532	-48897.8729	1264	1200	81.5223	96.1008
Botttom–Right	Leg 1	0	1.2D+1.0E+L	5827.532	63831.5334	1485.24	1200	81.5223	96.1008

Figure 47|Shear Wall A Detailing

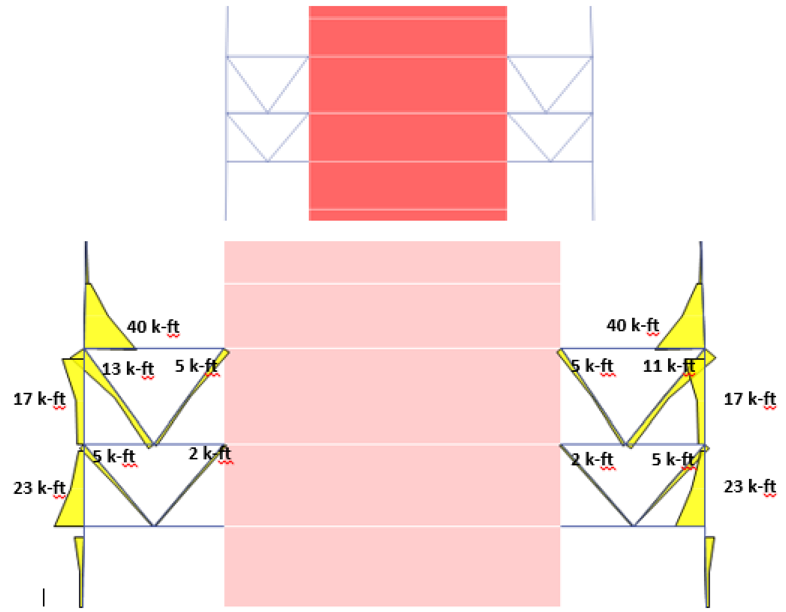
Appendix E: Outrigger Comparison Output

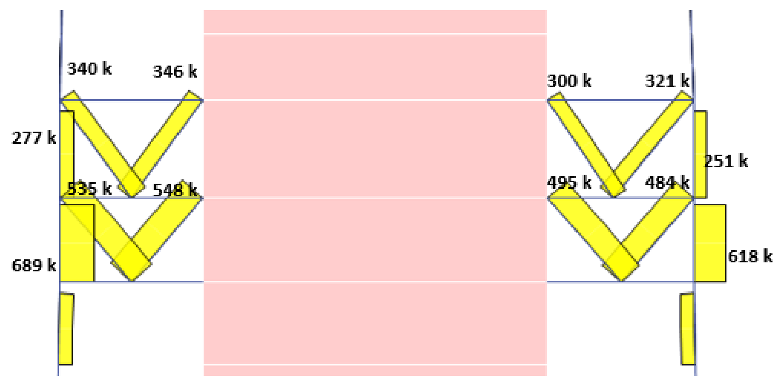
For each outrigger option listed below, the images provided consist of first, the option layout; second, the axial forces in the system; third, moments developed

- Option A –X bracing spanning two floors

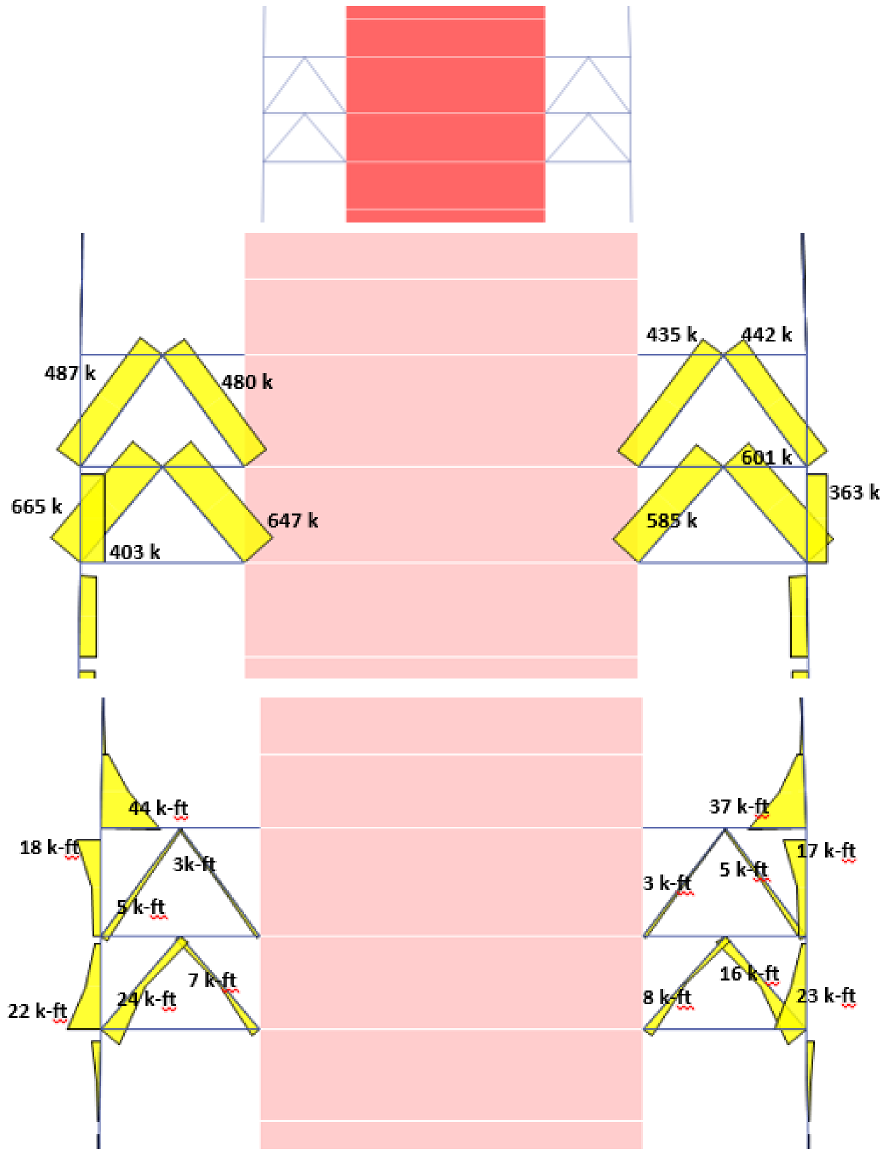


- Option B – V braces

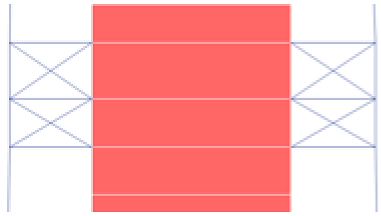


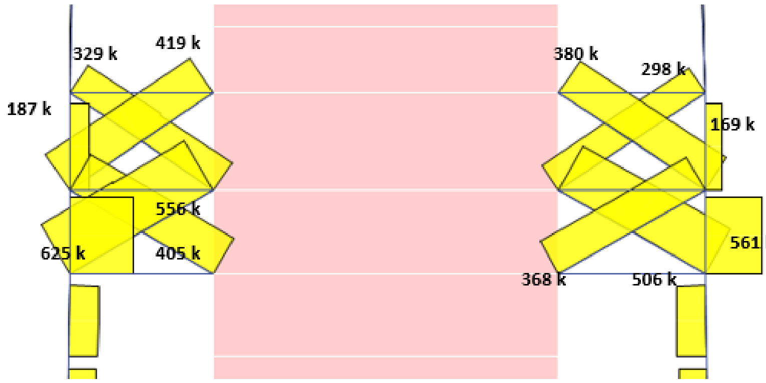


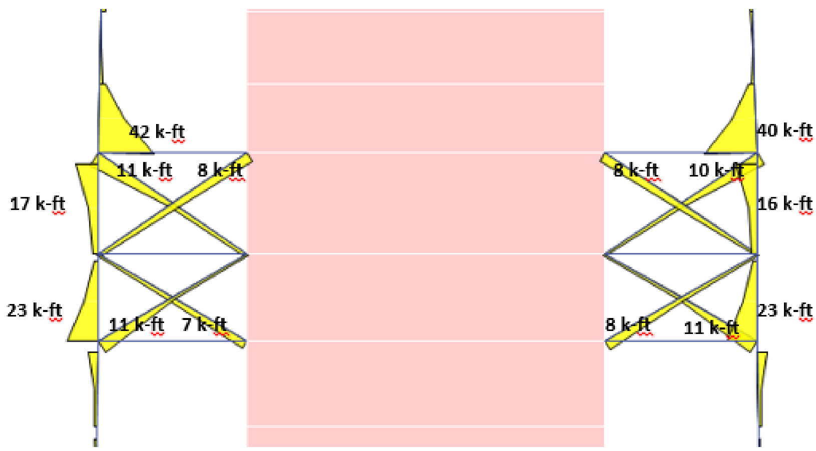
- Option C – Inverted V braces (Chevron)



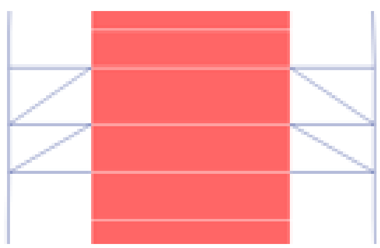
- Option D – X bracing both stories

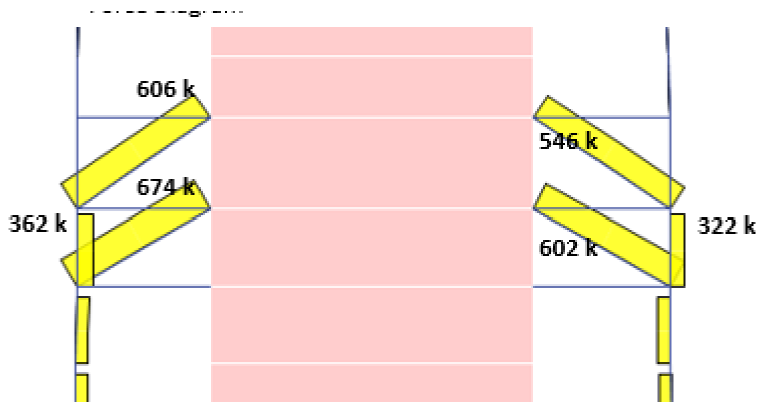


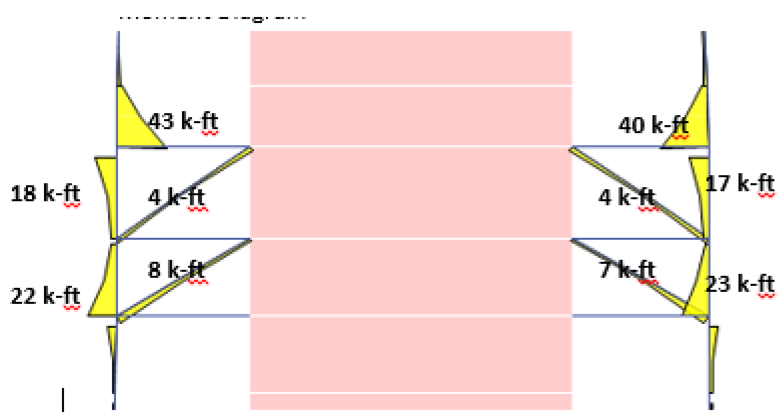




- Option E – diagonal bracing




606 k
546 k
674 k
362 k
322 k
602 k


43 k-ft
40 k-ft
18 k-ft
4 k-ft
4 k-ft
17 k-ft
8 k-ft
7 k-ft
22 k-ft
23 k-ft