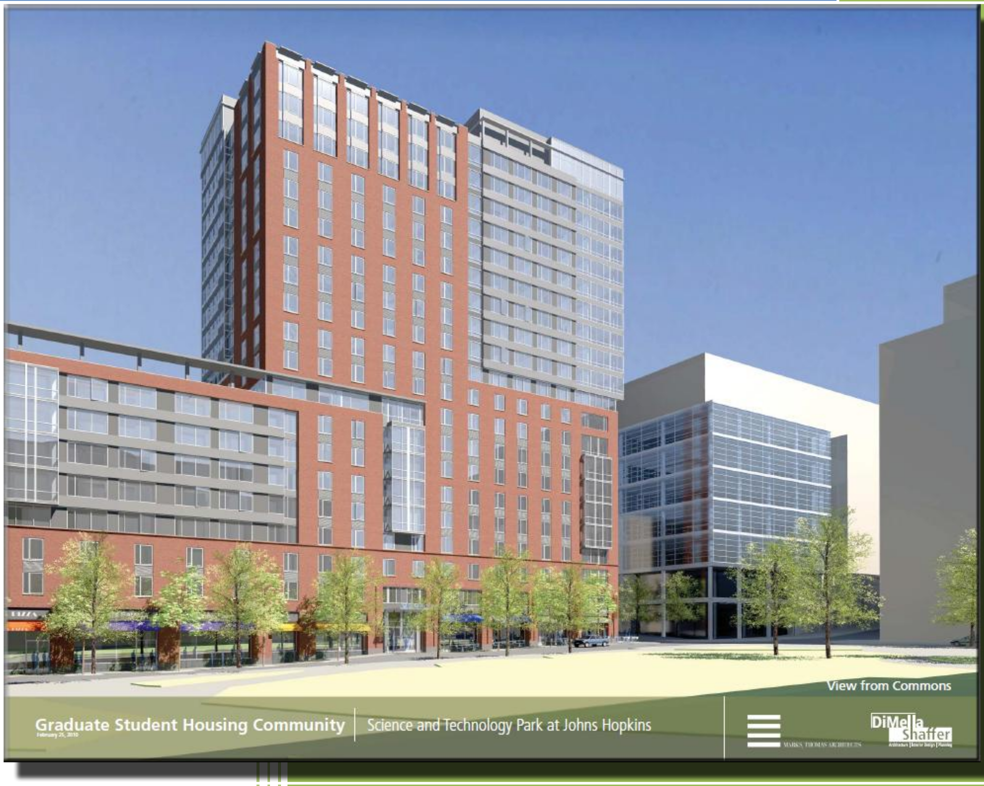
John Hopkins Graduate Student Housing

Final Report



929 North Wolfe Street
Baltimore, Maryland

Brad Oliver – Structural

Advisor: Professor Memari

04/04/2012

John Hopkins Graduate Student Housing

Baltimore, Maryland

Brad Oliver - Structural

Project Overview

- Size - 276,000 sq ft
- Construction - Aug '10 - June '12
- Cost - $44 million (hard costs)
- Contract - Single Prime
- Owner - Education Realty Trust
- Architect - Marks, Thomas Architects
- Contractor - Clark Construction
- Mechanical - BKM

Mechanical -

- 5 Air Handling units with an average flow of 4500 cfm
- VAV boxes with electric reheat coils located throughout the building
- Cooling is provided by two 350 ton water cooling towers

Architecture -

- Primarily residential use providing 929 rooms for John Hopkins Graduate students
- A 9 and 20 story tower composed of a brick and glass façade with metal panels to provide a modern look
- Accessible green roof terrace on the lower tower

Structure -

- Typical floor framing is an 8" thick two way post-tensioned slab system
- Deep foundation system consisting of Caissons ranging from 3 to 4.5 feet in diameter
- Ordinary reinforced concrete shear walls used to transfer lateral loads down to the foundation

Electrical -

- 2400 Amp 3 phase, 4 wire from utility company for normal 208Y/120V and 480Y/277V systems
- First 8 floors (208Y/120V systems) are serviced separately than the remaining primarily through 1000 amp bus ducts.
- 4000 KW 480Y/277V 3 phase, 4 wire generator for emergency systems on 1st floor

Brad Oliver's CPEP website: http://www.engr.psu.edu/ae/thesis/portfolios/2012/BRO5010/index.html

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Executive Summary -

Johns Hopkins Graduate Student Housing is a 20 story apartment complex located in Baltimore Maryland. The first floor is comprised of a three commercial spaces while the rest of the building is residential. The existing structure is composed of an 8 inch thick post-tensioned concrete slab. Lateral loads are resisted through one foot thick shear walls extending the whole height of the building.

In order to make a problem within the structure, a move to San Francisco was proposed. Moving to a high seismic region would cause the tall shear walls to no longer be code compliant. The proposed solution for this project then was to design a dual system of eccentric braced frames with moment connections capable of resisting at least 25% of the seismic load. These frames were designed according to AISC Seismic Provisions. At the Baltimore location, controlling wind deflections was the greatest challenge and caused the design to incorporate several frames.

In order to reduce seismic weight and prepare the structure for a seismic region, the gravity system was redesigned utilizing composite steel beams. Typical sizes for the beams were found to be W12X19 when sized by hand or Ram Structural Systems. A goal for designing the gravity system was to minimize the structural depth just as the original structure had done. This was achieved through small tributary areas and the composite system.

One the structure was designed at the current location, the move took place and was analyzed once again. Many of the structural elements, particularly columns, needed to be upsized by 10-20 pounds per foot. Unfortunately, the building was also found to once again be torsionally irregular despite the addition of several frames.

In order to compare and see if the steel system was viable, a cost a schedule analysis was done comparing the two structures. It was found that the steel system resulted in an expedited schedule and cost savings, but further investigation of the connections would need to be done to ensure accuracy. An architecture breadth was also performed. Minimizing the architectural impact was a goal throughout the design process but not all conflicts could be avoided. The lounge and fitness room were the locations studied and rendered for this project.

Acknowledgements –

I would like to thank everyone who made my thesis possible and contributed to a successful project including:

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Introduction –

Located just outside the heart of Baltimore, two blocks from John Hopkins campus, is the site for the new John Hopkins Graduate Student Housing. This housing project is being constructed in the science and technology park of John Hopkins. A developing "neighborhood", the science and technology park is over 277,000 sq. ft. which is planned to host at least five more buildings dedicated to research for John Hopkins University. The site is also directly across from a 3 acre green space. This location is ideal because it places graduate students within walking distance of the schools hospitals, shopping, dining and relaxing.

Figure 1 - Showing glass and brick facade along with curtain wall

John Hopkins Graduate Student Housing project is a new building constructed with brick and glass facades for a modern look. Upon completion, the building's main function is predominantly for graduate residential use, providing 929 bedrooms over 20 floors. There are efficiencies, 1, 2, and 4 bedroom apartments available. Other features include a fitness room and rooftop terrace. A secondary function of the building is three separate commercial spaces located on the first floor. Retail spaces provide a mixed use floor, creating a welcoming environment and bringing in additional revenue. At the 10th floor, the typical floor size decreases, creating a low roof and a tower for the remaining ten floors. Glass curtain walls on two corners of the building also begin on the 10th floor and extend to the upper roof.

The façade of John Hopkins GSH is composed mainly of red brick and tempered glass with metal cladding. Large storefront windows will be located on the first floor and approximately 6' x 6' windows in the apartments. The curtain wall is to be constructed of glass and metal cladding that can withstand wind loads without damage. There is a mechanical shading system in the windows to assist in the LEED silver certification.

Figure 2 - an overhead showing the green roof and large green area across the street

John Hopkins GSH is striving to achieve LEED silver certification. Most of the points accumulated to achieve this level come from the sustainable sites category. A total of 20/26 points were picked up in this category due to a number of achievements such as; community connectivity, public transportation access, and storm water design and quality control. Indoor air quality is the next largest category where the building picks up an additional 11 points for the use of low emitting materials throughout construction. Several miscellaneous points are picked up for using local materials and recycling efforts as well. Shading mechanisms are also implemented throughout the design as well as an accessible green roof.

There are three different types of roofs on this project. Above the concrete slab on the green roof is a hot rubberized waterproofing followed by polystyrene insulation, a composite sheet drying system, and finally the shrubbery. The sections of roof containing pavers will be constructed using the same waterproofing, a separation sheet, the insulation and finally pavers placed on a shim system. The remaining portions of the roof will be constructed using a TPO membrane system.

Structural Systems –

Foundations:

A geotechnical report was created based on 7 soil test borings drilled from 80’ to 115’ deep. Four soil types were found during these tests: man placed fill from previous construction 7-13 feet deep, Potomac group deposits of silty sands at 40-75 feet, and competent bedrock at 80-105 feet. Soil tests showed a maximum unconfined compressive strength of 12.37 ksi. The expected compression loads from the structure were 2400k and 1100k for the 20 and 9 floor towers, respectively. The foundation system will also have to support an expected uplift and shear force, respectively, of 1400k per column and 180k per column. Based on pre-existing soils and heavy axial loads it was determined that a shallow foundation system was neither suitable nor economical.

In order to reach the competent bedrock, John Hopkins GSH sits on deep caissons 71-91 feet deep. Caissons range in 36-54” in diameter and are composed of 4000psi concrete. Grade beams, 4000psi, sit on top of the caissons followed by the slab on grade. Slab on grade consists of 3500 psi reinforced with W2.9XW2.9 and rests on 6” of granular fill compacted to at least 95% of maximum dry density based on standard proctor.

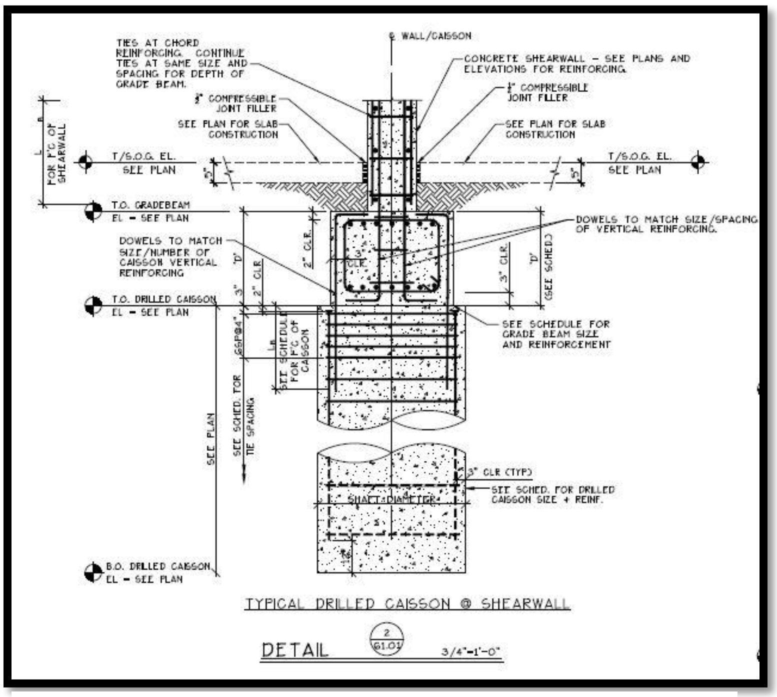


Figure 3 - a detail section of a caisson and column

According to the geotechnical report, the water table is approximately 10 feet below the first floor elevation, therefore a sub drainage system was not necessary.

Floor Framing:

Dead and live loads are supported in John Hopkins GSH through a 2-way post-tensioned slab. The slab is typically 8" thick normal weight 5000 psi concrete reinforced with #4 bars at 24" on center along the bottom in both directions. The tendons are low-relaxation composed of a 7-wire strand according to ASTM A-416. Effective post tensioning forces vary throughout the floor, but the interior bands are typically 240k and 260k. This system is typical for every floor except for the 9th which supports a green roof and accessible terrace. Higher loads on this floor require a 10" thick 2 way post tensioned slab reaching a maximum effective strength of 415k. The bottom layer of reinforcing in this area is also increased to #5 bars spaced every 18". One bay on the 9th floor (grid lines 7-8) is constructed with a 10" cast in place slab. Plans of this floor can be found in appendix E.

Mechanical penthouses exist on the 9th and 20th roof constructed with a steel moment frame. Typical sizes for the 9th floor penthouse are W10's and W12's with 1.5" 20 gage "B" metal deck. As for the 20th floor penthouse, the typical beam size is W16x26. Equipment will be supported on concrete pads typically 4" thick. Two air handling units and cooling towers on the roof will require 6" pads.

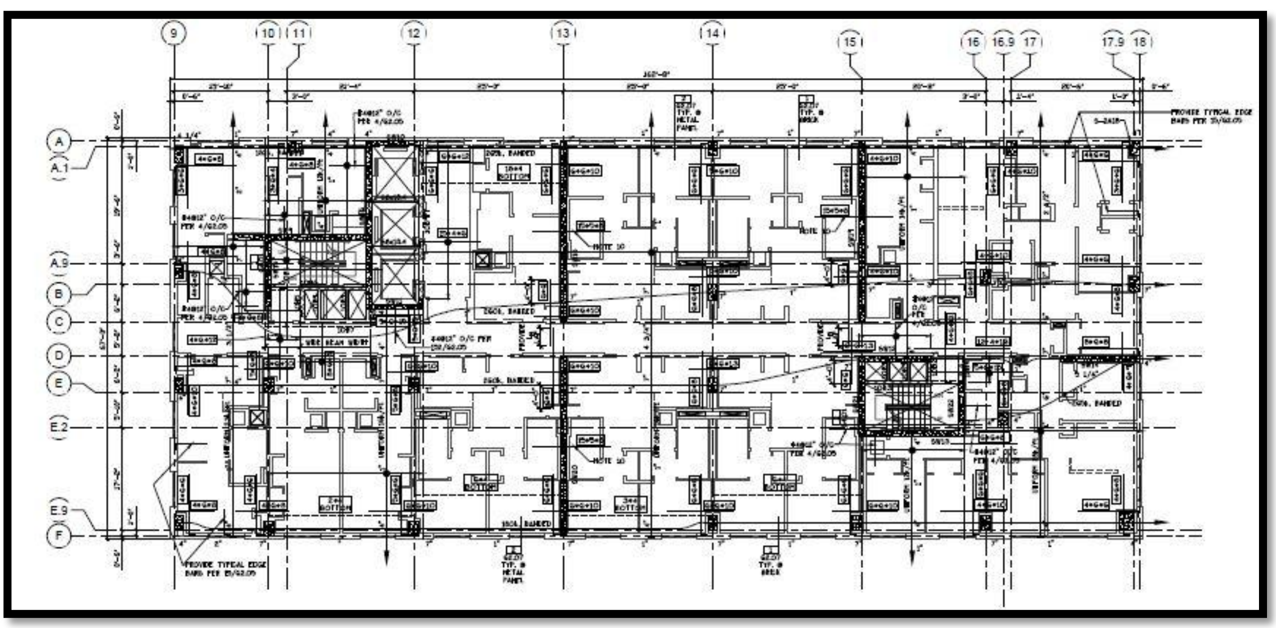


Figure 4 - Typical floor plan of upper tower

The loads will flow through the slab and reinforcement to the columns eventually making their way down to the foundation. To tie the slab and framing system into the columns, two tendons pass through the columns in each direction. To further tie the systems together, bottom bars have hooked bars at discontinuous edges. Dovetail inserts are installed every 2’ on center to tie the brick façade in with the superstructure. Columns are typically 30”x20” and composed of 4ksi strength in the northern tower (9 floors), while columns in the southern tower vary from 8ksi at the bottom, and 4 ksi at the top.

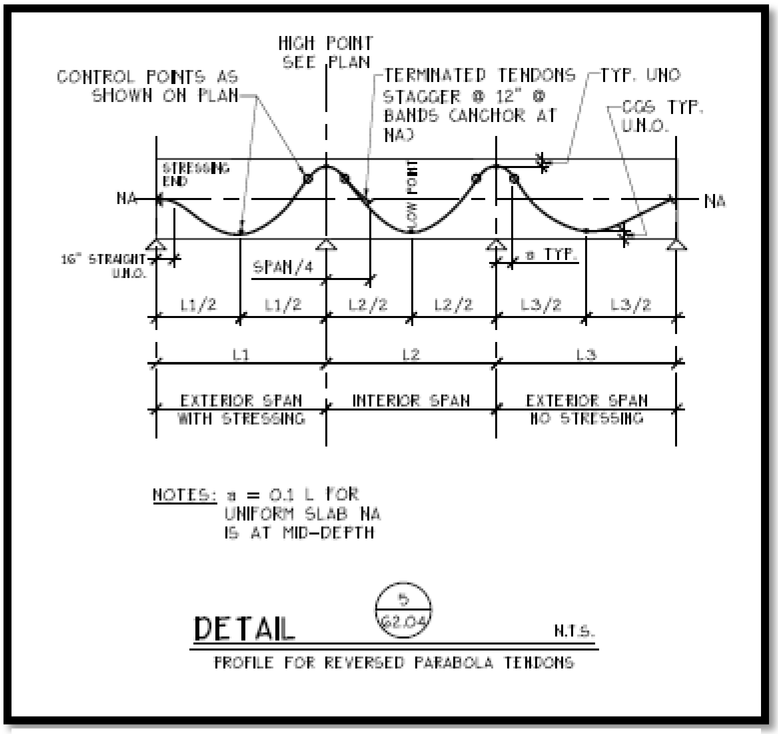


Figure 5- Typical detail for post tensioned tendon profile

Lateral System:

John Hopkins GSH is supported laterally through a cast in place reinforced concrete shear wall system. All of the shear walls are 12" thick and located throughout the building and around stairwells and elevator shafts. Shear walls in the 9 floor tower are poured with 4000psi strength concrete while shear walls in the 20 floor tower vary in three locations. From the foundation to 7th floor, 8ksi concrete is used, 6ksi from 7th to below 14th floor, and 4ksi for walls above the 14th floor. The shear walls are tied into the foundation system through bent vertical bars 1' deep into the grade beam as shown in figure 6. Shear walls are shown below in the figure with N-S walls highlighted in blue and E-W walls red. Walls in the center of the building will support lateral stresses directly, while those on the end support the torsion effects caused by eccentric loads.

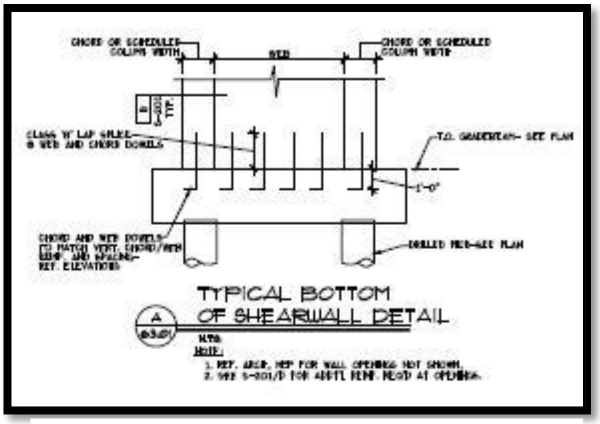


Figure 6 - detail tying shear wall into foundation

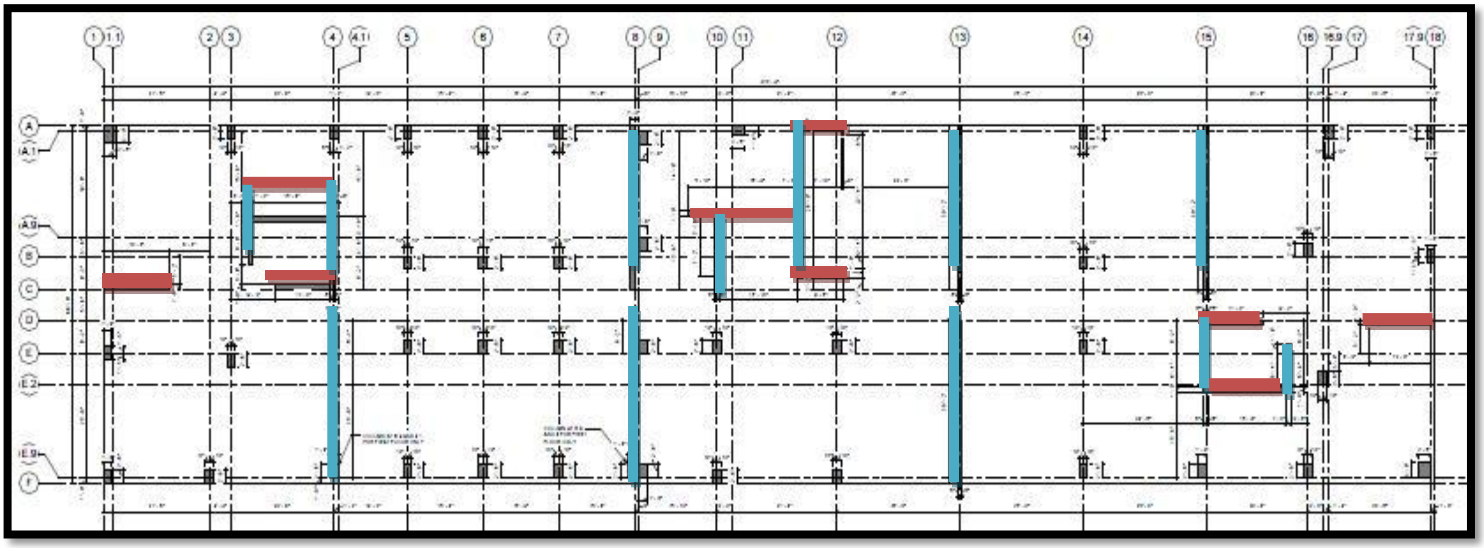


Figure 7 - Shear wall layout

Building Code Summary –

	John Hopkins GSH was designed to comply with:	My Thesis analysis/design will be based on:
General Building Code	IBC 2006	IBC 2006
Lateral Analysis	ASCE7	ASCE7-05
Concrete Specifications	ACI 301, 318, 315	ACI 318-08
Steel Specifications	AISC and AWS D1.1	AISC 2006
Masonry Specifications	ACI 530.1/ASCE 6	ACI 530.1-08/ASCE 6-08

Table 1- Building Code Comparison

Material Strength Summary –

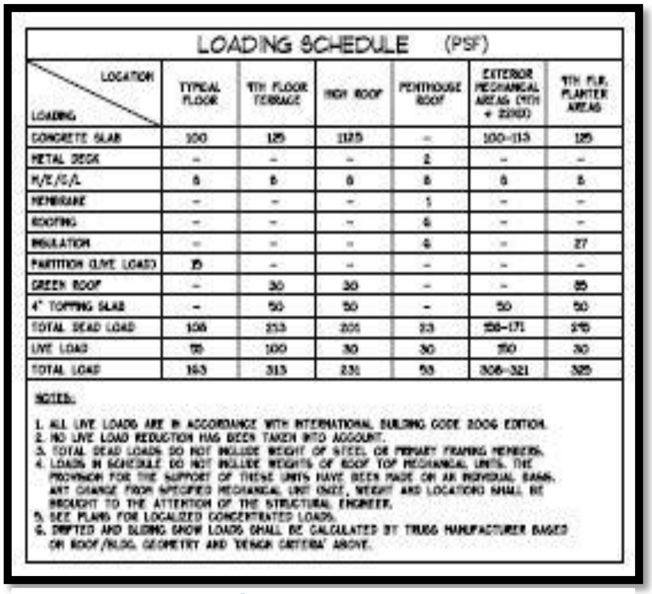
Material Strengths		
Concrete		
Material	**Weight (lbs/ft^3)**	**Strength (psi)**
Footings	145	4000
Pile Caps	145	4000
Caissons	145	4000
Grade Beams	145	4000
Slab-on-grade	145	3500
Slabs/beams	145	5000
Slab on metal deck	115	3500
Columns	145	Vary-see schedule
Shearwalls	145	Vary-see schedule
Steel		
Shape	**Grade**	**Yield Strength** (ksi)
W Shapes	A992	50
S, M and HP Shapes	A36	36
HSS	A500-GR.B	42
Channels, Tees, Angles, Bars, Plates	A36	36
Reinforcing Steel	GR. 60	60

Table 2 - Material Strength Summary

Load Calculations –

Dead Loads:

The dead loads calculated have confirmed the dead loads that were provided in the loading schedule as seen in figure 8. It appears that the designer used ASD in their analysis because the total load does not have any factors applied to it. The analysis in this tech report will be LRFD which typically results in a more aggressive design.



LOADING SCHEDULE (PSF)

LOCATION / LOADING	TYPICAL FLOOR	7TH FLOOR TERRACE	HIGH ROOF	PENTHOUSE ROOF	EXTERIOR MECHANICAL AREAS (7TH + 22ND)	7TH FLR. PLANTER AREAS
CONCRETE SLAB	100	125	112.5	–	100–115	125
METAL DECK	–	–	–	2	–	–
M/E/C/L	6	6	6	6	6	6
MEMBRANE	–	–	–	1	–	–
ROOFING	–	–	–	6	–	–
INSULATION	–	–	–	6	–	27
PARTITION (LIVE LOAD)	15	–	–	–	–	–
GREEN ROOF	–	30	30	–	–	85
4" TOPPING SLAB	–	50	50	–	50	50
TOTAL DEAD LOAD	106	213	201	23	156–171	295
LIVE LOAD	55	100	30	30	150	30
TOTAL LOAD	163	313	231	53	306–321	325

NOTES:

1. ALL LIVE LOADS ARE IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2006 EDITION.
2. NO LIVE LOAD REDUCTION HAS BEEN TAKEN INTO ACCOUNT.
3. TOTAL DEAD LOADS DO NOT INCLUDE WEIGHT OF STEEL OR PRIMARY FRAMING MEMBERS.
4. LOADS IN SCHEDULE DO NOT INCLUDE WEIGHTS OF ROOF TOP MECHANICAL UNITS. THE PROVISION FOR THE SUPPORT OF THESE UNITS HAVE BEEN MADE ON AN INDIVIDUAL BASIS. ANY CHANGE FROM SPECIFIED MECHANICAL UNIT SIZE, WEIGHT AND LOCATIONS SHALL BE BROUGHT TO THE ATTENTION OF THE STRUCTURAL ENGINEER.
5. SEE PLANS FOR LOCALIZED CONCENTRATED LOADS.
6. DRIFTED AND SLIDING SNOW LOADS SHALL BE CALCULATED BY TRUSS MANUFACTURER BASED ON ROOF/BLDG. GEOMETRY AND 'DESIGN CRITERIA' ABOVE.

Figure 8 - Summary of loads used by designer

Live Loads:

It seems John Hopkins used loads very similar to the ASCE7-05 standards. Exterior mechanical loads were not specified in the standard, but I am assuming the equipment can cause significant loads while operating. The 30psf on non-assembly roof areas is most likely a judgment call to account for the maintenance that would be required for a green roof. Although not specified on the table, the 100psf required in the corridor and stairwells are most likely balanced by the large banded post tensioned tendons running parallel to the corridor and around the stairwells.

Area	Designed for – (psf)	ASCE7-05 (psf)
Typical Floor	55 (includes partitions)	40 (residential) + 15 (partitions)
Corridors	N/A	100
Stairs	N/A	100
Assembly	N/A	100
First story retail	N/A	100
Roof used for garden/assembly	100	100
Exterior Mechanical areas	150	N/A
High Roof	30	N/A
Penthouse Roof	30	N/A
Planter Areas	30	N/A

Table 3 - Live Load Comparison

Problem Statement –

After performing a gravity and lateral analysis, the Johns Hopkins Graduate Student Housing project was found to be efficient and sufficient. In order to create problems in the structure and provide a learned experience in seismic area, a scenario has been proposed where the project site has been changed from Baltimore to San Francisco, California. The site change results in the structure being classified in seismic design category D.

Once the building location has been changed, the first problem occurs in the lateral system. ASCE 7-05 does not permit ordinary reinforced shear walls in SDC D; therefore, a dual system with moment frames capable of resisting at least 25% of the seismic loads will need to be designed. Lateral loads will be resisted primarily through eccentrically braced frames which need to be designed.

To reduce the seismic weight and loads on the building, the post-tensioned floor system will also need to be redesigned using a composite floor system. Using a steel frame will also provide more ductility to the structure as well.

The original design goals such as cost, minimal floor-floor depth, and appealing architecture, must also be of importance for the redesign. The project was found to be torsionally sensitive in Tech Report 3, so an additional goal for this redesign is to minimize torsional effects.

Problem Solution –

Structural Depth:

To solve the problems associated with moving the building to a seismic region, a steel framing system needs to be designed to withstand the gravity loads as defined by ASCE7-05. The steel structure will be designed to be as economical as possible while keeping the floor-to-floor heights at a minimum just like Tech Report 2. To minimize the structural depth, a composite system will be used to take advantage of concrete's strong compression properties. IBC 2006 mandates a 2-hour fire rating; therefore, the deck will also need to be designed accordingly. The gravity system also needs to satisfy strength and serviceability requirements such as L/240 for total load and L/360 for live load.

Once the gravity system has been designed, a lateral system needs to be designed to resist wind and seismic loads. Eccentrically braced frames will be the main lateral force resisting system. In order to reduce the torsional sensitivity of the building, braced and moment frames will be placed near the core of the building as well as the exterior. The frames also need to satisfy strength and serviceability requirements. To maximize the ductility in the system and the architectural flexibility, an eccentric braced frame, and moment frames will be designed. For eccentric frames the link element, the beam between braces, is the critical element because it will deform the most. Deformation will provide ductility for the system and absorb seismic loads and reduce the chances of a sudden failure. The lateral system will need to comply with ASCE standards regarding drift limits according to table 12.12-1.

Construction Management:

Changing the main construction method will significantly impact the schedule and cost. Steel erection typically results in quicker schedule than concrete because there is no need for formwork construction and tear down which would save the owner money. An expedited schedule would result in some cost savings for the owner also. Steel connections however would increase the cost of the structure, and if the building height isn't kept to a minimum, the façade will cost more money as well.

Comparisons will be made with regards to cost and schedule analysis at the current location between concrete and steel, and then again once the site is moved to a seismic region. The seismic region will result in more detailed connections, larger members, and possibly more members.

Architecture:

Altering the lateral system from shear walls to a steel braced frame will change numerous architectural features. Columns will need to be moved so they are centered on the grid lines, and added in several locations to limit the span of beams and girders. A steel system will make the most impact in the braced frames. An additional goal for the structural redesign will be to reduce torsion in the building, requiring braced frames in more locations than the current shear walls. These additional frames will cause functional changes to apartments near the outer walls and some of the public spaces such as fitness room and lounge.

Apartments and commercial spaces affected will be inspected to see if the frame can still be architecturally pleasing. If not, then the space will be redesigned to implement the frame while maintaining a functional and aesthetically pleasing space.

Structural Depth, Baltimore Location –

Gravity System:

The existing concrete system didn't have columns at every gridline, and sometimes were not centered on the gridlines. In order to make a more regular bay and layout, columns were added in some locations, or moved one to two feet, to create a geometrically clean and efficient layout. Moving the interior columns one foot towards the center of the building created 3 bays in the short direction of 24 feet on the edges, and 16 feet in the center. The beams were then designed to be spaced at 8' on center in order to minimize the tributary area to maintain low floor-floor heights. Figure 9 shows where new columns were added. The new columns were located where a wall used to be so that the architectural impact could be kept to a minimum. For analysis purposes, existing columns were moved to the nearest gridline and centered.

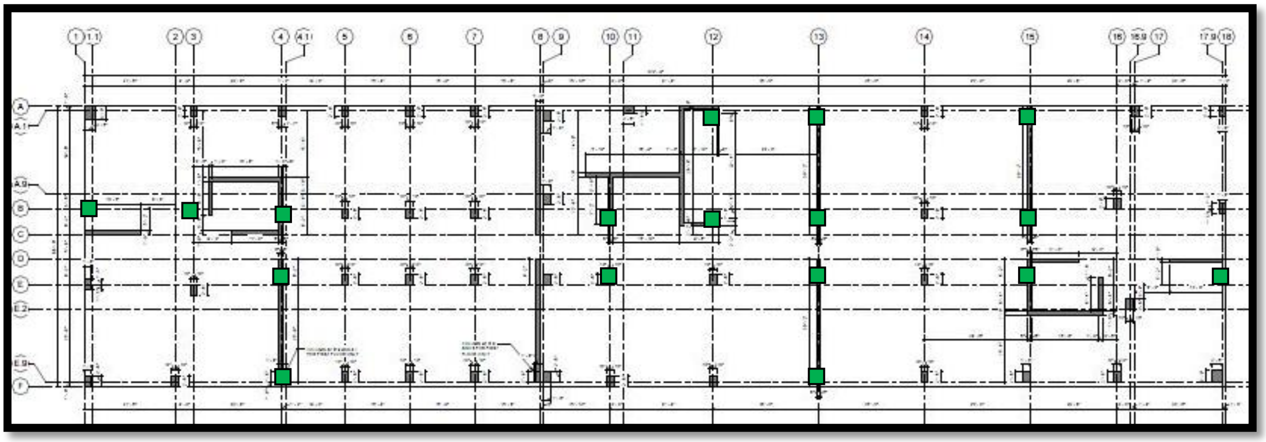


Figure 9 - Steel Column Locations

Composite steel beams were used in order to utilize the compression properties of concrete to resist part of the load. Using the concrete as a part of the gravity system would also help limit the depth of the structural system, ultimately reducing the overall height of the building. A composite beam system also helps maintain an economical design. 2 VLI composite deck with a 2 inch topping was chosen to be the floor system to maintain a two-hour fire rating in order to comply with building codes.

To start the design, beams were designed based on an iterative process to control deflections and meet strength requirements. With typical spans of 25 feet, the first step of the design process was to find the minimum moment of inertia required to meet serviceability requirements. Trial beam and stud designs were picked and compared with one another to determine which one was most economical before calculating the various strength requirements. Typical bays on the interior and exterior sides of the building were designed by hand and can be found in Appendix B.

In order to expedite the design process, the grids, columns, and loads were put into RAM Structural System. Before running the design process however, some assumptions needed to be made. Defaults were adjusted so the beams would be designed to include no camber, and to minimize the structural depth. Minimizing the structural depth was an original design goal so that the overall height of the building will be approximately the same and won't increase the cost of the façade.

After running the design process, typical sizes of beams were found to be 12X16. A full plan view of the short and tall tower can be seen below in figures 10 and 11 respectively. These designs were compared to the ones designed by Ram and were found to be very close. Ram was slightly more efficient because it's able to compare many more combinations of beams and studs quickly to determine the most economical pairing.

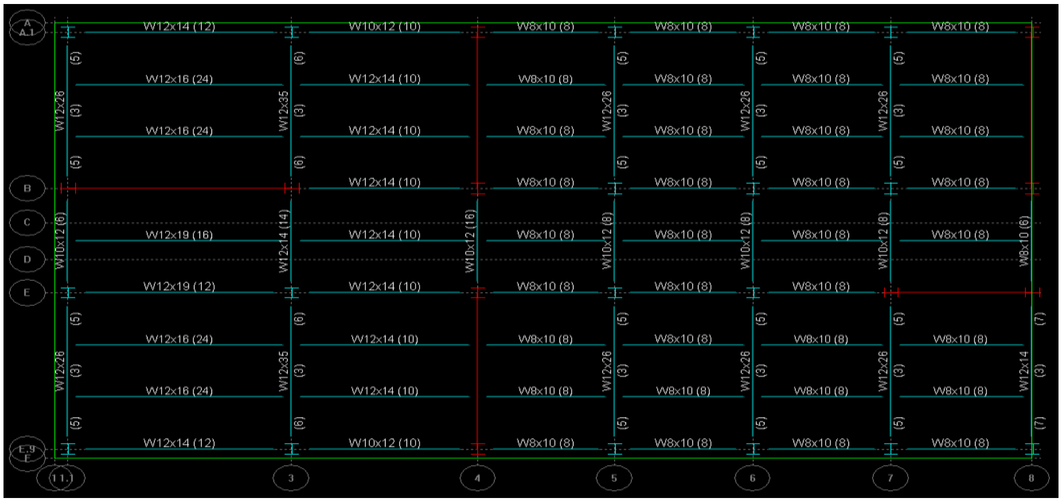


Figure 10 - Beam Design of Short Tower

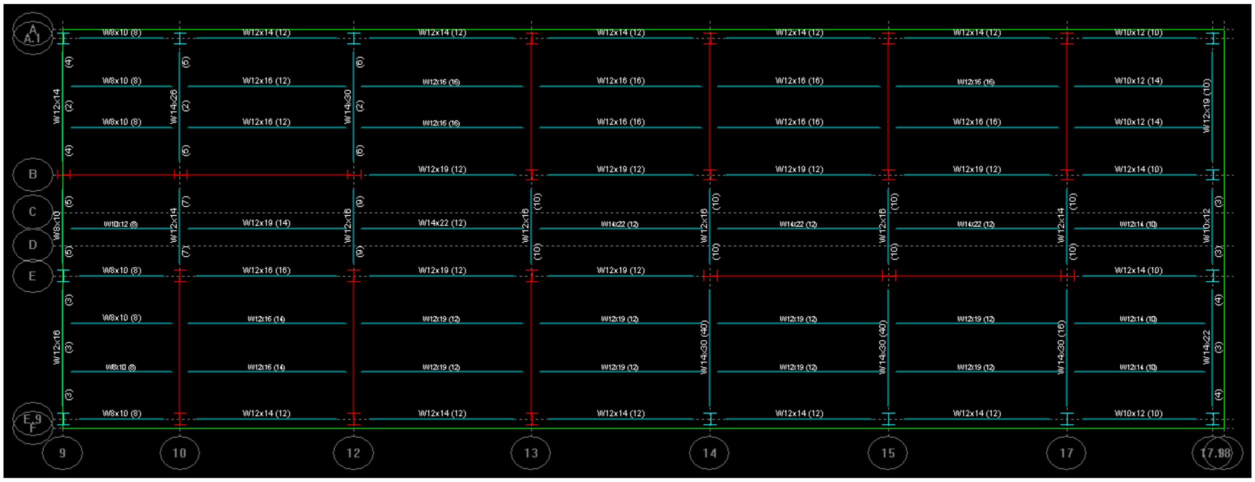


Figure 11 - Beam Design of Tall Tower

Preliminary designs of columns were also performed by hand and can be found in Appendix C. The factored ultimate load was compared to the reduced strength factor. Interaction equations were not done in the interest of time, but the second order effects were included in the computer design, and were close to the ones designed by hand. Column splices were included at every other floor for constructability purposes. OSHA won't allow work to be done more than two floors above grade or metal deck without fall protection, so columns will be erected and spliced as drawn in figure 12. This will also allow for the design to be economical. Once the initial gravity model was complete, the final height of the building was 207 feet, only 3 feet higher than the original.

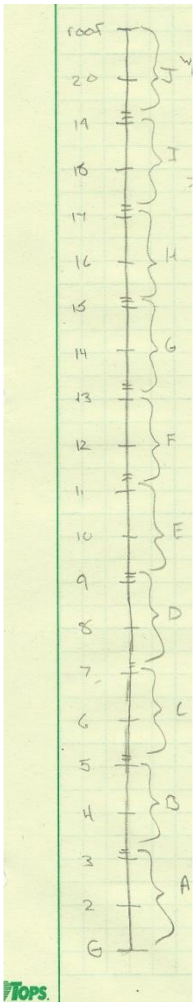


Figure 12 - Column Splice Locations

Eccentric Braced Frame Background:

Once the gravity system was designed, the lateral system was the next step in the design process. Eccentric braced frames are braces that do not stretch from column to column, but instead connect at the beam and have an eccentricity known as e. This is illustrated below in figure 13.

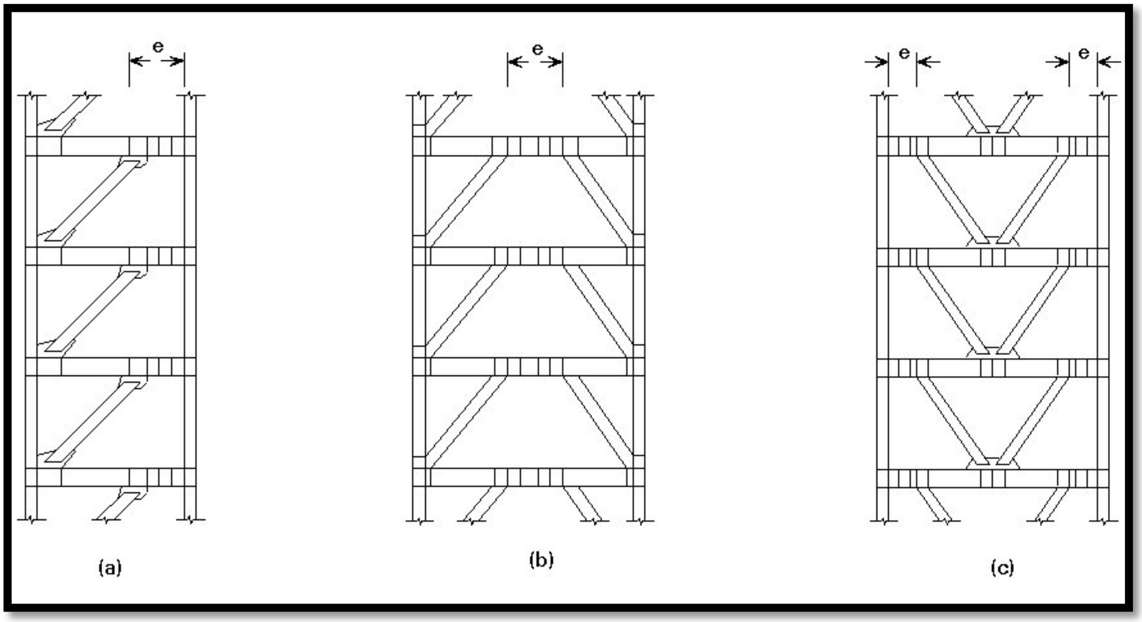


Figure 13 - Courtesy of http://www.fgg.uni-lj.si/kmk/esdep/master/wg01b/l0720.htm

For this particular project, a layout similar to "b" in figure 13 will be used in order to reduce the stresses on the connections. Layouts "a" and "c" put a lot of stress on the moment connection in those regions because of the high rotation at that location. In eccentric braced frames, the beam segment between the two braces is known as the link element and will be the most critical piece in frame design. Ideally, as lateral loads are applied to the structure, the brace will apply shear and axial loads on the link element beyond its elastic capacity. It will deform and dissipate energy which is an advantageous feature in a high-seismic region such as San Francisco.

Eccentric braced frames have several advantages when comparing them to typical chevron or moment frames. Chevron frames are very stiff, making deflections easier to control, but they inhibit the functionality of the architecture. Moment frames allow for the most flexibility of spaces, but are often too ductile for many situations. Due to the height of John Hopkins Graduate Student Housing, and the need for a flexible floor plan, eccentric braced frames were selected as the best option. The first design aspect of the braced frames is the link length. If the link is longer, the frame is less stiff and could be controlled by a combination of shear and flexure. The more desirable option is to have a shorter link length to increase stiffness, and have the design be controlled by shear. A graph representing the relative stiffness of a frame and link length can be found in figure 14. With a tall building, such as the John Hopkins Graduate Student housing, where serviceability will be an issue, especially with wind, the shorter link is an advantageous design.

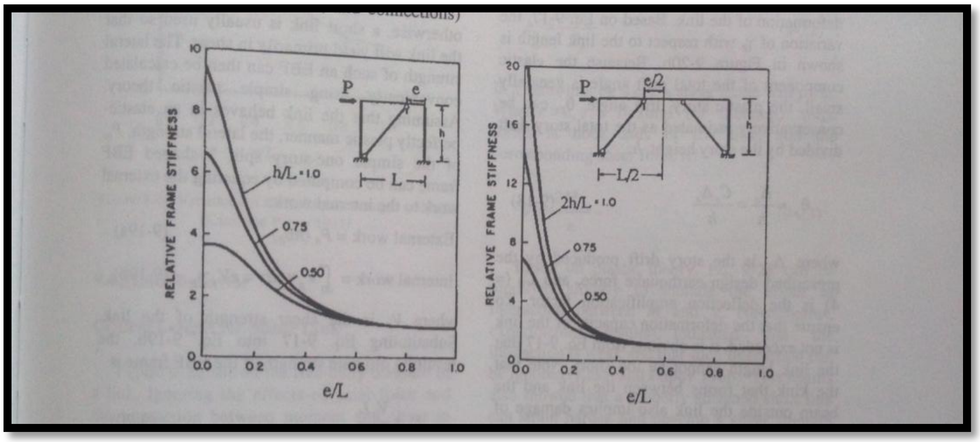


Figure 14 - Courtesy of Seismic Design Handbook

Figure 15 shows an idealized deformed shape of the eccentric braced frame. The link element is designed to deform greatly and dissipate most of the lateral loads. The rest of the beam is designed to remain elastic. Columns are designed to have a larger plastic moment capacity than the beam, known as strong columns weak beam design, to ensure that a pancake failure won't occur.

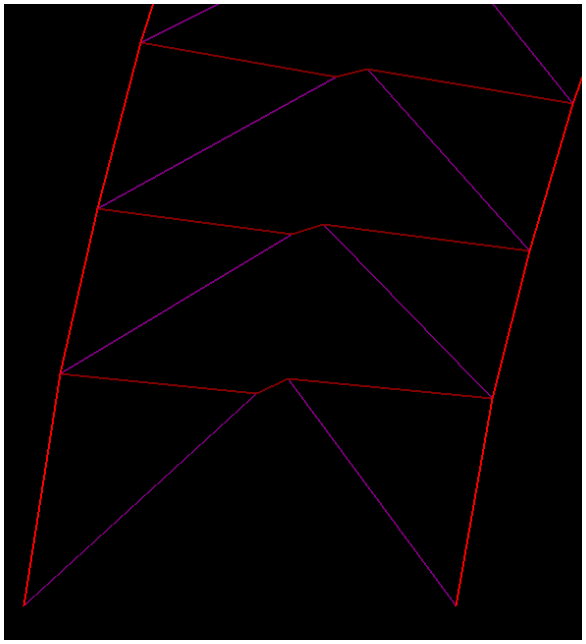


Figure 15 - EBF Deformed Shape

Load Combinations:

Before running the program, it was expected to see wind control the design because most of the concrete weight was removed, causing a decrease in seismic loads. The East-West direction was also predicted to be the most critical direction because it is a large, tall rectangular face that would collect a lot of force. After running the analysis, the predictions were confirmed that wind controlled in drift and strength in the East-West direction. Unlike the original structure however, the fourth wind load case including 15% eccentricity did not control. 100% of the wind load applied in the East-West direction controlled, which is an indicator that the torsional irregularity was removed. This will be discussed and calculated later with actual drift values.

Due to keeping the height of the new structure within three feet of the original, the wind loads were the same as the previous concrete design. The wind loads in the East-West direction are summarized below in tables 4 and 5.

Criteria		E-W Direction					
Tall Tower		**Floor**	**Height (ft)**	**K_z**	**q_z**	**p (windward) (psf)**	**p(leeward) (psf)**
G_f	0.87	Penthouse	208.42	1.21	21.327	18.68	-13.12
C_p (Windward)	0.8	Roof	194.25	1.19	20.974	18.37	-13.12
C_p (Leeward)	-0.5	20	183.9	1.17	20.622	18.06	-13.12
GC_{pi}	0.18	19	174.6	1.15	20.269	17.76	-13.12
Velocity (MPH)	90	18	165.3	1.13	19.917	17.45	-13.12
Lower Tower		17	155.9	1.12	19.741	17.29	-13.12
G_f	0.85	16	146.6	1.1	19.388	16.98	-13.12
C_p (Windward)	0.8	15	137.2	1.09	19.212	16.83	-13.12
C_p (Leeward)	-0.5	14	127.9	1.07	18.859	16.52	-13.12
GC_{pi}	0.18	13	118.6	1.04	18.331	16.06	-13.12
Velocity (MPH)	90	12	109.3	1	17.626	15.44	-13.12
		11	99.9	0.99	17.449	15.29	-13.12
		10	90.6	0.96	16.921	14.82	-13.12
		9	81.3	0.93	16.392	14.10	-9.92
		8	71	0.89	15.687	13.49	-9.92
		7	61.7	0.85	14.982	12.88	-9.92
		6	52.3	0.81	14.277	12.28	-9.92
		5	43	0.76	13.395	11.52	-9.92
		4	33.7	0.7	12.338	10.61	-9.92
		3	24.3	0.7	12.338	10.61	-9.92
		2	15	0.7	12.338	10.61	-9.92
		1	1	0.7	12.338	10.61	-9.92

Table 4 - Wind Load Calculations

E-W Direction Tall Tower					
Floor	Height (ft)	Height Below (ft)	Heigh Above (ft)	Trib Area (ft2)	Story Force (K)
Penthouse	208.42	15.2	0	1236.52	23.10
Roof	194.25	10.33	15.2	2076.87	38.16
20	183.9	9.33	10.33	1599.34	28.89
19	174.6	9.33	9.33	1517.99	26.95
18	165.3	9.33	9.33	1517.99	26.48
17	155.9	9.33	9.33	1517.99	26.25
16	146.6	9.33	9.33	1517.99	25.78
15	137.2	9.33	9.33	1517.99	25.55
14	127.9	9.33	9.33	1517.99	25.08
13	118.6	9.33	9.33	1517.99	24.38
12	109.3	9.33	9.33	1517.99	23.44
11	99.9	9.33	9.33	1517.99	23.20
10	90.6	9.33	9.33	1517.99	22.50
9	81.3	10.25	9.33	1592.83	22.45
8	71	9.33	10.25	1592.83	21.49
7	61.7	9.33	9.33	1517.99	19.56
6	52.3	9.33	9.33	1517.99	18.64
5	43	9.33	9.33	1517.99	17.49
4	33.7	9.33	9.33	1517.99	16.11
3	24.3	9.33	9.33	1517.99	16.11
2	15	14	9.33	1897.90	20.14
1	1	1	14	1220.25	12.95
				Base Shear (K)	**505**
				Overturning moment (k ft)	**58552**

Table 5 - Wind Force Distribution

In addition to the wind loads being applied 100% in each direction independently; the three other cases designated in figure 16 were also checked. Technical report three confirmed that case four controlled several of the deflections indicating a torsional irregularity. In order to check all of the combinations quickly and efficiently, Ram was utilized to calculate and input the wind loads. After inputting the criteria, the story shears were compared to the original spreadsheet to confirm the model was accurate.

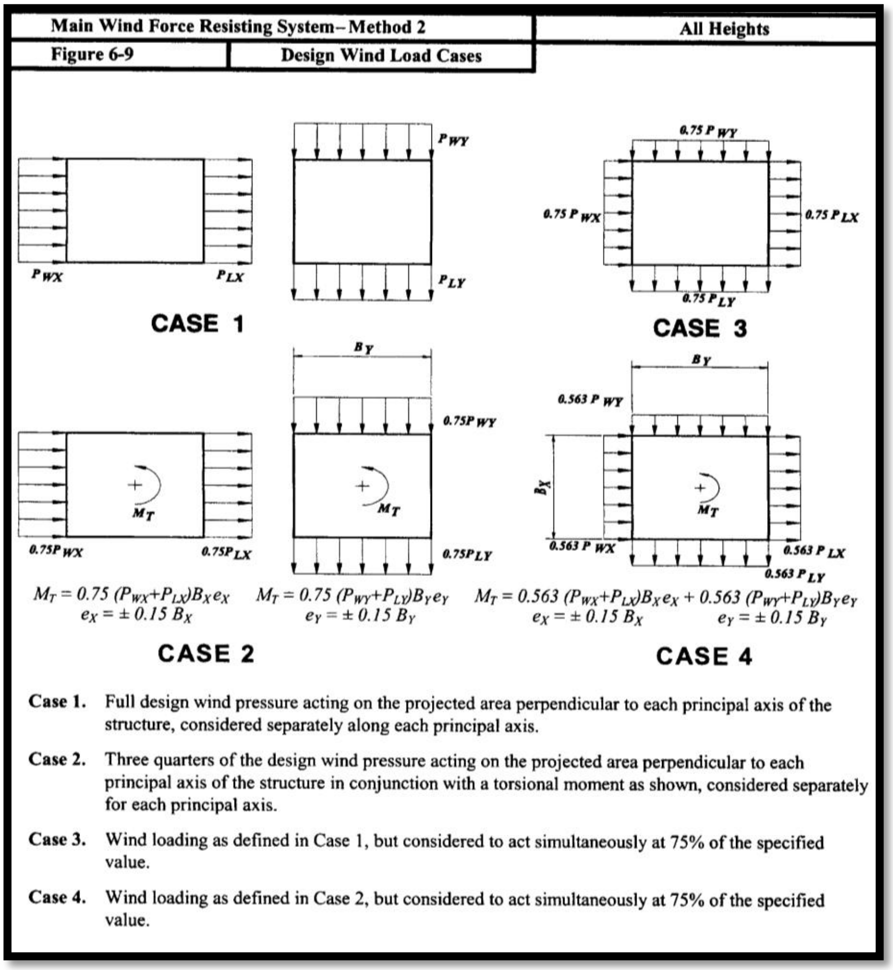


Figure 16- ASCE Wind Cases

After inputting the wind loads and updating the structure to meet the design criteria, a new seismic weight was established to determine earthquake loads. These loads were expected to be much lower than the previous building due to a lighter steel system, and a higher R value (8). A summary of these loads can be found in table 6, and show a large reduction in base shear. The new structural system decreased the base shear from 798 kips to 165 kips, a 79% reduction. Ram was once again utilized to expedite the analysis process when considering accidental and inherent torsion, but the main story shears were compared to ensure accuracy.

Seismic Force Distribution (Tall Tower) N-S						
Floor	Height (ft)	Weight (k)	$(w_x h_x)^k$	C_{vx}	F_x (K)	Overturning Moment (k ft)
Penthouse	208.42	205	1825519621	0.029	4.85	1009.82
Roof	194.25	458.8	7942713060	0.128	21.08	4094.95
20	183.9	467.1	7378756742	0.119	19.58	3601.50
19	174.6	466.5	6634249111	0.107	17.61	3074.36
18	165.3	466.5	5946329945	0.096	15.78	2608.80
17	155.9	466.8	5296072542	0.085	14.06	2191.38
16	146.6	467.1	4689080355	0.075	12.45	1824.48
15	137.2	467.8	4119349662	0.066	10.93	1500.04
14	127.9	468.5	3590544217	0.058	9.53	1218.85
13	118.6	469.5	3100563079	0.050	8.23	975.99
12	109.3	470.5	2644597478	0.042	7.02	767.18
11	99.9	471.7	2220561107	0.036	5.89	588.77
10	90.6	472.8	1834895481	0.029	4.87	441.22
9	81.3	476.2	1498855871	0.024	3.98	323.42
8	71	477.5	1149379506	0.018	3.05	216.59
7	61.7	476.2	863274893	0.014	2.29	141.37
6	52.3	477.1	622618772	0.010	1.65	86.43
5	43	478.7	423705173	0.007	1.12	48.36
4	33.7	480.3	261990157	0.004	0.70	23.43
3	24.3	483	137754822	0.002	0.37	8.88
2	15	492	54464400	0.001	0.14	2.17
	Sum	9659.6	62235275995	Base Shear (K)		**165**
			Base Overturning moment (k ft)			**24748**

Table 6- Seismic Load Distribution

Design Process:

Once again, Ram Structural Systems was used to assist in the design of the frames and speed up the iterative process. Before running the analysis however, the default settings of Ram needed to be adjusted. Braces were modeled as a pin connection at either end. Columns were orientated so that the strong axis of bending was orientated in the direction resisting the force. Centerline modeling was used in order to make rigid zone offsets and panel zone modeling negligible. P-Delta effects were also accounted for in the design and checks of columns. Another assumption made during modeling is the use of a rigid diaphragm. Figure 17 shows a 3D view of the modeled tall tower.

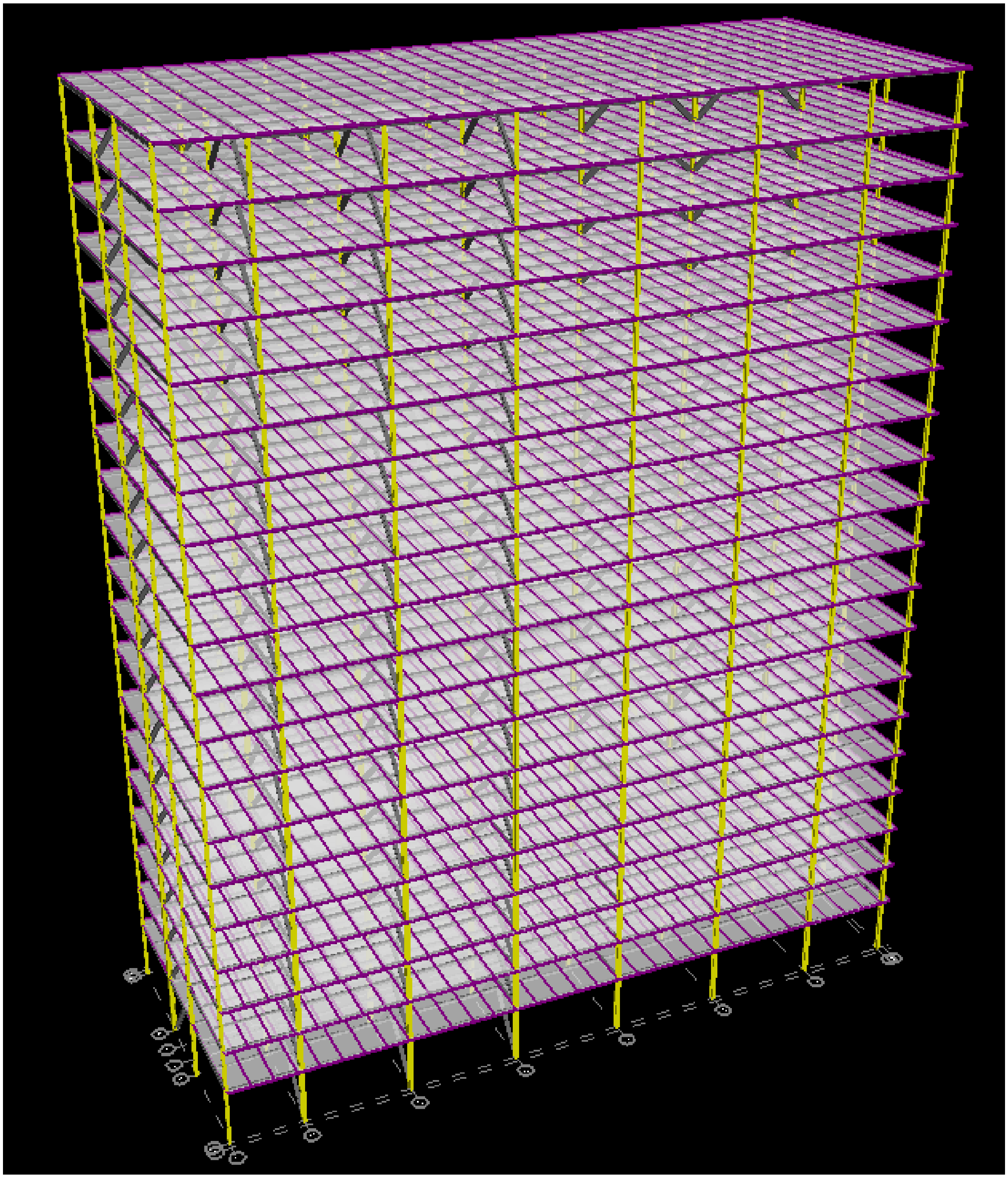


Figure 17- 3D Model of Tall Tower in RAM

The first attempt at designing the braced frames at the Baltimore location utilized an e/L ratio of .1. For a 24 foot span, a link length of 28 inches was used. The first attempt also investigated the use of light gauge bracing, such as C channels in order to reduce weight and cost of the structure. When the analysis ran however, deflections were calculated to be 27 inches, well above the L/400, 6.21 inches, recommendation for wind serviceability. To correct for such a large displacement, a second attempt was ran utilizing W12X26 braces, adding more frames overall, and shrinking the link element length to 20 inches, but the deflections were still not acceptable. In the end, the design ended up using W14X43 braces as well as W14X48 beams to limit overall deflection to 5.97 inches, within the recommended 6.21 inches. The layout of the frames for the tall and short tower respectively can be found below in figures 18 and 19. The star in the figure represents the most stressed frame and the one that was also checked by hand. A secondary reason so many frames were used was to try and remove the torsional irregularity that existed in the concrete system.

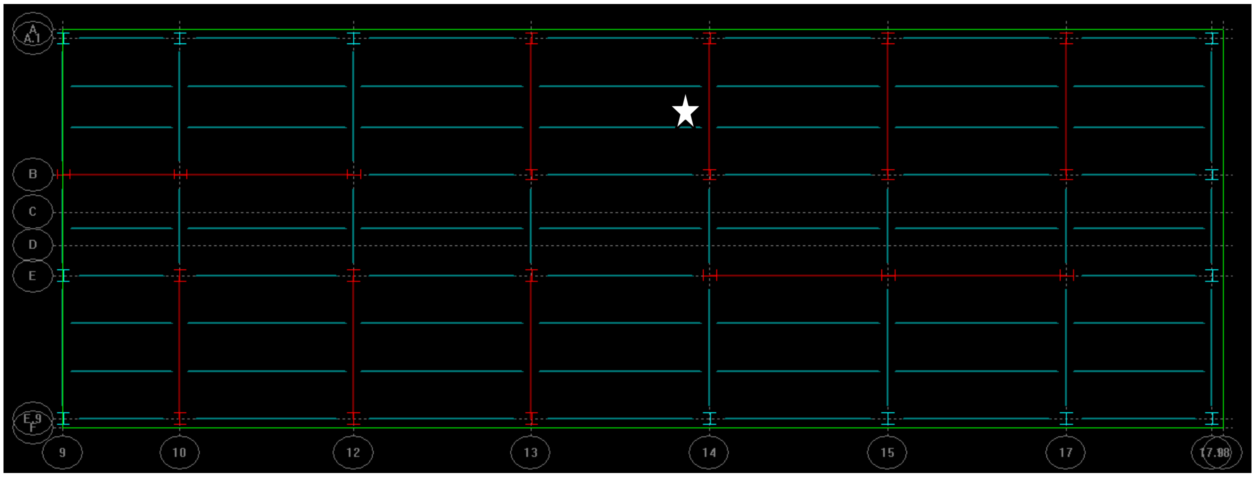


Figure 18 - Tall Tower EBF Layout

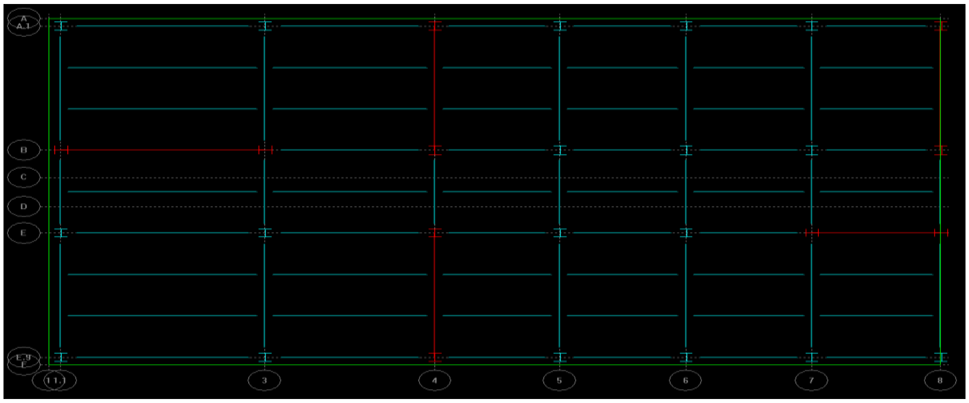


Figure 19 - Short Tower EBF Layout

After getting the design to be acceptable for serviceability, it needed to be checked for strength. Ram has a built in check, but a hand check was done to ensure accurate calculations and understanding of the frame. Figure 20 displays a visualization of all the strength checks that Ram performs on every member. Blue indicates the least stress and an acceptable interaction equation while red indicates a failed requirement. While designing this feature was utilized often to ensure the design was strong enough to resist the loads.

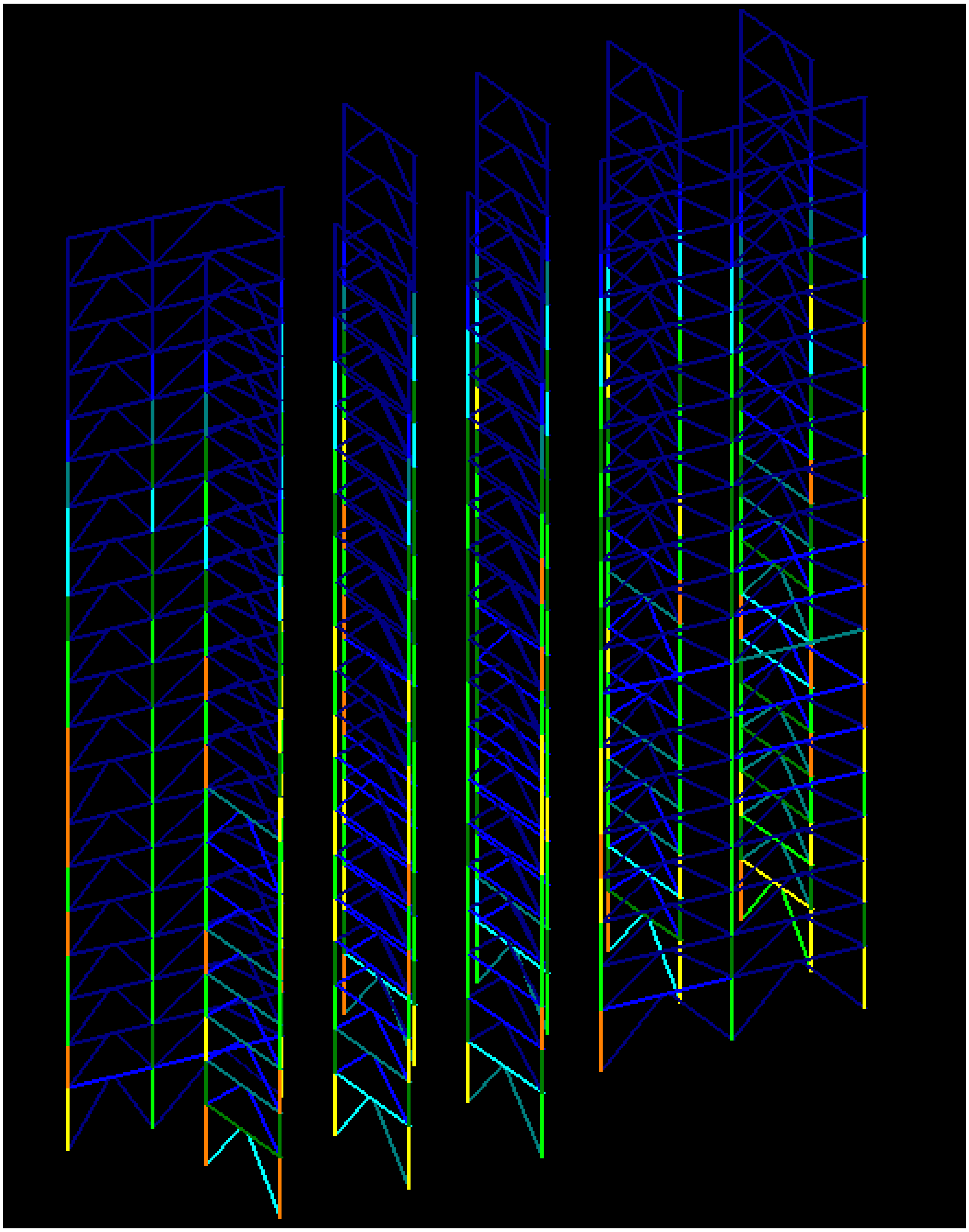


Figure 20 - Tall Tower Strength Check via Ram

The hand checks were then compared to the Ram model and were found to have the same conclusions. Due to the large amount of shear force being induced in the link element, and a small amount of area to resist the forces, local buckling needed to be investigated.

The seismic provisions printed by AISC require the link element to have web stiffeners. For the design in Baltimore, it was found that full depth double sided stiffeners 3/8” x 3.75” are required at the ends of the link element. Within the link element, the same size stiffeners are required on one side of the web spaced at 12”. A detail of this information can be found in figure 21. Several additional requirements were checked such as, rotation angle, shear strength, slenderness, and second order effects. Detailed calculations can be found in appendix D.

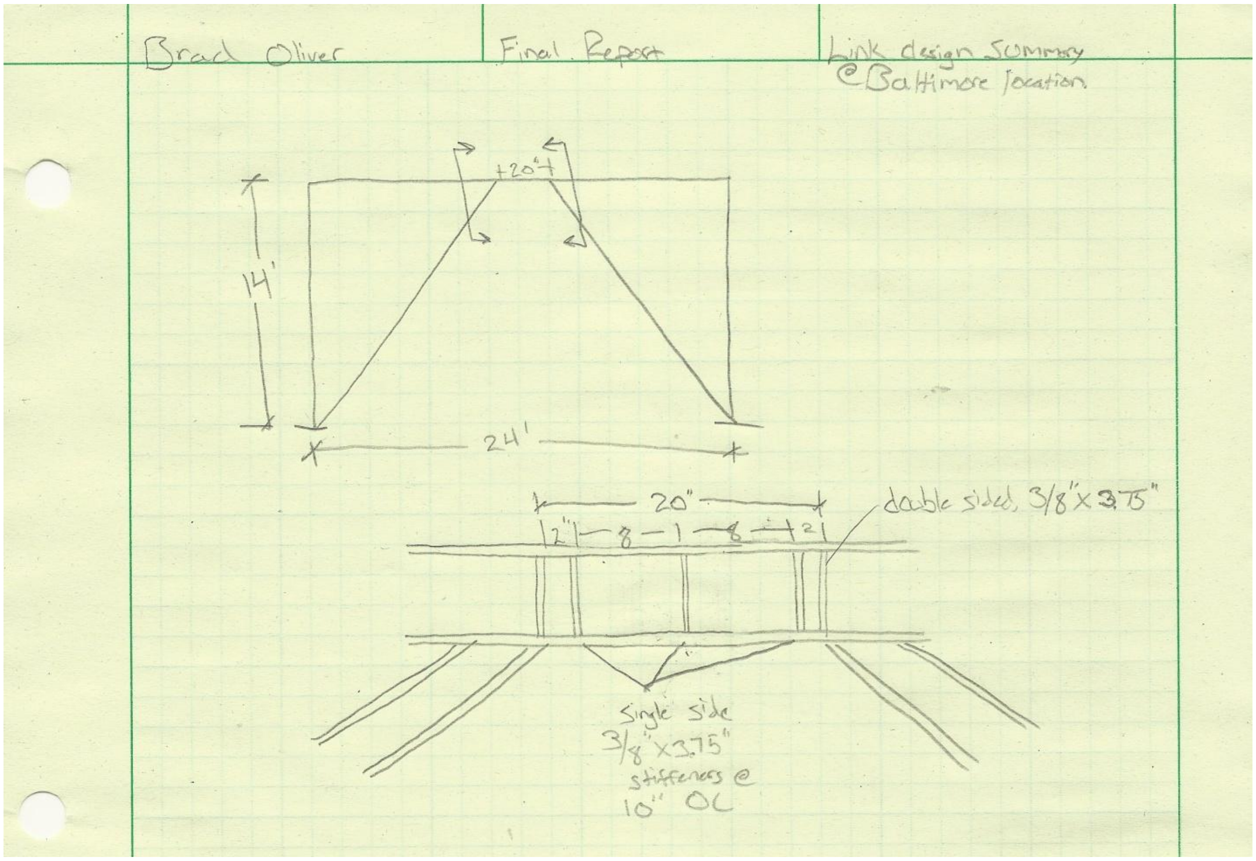


Figure 21 - Web Stiffener Detail

Structural Depth, San Francisco Location -

Upon completing the depth analysis and comparison at the original Baltimore location, it was time to hypothetically move the building to San Francisco. To keep most of the site factors similar to the original ones, a site was chosen in San Francisco University to mimic the one at Johns Hopkins, down to the college environment. Figure 22 displays this site. Several buildings around campus have similar architectural features such as glass and brick façade, and the site picked would be classified in wind exposure b.

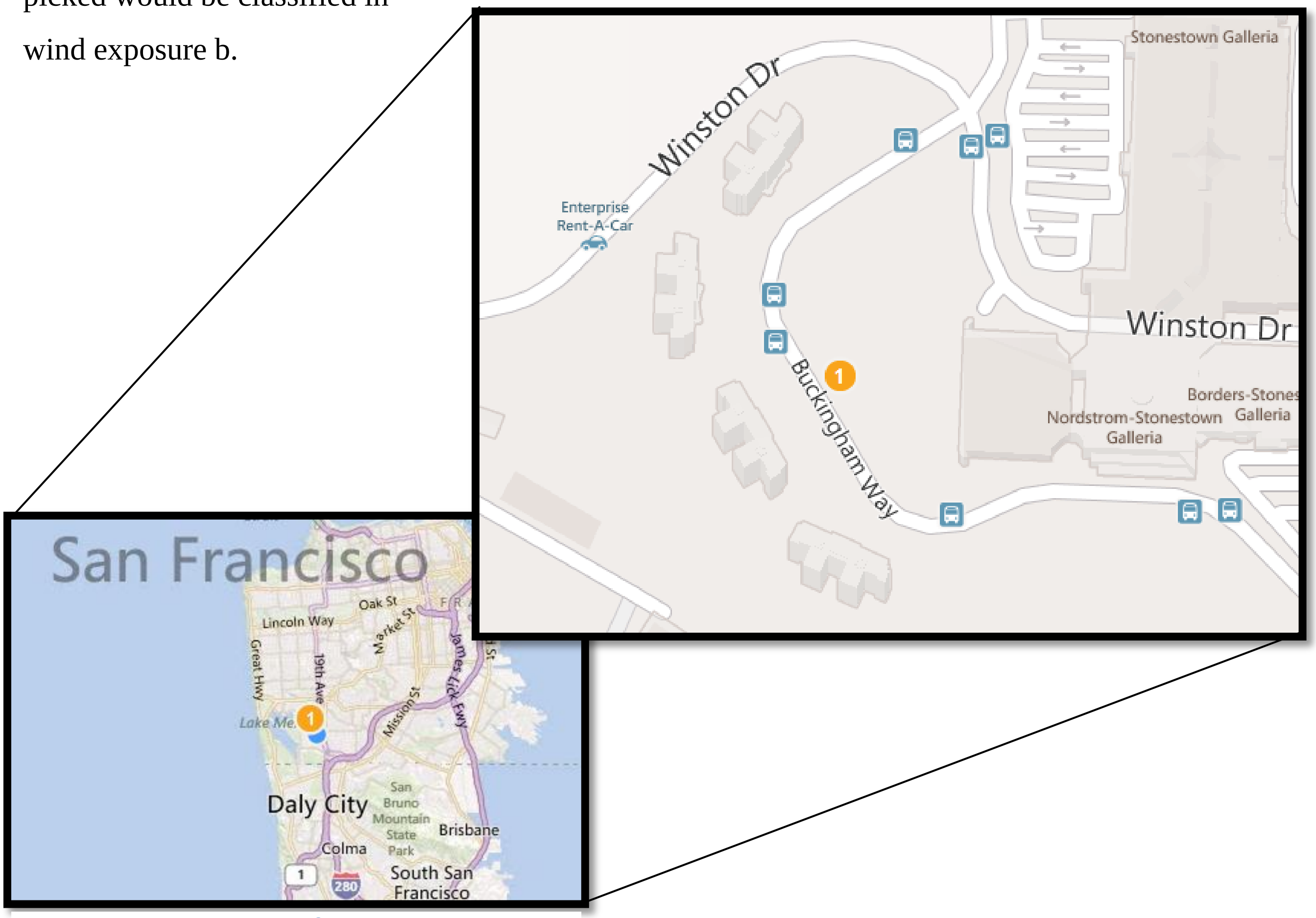


Figure 22 - Proposed location for new site

Load Combinations:

Moving the site to San Francisco will obviously increase the earthquake loads significantly, but it also decreased the wind velocity causing a decrease in base shear.

Criteria		E-W Direction					
Tall Tower		**Floor**	**Height (ft)**	**K_z**	**q_z**	**p (windward) (psf)**	**p(leeward) (psf)**
G_f	0.87	Penthouse	208.42	1.21	19.023	16.66	-11.70
C_p (Windward)	0.8	Roof	194.25	1.19	18.709	16.39	-11.70
C_p (Leeward)	-0.5	20	183.9	1.17	18.394	16.11	-11.70
GC_{pi}	0.18	19	174.6	1.15	18.080	15.84	-11.70
Velocity (MPH)	85	18	165.3	1.13	17.765	15.56	-11.70
Lower Tower		17	155.9	1.12	17.608	15.42	-11.70
G_f	0.85	16	146.6	1.1	17.294	15.15	-11.70
C_p (Windward)	0.8	15	137.2	1.09	17.137	15.01	-11.70
C_p (Leeward)	-0.5	14	127.9	1.07	16.822	14.74	-11.70
GC_{pi}	0.18	13	118.6	1.04	16.350	14.32	-11.70
Velocity (MPH)	85	12	109.3	1	15.722	13.77	-11.70
		11	99.9	0.99	15.564	13.63	-11.70
		10	90.6	0.96	15.093	13.22	-11.70
		9	81.3	0.93	14.621	12.57	-8.85
		8	71	0.89	13.992	12.03	-8.85
		7	61.7	0.85	13.363	11.49	-8.85
		6	52.3	0.81	12.734	10.95	-8.85
		5	43	0.76	11.948	10.28	-8.85
		4	33.7	0.7	11.005	9.46	-8.85
		3	24.3	0.7	11.005	9.46	-8.85
		2	15	0.7	11.005	9.46	-8.85
		1	1	0.7	11.005	9.46	-8.85

Table 7 - Wind Load Calculations at San Francisco

E-W Direction Tall Tower					
Floor	**Height (ft)**	**Height Below (ft)**	**Heigh Above (ft)**	**Trib Area (ft2)**	**Story Force (K)**
Penthouse	208.42	15.2	0	1236.52	20.61
Roof	194.25	10.33	15.2	2076.87	34.04
20	183.9	9.33	10.33	1599.34	25.77
19	174.6	9.33	9.33	1517.99	24.04
18	165.3	9.33	9.33	1517.99	23.62
17	155.9	9.33	9.33	1517.99	23.41
16	146.6	9.33	9.33	1517.99	23.00
15	137.2	9.33	9.33	1517.99	22.79
14	127.9	9.33	9.33	1517.99	22.37
13	118.6	9.33	9.33	1517.99	21.74
12	109.3	9.33	9.33	1517.99	20.91
11	99.9	9.33	9.33	1517.99	20.70
10	90.6	9.33	9.33	1517.99	20.07
9	81.3	10.25	9.33	1592.83	20.03
8	71	9.33	10.25	1592.83	19.17
7	61.7	9.33	9.33	1517.99	17.45
6	52.3	9.33	9.33	1517.99	16.62
5	43	9.33	9.33	1517.99	15.60
4	33.7	9.33	9.33	1517.99	14.37
3	24.3	9.33	9.33	1517.99	14.37
2	15	14	9.33	1897.90	17.96
1	1	1	14	1220.25	11.55
				Base Shear (K)	**450**
				Overturning moment (k ft)	**52227**

Table 8 - Wind Force Distribution at San Francisco

Tables 7 and 8 show a decrease in base shear from 505 kips to 450 kips due to wind in the critical direction. Seismic loads also needed to be recalculated using higher acceleration values obtained from ASCE7-05. Detailed calculation of the criteria can be found in appendix E, but table 9 summarizes the results. The base shear increased 120% from 165 kips to 362 kips.

Seismic Force Distribution (Tall Tower) N-S						
Floor	Height (ft)	Weight (k)	$(w_x h_x)^k$	C_{vx}	F_x (K)	Overturning Moment (k ft)
Penthouse	208.42	205	628517219	0.031	11.08	2308.94
Roof	194.25	458.8	2540799702	0.124	44.78	8699.35
20	183.9	467.1	2369103495	0.115	41.76	7679.29
19	174.6	466.5	2141421635	0.104	37.74	6590.25
18	165.3	466.5	1929907763	0.094	34.02	5622.96
17	155.9	466.8	1728845679	0.084	30.47	4750.70
16	146.6	467.1	1540044576	0.075	27.14	3979.45
15	137.2	467.8	1361718146	0.066	24.00	3293.04
14	127.9	468.5	1195094549	0.058	21.06	2694.19
13	118.6	469.5	1039605438	0.051	18.32	2173.25
12	109.3	470.5	893802573	0.043	15.75	1721.94
11	99.9	471.7	757076198	0.037	13.34	1333.09
10	90.6	472.8	631583484	0.031	11.13	1008.59
9	81.3	476.2	521160965	0.025	9.19	746.82
8	71	477.5	404986249	0.020	7.14	506.82
7	61.7	476.2	308561441	0.015	5.44	335.57
6	52.3	477.1	226209636	0.011	3.99	208.53
5	43	478.7	156931663	0.008	2.77	118.94
4	33.7	480.3	99396416	0.005	1.75	59.04
3	24.3	483	53969882	0.003	0.95	23.12
2	15	492	22351521	0.001	0.39	5.91
	Sum	9659.6	20551088230	Base Shear (K)		**362**
			Base Overturning moment (k ft)			**53860**

Table 9 - Seismic Force Distribution in Tall Tower

Results:

The idea behind moving the structure to California was to investigate how many more additional members would be required or upsized. Upon running the analysis as designed for the Baltimore area, several members needed 10-20 lbs/ft of additional weight, but nothing too drastic. It was confirmed that earthquake loads controlled in the North-South direction for strength and deflections. Unfortunately the structure was also found to still have a torsional irregularity. Upon performing the calculations found in figure23 the structure was found to have the horizontal irregularity 1a, but not extreme 1b as defined in figure 24.

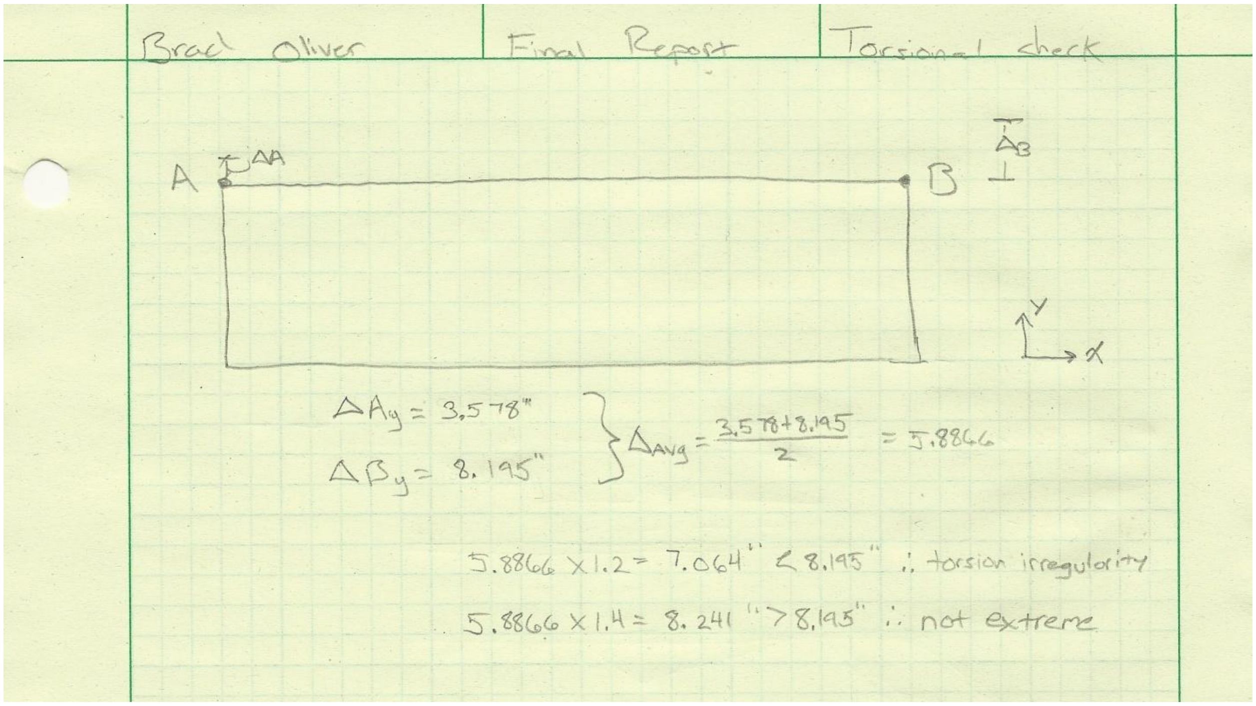


Figure 23 - Torsion Irregularity Check

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D

Figure 24 - ASCE Irregularity's

Due to the torsional irregularity, the story drifts were no longer permitted to be calculated at the center of mass, but at the point of largest displacement. The point "B" in figure 23 represents the point used for calculating story drifts and comparing them to the acceptable limits. The story drift ratio was within the acceptable limits as prescribed by ASCE 7-05, figure 25, and can be seen below in table 10.

Drift Ratios at Point B Including Accidental Torsion - Earthquake					
			N-S Loading		
Story	Height (in)	Allowable story Drift (inches)	Story Drift (inches)	Story Drift (inches) with Amplification	Compliant?
Roof	2484	2.64	0.6454	2.5816	ok
20	2352	2.4	0.5982	2.3928	ok
19	2232	2.4	0.5979	2.3916	ok
18	2112	2.4	0.5976	2.3904	ok
17	1992	2.4	0.5976	2.3904	ok
16	1872	2.4	0.5845	2.338	ok
15	1752	2.4	0.567	2.268	ok
14	1632	2.4	0.5422	2.1688	ok
13	1512	2.4	0.517	2.068	ok
12	1392	2.4	0.4848	1.9392	ok
11	1272	2.4	0.4538	1.8152	ok
10	1152	2.4	0.4172	1.6688	ok
9	1032	2.88	0.4532	1.8128	ok
8	888	2.4	0.33	1.32	ok
7	768	2.4	0.291	1.164	ok
6	648	2.4	0.2478	0.9912	ok
5	528	2.4	0.2067	0.8268	ok
4	408	2.4	0.1624	0.6496	ok
3	288	2.4	0.0191	0.0764	ok
2	168	3.36	0.0158	0.0632	ok
1	0	0	0	0	

Table 10 - Story Drift Ratio Check

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 25- ASCE Allowable Story Drift Ratio

Torsion was a very difficult issue to remove from this building due the height and geometry of the building. The center-of-mass and center-of-rigidity were within two feet of one another on every floor, but added up over so many floors caused significant torsion. With the building being advertised as a graduate level student housing, putting frames on the exterior of the building seemed detrimental to the views of tenants. Every apartment has large windows overlooking the city, but with frames on the exterior, that view would be ruined and a potential eye sore. With those limitations, the bracing in the North-South direction was close to the center of the building, which decreases the buildings ability to resist torsional shears. With fewer frames in the North-South direction, the most critical frame due to earthquake loads was found. It is denoted in figure 26 with a star and was checked to make sure it complied with the seismic provisions.

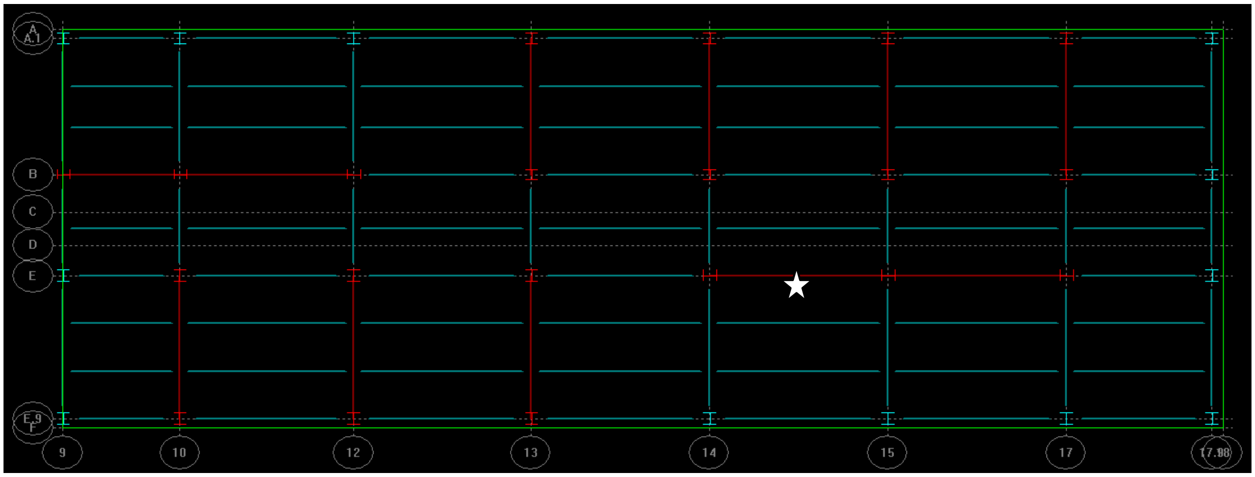


Figure 26 - Critical Frame

The details of these calculations can be viewed in appendix F. The rest of the frames were designed using Ram to expedite the design process. This design did need larger and more closely spaced web stiffeners and is summarized below visually in figure 27.

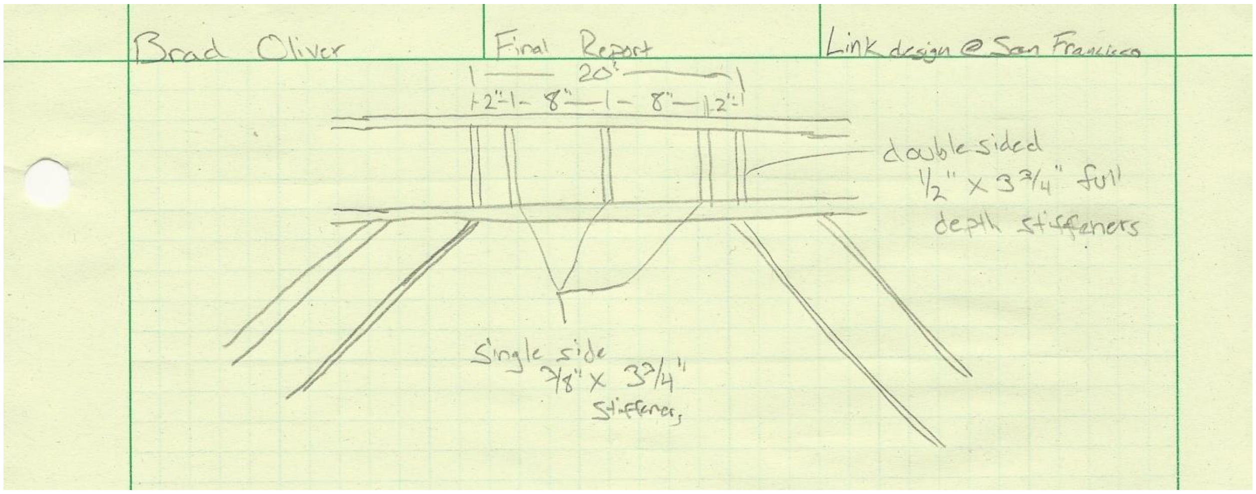
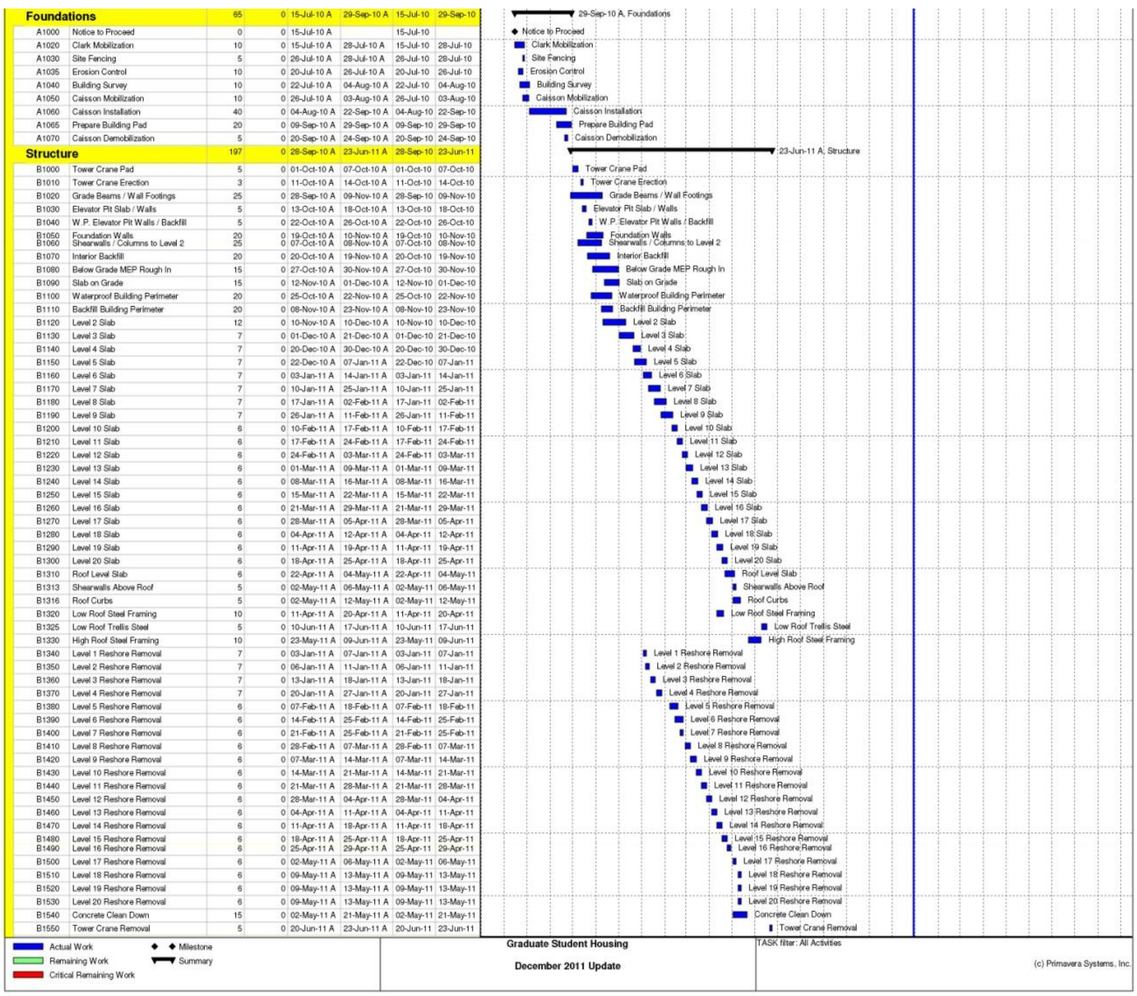


Figure 27 - Web Stiffeners at San Francisco

Construction Management Breadth –

It was predicted that a steel system would lead to a quicker schedule given the height and repetition of every floor. By performing a schedule and cost analysis, it was proved that not only was the schedule expedited, but it was also significantly cheaper to build. The original concrete structure including foundations began on July 15, 2010 and ended June 23, 2011. A copy of this existing schedule can be found in figure 28.

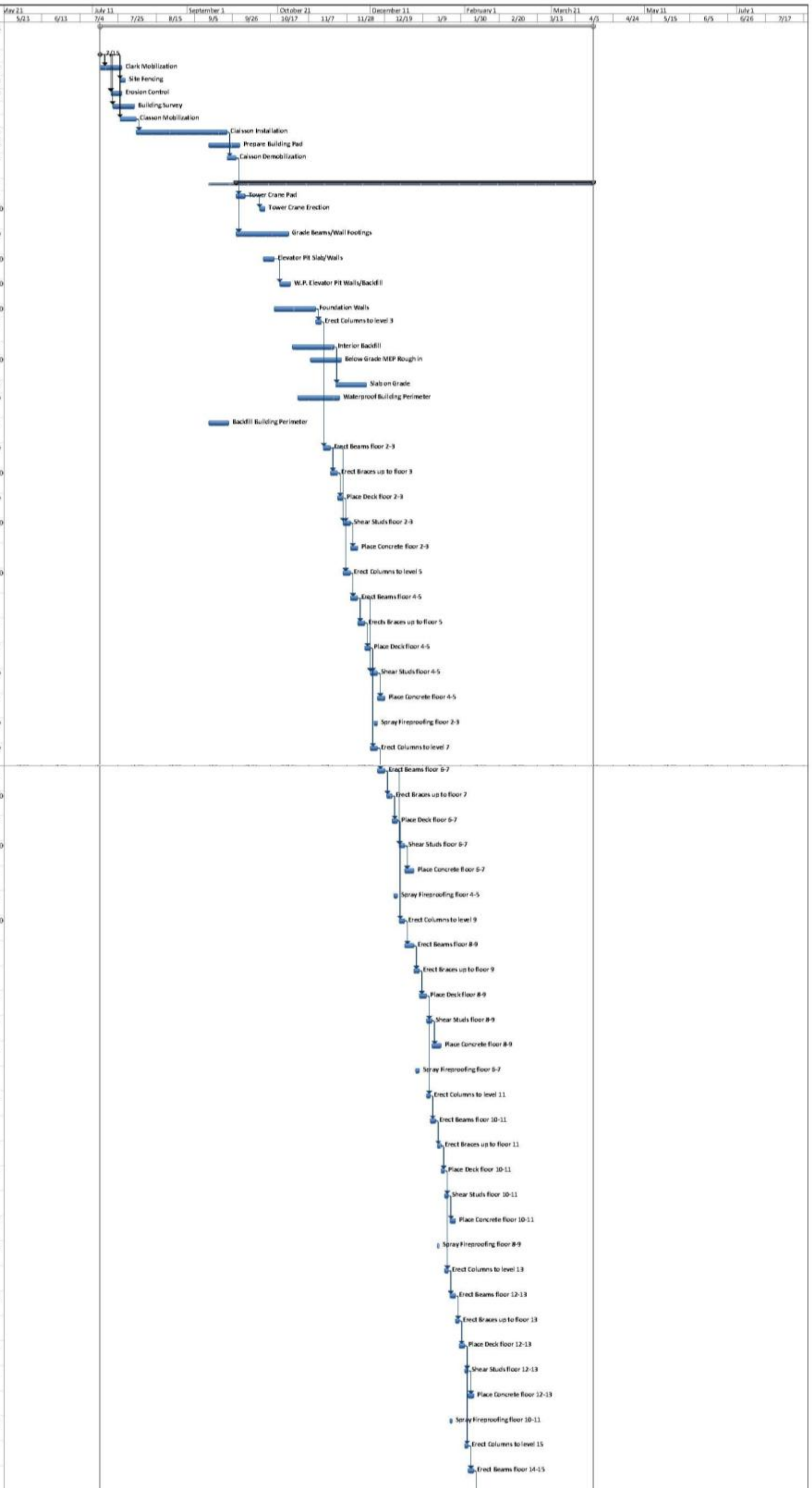


Foundations		65	0	15-Jul-10 A	29-Sep-10 A	15-Jul-10	29-Sep-10
A1000	Notice to Proceed	0	0	15-Jul-10 A		15-Jul-10	
A1020	Clark Mobilization	10	0	15-Jul-10 A	28-Jul-10 A	15-Jul-10	28-Jul-10
A1030	Site Fencing	5	0	26-Jul-10 A	28-Jul-10 A	26-Jul-10	28-Jul-10
A1035	Erosion Control	10	0	20-Jul-10 A	26-Jul-10 A	20-Jul-10	26-Jul-10
A1040	Building Survey	10	0	22-Jul-10 A	04-Aug-10 A	22-Jul-10	04-Aug-10
A1050	Caisson Mobilization	10	0	26-Jul-10 A	03-Aug-10 A	26-Jul-10	03-Aug-10
A1060	Caisson Installation	40	0	04-Aug-10 A	22-Sep-10 A	04-Aug-10	22-Sep-10
A1065	Prepare Building Pad	20	0	09-Sep-10 A	29-Sep-10 A	09-Sep-10	29-Sep-10
A1070	Caisson Demobilization	5	0	20-Sep-10 A	24-Sep-10 A	20-Sep-10	24-Sep-10
Structure		197	0	28-Sep-10 A	23-Jun-11 A	28-Sep-10	23-Jun-11
B1000	Tower Crane Pad	5	0	01-Oct-10 A	07-Oct-10 A	01-Oct-10	07-Oct-10
B1010	Tower Crane Erection	3	0	11-Oct-10 A	14-Oct-10 A	11-Oct-10	14-Oct-10
B1020	Grade Beams / Wall Footings	25	0	28-Sep-10 A	09-Nov-10 A	28-Sep-10	09-Nov-10
B1030	Elevator Pit Slab / Walls	5	0	13-Oct-10 A	18-Oct-10 A	13-Oct-10	18-Oct-10
B1040	W.P. Elevator Pit Walls / Backfill	5	0	22-Oct-10 A	26-Oct-10 A	22-Oct-10	26-Oct-10
B1050	Foundation Walls	20	0	19-Oct-10 A	10-Nov-10 A	19-Oct-10	10-Nov-10
B1060	Shearwalls / Columns to Level 2	25	0	07-Oct-10 A	08-Nov-10 A	07-Oct-10	08-Nov-10
B1070	Interior Backfill	20	0	20-Oct-10 A	19-Nov-10 A	20-Oct-10	19-Nov-10
B1080	Below Grade MEP Rough In	15	0	27-Oct-10 A	30-Nov-10 A	27-Oct-10	30-Nov-10
B1090	Slab on Grade	15	0	12-Nov-10 A	01-Dec-10 A	12-Nov-10	01-Dec-10
B1100	Waterproof Building Perimeter	20	0	25-Oct-10 A	22-Nov-10 A	25-Oct-10	22-Nov-10
B1110	Backfill Building Perimeter	20	0	08-Nov-10 A	23-Nov-10 A	08-Nov-10	23-Nov-10
B1120	Level 2 Slab	12	0	10-Nov-10 A	10-Dec-10 A	10-Nov-10	10-Dec-10
B1130	Level 3 Slab	7	0	01-Dec-10 A	21-Dec-10 A	01-Dec-10	21-Dec-10
B1140	Level 4 Slab	7	0	20-Dec-10 A	30-Dec-10 A	20-Dec-10	30-Dec-10
B1150	Level 5 Slab	7	0	22-Dec-10 A	07-Jan-11 A	22-Dec-10	07-Jan-11
B1160	Level 6 Slab	7	0	03-Jan-11 A	14-Jan-11 A	03-Jan-11	14-Jan-11
B1170	Level 7 Slab	7	0	10-Jan-11 A	25-Jan-11 A	10-Jan-11	25-Jan-11
B1180	Level 8 Slab	7	0	17-Jan-11 A	02-Feb-11 A	17-Jan-11	02-Feb-11
B1190	Level 9 Slab	7	0	26-Jan-11 A	11-Feb-11 A	26-Jan-11	11-Feb-11
B1200	Level 10 Slab	6	0	10-Feb-11 A	17-Feb-11 A	10-Feb-11	17-Feb-11
B1210	Level 11 Slab	6	0	17-Feb-11 A	24-Feb-11 A	17-Feb-11	24-Feb-11
B1220	Level 12 Slab	6	0	24-Feb-11 A	03-Mar-11 A	24-Feb-11	03-Mar-11
B1230	Level 13 Slab	6	0	01-Mar-11 A	09-Mar-11 A	01-Mar-11	09-Mar-11
B1240	Level 14 Slab	6	0	08-Mar-11 A	16-Mar-11 A	08-Mar-11	16-Mar-11
B1250	Level 15 Slab	6	0	15-Mar-11 A	22-Mar-11 A	15-Mar-11	22-Mar-11
B1260	Level 16 Slab	6	0	21-Mar-11 A	29-Mar-11 A	21-Mar-11	29-Mar-11
B1270	Level 17 Slab	6	0	28-Mar-11 A	05-Apr-11 A	28-Mar-11	05-Apr-11
B1280	Level 18 Slab	6	0	04-Apr-11 A	12-Apr-11 A	04-Apr-11	12-Apr-11
B1290	Level 19 Slab	6	0	11-Apr-11 A	19-Apr-11 A	11-Apr-11	19-Apr-11
B1300	Level 20 Slab	6	0	18-Apr-11 A	25-Apr-11 A	18-Apr-11	25-Apr-11
B1310	Roof Level Slab	6	0	22-Apr-11 A	04-May-11 A	22-Apr-11	04-May-11
B1313	Shearwalls Above Roof	5	0	02-May-11 A	06-May-11 A	02-May-11	06-May-11
B1316	Roof Curbs	5	0	02-May-11 A	12-May-11 A	02-May-11	12-May-11
B1320	Low Roof Steel Framing	10	0	11-Apr-11 A	20-Apr-11 A	11-Apr-11	20-Apr-11
B1325	Low Roof Trellis Steel	5	0	10-Jun-11 A	17-Jun-11 A	10-Jun-11	17-Jun-11
B1330	High Roof Steel Framing	10	0	23-May-11 A	09-Jun-11 A	23-May-11	09-Jun-11
B1340	Level 1 Reshore Removal	7	0	03-Jan-11 A	07-Jan-11 A	03-Jan-11	07-Jan-11
B1350	Level 2 Reshore Removal	7	0	06-Jan-11 A	11-Jan-11 A	06-Jan-11	11-Jan-11
B1360	Level 3 Reshore Removal	7	0	13-Jan-11 A	18-Jan-11 A	13-Jan-11	18-Jan-11
B1370	Level 4 Reshore Removal	7	0	20-Jan-11 A	27-Jan-11 A	20-Jan-11	27-Jan-11
B1380	Level 5 Reshore Removal	6	0	07-Feb-11 A	18-Feb-11 A	07-Feb-11	18-Feb-11
B1390	Level 6 Reshore Removal	6	0	14-Feb-11 A	25-Feb-11 A	14-Feb-11	25-Feb-11
B1400	Level 7 Reshore Removal	6	0	21-Feb-11 A	25-Feb-11 A	21-Feb-11	25-Feb-11
B1410	Level 8 Reshore Removal	6	0	28-Feb-11 A	07-Mar-11 A	28-Feb-11	07-Mar-11
B1420	Level 9 Reshore Removal	6	0	07-Mar-11 A	14-Mar-11 A	07-Mar-11	14-Mar-11
B1430	Level 10 Reshore Removal	6	0	14-Mar-11 A	21-Mar-11 A	14-Mar-11	21-Mar-11
B1440	Level 11 Reshore Removal	6	0	21-Mar-11 A	28-Mar-11 A	21-Mar-11	28-Mar-11
B1450	Level 12 Reshore Removal	6	0	28-Mar-11 A	04-Apr-11 A	28-Mar-11	04-Apr-11
B1460	Level 13 Reshore Removal	6	0	04-Apr-11 A	11-Apr-11 A	04-Apr-11	11-Apr-11
B1470	Level 14 Reshore Removal	6	0	11-Apr-11 A	18-Apr-11 A	11-Apr-11	18-Apr-11
B1480	Level 15 Reshore Removal	6	0	18-Apr-11 A	25-Apr-11 A	18-Apr-11	25-Apr-11
B1490	Level 16 Reshore Removal	6	0	25-Apr-11 A	29-Apr-11 A	25-Apr-11	29-Apr-11
B1500	Level 17 Reshore Removal	6	0	02-May-11 A	06-May-11 A	02-May-11	06-May-11
B1510	Level 18 Reshore Removal	6	0	09-May-11 A	13-May-11 A	09-May-11	13-May-11
B1520	Level 19 Reshore Removal	6	0	09-May-11 A	13-May-11 A	09-May-11	13-May-11
B1530	Level 20 Reshore Removal	6	0	09-May-11 A	13-May-11 A	09-May-11	13-May-11
B1540	Concrete Clean Down	15	0	02-May-11 A	21-May-11 A	02-May-11	21-May-11
B1550	Tower Crane Removal	5	0	20-Jun-11 A	23-Jun-11 A	20-Jun-11	23-Jun-11

Figure 28 - Existing Schedule

A new schedule was made using Microsoft Project starting also starting on July 15, 2010. Using durations obtained from RS means and takeoffs from Ram, the new schedule lasted until April 12, 2011, causing a time savings of over two months. The original schedule was drawn up to have an entire level poured in seven days, but would often take over two weeks in the middle of winter. Pouring concrete during cold days is often difficult and sometimes impossible to work through, which seemed to be the case on this project. A copy of the new schedule can be found in figure 29.

ID	Task Name	Duration	Start	Finish
0	**Foundations and structure**	**195 days**	**Thu 7/15/10**	**Tue 4/12/11**
1	Notice to Proceed	0 days	Thu 7/15/10	Thu 7/15/10
2	Clark Mobilization	10 days	Thu 7/15/10	Mon 7/26/10
3	Site Fencing	3 days	Mon 7/26/10	Wed 7/28/10
4	Erosion Control	5 days	Wed 7/21/10	Mon 7/26/10
5	Building Survey	10 days	Thu 7/22/10	Mon 8/2/10
6	Ciasson Mobilization	8 days	Mon 7/26/10	Tue 8/3/10
7	Ciaisson Installation	43 days	Wed 8/4/10	Wed 9/22/10
8	Prepare Building Pad	15 days	Mon 9/13/10	Wed 9/29/10
9	Caisson Demobilization	4 days	Thu 9/23/10	Mon 9/27/10
10	**Structure**	**141 days**	**Tue 9/28/10**	**Tue 4/12/11**
11	Tower Crane Pad	5 days	Tue 9/28/10	Sat 10/2/10
12	Tower Crane Erection	3 days	Mon 10/11/10	Wed 10/13/10
13	Grade Beams/Wall Footings	25 days	Tue 9/28/10	Tue 10/26/10
14	Elevator Pit Slab/Walls	5 days	Wed 10/13/10	Mon 10/18/10
15	W.P. Elevator Pit Walls/Backfill	5 days	Fri 10/22/10	Wed 10/27/10
16	Foundation Walls	20 days	Tue 10/19/10	Wed 11/10/10
17	Erect Columns to level 3	3 days	Thu 11/11/10	Sat 11/13/10
18	Interior Backfill	20 days	Fri 10/29/10	Sat 11/20/10
19	Below Grade MEP Rough in	15 days	Mon 11/8/10	Wed 11/24/10
20	Slab on Grade	15 days	Mon 11/22/10	Wed 12/8/10
21	Waterproof Building Perimeter	20 days	Mon 11/1/10	Tue 11/23/10
22	Backfill Building Perimeter	10 days	Mon 9/13/10	Thu 9/23/10
23	Erect Beams floor 2-3	4 days	Mon 11/15/10	Thu 11/18/10
24	Erect Braces up to floor 3	3 days	Fri 11/19/10	Mon 11/22/10
25	Place Deck floor 2-3	3 days	Tue 11/23/10	Thu 11/25/10
26	Shear Studs floor 2-3	3 days	Fri 11/26/10	Mon 11/29/10
27	Place Concrete floor 2-3	4 days	Tue 11/30/10	Fri 12/3/10
28	Erect Columns to level 5	3 days	Fri 11/26/10	Mon 11/29/10
29	Erect Beams floor 4-5	4 days	Tue 11/30/10	Fri 12/3/10
30	Erects Braces up to floor 5	3 days	Sat 12/4/10	Tue 12/7/10
31	Place Deck floor 4-5	3 days	Wed 12/8/10	Fri 12/10/10
32	Shear Studs floor 4-5	3 days	Sat 12/11/10	Tue 12/14/10
33	Place Concrete floor 4-5	4 days	Wed 12/15/10	Sat 12/18/10
34	Spray Fireproofing floor 2-3	2 days	Mon 12/13/10	Tue 12/14/10
35	Erect Columns to level 7	3 days	Sat 12/11/10	Tue 12/14/10
36	Erect Beams floor 6-7	4 days	Wed 12/15/10	Sat 12/18/10
37	Erect Braces up to floor 7	3 days	Mon 12/20/10	Wed 12/22/10
38	Place Deck floor 6-7	3 days	Thu 12/23/10	Sat 12/25/10
39	Shear Studs floor 6-7	3 days	Mon 12/27/10	Wed 12/29/10
40	Place Concrete floor 6-7	4 days	Thu 12/30/10	Mon 1/3/11
41	Spray Fireproofing floor 4-5	2 days	Fri 12/24/10	Sat 12/25/10
42	Erect Columns to level 9	3 days	Mon 12/27/10	Wed 12/29/10
43	Erect Beams floor 8-9	4 days	Thu 12/30/10	Mon 1/3/11
44	Erect Braces up to floor 9	3 days	Tue 1/4/11	Thu 1/6/11
45	Place Deck floor 8-9	3 days	Fri 1/7/11	Mon 1/10/11
46	Shear Studs floor 8-9	3 days	Tue 1/11/11	Thu 1/13/11
47	Place Concrete floor 8-9	4 days	Fri 1/14/11	Tue 1/18/11
48	Spray Fireproofing floor 6-7	2 days	Wed 1/5/11	Thu 1/6/11
49	Erect Columns to level 11	2 days	Tue 1/11/11	Wed 1/12/11
50	Erect Beams floor 10-11	3 days	Thu 1/13/11	Sat 1/15/11
51	Erect Braces up to floor 11	2 days	Mon 1/17/11	Tue 1/18/11
52	Place Deck floor 10-11	2 days	Wed 1/19/11	Thu 1/20/11
53	Shear Studs floor 10-11	2 days	Fri 1/21/11	Sat 1/22/11
54	Place Concrete floor 10-11	3 days	Mon 1/24/11	Wed 1/26/11
55	Spray Fireproofing floor 8-9	1 day	Mon 1/17/11	Mon 1/17/11
56	Erect Columns to level 13	2 days	Fri 1/21/11	Sat 1/22/11
57	Erect Beams floor 12-13	3 days	Mon 1/24/11	Wed 1/26/11
58	Erect Braces up to floor 13	2 days	Thu 1/27/11	Fri 1/28/11
59	Place Deck floor 12-13	2 days	Sat 1/29/11	Mon 1/31/11
60	Shear Studs floor 12-13	2 days	Tue 2/1/11	Wed 2/2/11
61	Place Concrete floor 12-13	3 days	Thu 2/3/11	Sat 2/5/11
62	Spray Fireproofing floor 10-11	1 day	Mon 1/24/11	Mon 1/24/11
63	Erect Columns to level 15	2 days	Tue 2/1/11	Wed 2/2/11
64	Erect Beams floor 14-15	3 days	Thu 2/3/11	Sat 2/5/11



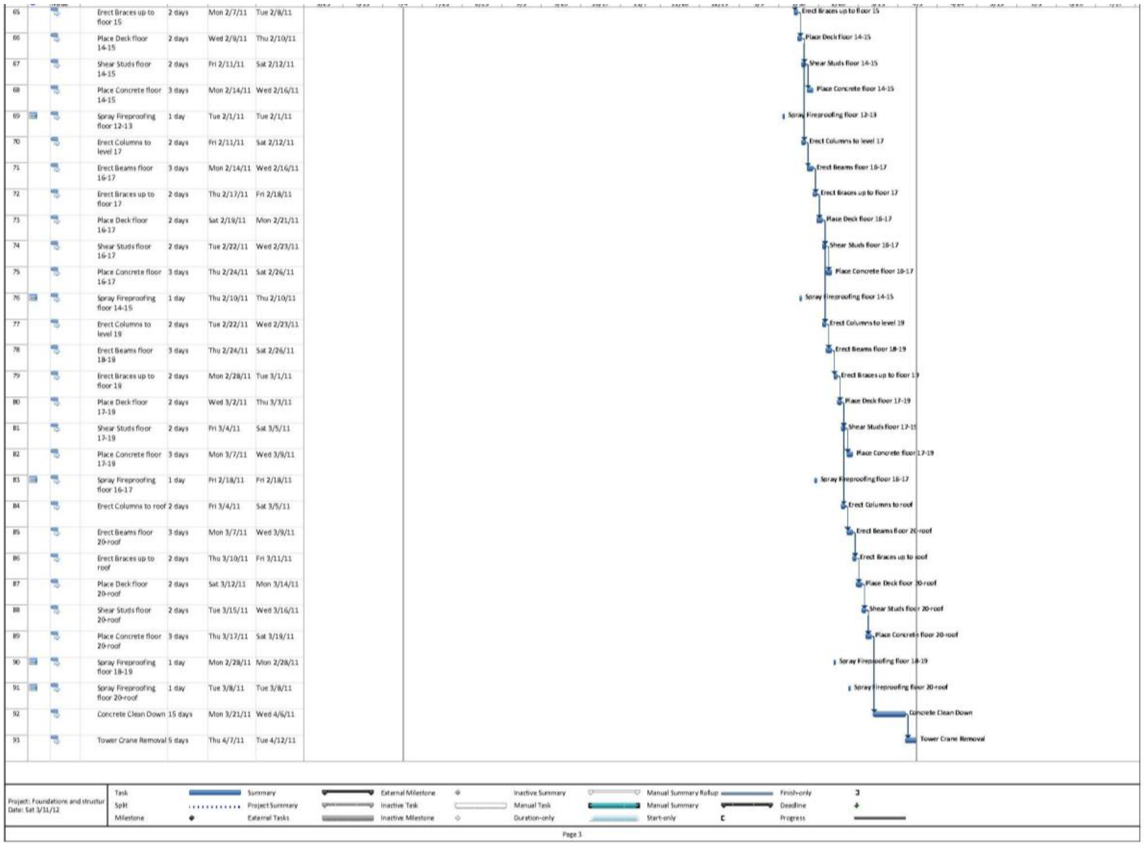


Figure 29 - New Schedule Using Steel

Using RS Means, a detailed cost estimate was also done to compare the two structures. Information on the members, crews, decking, fireproofing, and concrete were put into an excel sheet. For steel members, the default equipment was a 90 ton lattice boom crane which would be insufficient for this project. Information for a tower crane was found and prices were adjusted to reflect this change. Next, takeoffs were taken using Ram including length and weight. For sizes that RS Means didn't have, values were interpolated. The final steps were multiplying the prices by the lengths for each element. RS Means said to estimate the price of connections, it is permitted to take 10% of the weight of steel as a rule of thumb estimate, which was incorporated in the calculations. Column splices were also taken into account by estimating 500 lbs of steel for each splice. A copy of the spreadsheet used to calculate these numbers can be found in appendix G.

The original concrete structure cost $5.75 million, and the new system cost approximately $4.367 million. A savings of $1.38 million was achieved by switching to a composite steel design. Since the final height of the new structure was within three feet of the original, the additional cost of the façade was considered negligible. Moving the structure to San Francisco resulted in some heavier members, particularly in the columns. The largest difference in cost of the move would be seen in the connections. Due to the dual system and seismic design category D, the moment connections would need to be capable of resisting 25% of the lateral loads. San Francisco would also be more willing to weld the connections which would results in a slightly longer schedule and higher costs. Another source of cost increase that wasn't estimated in this report is the connections between the diaphragm and lateral members. According to ASCE, a torsionally irregular building in seismic design category D must have the forces on those connections increased by 20%. A more detailed analysis of connections would need to be done in order to truly say the steel system is cheaper.

Architecture –

When designing the structure, the architecture was kept in mind throughout the process. Columns were added along walls or moved to an area that could minimize the impact of the functionality and aesthetics of the space. Due to the number of frames, not all areas could be preserved perfectly and this study focused on two spaces, the lounge and fitness area. Located on the 9th floor near the edge of the building as indicated in figure 30, frames were being designed to cut through the middle of open floor plans as indicated in blue. A Revit model was made of the areas as they were currently designed, and then another model was made indicating the changes so they could be compared.

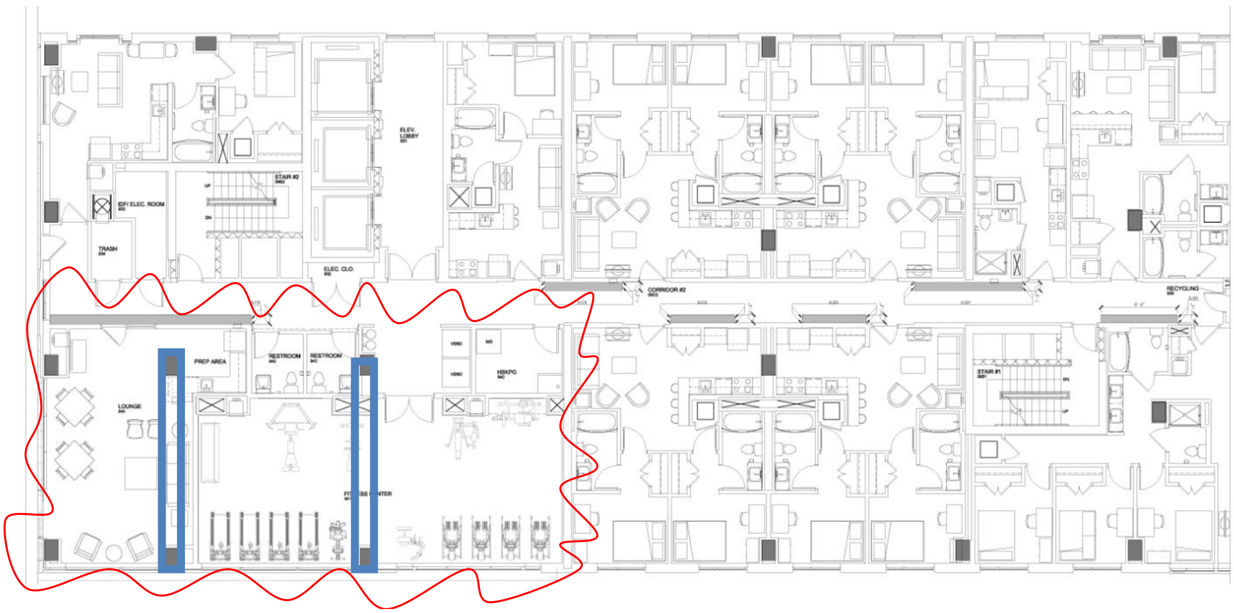


Figure 30 - Floor Plan With Planned Frame Location

Starting with the fitness room, the original design calls for one large open room with cardio equipment along the windows, and weight machines along the walls. The lounge was designed to allow for plenty of seating space, access to the green roof, and a place to relax. Figures 31, 32, and 33 show the models to gain a visual representation of the space.

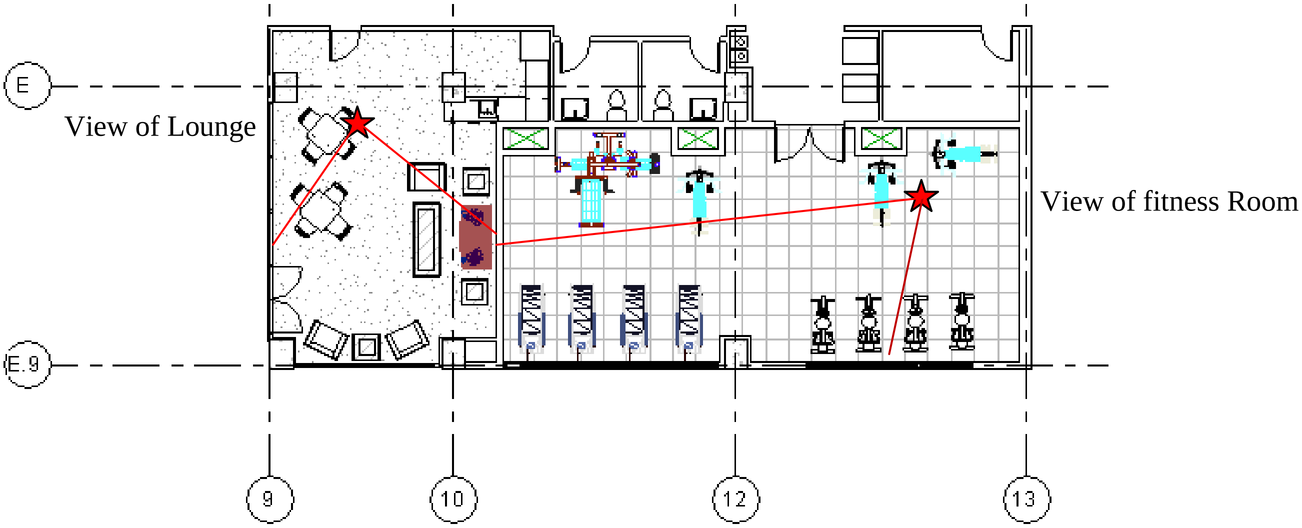


Figure 31 - Plan View of Existing

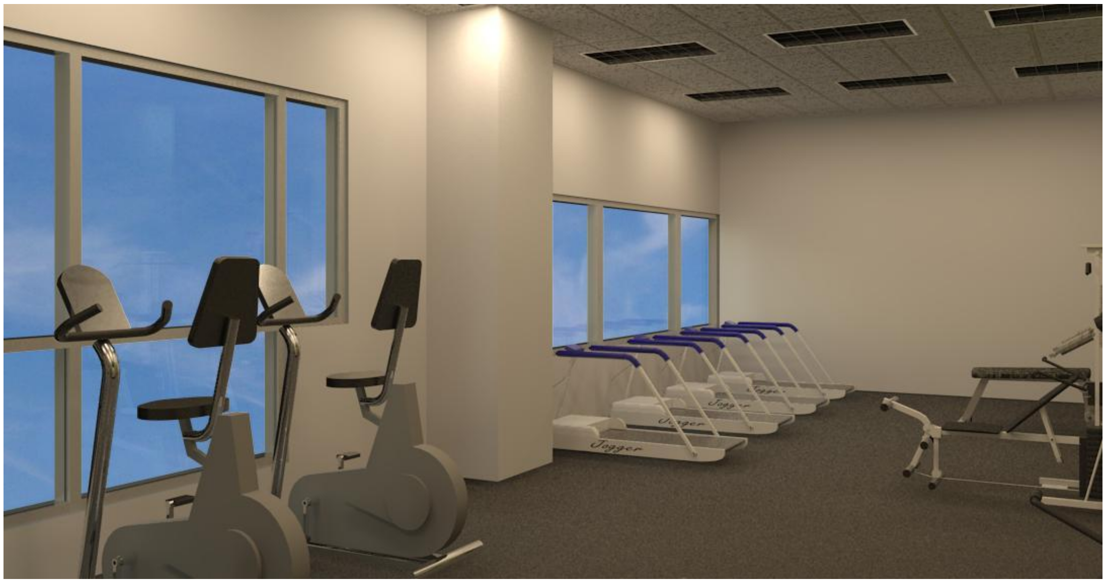


Figure 32 - Rendering of Fitness Room

Figure 33 - Rendering of Lounge

Placing the new frames as designed caused issues in both rooms. To address the issues at hand it was decided to expand the size of the fitness room to reach column line 10. This would allow for best flow of people in both spaces. A large opening was cut into the wall of the fitness room to make it appear open, but two separate distinct spaces. These spaces could be better utilized by converting one into a cardio room with the other being a weight room. This distinction of spaces along with additional wall space ended creating room for more equipment. The lounge ended up getting the short end of the stick however. One set of table and chairs were removed in order to prevent the space from becoming cluttered. Once the table was removed, the space has a similar feel as it did before, just with four less seats. This was deemed an acceptable tradeoff in order to keep the frames as designed. Figures 34, 35, 36, and 37 show what these spaces look like after the modifications.

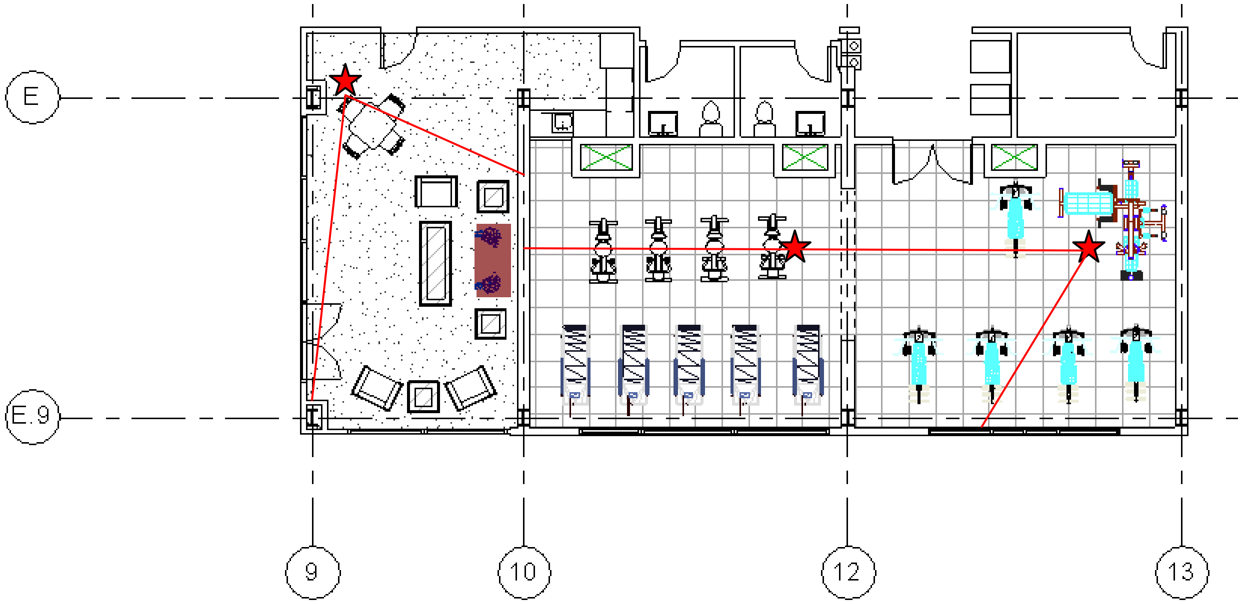


Figure 34 - Plan of New Layout

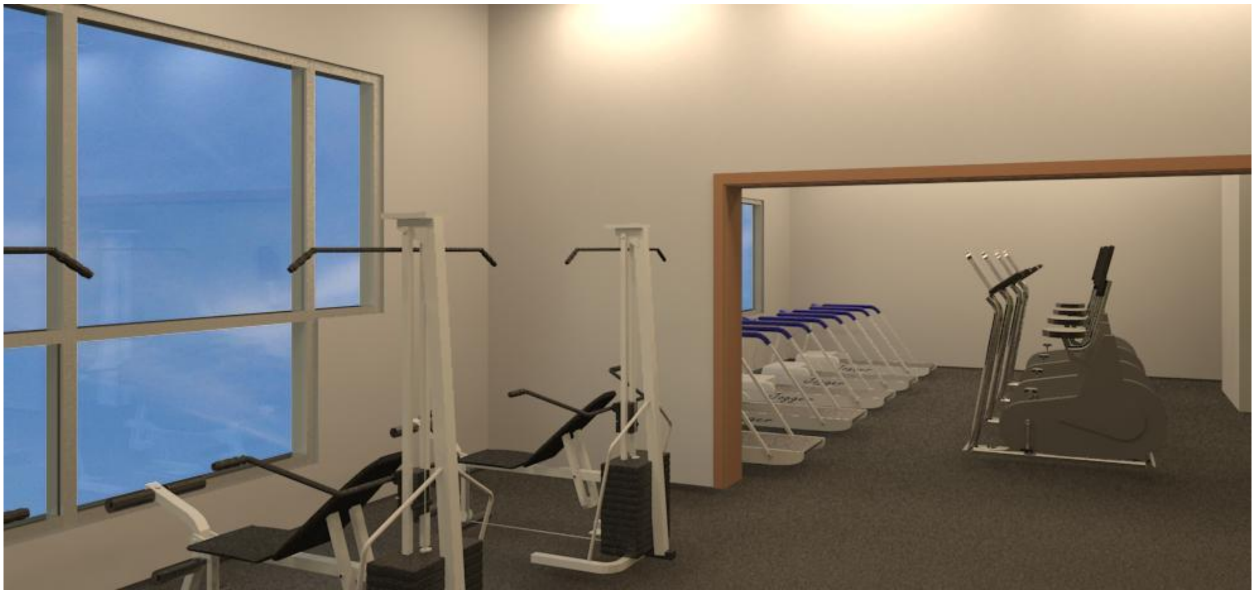


Figure 35 - View 1 Cardio Room from Weight Room

Figure 36 - View of Weight Room from Cardio Room

Figure 37 - View of New Smaller Lounge

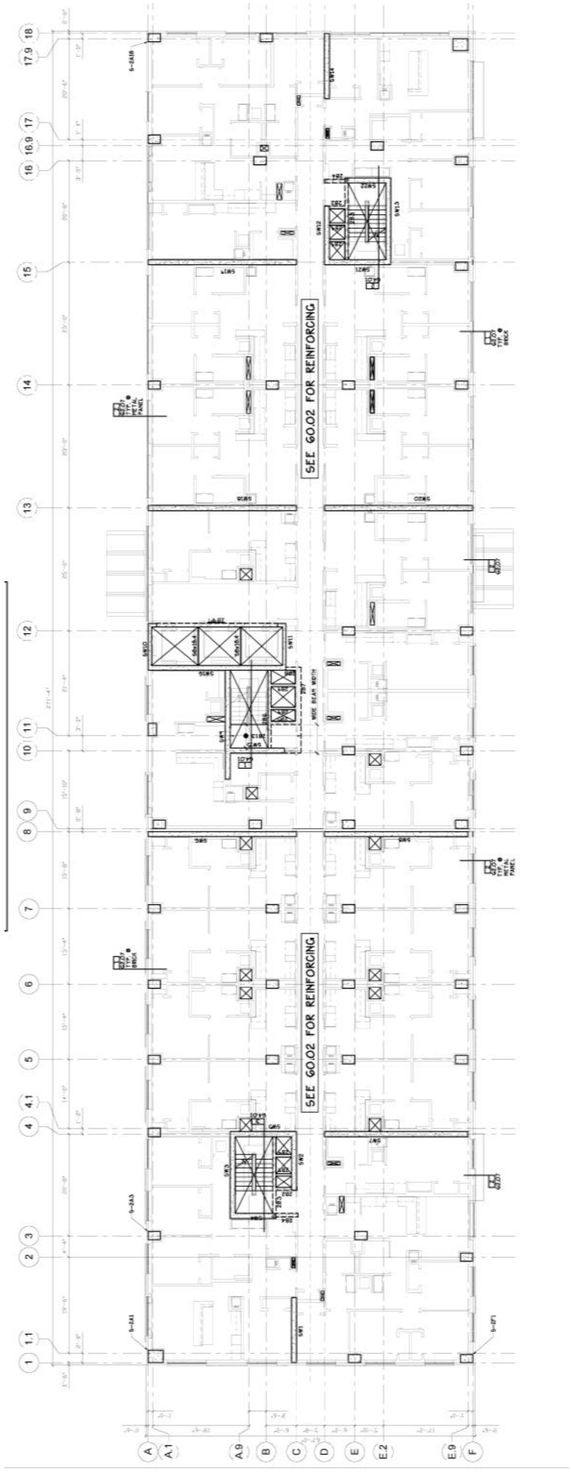
Conclusion –

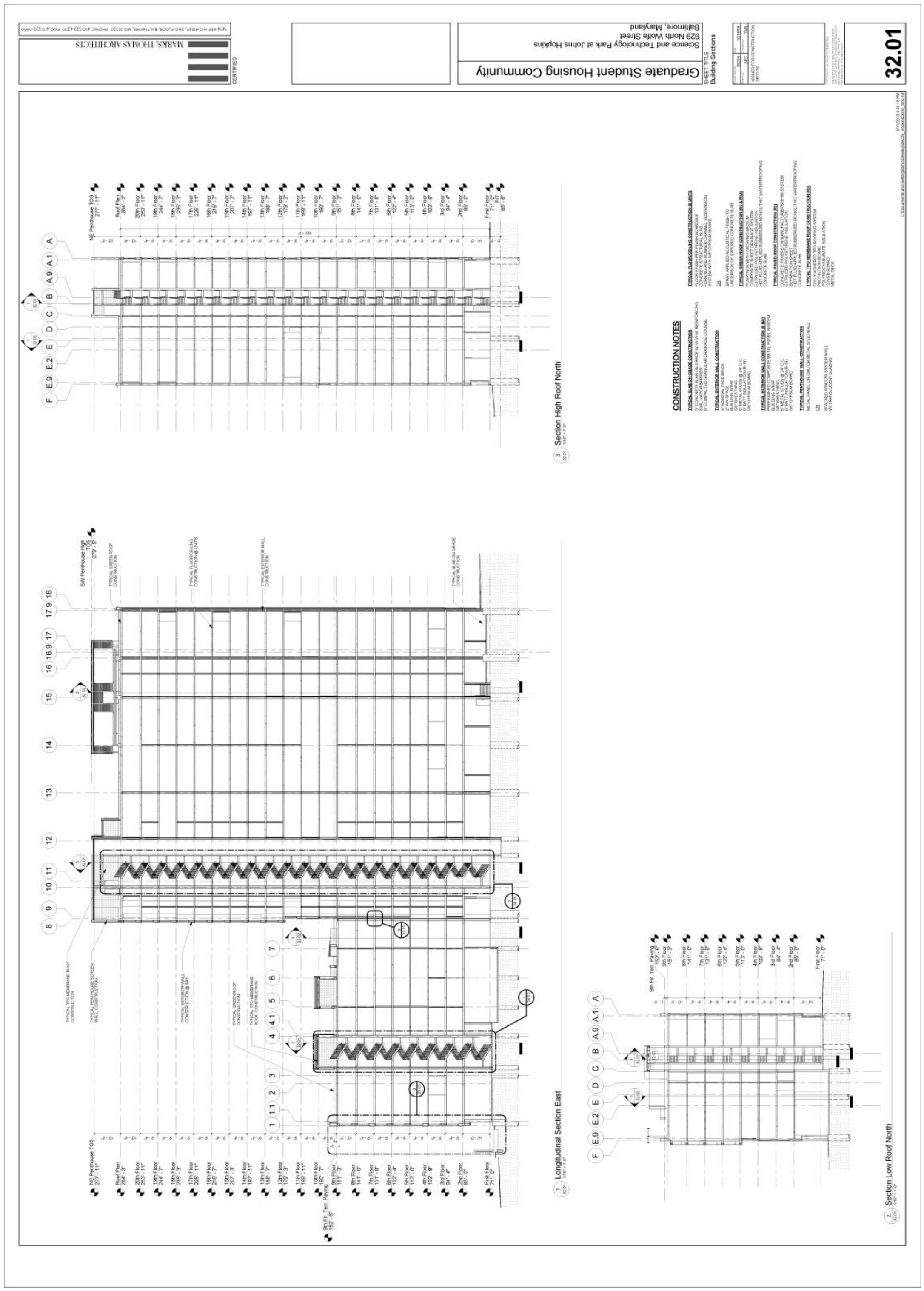
Eccentrically braced frames were successfully designed at the Baltimore and San Francisco locations. Due to the high wind loads in the East-West direction, the design was primarily driven by serviceability requirements. Using C-Braces in the design resulted in deflections of 27 inches. The design ended up utilizing W14X43 braces and W14X48 beams for the link elements. The overall building height was kept within three feet of the original structure, which was a design goal so that the wind loads and façade cost would be comparable. Once the structure was moved to San Francisco, the structure was found to be torsionally irregular. This report didn't involve the design of connections, but this means when they are designed, the forces must be increased by 20%, resulting in a significant cost increase.

Without the detailed connection cost and using the 10% weight estimate, the building was estimated to cost approximately $4.37 million. This results in a saving of $1.38 million. A schedule was also created using RS Means as a guide as well as takeoffs. When the new schedule was compared to the original schedule, there was a time savings of over two months. When analyzed closely, most of the time was made up during winter when steel could be erected, but concrete could not be casted.

Throughout the entire design process the architecture was kept in mind. Columns were placed near walls or moved to locations where the impact would be negligible. The public lounge and fitness rooms were chosen to be studied more closely because the re-design called for braced frames running through the middle of both rooms. A Revit model was made of both rooms for the existing rooms and rendered. Once the new frames were in place, the fitness room dimensions were increased and separated into two rooms. The lounge was decreased in size, but maintained its functionality. A model and renderings were also created of the new design.

Appendix A – Existing Drawings





Appendix B – Composite Steel Beams Calculations

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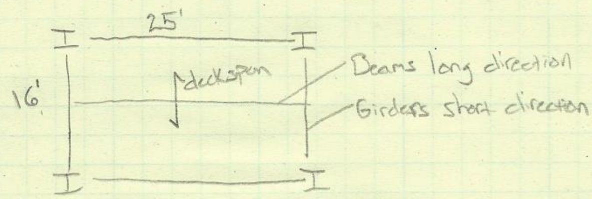
Typical Bay in Tall Tower - Interior



Assume 8' spacing of beams, Normal weight 5000 psi concrete

Interior Loads - corridor - LL - 100 psf not reduceable

Load case 1.2D + 1.6L

decking for 2 hour fire rating - $\boxed{\text{2VLI deck } 4\frac{1}{2}\text{" topping, gage 22}}$

Max unshored construction span - 8'10" > 8' ✓

@ 8' Max LL allowable - 129 psf > 100
weight - 39 psf + 1.6 psf for deck = 40 psf

Beam - DL - self weight + 8 psf SDL + 40 psf = 48 psf + self
LL - 100 psf not reduceable

$W_u = 1.2(48) + 1.6(100) = 218 \text{ psf} \times 8' = 1741 \text{ lb/ft}$

$b' < \text{span}/8 = \frac{25 \times 12}{8} = 37.5"$ controls

$b' < 1/2 \text{ dist to Adj beam} = 1/2(8 \times 12) = 48"$

$b_{eff} = 75"$

Find minimum I

$\Delta_{TL} < L/240 = 25 \times 12/240 = 1.25 > \frac{5 w_{TL} L^4}{384EI} = \frac{5(148.2)(25)^4(1728)}{384(29000)I}$ $I > 287 \text{ in}^4$

$\Delta_{LL} < L/360 = 25 \times 12/360 = .833 > \frac{5(1.8)(25^4)(1728)}{384(29000)I}$ $I > 291 \text{ in}^4$ controls

Try W 10x22 $y_2 = 3.5$ $I_{LB} = 312 \text{ in}^4 > 291 \text{ in}^4$

$M_u = 1.741(25^2)/8 = 136'K$ $\phi M_n = 181'K > 136'K$ ✓

Check Assumption

$a = 221/.85(5)(75) = .69$ y_2 is actually $4 - \frac{.69}{2} = 3.65 > 3.5$ ✓

Studs - $221/17.2 = 12.8 \Rightarrow 26$ studs/beam > 25' Not ok
2 studs/rib $221/14.6 = 15.13 \Rightarrow 32$ studs/beam < 30 Okay

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Try W10X19 with $y_2 = 3.5$ as assumption. $I_{LB} = 305\ in^4 > 291\ in^4$

$\Phi M_n = 182'K > 136'K$

Verify Assumption - $a = 281/.85(5)(75) = .88$

$y_2 = 4 - .88/2 = 3.56 > 3.5 \therefore OK$

$\Sigma Q_n = 281$ Kips

strength for stud assuming weak position, ~ 17.2K

$281/17.2 = 16.3 \Rightarrow 34$ studs/beam > 25 No Good

2 studs/rib strength - 14.6K

$281/14.6 = 19.2 \Rightarrow 40$ studs/beam < 50 OK

Try W10x26 with $y_2 = 3.5$ $I_{LB} = 334\ in^4 > 291\ in^4$

$\Phi M_n = 195'K > 136'K$

Verify Assump- $a = 190/.85(5)(75) = .596$

$y_2 = 4 - .596/2 = 3.7 > 3.5$

$\Sigma Q_n = 190K$ Studs- $190/17.2 = 11.04 \Rightarrow 24$ Studs/beam

Weight comparisons- $\boxed{10x22\ w/32\ \text{studs} - 22(25) + 32(10) = 870\ lbs}$ ☆

10x19 w/ 40 studs $19(25) + 40(10) = 875$ lbs

10x26 w/ 24 studs $26(25) + 24(10) = 890$ lbs

Check Beam for unshored strength- DL only $1.4(40 \cdot 8) + 1.4(22) = 479$ lb/ft

Const loads $1.2(40 \cdot 8) + 1.2(22) + 1.6(20 \cdot 8) = 666$ lb/ft ☆

$M_u = .666(25^2)/8 = 52'K$

ΦM_n of Beam $= 97.5'K > 52'K$

check wet concrete deflection Service loads $39(8) + 22 = 334$ lb/ft

$I_{beam} = 118\ in^4$

$$\Delta_{wc} = \frac{5(.334)(25^4)(1728)}{384(29000)(118)} = .857''$$

$\Delta_{Allow} = L/240 = 25 \cdot 12/240 = 1.25'' > .857''$

Total load deflection including self weight $.148\ k/ft^2(8) + .022 = 1.206$ k/ft

$$\Delta = \frac{5(1.206)(25^4)(1728)}{384(29000)(305)} = 1.2'' < 1.25''\ \text{Allowable}$$ ✓

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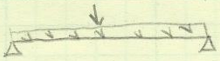
Girder Span 16'



$P = 1.206\ ^{k}/_{ft}(25') = 30.15K$
$P_u = (1741\ ^{lb}/_{ft} + 22 \cdot 1.2) \cdot 25 = 44K$
W_u = self weight

$\Delta_{LL\ Allow} = {}^{16 \cdot 12}/_{360} = .533''$ $\quad P_{Live} = 100(8)(25) = 20{,}000$ lbs $= 20K$

$\Delta = \frac{20(16^3)(1728)}{48(29000)I} < .533'$ $\quad I > 191\ in^4$

$\Delta_{TL\ Allow} = {}^{16 \cdot 12}/_{240} = .8''$

$\Delta = \frac{30(16^3)(1728)}{48(29000)I} < .8''$ $\quad I > 191\ in^4$ ✱ controls b/c doesn't include self weight yet.

$b' < {}^{16 \cdot 12}/_{8} = 24''$ ✱ $\quad b_{eff} = 48''$
$< {}^{1}/_{2}(25 \cdot 12) = 150''$

$M_u = {}^{P_uL}/_{4} = {}^{44(16)}/_{4} = 176'K$

Try W10X22 w/ $y_2 = 3$ as assumption $I_{LB} = 312\ in^4 > 191\ in^4$

$\phi M_n = 185'K$ $\quad M_u = 176 + \frac{1.2(.022)(16^2)}{8} = 177K < 185$ ✓

Check Assumption

$a = {}^{273}/_{.85(5)(48)} = 1.34''$
$y_2 = 4 - {}^{1.34}/_{2} = 3.33 > 3''$ ✓

Stud Str - 18.3K b/c // ribs
Studs - ${}^{273}/_{183} = 14.9 \Rightarrow 30$ studs/girder < 64 max studs ✓

Try W10X26 w/ $y_2 = 3$ as assumption $I_{LB} = 313\ in^4 > 191\ in^4$

$\phi M_n = 188'K$ $\quad M_u = 176 + \frac{1.2(.026)(16^2)}{8} = 177'K < 188'K$ ✓

Check Assumption

$a = {}^{190}/_{.85(5)(48)} = .931$
$y_2 = 4 - ({}^{.931}/_{2}) = 3.53 > 3$ ✓

Stud Strength - 18.3K
${}^{190}/_{18.3} = 10.3 \Rightarrow 22$ studs/girder

Weight comparisons
W10X22 - 22x16 + 30(10) = 652 lbs

$\boxed{\text{W 10x26 - 26x16 + 22(10) = 636 lbs ✱}}$

Check Beam for unshored strength

DL only - $P = 479(25) = 12K$ $\quad w = 1.4(26) = 36\ ^{lb}/_{ft}$
Const loads - $P = 666(25) = 16.7K$ $\quad w = 1.2(26) = 31.2\ ^{lb}/_{ft}$

$M_u = \frac{12(16)}{4} + \frac{36(16^2)}{8} = 49'K$
$M_u = \frac{16.7(16)}{4} + \frac{31.2(16^2)}{8} = 67.8'K < \phi_b M_p = 117'K$ ✓

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Check Wet concrete deflection

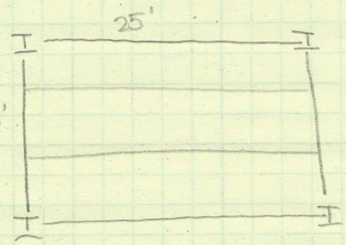
Service loads - $P = 334(25) = 8.4K$ $W = .026\ K/ft$ $I = 144\ in^4$

$$\Delta = \frac{8.4(16^3)(1728)}{48(29000)(144)} + \frac{5(.026)(16^4)(1728)}{384(29000)(144)} = .31 < .8'' \checkmark$$

Check Total load Δ w/ self weight

$$\Delta = \frac{(25)(1.206)(16^3)(1728)}{48(29000)(313)} + \frac{5(.026)(16^4)(1728)}{384(29000)(313)} = .49'' < .8'' \checkmark$$

Typical Bay Tall Tower - Exterior



Loads in exterior
LL - 40 psf residential
15 psf - Partitions
DL - 40 psf conc & deck
8 psf SDL
Self Weight

Total Load - 103 psf $\cdot$ 8' = .824 K/ft

$$\Delta_{TL} = \frac{5(.824)(25^4)1728}{384(29000)I} > 1.25'' \quad I > 200\ in^4$$

$$\Delta_{LL} = \frac{5(.055 \cdot 8)25^4(1728)}{384(29000)I} > .833 \quad I > 160\ in^4$$

$W_u = 1.2(48) + 1.6(55) = 146\ psf \cdot 8 = 1.168\ K/ft$
$M_u = 1.168(25^2)/8 = 91'K$

Try W10x15 Assuming $y = 3.5$ $I_{LB} = 218\ in^4$
$\phi M_n = 133'K$
$M_u = 91 + \frac{1.2(.015)25^2}{8} = 92'K < 133'K$

Verify Assumption
$a = 194/.85(5)(75) = .6$
$y_2 = 4 - .6/2 = 3.7'' > 3.5'' \checkmark$

Stud Strength - 17.2 K
$194/17.2 = 11.2 \Rightarrow 24$ studs/beam $< 25 \checkmark$

Try W10x17 Assuming $y = 3.5$ $I_{LB} = 219\ in^4$
$\phi M_n = 133'K > 92'K$
Verify Assum
$a = 150/.85(5)(75) = .47$
$y_2 = 4 - .47/2 = 3.76'' > 3.5'' \checkmark$
Studs - $150/17.2 = 8.7 \Rightarrow 18$ studs/beam < 25

Try W10x14 Assuming $y = 3.5$ $I_{LB} = 223\ in^4$

$\phi M_n = 137'K > 92'K \checkmark$
$a = 122/.85(5)(75) = .382 \therefore y_2 = 3.8 > 3.5 \checkmark$

Studs - $122/17.2 = 7.1 \Rightarrow 16$ studs/beam $< 25 \checkmark$

Weight comparison

W10x15 – $15(25) + 24(10) = 615$ lbs

$\boxed{\text{W }10\times17 - 17(25) + 18(10) = 605 \text{ lbs } \star}$

W 10x19 – $19(25) + 16(10) = 635$ lbs

Check Beam for unshored strength

DL only – $1.4(40\cdot 8) + 17(1.4) = 472$ lb/ft

Const loads – $1.2(40\cdot 8) + 1.2(17) + 1.6(20\cdot 8) = 660$ lb/ft ✱

$M_u = .66(25^2)/8 = 52'K$

$\phi_b M_p = 70.1'K > 52'K$ ✓

Check for wet concrete deflection $w = 334$ lb/ft $I = 82 \text{ in}^4$

$\Delta_{WC} = \frac{5(.334)(25^4)1728}{384(29000)(82)} = 1.23'' < 1.25''$ ✓

Total load deflection

$\Delta_{TL} = \frac{5(.824+.017)25^4(1728)}{384(29000)(219)} = 1.16'' < 1.25''$ ✓

Girder 24' span

$P = 21K$
$P_u = 29K$

$\Delta_{TL\ Allow} = 24\cdot 12/240 = 1.2''$

$\Delta_{TL} = \frac{21(24^3)(1728)}{28(29000)I} < 1.2''$ $\therefore I > 515 \text{ in}^4 \star$

$\Delta_{LL\ Allow} = 24\cdot 12/360 = .8''$

$\Delta_{LL} = \frac{11(24^3)(1728)}{28(29000)I} < .8$ $\therefore I > 404 \text{ in}^4$

$M_u = 29(8) = 232'K$

Try W12x26 Assuming $y_2 = 3.5$ $I_{LB} = 526 \text{ in}^4$

$\phi M_n = 259'K > 232'K + \frac{.026(1.2)(24^2)}{8} = 234'K$ ✓

Verify Assumption $a = 321/.85(5)(72) = 1.04''$ $y_2 = 3.5''$ ✓

Studs $321/18.3 = 17.5 \Rightarrow 36$ studs/girder

Try W12x30 Assuming $y_2 = 3.5$ $I_{LB} = 569 \text{ in}^4$

$\phi M_n = 280'K > 234'K$

Verify Assum $a = 296/.85(5)(72) = .97$ $\therefore y_2 = 3.52 > 3.5$ ✓

Studs – $296/18.3 = 16.2 \Rightarrow 34$ studs/girder

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Weight Comparison

$\boxed{\text{W12x26} \quad 26(24)+36(10) = 984 \text{ lbs} \star}$

W12x30 $\quad 30(24)+34(10) = 1060$ lbs

check for unshored strength

DL Only $\quad P = 472(25) = 11.8k \quad w = 1.4(.026) = .036$ k/ft

constr. loads $\quad P = 660(25) = 16.5k \quad w = 1.2(.026) = .031$ k/ft

$$M_u = 16.5(8) + \frac{.031(24^2)}{8} = 134'k$$

$$\phi_B M_p = 140'k > 134'k \checkmark$$

Check wet concrete deflection $\quad P = .334(25) = 8.35k \quad I_{beam} = 204$

$$\Delta = \frac{8.35(24^3)(1728)}{28(29000)(204)} = 1.2'' \text{ Also } \Delta \text{ is allowable} \checkmark$$

Check total load Δ w/ self weight

$$\Delta = \frac{21(24^3)(1728)}{28(29000)(526)} + \frac{.026(5)(24^4)(1728)}{384(29000)(526)} = 1.19'' < 1.2'' \checkmark$$

Appendix C – Preliminary Column Design

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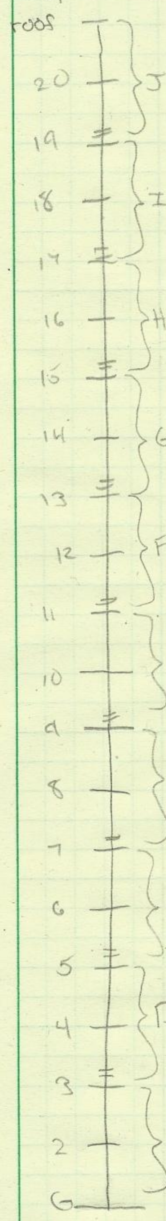
Columns will be spliced @ every 2 floors for an expedited schedule

& negate the use of fall protection, also being more economic

Typical exterior corner column



= indicate column splice

$A_t = \frac{1}{2}(20.5') \times \frac{1}{2}(24') = 123\ ft^2$

$P_d = 8\,psf(SDL) + 40\,psf(\text{conc deck}) + 2.125\,psf(\text{beams}) + 2.53\,psf(\text{girders})$
$= 52.655\,psf \times 123\,ft^2 = 6476.6\ lbs$

$P_L = 40\,psf(\text{resid}) + 15\,psf(\text{partition}) = 55\,psf$
$= 55\,psf \times 123\,ft^2 = 6765\ lbs$

P_u typical floor $= 1.2(6476.6) + 1.6(6765)$
$\simeq 18.6$ Kips

@ Roof $P_d = 6476.6$ lbs
$P_L = 30\,psf(123\,ft^2) = 3690$ lbs
$P_S = 16\,psf(123\,ft^2) = 1968$ lbs
$P_u = 1.2(6476.6) + 1.6(3690) + .5(1968)$
$= 14.7$ Kips

P_u @ base $= 368.1$ Kips

All columns A–J will be $\boxed{W12 \times 40}$
$40(10')(20\,flrs)\,1.2 = 9.6\,K$
including self weight $P_u = 377.1$

$\phi P_n = 394\,K > 377\,K$

Won't use smaller than W12 b/c you then have thin webs & flanges & can create expensive connections referenced by Charlie Carter on Modern Steel.

ϕP_n is based on 10' length.

TOPS.

Brad Oliver | Final Report | Col design (2)

interior column along exterior wall.

$A_t = 25' \times \frac{1}{2}(24') = 300\ ft^2$

$P_d = 52.655\ psf\ (300 ft^2) = 15,796.5\ lbs$

$P_L = 55\ psf (300 ft^2) = 16,500\ lbs$

P_u typical floor $= 1.2(15,796.5) + 1.6(16,500) = 45.4$ Kips

P_u roof $= 1.2(16,500) + 1.6(30 \cdot 300) + .5(16 \cdot 300) \approx 36.6$ Kips

Floors	Tier	Load	Section	Capacity
roof – 20	J	$P_u = 82K$; w/ self W $P_u = 83K$	W12X40	$\phi P_n = 397K$
19 – 18	I	$P_u = 173.8$; $= 174.7K$ w/ self weight	W12X40	$\phi P_n = 397K$
17 – 16	H	$P_u = 265.2K$; $= 266.5K$ w/ self	W12X40	$\phi P_n = 397K$
15 – 14	G	$P_u = 357.3K$; $= 358.2K$ w/ self	W12X40	$\phi P_n = 397K$
13 – 12	F	$P_u = 449K$; $= 450K$ w/ self	W12X50	$\phi P_n = 499K$
11 – 10	E	$P_u = 541K$; $= 543K$ w/ self	W12X53	$\phi P_n = 590K$
9 – 8	D	$P_u = 634K$; $= 635.4K$ w/ self	W12X58	$\phi P_n = 649K$
7 – 6	C	$P_u = 726.2K$; $= 727.8K$ w/ self weight	W12X65	$\phi P_n = 765K$
5 – 4	B	$P_u = 818.6K$; $= 820.3K$ w/ self weight	W12X72	$\phi P_n = 849K$
3 – 2 – G	A	$P_u = 911K$; $= 913K$ w/ self	use W12X79	$\phi P_n = 932K$

All ϕP_n are based on 10' height

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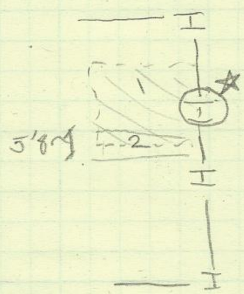
Brad Oliver | Final Report | Col. design (3)

Typical col. on the exterior near corridor. Col. has part of trib area in 100 psf LL area, & other part in residential 55 psf area.

$A_t = \frac{1}{2}(20.5') \times \frac{1}{2}(16') = 82\ ft^2$

A_t in 100 psf LL: $\frac{1}{2}(20.5) \times \frac{1}{2}(5.67') = 29\ ft^2$



$A_{T1} = 82 - 29 = 53\ ft^2$

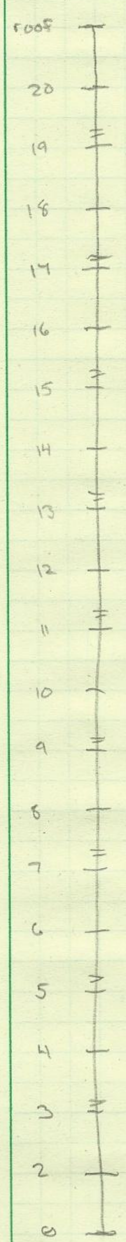
$A_{T2} = 29\ ft^2$

$P_{d1} = 52.665\ psf(53\ ft^2) = 2.79K$

$P_{d2} = 50.75\ psf(29\ ft^2) + 208 = 1.68K$

$P_{L1} = 55\ psf\ (53) = 2.915K$

$P_{L2} = 100\ (29) = 2.90K$



$P_u = 1.2(2.79 + 1.68) + 1.6(2.915 + 2.9) = 14.67K$ typ floor

@ roof $P_u = 1.2(2.79 + 1.68) + 1.6(30(82)) + .5(16)(82)$
$= 4.6K$

@ Base level,
$P_u = 14.67(19\ flrs) + 4.6\ (roof)$
$= 283\ K$

W12X40 for all columns @ this location

including self weight
$P_u = 283 + 1.2(40 \cdot 10' \cdot 20)/1000$
$= 292.6$

$\phi P_n = 397K$ @ 10' lengths

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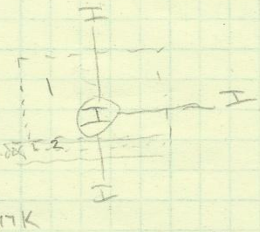
Typical interior column near central corridor

$A_t = 25' \times (8' + 12') = 500 ft^2$



1 is resid loads
2 is corridor loads.

$A_{T2} = 25' \times \frac{1}{2}(5.66) = 70.8 ft^2$
$A_{t1} = 500 - 70.8 = 429.2 ft^2$

$P_{d1} = 429.2(52.665) = 22.6K$
$P_{d2} = 70.8(50.75) = 3.59K$

$P_{L1} = 429.2(55) = 23.6K$
$P_{L2} = 70.8(100) = 7.1K$

P_u typ floor $= 1.2(22.6 + 3.59) + 1.6(23.6 + 7.1) = 80.5K$

$P_u e$ roof $= 1.2(22.6 + 3.59) + 1.6(30 \cdot 500) + .5(16 \cdot 500) = 28K$

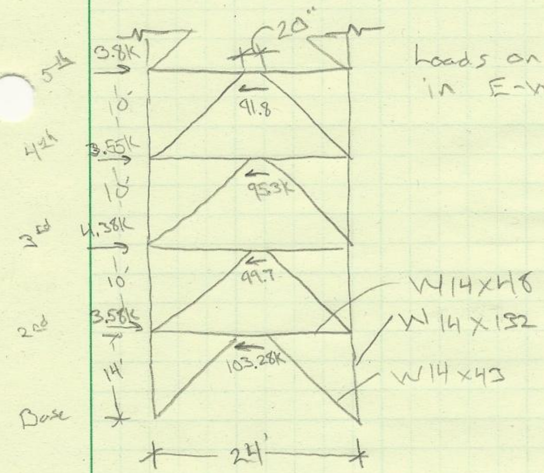
roof
20 — $P_u = 108.5K$ W14x48 $= 109.7K$ w/self $\phi P_n = 477K$
19
18 — $P_u = 270.7K$ W14x48 $\phi P_n = 477K$ $= 271.8K$ w/self
17
16 — $P_u = 432.8K$ W14x48 $\phi P_n = 477K$ $= 434K$ w/self
15
14 — $P_u = 595K$ W14x61 $\phi P_n = 677K$ $= 595.7K$ w/self
13
12 — $P_u = 756.7K$ W14x74 $\phi P_n = 926K$ $= 758.5K$ w/self
11
10 — $P_u = 919.5K$ W14x90 $\phi P_n = 1100K$ $= 921.6K$ w/self
9
8 — $P_u = 1082.6K$ W14x90 $\phi P_n = 1100K$ $= 1084.8K$ w/self
7
6 — $P_u = 1245K$ W14x109 $\phi P_n = 1340K$ $= 1248.4K$ w/self
5
4 — $P_u = 1409.4K$ W14x120 $\phi P_n = 1470K$ $= 1412.3K$
3
2 — $P_u = 1573.3K$ W14x145 $\phi P_n = 1687K$ @14' $= 1576.6K$ w/self
G

All strengths based on 10' length.

TOPS 35502

Appendix D – EBF Checks at Baltimore

Brad Oliver | Final Report | Braced Frame checks | 1



Loads on frame @ col line 14 due to Wind in E-W direction
L.C. 1.2D + 1.0L + 1.6W

inspect link rotation angle

$e = X \frac{M_p}{V_p}$

$M_p = Zf_y = 78.4(50) = 3920''k$
$V_p = .6f_y A_w = .6(50)(13.8-2(.595))(.34)$
$= 128.6\,K$

$\alpha = \frac{eV_p}{M_p} = 20(128.6)/3920 = .66 < 1.6$ ∴ failure controlled by shear.

$\gamma_p = h/e\, \theta_p < .08$ rad
$\theta_p = \frac{\Delta_p}{h}$

$\Delta_p = \frac{C_d \delta_e}{I} = \frac{4(.1222'')}{1} = .493''$

$\theta_p = \frac{.493}{14' \times 12''/ft} = .003$ rad

$\gamma_p = \frac{(24' \times 12''/ft)}{20''}(.003) = .04$ rad $< .08$ rad ✓

Lateral Bracing requirements - Flanges @ end of link must be braced for

$R_u = .06 R_y F_y Z / h_o$ (expected str.)
$= .06(1.1)(50)(78.4)/(13.8 - .595)$
$= 19.6\,K$ of bracing force will need to be applied @ top & bottom flange outside of link element.

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Brad Oliver | Final Report | Braced frame checks | 8

Check stiffner requirement
Sec 15.3 requires double sided, full depth stiffners @ ends of link.

Min req width - $W_{min} = \frac{b_f - 2t_w}{2} = \frac{8.03 - 2(.34)}{2} = 3.68"$

min req thickness $t_{min} = .75 t_w \geq 3/8"$
$= .75(.34) = .255" < 3/8"$ Not OK
$\therefore$ use 3/8" thick stiffners.

To accomodate connection link $b_f \geq$ brace b_f

$8.03" \geq 8.0"$ ✓

Check link element slenderness $\lambda_f = \frac{b_f}{2t_f} = 6.75$

for flange compactness - $\lambda_{ps} = .3\sqrt{\frac{E}{F_y}} = .3\sqrt{\frac{29000}{50}} = 7.22$

$\lambda_f < \lambda_{ps}$ ✓ No local buckling issues

Now web check - $\lambda_w = h/t_w = 33.6$

$C_a = \frac{P_u}{\phi_b P_y} = \frac{P_u}{.9 F_y F_y} = \frac{50.09}{.9(50)(14.1)} = .0789$

$\lambda_{ps} = 3.14\sqrt{\frac{E}{F_y}}(1 - 1.54 C_a)$

$= 3.14\sqrt{\frac{29000}{50}}(1 - 1.54(.0789)) = 66.4$

$\lambda_w < \lambda_{ps}$ ✓

Determine Shear Strength of the link
$.15 P_y > .15 F_y A_g = .15(50)(14.1) = 106K$

B/C $P_u < .15 P_y$ Axial effects may be ignored. Also nominal shear strength of link is lesser of V_p or $2M_p/e$

$V_p = .6 F_{yw} A_w$
$V_p = .6(50)(13.8 - 2(.595)) = 128.6$ ☆

$2M_p/e = 2(3920)/20 = 392K$

$\phi V_n = .9(128.6)$
$= 116K > V_u = 60.06K$ ✓

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Design of Beam outside link element

Beam must be capable of holding factored gravity loads & forces generated times 1.1 the expected shear strength of the link.

$$1.1 R_y V_p = 1.1(1.1)(128.6) = 156\ K$$

Axial force beam is required to hold is based on

$$P_e = \frac{1.1 R_y V_p L}{2h} = \frac{156(24')}{2(14')} = 133.7K$$

$$\phi P_n = 140K \text{ @ } 24' \text{ unbraced length} > 133.7K\ \checkmark$$

Web stiffners within the link - spacing

for rotation $\angle = .08$ rad $\quad 30t_w - d/5 = 30(.34) - 13.8/5$
$= 7.44''$

for rotation $\angle < .02$ rad $\quad 52t_w - d/5 = 50(.34) - 13.8/5$
$= 14.24''$

Actual rotation - .04 rad

Interpolation

7.44"	.08 rad
x	.04 rad
14.24"	.02 rad

$$\frac{(14.24 - 7.44)}{x - 7.44} = \frac{(.02 - .08)}{(.04 - .08)} \Rightarrow \boxed{x = 12'' \text{ spacing}}$$

only req on 1 side of web b/c link d < 25"
Same thickness & width as previously calculated

Second Order Effects

determine unbraced length - it is braced @ end of link element

$$L_b = \frac{L - e - 2d_c/2}{2} = \frac{24'(12) - 20 - 2(14)/2}{2} = 127'' = 10'7''$$

$$B_1 = \frac{C_M}{1 - \left(\frac{\alpha P_r}{P_{e1}}\right)} \geq 1 \qquad B_2 = 1 \text{ b/c end points assumed not to translate}$$
$C_M = 1$ to be conservative

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2(29000)(484)}{(1(124))^2} = 8589\ K$$

$$B_1 = \frac{1}{1 - \left(\frac{1.0(81.3)}{8589}\right)} = 1.01$$

$$M_{rx} = B_1 M_{NT} + B_2 M_{LT}$$
$$= 1.01(145.68) = 147\ K'$$

Using combined loading table - UBL = 11' conservative - $p = 2.23e^{-3}$ $\quad b_x = 3.4e^{-3}$

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$$\frac{P_r}{P_c} = \frac{pP_r}{P_y} = \frac{2.23e^{-3}(81.3)}{1.1} = .1648 < .2$$

$$\left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}}\right) = \frac{b_x M_{rx}}{P_y} = \frac{(3.4e^{-3})(147)}{1.1} = .454$$

b/c $P_r/P_c < .2$

$$\frac{1}{2}pP_r + \frac{9}{8}B_x M_{rx} \le 1$$

$$\frac{1}{2}(.1648) + \frac{9}{8}(.454) = .6 \checkmark$$ This corresponds to RAM model as well

Brace Design W 14X43

Check Brace Slenderness

$$\lambda_f = b_f/2t_f = 7.54$$

$$\lambda_{pf} = .38\sqrt{E/f_y} = .38\sqrt{\frac{29000}{50}} = 9.15$$

$$\lambda_w < \lambda_p \checkmark$$

$$\lambda = h/t_w = 37.4$$

$$\lambda_p = 3.76\sqrt{E/F_y} = 3.76\sqrt{29000/50} = 90.6$$

$$\lambda_w < \lambda_p \checkmark$$

Consider 2nd order effects -

Unbraced length $\sqrt{14^2 + 10.58^2} = 17.55'$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \ge 1$$ B/c ends Assume not to translate, $B_2 = 1.0$

Assume $K=1$

$$P_{e1} = \frac{\pi EI}{(KL)^2} = \frac{\pi(29000)(428)}{(17.55 \times 12)^2} = 879.2K$$

$C_m = 1.0$ to be conservative

$$P_r = 157(1) = 157K$$

$$B_1 = \frac{1}{1 - \left(\frac{1(157)}{879.2}\right)} = 1.22 \ge 1 \checkmark$$

$$M_r = 1.22(0) = 0$$

$KL = 18'$ $p = 4.58e^{-3}K$ $b = 4.97e^{-3}K$

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Brad Oliver | Final Report | Braced Frame Check | 5

$\frac{P_r}{P_c} = pP_r = 4.58e^{-3}(157) = .72$

$(8/9)\left(\frac{M_{rx}}{M_{cx}}\right) - b_x M_{rx} = 0$

since $P_r/P_c > 2$ interaction $u = .72 + 0 = .72 < 1$ ✓

Corresponds closely to RAM

For col - Strong col weak Beam design to avoid pancaking

$M_{pc} > M_{pb}$

$F_y Z$

$50(1900) > 3920$

$9500 > 3920$ ✓

Appendix E – New Seismic Criteria

Brad Oliver | Final Report | Earthquake - San Fran.

$S_1 = 60\% g = .6g$
$S_s = 150\% g = 1.5g$

Assume Site class D - unknown

$T_L = 12$ sec

$S_{MS} = F_a S_s \quad F_a = 1.0 \quad \therefore S_{MS} = 1.5g$

$S_{M1} = F_v S_1 \quad F_v = 1.5 \quad \therefore S_{M1} = .6g(1.5) = .9g$

$S_{ds} = 2/3\, S_{MS} = 2/3(1.5g) = 1g$
$S_{D1} = 2/3\, S_{M1} = 2/3(.9g) = .6g$

Based on occupancy category II & $S_{ds} > .5$ & $S_{d1} > .2$ Seismic Design category D

$V = C_s W$

$C_s = S_{ds}/R/I = 1/8/1 = .125 > .01$ ✓ $\cancel{\&}$ $> .0375$

$C_u = 1.4$

$T_a = C_t h_n^x = .03(207)^{.75} = 1.64$ upper limit for $T = 1.64(1.4) = 2.29$ sec

$C_s < \frac{S_{d1}}{T(R/I)} = \frac{.6}{2.29(8)} = .0328$

$< \frac{S_{d1} T_L}{T^2(R/I)} = \frac{.6(12)}{2.29^2(8)} = .1716$ ✓

$C_s > \frac{.5 S_1}{R} = \frac{.5(.6)}{8} = .0375$ ☆ controls

$K = 1.9$ through interpolation.

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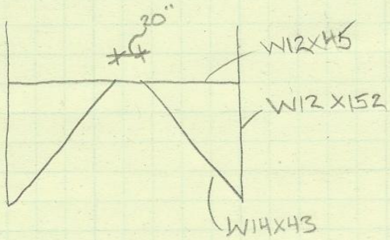
Appendix F – EBF Checks at San Francisco

Brad Oliver | Final Report | Brace Frame V @ San Fran (1)



Check story drift

$\delta_e = .11667"$ $C_d = 4$

Allowable drift - $\Delta_a = .025 h_{sx} = .025(14' \times 12) = 4.2"$

$\delta_x = \delta_e C_d = .1167"(4) = .4668" < 4.2"$ ✓

b_f of beam $\geq$ b_f of brace $- 8.05" \geq 8"$ ✓

Check link element slenderness

$\lambda_f = b_f/2t_f = 7.0$

Check for compactness $\lambda_B = .3\sqrt{E/F_y} = .3\sqrt{\frac{29000}{50}} = 7.22 > 7.0$ ✓

∴ Meets local Buckling req.

width thickness ration

$\lambda_w = h/t_w = 29.6$

$C_a = P_u/\phi_b P_y = \frac{P_u}{.9(F_y A_g)} = \frac{45.8}{.9(50)(13.1)} = .0777$

with $C_a < .125$

$\lambda_{ps} = 3.14\sqrt{\frac{E}{F_y}}(1 - 1.54 C_a) = 3.14\sqrt{\frac{29000}{50}}(1 - 1.54(.0777))$

$= 66.57$

$\lambda_w < \lambda_{ps}$ ✓

Shear strength of link

$.15 P_y = .15 F_y A_g = .15(50)(13.1) = 98.25$

since $P_u < .15 P_y$ $V_p = .6 F_{yw} A_w$

$A_w = (d_b - 2t_f)t_w = (12.1 - 2(.575)).335 = 3.67$

$V_p = .6(50)(3.67) = 110.1$

$M_p = F_y Z_x = 50(64.2) = 3210K" = 268'K$

$\phi V_n = \phi V_p < \phi^{2M_p}/e$

$= .9(110.1) < .9(3210(2)/20)$

$= 99.1K < 289$

$= 99.1K$

$V_u = 51K < 99.1K$ ✓

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Brad Oliver | Final Report | Brace Frame @ San Fran (2)

Check Link rotation Angle

$e = x\, M_r/V_p \qquad \frac{V_p e}{M_p} = \frac{110.1(20)}{3210} = .686 < 1.6 \therefore$ shear controls

$\gamma_r = \frac{L}{e}\theta_p \qquad \theta_p = \Delta_r/h = .4668/14 \times 12 = .00266$ rad

$\gamma_p = \frac{25' \times 12\,(.00266)}{20} = .04$ rad $\leq .08$ rad ✓

∴ W12X45 is adequate

Lateral Bracing Requirements

Bean Flanges @ both ends of link to be braced for following force

$R_u = .06\, R_y F_y Z/h_o = .06(1.1)(50)(64.2)/(12.1-.575)$
$= 18.4$ Kips

Stiffner requirements

$W_{min} = \frac{b_f - 2t_w}{2} = \frac{8.05-2(.335)}{2} = 3.67''$

Min req thickness of stiffner is

$t_{min} = .75\,t_w \geq 3/8''$
$= .75(.575)$
$\approx .431''$

Full depth 1/2" X 3 3/4" stiffners on both sides of web @ link ends

Intermediate stiffners

Spacing if link rotation $\leq .08$ rad
$30t_w - d/5 = 30(.335) - 12.1/5 = 7.63''$

spacing if link rotation = .02 rad
$52t_w - d/5 = 52(.335) - 12.1/5 = 15''$

Through interpolation - spacing = 12.54'' Maximum ∴ only req. on 1 side of web b/c $d < 25''$

Min thickness of stiffners = $t_w \geq 3/8''$
$= .335 \geq .375 \;\therefore 3/8''$

Req width of stiffners = $b_f/2 - t_w = 8.05/2 - .335 = 3.69''$

Full depth 3/8" X 3 3/4" intermediate web stiffner on 1 side

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Appendix G – Construction Management Calculations

	05 12 23.17 Columns, Structural	Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
7000	W10x45	E2	1032	0.054	L.F.	55.5	2.57	1.57	59.64
7050	W10x68	E2	984	0.057	L.F.	84	2.7	1.65	88.35
7100	W10x112	E2	960	0.058	L.F.	139	2.76	1.69	143.45
7150	W12x50	E2	1032	0.054	L.F.	62	2.57	1.57	66.14
	W12X53	E2	1028	0.054	L.F.	66	2.58	1.58	70.16
	W12X58	E2	1022	0.055	L.F.	72	2.6	1.59	76.19
	W12X65	E2	1013	0.055	L.F.	81	2.62	1.6	85.22
	W12X72	E2	1003	0.056	L.F.	89	2.65	1.62	93.27
	W12X79	E2	994	0.056	L.F.	98	2.67	1.63	102.3
7200	W12X87	E2	984	0.057	L.F.	108	2.7	1.65	112.35
	W12X96	E2	977	0.057	L.F.	119	2.72	1.67	123.39
	W12X106	E2	970	0.058	L.F.	132	2.73	1.68	136.41
7250	W12X120	E2	960	0.058	L.F.	149	2.76	1.69	153.45
	W12X136	E2	949	0.059	L.F.	169	2.79	1.71	173.5
	W12X152	E2	938	0.06	L.F.	188	2.83	1.73	192.56
7300	W12x190	E2	912	0.061	L.F.	235	2.91	1.78	239.69
7350	W14x74	E2	984	0.057	L.F.	91.5	2.7	1.65	95.85
	W14X82	E2	980	0.057	L.F.	101.5	2.71	1.66	105.87
	W14X90	E2	976	0.057	L.F.	111.5	2.72	1.66	115.88
	W14X99	E2	971	0.058	L.F.	123	2.73	1.67	127.4
	W14X109	E2	966	0.058	L.F.	135	2.75	1.68	139.43
7400	W14x120	E2	960	0.058	L.F.	149	2.76	1.69	153.45
	W14X132	E2	950	0.059	L.F.	164	2.79	1.71	168.5
	W14X159	E2	927	0.06	L.F.	197	2.86	1.75	201.61
7450	W14x176	E2	912	0.061	L.F.	218	2.91	1.78	222.69
8090	For Projects 75-99 tons, add				L.F.	10%			
8092	For Projects 50-74 tons, add				L.F.	20%			
8094	For Projects 24-49 tons, add				L.F.	30%	10%		
8096	For Projects 10-24 tons, add				L.F.	50%	25%		
8098	For Projects 2-9 tons, add					75%	50%		
8099	For Projects < 2 tons, add					100%	100%		
	01 50 19.60 Monthly Tower Crane Crew								
Static Tower Crane - 6200 lb Capacity		A-3N	0.05	176	Month		8,175	21700	29875
	05 12 23.75 Structural Steel Members								
702	W10x22	E2	600	0.093	L.F.	27	4.42	2.7	34.12
	W10X30	E2	585	0.096	L.F.	37	4.55	2.78	44.33
	W10X33	E2	580	0.097	L.F.	41	4.6	2.8	48.4
902	W10X39	E2	569	0.099	L.F.	48	4.7	2.86	55.56
1102	W10x49	E2	550	0.102	L.F.	60.5	4.82	2.95	68.27
	W12x16	E2	880	0.064	L.F.	19.8	3.01	1.84	24.65
1302	W12X19	E2	880	0.064	L.F.	23.4	3.01	1.84	28.25
1502	W12x22	E2	880	0.064	L.F.	27	3.01	1.84	31.85
	W12x26	E2	880	0.064	L.F.	32	3.01	1.84	36.85
	W12X30	E2	859	0.066	L.F.	37	3.11	1.9	42.01
	W12X35	E2	833	0.069	L.F.	43	3.23	1.98	48.21
1702	W12X45	E2	781	0.074	L.F.	56	3.48	2.13	61.61
1902	W12x72	E2	640	0.088	L.F.	89	4.14	2.53	95.67
2102	W14x26	E2	990	0.057	L.F.	32	2.68	1.64	36.32
2302	W14x30	E2	900	0.062	L.F.	37	2.95	1.8	41.75
	W14x34	E2	810	0.069	L.F.	42	3.27	2	47.27
	W14X43	E2	800	0.069	L.F.	53	3.32	2.02	58.34
	W14X48	E2	795	0.07	L.F.	59	3.34	2.04	64.38
2502	W14x120	E2	720	0.078	L.F.	149	3.68	2.25	154.93
8490	75-99 tons, add				L.F.	10%			
8492	50-74 tons, add				L.F.	20%			
8494	25-49 tons, add				L.F.	30%	10%		
8496	10-24 tons, add				L.F.	50%	25%		
3.50 Floor Decking									
5200	2" Deep, 22 gauge, composite	E-4	3860	0.008	S.F.	1.35	0.41	0.03	1.79
Sprayed Fireproofing									
400	Beams	G-2	1500	0.016	S.F.	0.53	0.58	0.08	1.19
800	Columns	G-2	700	0.034	S.F.	1.13	1.24	0.18	2.55
ormal Weight Concrete									
400	5000 psi				C.Y.	111			111
70 Placing Concrete									
1400	Elevatd Slabs < 6" thick, pumped	C-20	140	0.457	C.Y.		16.8	5.6	22.4
3500	High Rise, more than 5 stories, add/floor	C-20	2100	0.03	C.Y.		1.12	0.37	1.49

Crew E2	Hr.	Daily
1 Struc Foreman	50.55	404.4
4 Steel workers	48.55	1533.6
1 Equip Oper. (Crane)	46.5	372
1 Equip Oper. Oiler	10.3	322.4
1 Lattice Boom Crane, 90 Ton		1622
56 Labor Hour Daily Total		4254.4
Crew A-3N		
1 Equip Oper. (Crane)	46.5	372
1 Tower Crane (monthly)		987.2
8 L.H. Daily Total		1359.2
Crew G-2		
1 Plasterer	39.4	315.2
1 Plasterer Helper	35.05	280.4
1 Building Laborer	34.35	274.8
1 Grout Pump, 50 C.F./hour		125.8
24 L.H. Daily Total		996.2
Crew C-20		
1 Labor Foreman	36.35	290.8
5 Laborers	34.35	1374
1 Cement Finisher	40.85	326.8
1 Equip. Oper. (med.)	45.35	362.8
2 Gas Engine Vibrators		46.4
1 Concrete Pump (small)		741
64 L.H. Daily totals		3141.8

Takeoffs

Tall Tower

Gravity Beams	Quanity	Length (ft)	Weight (lbs)
W8X10	161	2643	26621
W10X12	122	2351.83	28330
W12X14	360	8174.5	115715
W12X16	370	525	136631
W12X19	312	7746.33	146820
W14X22	96	2379	52538
W14X26	21	484	12665
W14X30	78	1853	55802
Studs	19949		
Total		26156.66	575122
Gravity Columns	Quanity	Length (ft)	Weight (lbs)
W12X40	73	1482	59002
W12X45	6	128	5706
W12X50	6	120	5962
W12X53	10	214	11360
W12X58	6	128	7404
W12X65	6	124	8059
W12X72	7	152	10913
W12X79	2	40	3158
W12X87	2	48	4181
W12X96	2	48	4606
Total		2484	120351
Lateral Beams	Quanity	Length (ft)	Weight (lbs)
W14X48	220	5093.3	244370
Total			244370
Lateral Braces	Quanity	Length (ft)	Weight (lbs)
W14X43	440	6587.1	282638
Total			282638
Lateral Columns	Quanity	Length (ft)	Weight (lbs)
W12X40	102	1037	41285
W12X45	8	80	3566
W12X53	20	206	10935
W12X65	28	288	18718
W12X58	16	162	9371
W12X50	22	220	10930
W12X72	12	120	8616
W12X79	18	192	15157
W12X87	14	150	13066
W12X96	18	196	18808
W12X106	8	80	8493
W12X120	14	148	17777
W12X136	2	24	3258
W14X43	22	225	9647
W12X152	8	96	14602
W14X48	8	80	3838
W14X61	10	100	6091
W14X68	12	120	8167
W14X90	14	150	13526
W14X74	2	20	1484
W14X99	8	80	7922
W14X109	4	40	4355
W14X82	10	102	8330
W14X120	10	104	12492
W14X132	8	96	12674
W14X159	2	24	3814

Short Tower

Gravity Beams	Quanity	Length (ft	Weight (lbs)
W8X10	282	4336.67	43680
W10X12	58	1000.67	12054
W12X14	88	1849	26174
W12X16	30	745	11940
W12X19	16	387.67	7348
W12X26	56	1344	34986
W12X22	6	149	3285
W12X30	8	192	5743
W12X45	2	48	2140
W12X35	14	336	11776
Studs	6111		
Total		10388.01	159126
Gravity Columns	Quanity	Length (ft	Weight (lbs)
W12X40	71	1524	60674
W12X50	1	24	1192
Total		1548	61866
Lateral Beams	Quanity	Length (ft	Weight (lbs)
W8X10	7	109.7	1105
W10X12	1	15.7	189
W10X39	14	336	13148
W10X22	7	173.8	3839
W12X14	7	168	2378
W12X19	1	24	455
W14X22	1	24.8	548
W16X26	2	48	1254
Total		900	22916
Lateral Braces	Quanity	Length (ft	Weight (lbs)
W10X33	4	71.6	2367
W10X30	76	1135.3	34150
Total			36517
Lateral Columns	Quanity	Length (ft	Weight (lbs)
W12X40	74	780	31053
W12X45	4	56	2496
W12X50	2	24	1192
Total		860	34741

Beams	Duration	Cost			$/lb of steel		Total Weight
W8X10	11.81561667	241889.3044			1.77072695		1824569
W10X12	5.613666667	114922.984					
W10X22	0.289666667	5930.056					
W10X39	0.590509666	18668.16					
W12X14	11.58125	251220.475					
W12X16	1.443181818	31305.5					
W12X19	9.270454545	230463.5					
W14X22	2.476565657	89049.376					
W14X26	0.488888889	17578.88					
W14X30	2.058888889	77362.75					
W12X26	1.527272727	49526.4					
W12X22	0.169318182	4745.65					
W12X30	0.223515716	8065.92					
W12X45	0.061459667	2957.28					
W12X35	0.403361345	16198.56					
W14X48	6.406666667	327906.654					
Columns	**Duration**	**Cost**					
W12X40	4.673449612	318993.22					
W12X45	0.255813953	17460.96					
W12X53	0.406976744	27778.8					
W12X65	0.406712734	59796.3174					
W12X58	0.283757339	22095.1					
W12X50	0.375968992	25662.32					
W12X72	0.271186441	25369.44					
W12X79	0.233400402	23733.6					
W12X87	0.201219512	22245.3					
W12X96	0.249744115	30107.16					
W12X106	0.082474227	10912.8					
W12X120	0.154166667	22710.6					
W12X136	0.025289779	4164					
W12X152	0.102345416	18485.76					
W14X43	0.228658537	21566.25					
W14X48	0.081300813	7668					
W14X61	0.101626016	9585					
W14X68	0.12195122	11502					
W14X74	0.020325203	1917					
W14X90	0.153688525	17382					
W14X99	0.081967213	10192					
W14X109	0.041407867	5577.2					
W14X82	0.104081633	10798.74					
W14X120	0.108333333	15958.8					
W14X132	0.101052632	16176					
W14X159	0.025889968	4838.64					
Braces	**Duration**	**Cost**					
W10X30	1.940683761	50327.849					
W10X33	0.123448276	3465.44					
W14X43	8.233875	384291.414					
Decking	**Duration**	**Cost**					
2" VLI	71.53608808	494271.447					
Splices	**Duration**	**Cost**					
		329355.2124					
Connections	**Duration**	**Cost**					
		323081.3498					
Concrete	**Duration**	**Cost**					
	18.26251984	283799.5583					
Fireproofing	**Duration**	**Cost**					
Beams		159716.72					
Columns		88287.8					
		$4,367,065					

Appendix H – References

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