[Helios Plaza]

Houston, Texas

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[TECHNICAL REPORT III:]

Lateral System Analysis and Confirmation Design

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Executive Summary

The purpose of this report is to confirm the lateral system design of Helios Plaza. Three main points that will be touched upon are:

1. Load determination for the building
2. Distribution of loads throughout the building
3. Strength checks of critical members

The report generally follows the above order with minor deviations as they become necessary to the discussion of the analysis.

Helios Plaza is an office building that houses the IST and oil trading divisions of its owner BP. The plaza is located in Houston, Texas near other office buildings and suburban housing. The overall building height is 113' with a typical floor-to-floor height of 15'.

With respect to the overall structural system of Helios Plaza, the gravity system is a mixture of concrete pan joists supported on concrete columns and composite steel deck supported on long-span, castellated steel wide flanges. Lateral forces in the building are resisted by concrete moment frames.

After making assumptions based upon the structural notes for Helios Plaza, the loads on the building were compiled. From these calculations, it was found that wind forces control in the North-South direction and seismic forces control in the East-West direction. The overall building torsion was determined and applied to find the controlling load case, which happens to be ASCE7-10 basic load combination 5 in the x-direction and load combination 4 in the y-direction.

The relative stiffness of each frame was then calculated and the controlling load cases' forces were assessed on the frames. From these calculations, the forces in each member as well as the lateral drifts could be extracted from the computer model analysis. The drifts were compared to code and industry standards and were found to be acceptable.

Strength checks on critical members were prepared by hand to compare to the computer analysis loadings. The comparisons confirmed that the members in Helios Plaza are adequate for the controlling lateral load cases.

Introduction

Helios Plaza is a corporate campus located in Houston, Texas that is comprised of three main structures. The first structure, which is the focus of this report, is a six-story office building that houses the IST and oil trading divisions of BP, the building owner. In addition to the office building, there is a 1,909 car capacity parking deck adjacent to a five megawatt combined heat and power plant separate from the office building. Construction was completed in September 2009. The office building will be referred to as Helios Plaza throughout the rest of this document.

The six-story office building is 423,500 gross square feet with an overall building height of 113 feet. The typical floor-to-floor height is 15 feet with exception at the first floor, the lower roof level and the roof level. The first floor height is 21.5 feet, the lower roof level is 17 feet and the roof level is 14 feet higher than the lower roof. Figure 1 represents these dimensions below.

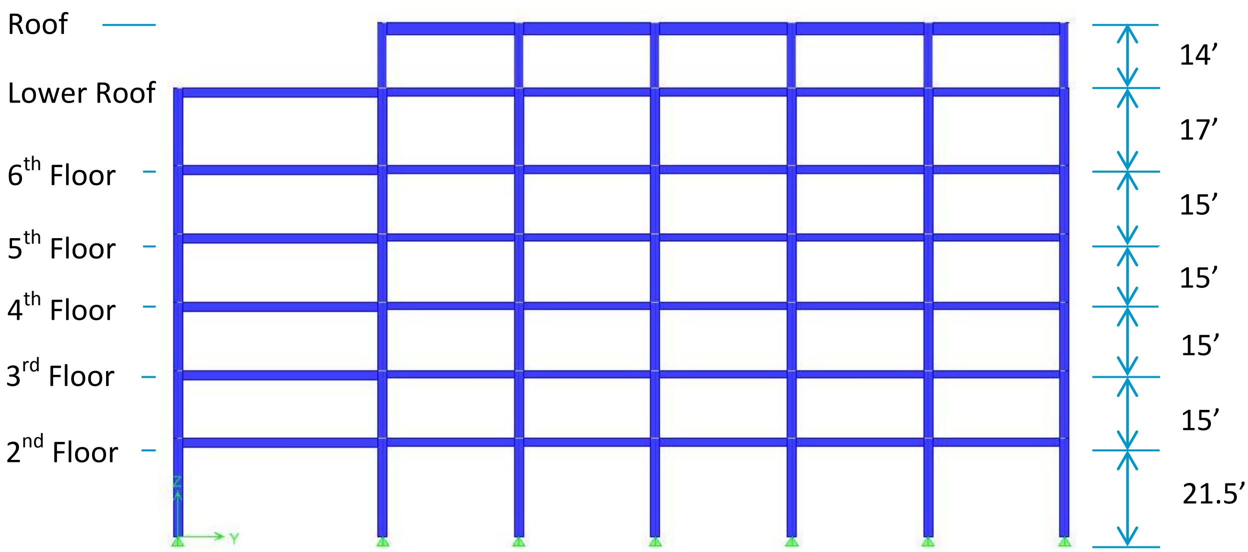

Figure 1: Building Frame Section

One of the more unique aspects of the office building is a result of the oil trading division wishes. The traders requested large, open areas to work in and these spaces are accommodated on the second, fourth and sixth floors. To make these areas more open, the floors above (i.e. the third floor, fifth floor, and lower roof level) are cut out over the trading floors to create double story spaces. To further the open feeling, the number of columns used is limited, which in turn creates long-span situations. Simplified floor plans of these situations are shown in Figures 2 and 3 below. Further visuals that may be helpful can be found in Appendix A.

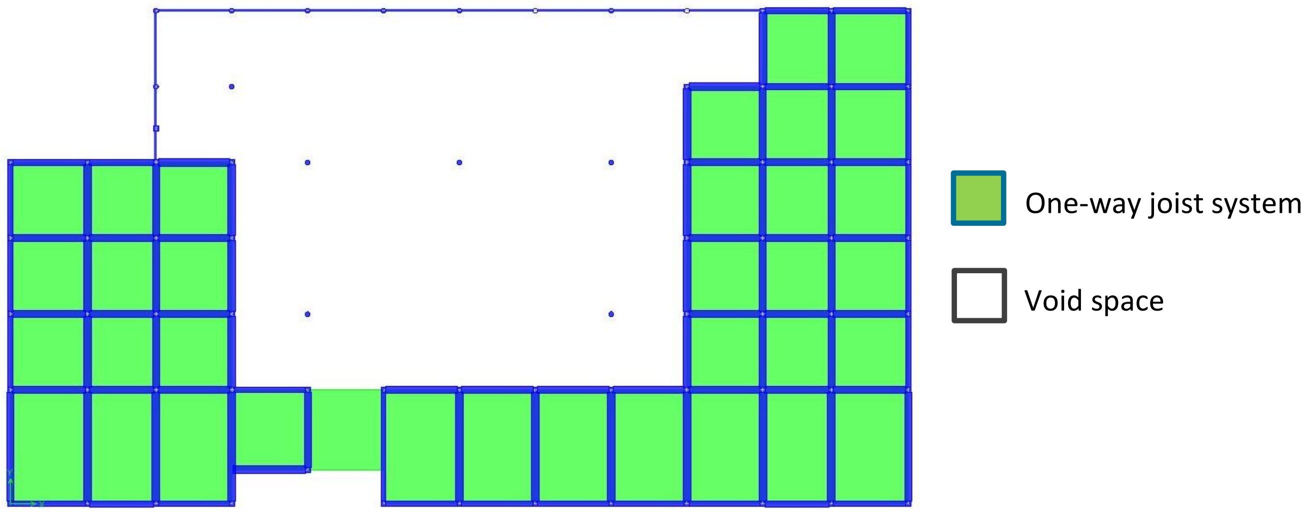

Figure 2: Cut-out Floor over Trading Floor

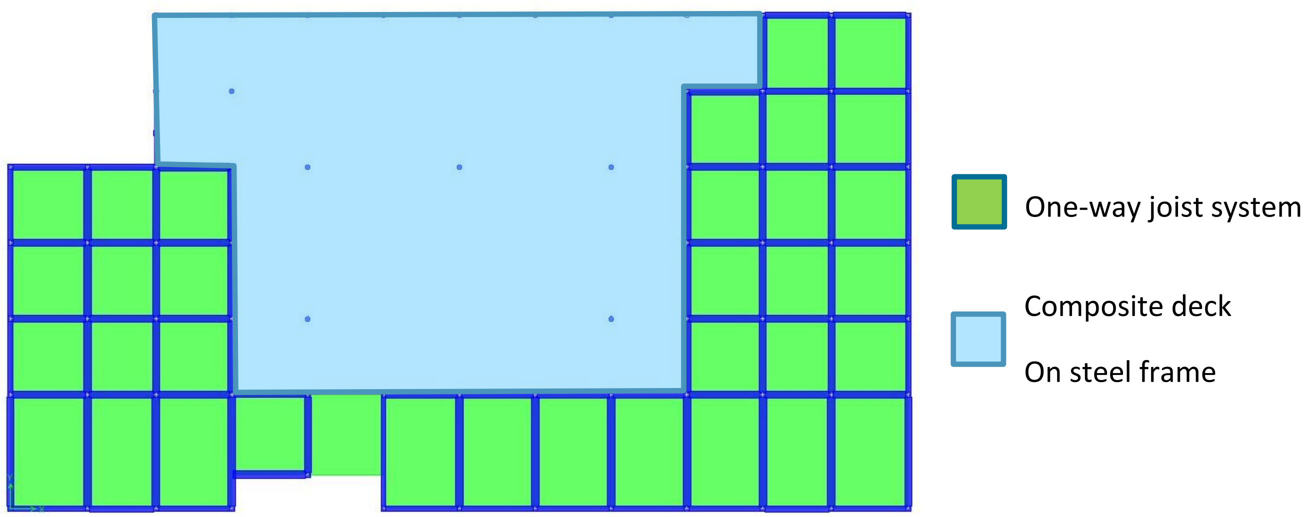

Figure 3: Composite Deck at Trading Floor

Structural System Overview

The main structural system of Helios Plaza is framed in reinforced concrete. Gravity loads are handled largely by square concrete columns, although concrete filled HSS columns are used for aesthetics in larger spaces. For shorter spans, averaging thirty feet, concrete girders in combination with pan beams are used. For longer spans of forty-five feet, post tensioned girders are employed. Finally, for spans of sixty feet, castellated wide flanges shapes are used to reduce the weight-span ratio while maintaining strength.

The floor is mainly a concrete one-way system that uses 66" span, 6" wide skip joists typically. In mechanical rooms, two-way slabs are used to distribute the larger loads more evenly to the supporting members. Composite decking with lightweight concrete is used over the long span steel members in the trading rooms.

To resists lateral loads, the building relies on the typical framing members to perform as concrete moment frames. In the trading floor areas, 2' diameter HSS columns are filled with 7000 psi concrete to take as much lateral load possible since the framing members are not moment connected.

Foundation

The site had to be extensively dewatered prior to the excavation for the project because of the porosity of the soil in Houston. Also, the soil has a high clay content which required the delivery of soils with better bearing capacity to the site.

Spread concrete footings are placed at the base of all grade level columns. The typical depth of the footings is three feet below the member that they are supporting. Their sizes range from 4' x 4' x 15" to 17' x 17' x 57".

Retaining walls are only used in the southeast corner of the building where there is a sub-grade basement with access to the adjacent parking structure via a tunnel. This can be seen below in Figure 4.

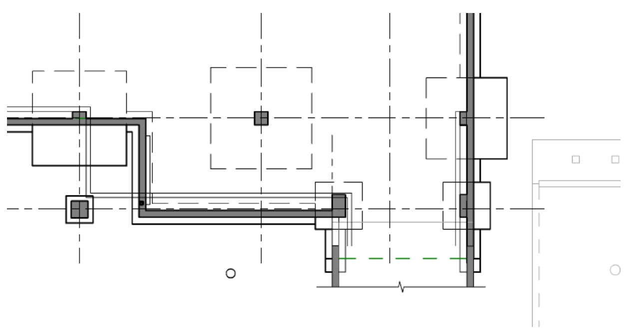

Figure 4: Basement Tunnel Entrance to Parking Structure

At level one, the floor is a slab on grade with thickness ranging from 5" to 12". Grade beams are also implemented at level one sized at 42" x 30".

Columns

Rectangular concrete columns are the predominant system used in Helios Plaza. For the most part these normal weight columns are 24" x 24" in size at all floors except level one where there is an increase in size to 30" x 30". The concrete strength decreases as the levels increase from 6000 psi at the basement level and level one to 5000 psi at levels two and three to 4000 psi for levels four through six. The basement level only occurs in the southeast corner of the building to allow access from the underground tunnel to the rest of the building. The basement area is only fifteen percent of the ground floor area. This space is spanned at level one by post-tensioned girders and one-way pan joists.

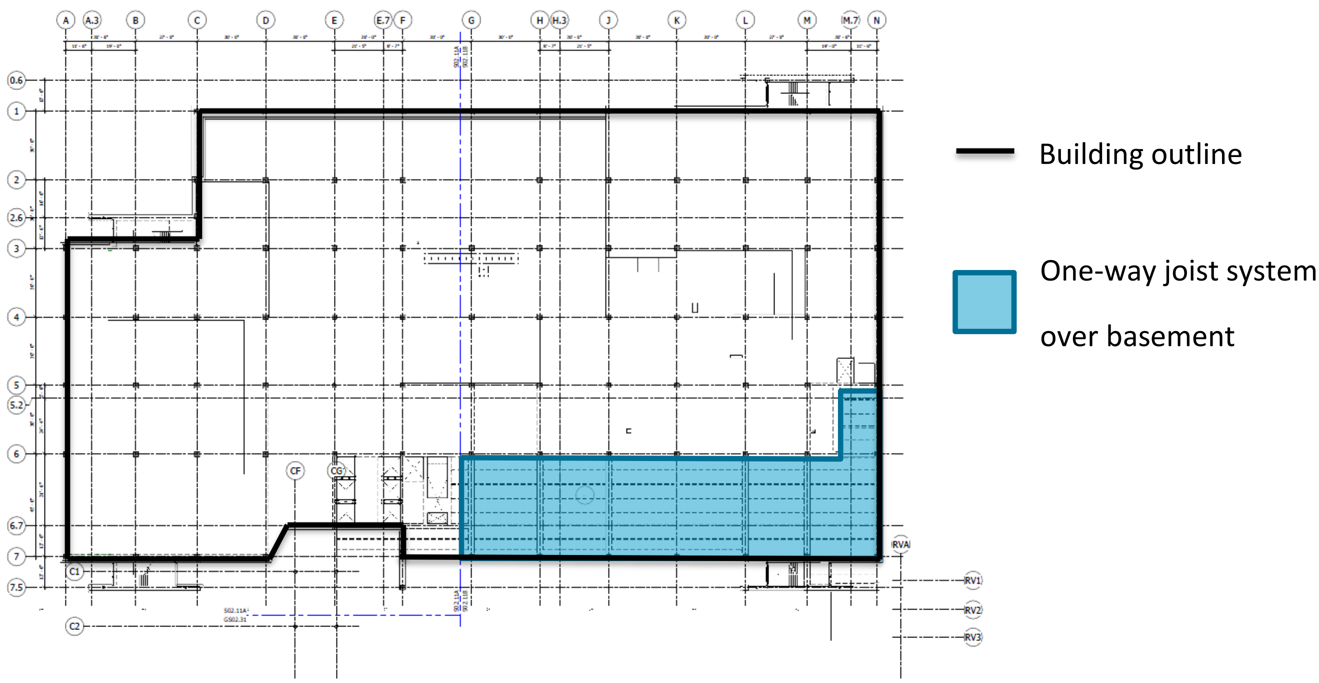

Figure 5: Basement Area

In addition to the rectangular concrete columns, concrete filled HSS columns are used in the double story trading spaces. These columns are 24Ø and are fillet welded to a metal plate at the base. This plate is then tied to the floor or foundations with anchor rods as is evidenced in Figure 6 on the next page. The same concrete strengths apply to these HSS columns as the rectangular columns listed above.

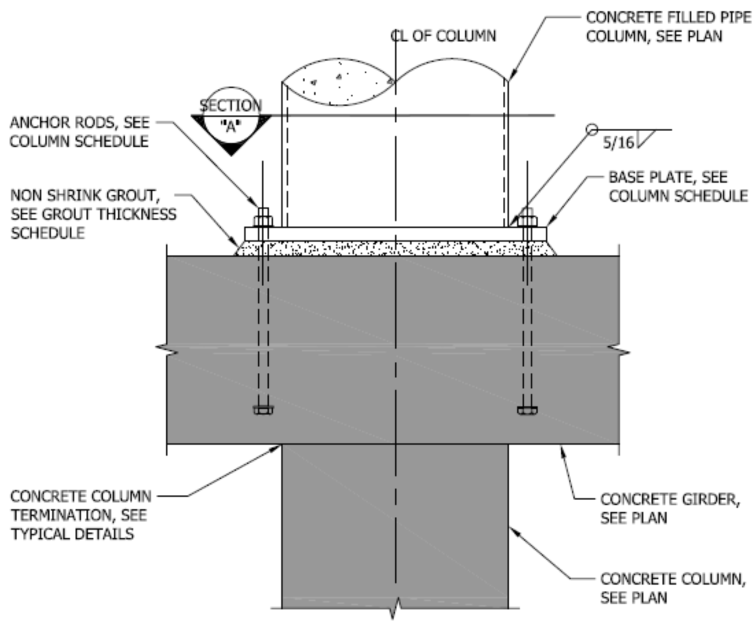

Figure 6: Typical Detail of Concrete Filled HSS Anchorage

Floor Systems

As with the rest of the structural systems in Helios Plaza, the floor system is split into two main categories, one-way pan joists and composite deck. The one-way pan joist system has a welded-wire reinforced (WWR), 4" slab that rests on 16" deep pan typically. The one-way system frames into girders that range from 20" to 33" deep with a width ranging from 24" to 36". Girders also span in the same direction as the one-way joist system, but these are there to create concrete moment frames to resist lateral loads.

Post-tensioned girders are used all along the south face of the building that span in the North-South direction. This is necessary to meet the strength requirements for the 45' distance that these members span. The tendons are typically bundled in groups of four and the minimum final post-tension force is 351 kips. Their locations can be seen in Figure 7 on the next page.

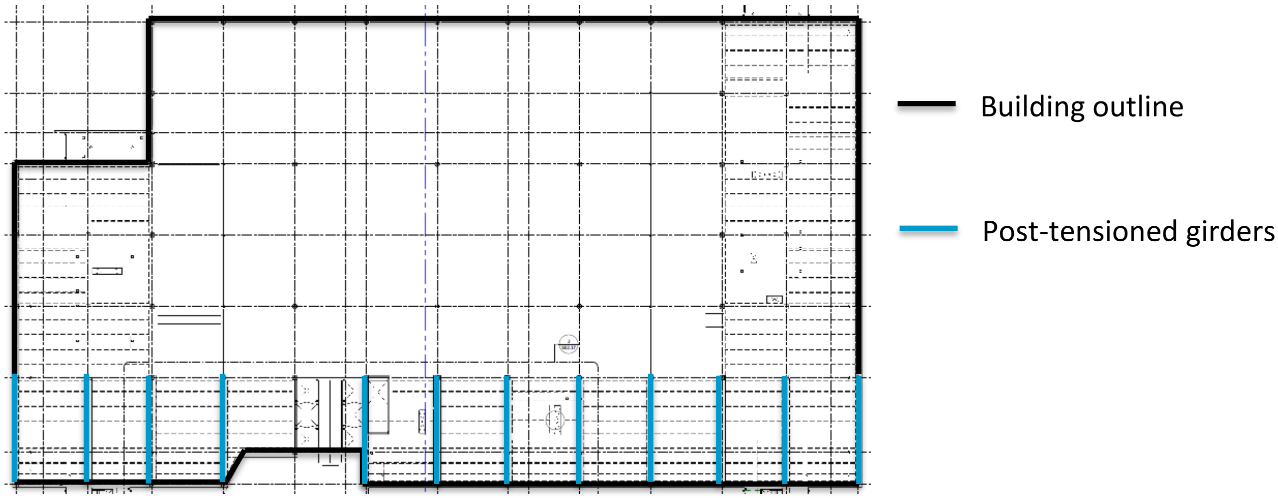

Figure 7: Post-Tensioned Girder Locations

Two-way slabs are implemented in areas where mechanical equipment is housed on every floor. The slabs are typically 10” thick, but in some cases they are 12” thick. Bathrooms usually share the same bays as the mechanical rooms because cutting holes in this system is efficiently achievable.

The second main floor system used in Helios Plaza is a composite deck on w-shapes. The change occurs because of the move to long span castellated beams to accommodate open, double story spaces for the trading floors. Spans of 60’ dominate these spaces and the castellated beams vary between CB24x100 and CB30x44/62. In addition to the weight saving caused by punching out parts of the web, the beams are cambered 1.5” and 1.75” to meet deflection limits. The composite section used is typically 3 1/2” light weight concrete over 2” composite deck. The concrete is reinforced with additional WWR. Figure 8 below shows all three of the floor systems in adjacent bays of the building.

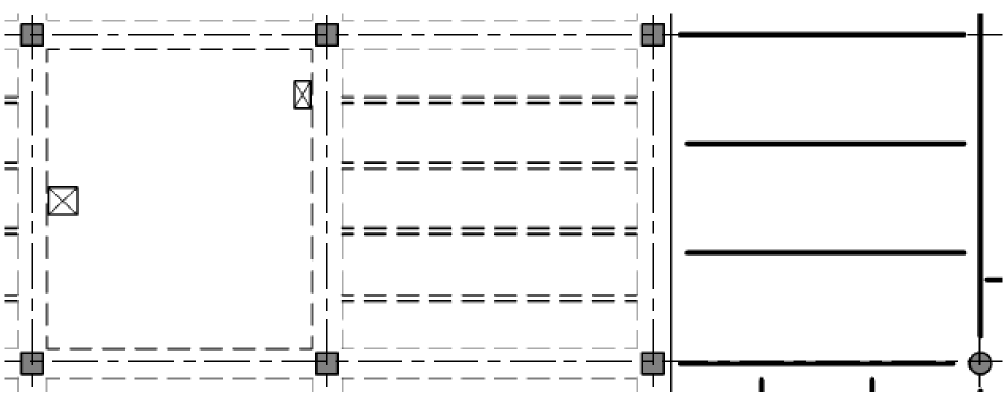

Figure 8: All Three Floor Systems in Adjacent Bays

Lateral Systems

Lateral forces are resisted in Helios Plaza by concrete moment frames. As mentioned before, girders run in the same direction as the one-way joist system to make up the frames in the East-West direction. In the North-South direction the same system is in place, however, the moment frames are broken up by the trading spaces. When a double story occurs, the floor that gets cut out is no longer there to distribute lateral forces from the building's enclosure to the moment frames. The force is instead transferred perpendicularly by horizontal circular HSS members to the one-way joists or to the floors above and below by the columns. These members are welded to the HSS columns and their arrangement can be seen below in Figure 9.

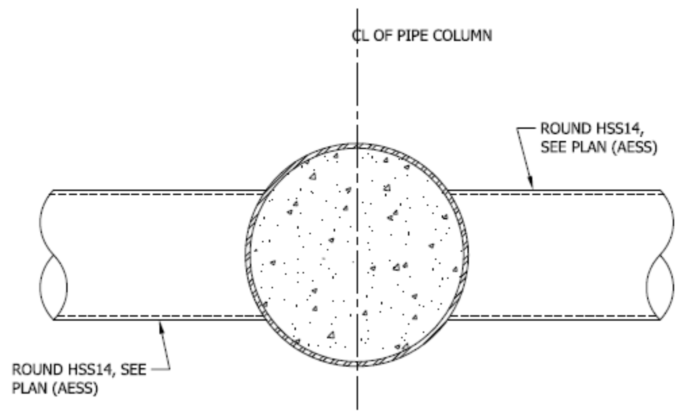

Figure 9: Round HSS Members Framing Into Each Other

Steel members that compose the floor system for the trading areas are not effective lateral members. They are not framed with moment connections and essentially only function to make a rigid diaphragm and to carry gravity loads. The section shown in Figure 10 illustrates that despite the size of the connection, the flanges are not restrained and do not transfer moment. Because of this, all of the steel wide-flange beams have been ignored when it comes to the computer modeling of the lateral system for analysis.

Overall, the building consists of twenty-two moment frames, with the majority of vertical members working in two frames. These frames will be explained further in the later section discussing the distribution of lateral loads.

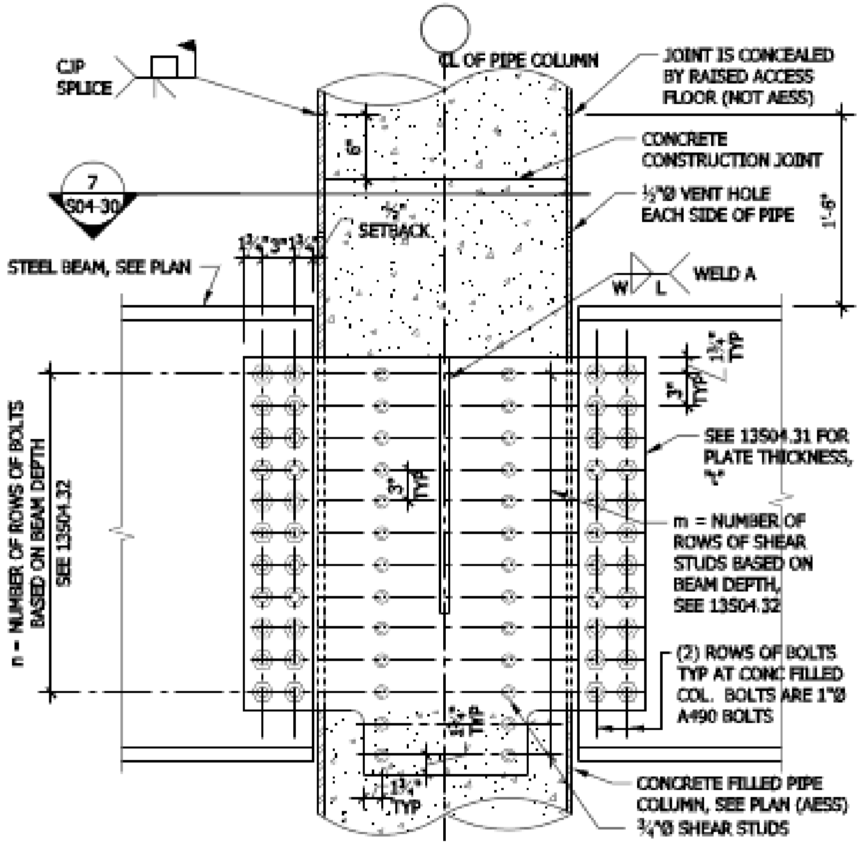

Figure 10: Steel Beam Connection to Concrete Filled Pipe Column

Order of Discussion

The topics of discussion for this technical assignment are organized as follows. First, the loads and load combinations used in analysis will be explained. Secondly, the distribution of lateral loads to their respective frames will be discussed. Thirdly, computer analysis and hand calculations will be compared in addition to a discussion of the relative stiffness and drifts of the frames. The final topic will be strength checks for members. Before any of the lateral discussion begins some further preliminary information in the form of design codes and material properties will addressed.

Codes and References

Helios Plaza was designed following all of the applicable guidelines for the state of Texas as well as the city of Houston. For the purpose of this technical assignment, the latest design codes will be utilized without specific regional additions.

Original Design Codes

- National Model Code:
  - 2003 International Building Code with City of Houston Amendments
- Design Codes:
  - Texas Architectural Barrier Act Standard
  - ANSI/AWS Structural Welding Code
- Structural Standards:
  - American Society of Civil Engineers, SEI/ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Code:
  - 2009 International Building Code
- Design Codes:
  - Steel Construction Manual 13th edition, AISC
  - ACI 318-05, Building Code Requirements for Structural Concrete
- Structural Standards:
  - American Society of Civil Engineers, SEI/ASCE 7-10, Minimum Design Loads for Buildings and Other Structures

Materials

Concrete		f'c (psi)
Spread Footings		4000
Basement Walls		6000
Slabs	On-Grade	3500
	Level 2	5000
	Level 3-6	4000
	Metal Deck	3500
Columns	Basement	6000
	Level 1	6000
	Levels 2-3	5000
	Levels 4-6	4000
	Steel Pipe Infill	7000
Beams		Same As Columns
Girders		Same As Columns
Reinforcement		**Fy (ksi)**
Rebar	#7 to #18	75
	All Other Sizes	60
Welded Wire	Smooth	65
	Deformed	75
Post-Tensioning Steel		**fs (ksi)**
Tendons		270
Concrete Masonry		**f'm (psi)**
All Types		1500
Structural Steel		**Fy (ksi)**
Wide Flange Shapes		50
Edge Angles/Bent Plates		36
HSS		42
Baseplates		36

Table 1: Material Strengths

Load Determinations

Dead Loads

For the analysis of the dead loads acting upon Helios Plaza, several assumptions were made. Although depth of metal deck and topping was specified, a specific deck type was not mentioned. The weight of lighting, electrical, and plumbing equipment was also not specified. Decks were chosen from the Vulcraft catalog and due to the nature of the building's function (IST) a superimposed load was added for cabling. A summary of the dead loads is tabulated below.

Floor Dead Load	
Load Source	Design Load
Normal Weight Concrete	150 PCF
Light Weight Concrete	115 PCF
Composite Decking	44 PSF
MEP	20 PSF
Cabling	5 PSF
Roof Dead Load	
Load Source	Design Load
Roof Decking	23 PSF
Roof Cladding	5PSF

Table 2: Dead Loads

Live Loads

Since Helios Plaza is an IST and trading office, many of the loads used are not prescribed directly in the ASCE 7-10 Code. The following table shows the comparison of the ASCE 7-10 live loads and the loads used by the designer.

Live Load		
Load Source	Design Load	ASCE 7-10 Load
First Floor Corridors	100 PSF	100 PSF
Corridors Above First Floor	80 PSF	80 PSF
Lobbies	100 PSF	100 PSF
Office	80 PSF	50 PSF
Partitions	20 PSF	20 PSF
Server Rooms	100 PSF	-
Mechanical Rooms	100 PSF	-
Roof	20 PSF	20 PSF

Table 3: Live Loads

With respect to the live loads used in the computer model, the live load was chosen to be 80 psf everywhere in the building for simplicity.

Snow Loads

Due to the location of Helios Plaza, there are no snow loads to be calculated, as Figure 7 shows.

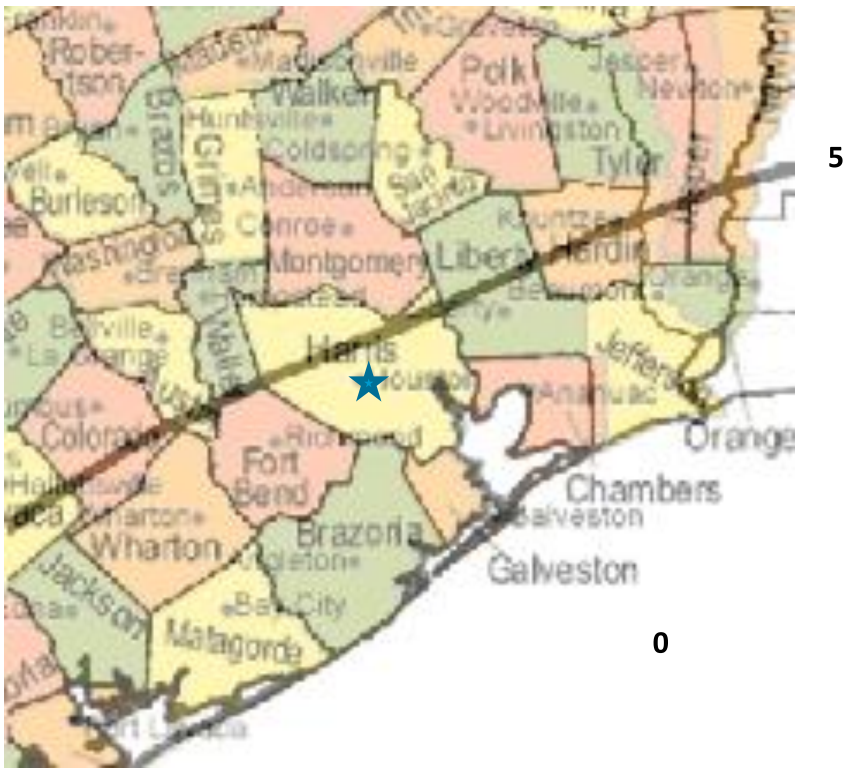

Figure 11: ASCE 7-10 Figure 7-1 Overlay on Texas County Map

Wind Analysis

The basis for the wind analysis comes from ASCE 7-10 Chapters 26 and 27. From these sections, the building was determined to have an occupancy category of III and a basic wind speed of 140 mph. Helios Plaza is surrounded by suburban housing subdivisions as well as a highway and other office buildings. Based upon the site location and geometry, the building exposure was determined to be category B. From the approximate natural frequency section of the code, it was determined that Helios Plaza is a flexible building which meant that it could be subjected to wind gusts. Further calculations and parameters can be found in Appendix B. The following table is a summary of the wind story forces calculated as result of the above procedure.

Wind Forces					
Level	Height (ft)	Load (k)		Shear (k)	
		N-S	E-W	N-S	E-W
r_2	113.0	80.8	47.4	80.8	47.4
r_1	98.5	222.4	120.2	303.1	167.6
6	81.5	262.6	134.8	565.7	302.3
5	66.5	237.9	121.5	803.5	423.9
4	51.5	229.0	116.4	1032.6	540.3
3	36.5	218.4	110.3	1251.0	650.6
2	21.5	247.1	123.4	1498.0	774.0
1	0.0	0.0	0.0	1498.0	774.0

Table 4: Wind Design Forces

From Table 4 above, it can be seen that the base shear is 865.8 kips in the North-South direction and 508.2 kips in the East-West Direction. Although there are no values to compare these calculations to, it is possible that these values could be higher than the ones calculated by the designer. According to structural specifications, the ASCE 7-02 basic wind speed the designer used was 110 mph, as compare to the ASCE 7-10 basic wind speed of 140 mph. An illustration of the wind pressure used to calcualate the values in table 4 can be seen below in Figures 12 and 13.

Figure 12: North-South Wind Pressure Diagram

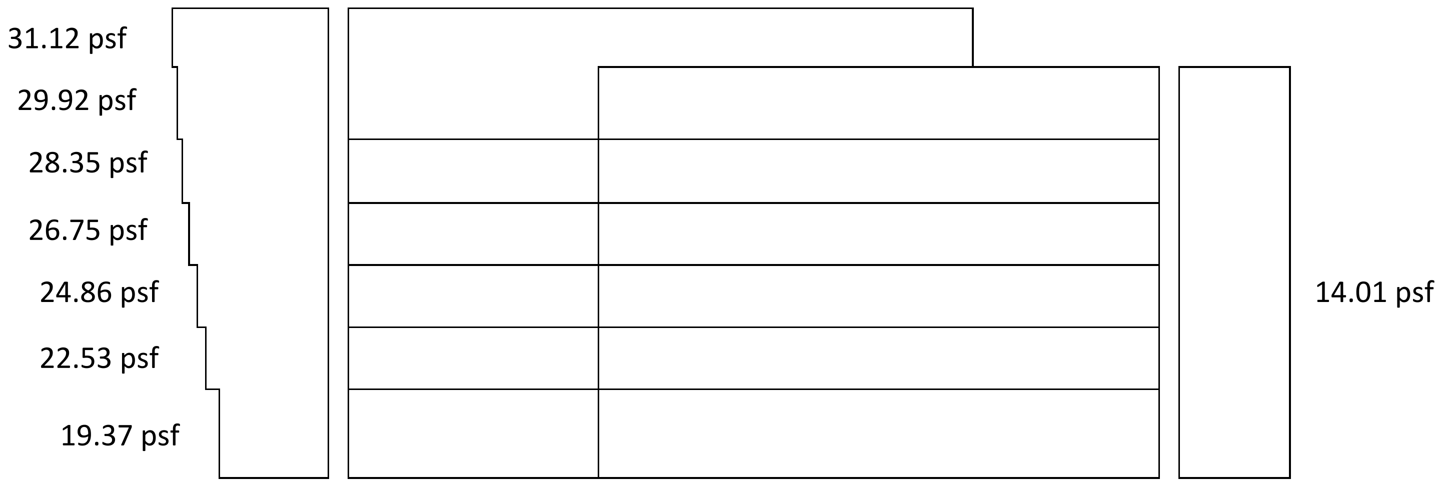

Figure 13: East-West Wind Pressure Diagram

In both Figure 12 and 13, the windward pressures are shown on the left side of the elevations and the leeward pressures on the right. Since these figures are not drawn to scale it is not apparent why the wind pressures in the East-West direction are larger than in the North-South direction. The footprint of the building is 355' by 195' and this ratio of approximately 1.8 accounts for small alterations in the wind pressures, but result in large differences in the story forces as can be seen in the following to figures, Figures 14 and 15.

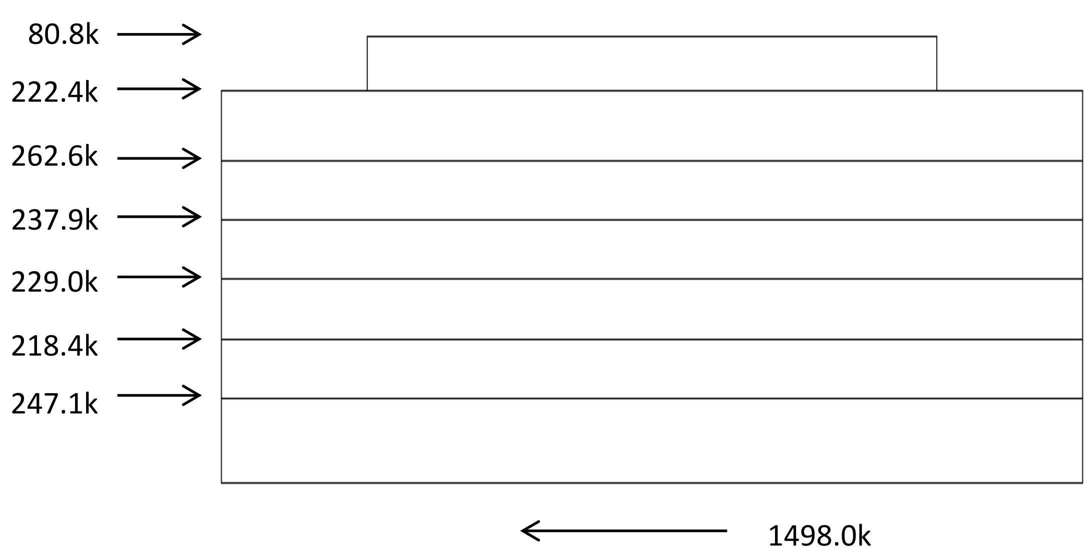

Figure 14: North-South Wind Story Forces

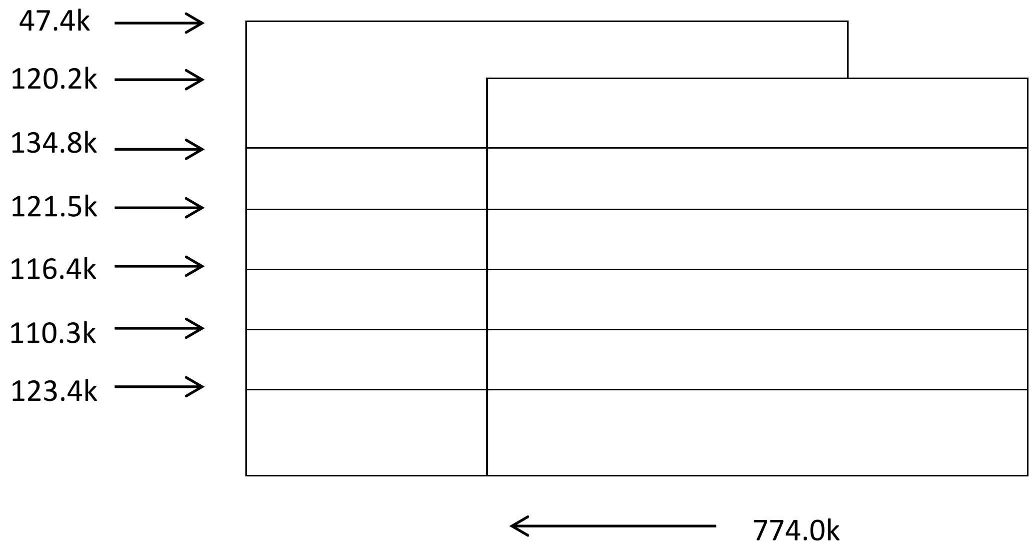

Figure 15: East-West Wind Story Forces

Seismic Analysis

The basis for the seismic design is from ASCE 7-10 Chapters 11 and 12. After finding the S_s and S_1 values from ASCE 7-10 Figures 22-1 and 22-2, the site class needed to be determined. Based upon Table 20.3-1, the site class was assumed to be Class E due to the high clay content of the soil at the site. Another assumption that was made was the R value, which was taken to be three since the lateral resistance system is a regular concrete moment frame. The weight of all the floors was also necessary to perform the seismic load analysis and using the assumed dead and live loads from the previous sections, these weights were tabulated. Calculations can be found in Appendix C.

Seismic Forces						
Level	Height (ft)	Weight (k)	$w_x h_x^k$	C_{vx}	F_x (k)	Shear (k)
roof	113	1089	545539.3	0.0860	78.8	78.8
lower roof	98.5	2672	1117390.8	0.1762	161.4	240.1
6	81.5	5701	1858332.0	0.2931	268.4	508.5
5	66.5	3957	987134.9	0.1557	142.6	651.1
4	51.5	5701	1016196.2	0.1603	146.8	797.8
3	36.5	3957	448519.3	0.0707	64.8	862.6
2	21.5	6496	367115.4	0.0579	53.0	915.6
Total		29573	6340227.9	1.0000	915.6	-

Table 5: Seismic Design Forces

As seen above in Table 5, the base shear due to seismic loading is larger than the base shear from wind loading in the East-West direction. This could be a point of contention because this region is not highly active seismically, but the low building height may account for the lack of wind controlling. The following Figure 16 is made by extracting the F_x values from Table 5.

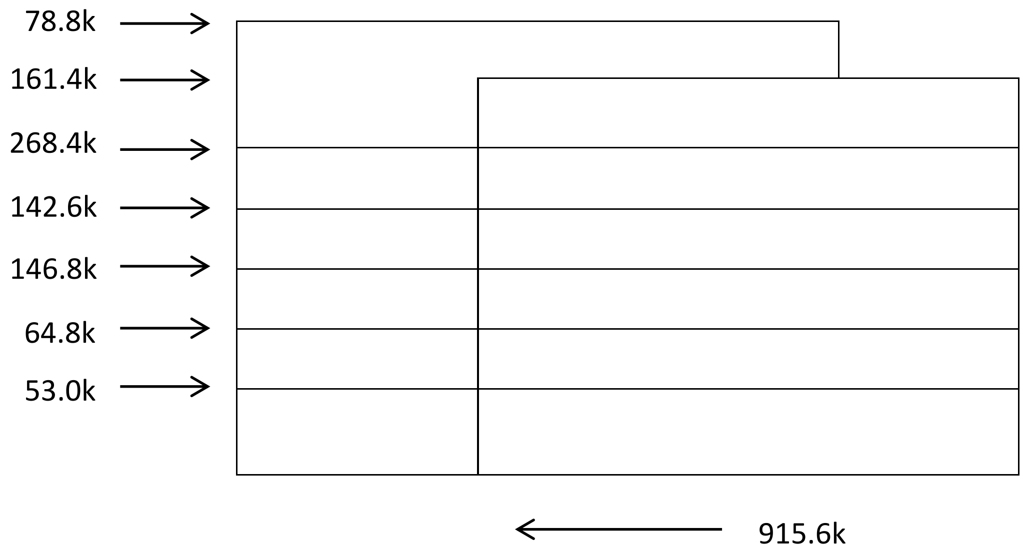

Figure 16: Seismic Force Diagram

Overall Building Torsion

Helios Plaza has several features that qualify as torsion irregularities. As per ASCE7-10 Table 12.3-2, Helios Plaza has an extreme soft story irregularity since the ground story lateral stiffness is less than 60% of the second story lateral stiffness. The building also has a weight (mass) irregularity between the second and third floor since the second story is more than 150% the weight of the third floor. Surprisingly, Helios Plaza does not have a diaphragm discontinuity irregularity since the cutout floor area is only 45% of the gross floor area as per ASCE7-10 Table 12.3-1. The results of these irregularities do not affect the analysis of the building though, because the load penalties only apply to the type of analysis allowed by the code. Even with the torsional irregularities, all types of seismic analysis are still permitted.

For the calculation of the center of mass, the only simplification made was in regards to the floor mass. The floor mass was applied to the diaphragm as a distributed load based on the average floor weight. With this assumption made, the center of mass (COM) was calculated with the distribution of mass based on member location. The results are summarized in the following Table 6. The center of rigidity, which is also in Table 6, was calculated using the relative stiffness values from Tables 9 and 10.

Story	Center of Mass (ft)		Cumulative COM (ft)		Center of Rigidity (ft)	
	X-Direction	Y-Direction	X-Direction	Y-Direction	X-Direction	Y-Direction
roof	177.500	120.000	177.500	120.000	187.409	89.383
lower roof	196.895	64.033	191.293	80.198	190.692	79.632
6th floor	185.302	93.960	187.667	88.528	186.215	87.726
5th floor	196.110	70.203	190.145	83.150	188.057	84.749
4th floor	185.302	93.960	188.694	86.388	186.215	87.726
3rd floor	196.110	70.203	189.959	83.628	184.277	90.647
2nd floor	185.302	93.960	188.931	85.908	186.215	87.726

Table 6: Building Center of Mass and Rigidity

Based upon the eccentricity between the centers of mass and the centers of rigidity, there is inherent torsion in the building. This was accounted for in the wind load calculations earlier and results in minor torsion in the building overall. The main way that the torsion affects the performance of the building is in response to earthquake loading. In mode three of the earthquake response, the building's deflection is controlled by rotation about the z-axis.

Load Combination Determination

Load Combinations

With the loadings from above, the controlling load combination needed to be determined using the basic combinations found in Section 2.3.2 of the ACSE7-10 code. These basic combinations are summarized in Table 7 below.

Load Combination	Equation
1	1.4D
2	1.2D + 1.6L + 0.5(L_r or S or R)
3	1.2D + 1.6(L_r or S or R) + (L or 0.5W)
4	1.2D + 1.0W + L + 0.5(L_r or S or R)
5	1.2D + 1.0E + L + 0.2S
6	0.9D + 1.0W
7	0.9D + 1.0E

Table 7: ASCE7-10 2.3.2 Basic Load Combinations

These combinations can be further simplified in the instance of Helios Plaza because there is no snow load and the rain load is neglected. With respect to the wind load factor within these combinations, the number of possibilities expands when section 27.4 of ASCE7-10 is taken into consideration. Figure 17 shows an excerpt from Figure 27.4-8 from the ASCE7-10 which illustrates the varied wind force load cases.

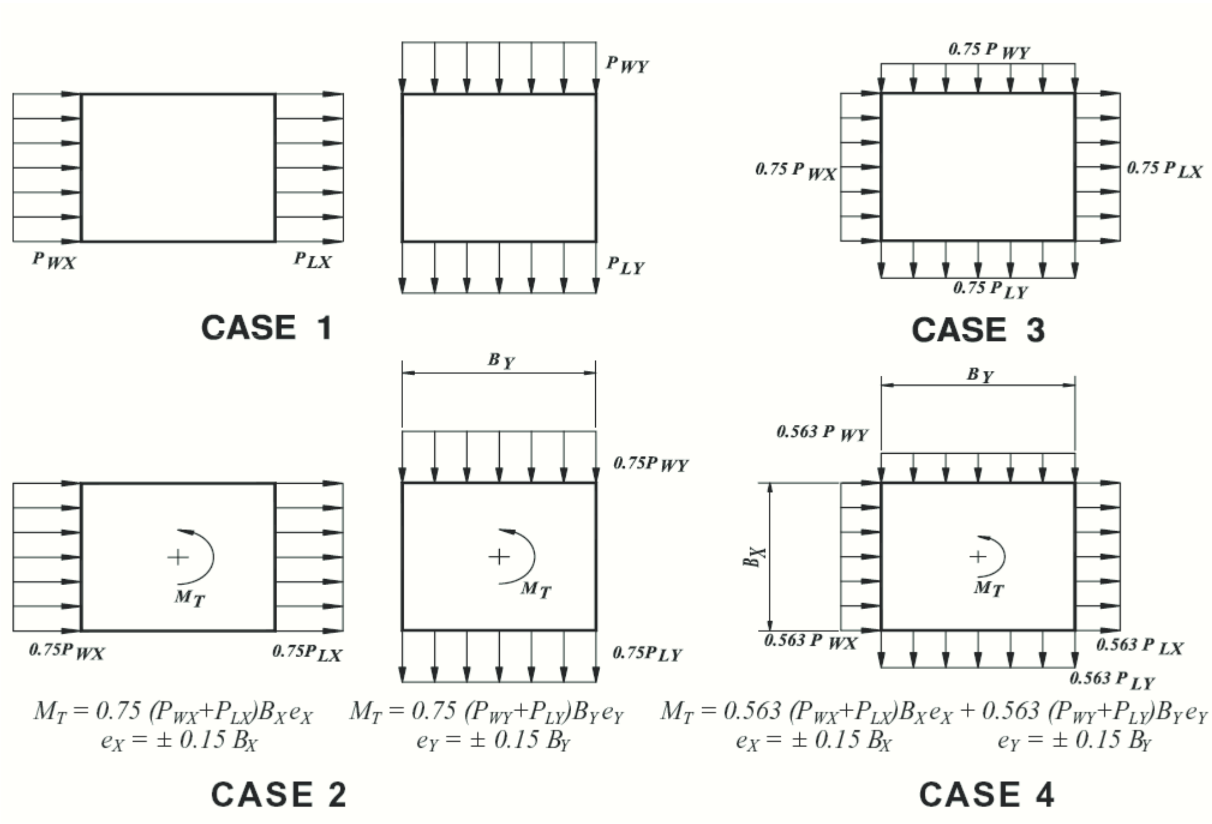

Figure 17: Excerpt from ASCE7-10 Figure 27.4-8

When every variation on the seven basic combinations from Table 6 is taken into account, Helios Plaza has sixty-four potential cases that can control the building design of the lateral system. These loads were then assessed on a computer model that would help determine which case controlled.

Computer Model

Moving forward with the load combinations discussed above, a computer model was developed to determine what combination was the controlling instance. To build the model, several assumptions were made. One of the most influential assumptions was to model the floors as a rigid diaphragm. This assumption is no stretch of the imagination though due to the highly stiff nature of monolithically poured concrete slabs and beams. The issue that would arise with diaphragm rigidity is the transition between one-way pan joist and the composite deck with concrete topping. The following Figure 18 shows that the typical transition between composite deck with concrete topping to one-way pan joists calls for sufficient development length of rebar between the two elements as well as embedded plates to take double angle shear connections. These measures ensure that the diaphragm moves as one entity and is rigid.

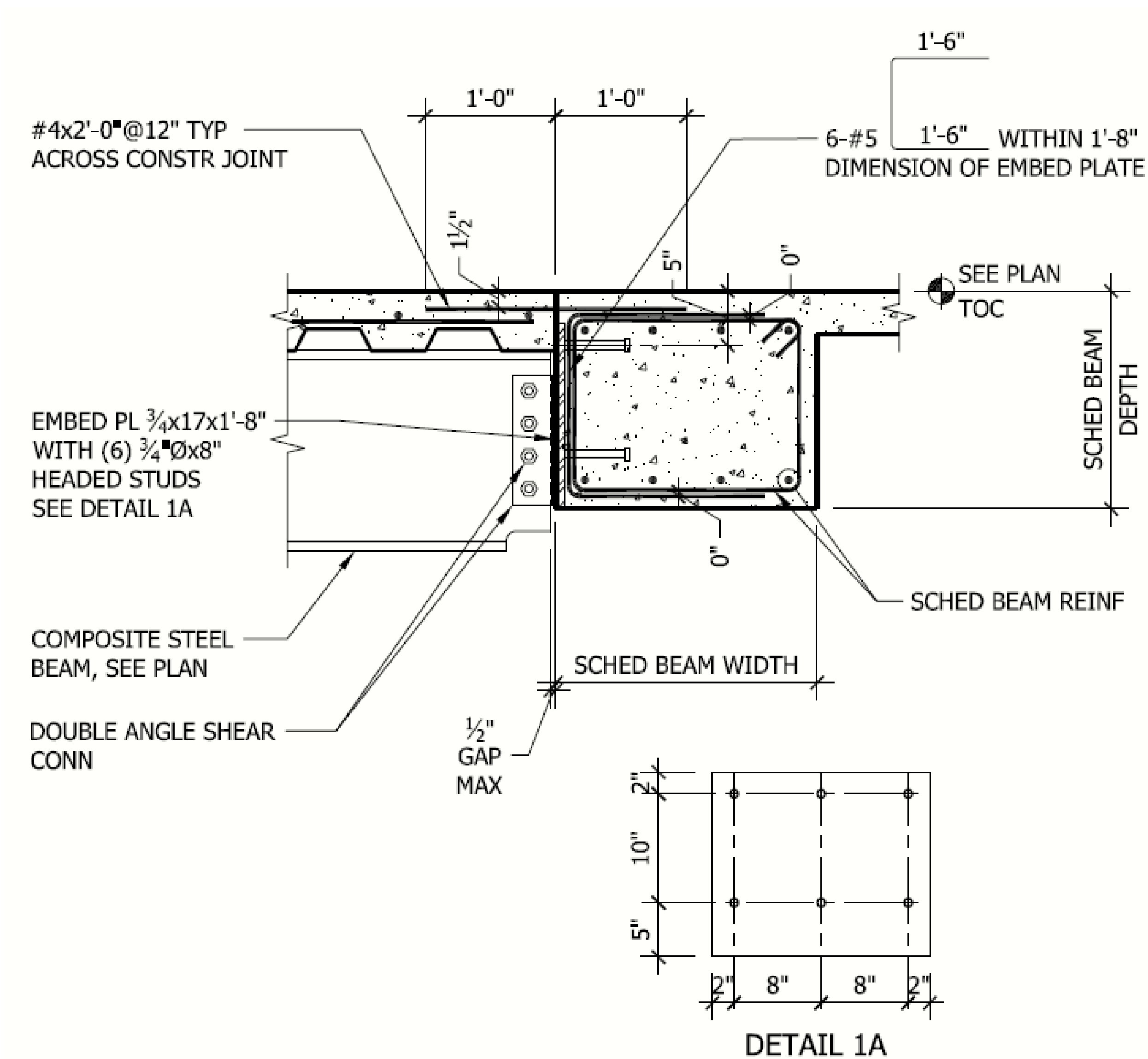

Figure 18: Typical Composite Deck with Concrete Topping to Concrete Beam Detail

When choosing the restraint type for the base, the joints were chosen to be pins because achieving a fixed base requires extensive detailing and runs up the cost of the design. The details for Helios Plaza call for simple anchor rod connections when attaching the steel pipe columns as discussed earlier and the concrete columns are integrally poured with the spread footings. For the concrete spread footings, the soil bearing capacity is not high enough to justify considering the supports fixed.

The cover distance used when defining the section properties was 2.75” for both rectangular and circular concrete columns. This value accounts for the minimum cover distance of 1.5” plus the diameter of the stirrups (which are generally #4 bars, but #5 bars at max) and approximately half of the bar diameter (ranging from #9 bars to #14 bars). Cover for the beams is also 1.5" for the top and bottom and a value of 2.5" was chosen for the location of the main reinforcing because the rebar in the beams was generally smaller than the columns (ranging from #7 bars to #11 bars).

Placing the lateral elements required choosing which elements were active in resisting the loads. All of the concrete beams and columns were modeled since the specified lateral system is called out as concrete moment frame. Being integrally poured and having sufficient development length between all the members ensures that the concrete members can transfer moment. In addition to these concrete elements, the concrete filled steel pipe columns were modeled as well as the HSS 14 x 0.375 steel beams that run the perimeter of the trading rooms. The HSS sections were included because they are welded to the pipe columns, enabling them to transfer moment and actively participate in lateral load resistance.

Controlling Case

After running the computer model in ETABS, the analysis showed that load combination 4 controls in the y-direction and load combination 5 controls in the x-direction. The load cases are as follows:

Load case 4: $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$

Load case 5: $1.2D + 1.0E + L + 0.2S$.

These results show that major contributing factor in the strength design of the lateral system in the y-direction is wind and in the x-direction is earthquake load. For the analysis, representative frames were chosen (Frames 6 and L) and the story shear forces were compared between all of the load cases. The results of this comparison can be seen on the following page in Table 8. These results are not surprising based upon the controlling base shears that were calculated earlier.

X-Direction	
Load Case	Shear Force (k)
1	0.0856
2	0.0734
3A	0.0734
3BX1	35.2822
3BX2	28.6654
3BX3	30.3981
3BX4	27.0145
3BX5	25.9388
3BX6	27.6815
3BX7	15.4111
3BX8	12.9327
3BX9	23.5879
4X1	70.4910
4X2	57.2574
4X3	60.7227
4X4	53.9556
4X5	51.8042
4X6	55.2897
4X7	30.7488
4X8	25.7920
4X9	47.1024
5X	85.8529
6X1	70.4727
6X2	57.2391
6X3	60.7044
6X4	53.9373
6X5	51.7859
6X6	55.2713
6X7	30.7304
6X8	25.7736
6X9	47.0841
7X	85.8345

Y-Direction	
Load Case	Shear Force (k)
1	0.3199
2	0.2742
3A	0.2742
3BY1	39.4786
3BY2	38.9450
3BY3	35.5270
3BY4	27.8725
3BY5	-30.9359
3BY6	29.9392
3BY7	-28.1153
3BY8	12.0392
3BY9	-18.1907
4Y1	78.6830
4Y2	77.6159
4Y3	70.7797
4Y4	55.4709
4Y5	-62.1460
4Y6	59.6042
4Y7	-56.5048
4Y8	23.8043
4Y9	-36.6555
5Y	53.1750
6Y1	78.6144
6Y2	77.5473
6Y3	70.7112
6Y4	55.4024
6Y5	-62.2145
6Y6	59.5356
6Y7	-56.5733
6Y8	23.7357
6Y9	-36.7241
7Y	53.1064

Table 8: Story Shear Force Results

Load Distribution

To determine the relative stiffness of each frame, a unit load was placed at the lower roof level for each frame, and their deflections were measured. Utilizing the relationship of P=kΔ, the stiffness can be found for each frame and these stiffness values can be compared to each other to get relative ratios. For the purpose of this technical assignment, a 1000 kip load was applied at the lower roof level and the deflection of the frame was measured at this same level. The following Tables 9 and 10 show the relative stiffness values from analysis.

1000 k Load In X-Direction				
Frame	Δ	K (k/in)	$K_{relative}$	$K_{relative}$ (%)
1	16.310	61.3	0.1441	14.41
2	29.629	33.8	0.0793	7.93
3	27.527	36.3	0.0854	8.54
4	28.044	35.7	0.0838	8.38
5	28.032	35.7	0.0838	8.38
6	15.140	66.0	0.1552	15.52
7	19.770	50.6	0.1189	11.89
8	9.413	106.2	0.2496	24.96
	Total	425.6	1	100

Table 9: Relative Stiffness of Frames in X-Direction

1000 k Load In Y-Direction				
Frame	Δ	K (k/in)	$K_{relative}$	$K_{relative}$ (%)
A	16.213	61.7	0.1238	12.381
B	29.308	34.1	0.0685	6.849
C	20.847	48.0	0.0963	9.629
D	29.599	33.8	0.0678	6.782
E	47.613	21.0	0.0422	4.216
F	37.353	26.8	0.0537	5.374
G	59.982	16.7	0.0335	3.347
H	60.068	16.6	0.0334	3.342
J	60.068	16.6	0.0334	3.342
K	27.210	36.8	0.0738	7.377
L	20.870	47.9	0.0962	9.619
M	20.503	48.8	0.0979	9.791
N	11.183	89.4	0.1795	17.950
	Total	498.2	1	100

Table 10: Relative Stiffness of Frames in Y-Direction

Plan views of Frames 1 through 8 can be seen figure 19 and Frames A through N can be seen in Figure 20. Sections of Figures 19 and 20 can be found in Appendix E.

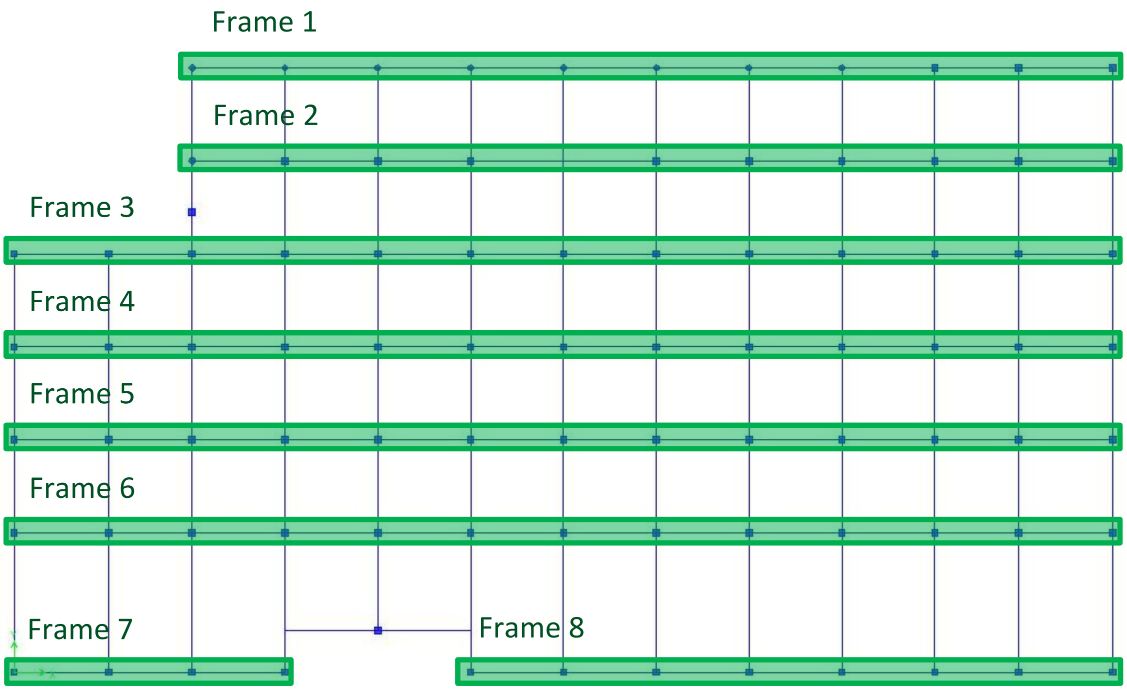

Figure 19: Frames in the X-Direction

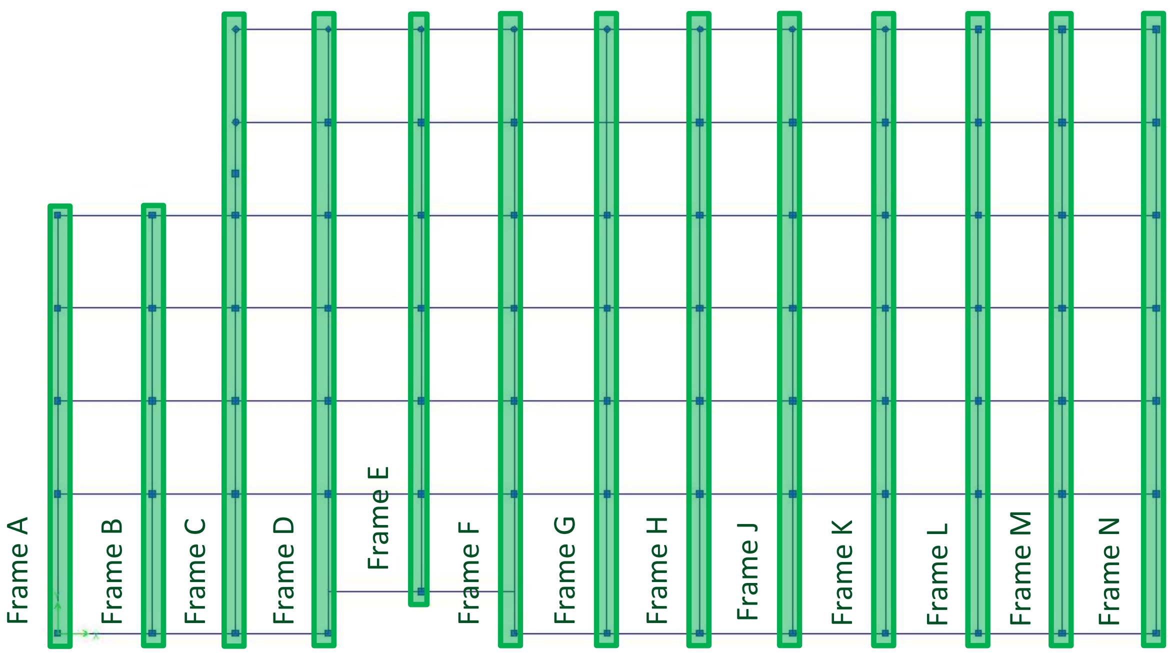

Figure 20: Frames in the Y-Direction

Relative Stiffness

Based upon the geometries of the frames, the relative stiffness values obtained seem logical. The frames that cross the trading floors take noticeably less load than the frames that are able to run continuously from one end of the building to the other. In particular, this trend can be seen in the x-direction with Frame 1. This is an exterior frame enclosing the trading floors that contains all of the moment connected HSS14 x 0.375 to concrete steel pipe columns. Frame 1 takes 14.41% of the lateral load in the x-direction especially since half of the frames pass through the trading floor open spaces.

In the y-direction, the one frame that seems to take less load than it should is Frame B. This can be explained by the shorter dimension of Frame B longitudinally as compared to all the other frames save one. Frame A is the same width and height as Frame B, but being an exterior frame gives is extra room for substantially larger columns and beams for higher rigidity.

To confirm the stiffness values retrieved from the computer model, a representative frame was chosen from both the x- and y-direction to be analyzed by hand. The two frames picked for analysis were Frames A and 8. The results of the hand calculations were 68.995 k/in and 110.391 k/in respectively. Both of these values are higher, with Frame A having a 10.57% increase in strength and Frame 8 having a 3.79% increase in strength. The explanation for the discrepancies comes down to the simplifications made for hand analysis. First of all, the hand calculations assume ideal circumstances, such as no axial deformation of the supporting beams and perfect fixity of connections. A second contributing factor involves the moment of inertia not accounting for the steel rebar in the columns used in the hand checks. A final reason for higher values is the idealized form of the stiffness equations used. The equations are based upon theoretical curvatures that rarely occur practically. Despite these differences, the values from the computer and from hand analysis are comparable.

Lateral Drifts

Analysis was carried out to determine the lateral drifts of the building to check against industry standards. Both the overall drift and the inter-story were checked to ensure that the lateral system performs to code stipulations.

Wind drifts were checked against the industry standard limit of H/400. Since this is a serviceability issue, the loads applied were unfactored to simulate the actual experience of the tenants. The following Tables 11 and 12 summarize the drifts from ETABS analysis and compare them to the allowable limits. As can be seen in the tables, the frames provide sufficient strength to limit the drifts and deflections of the building.

Drift Comparison North-South Wind Load					
Story	Height (in)	Δ_{actual} (in)	Δ_{allow} (in)	δ_{story} (in)	δ_{allow} (in)
roof	1356	1.517	3.390	0.087	0.435
lower roof	1182	1.429	2.955	0.110	0.510
6th	978	1.319	2.445	0.121	0.450
5th	798	1.198	1.995	0.160	0.450
4th	618	1.038	1.545	0.207	0.450
3rd	438	0.831	1.095	0.194	0.450
2nd	258	0.638	0.645	0.638	0.645

Table 11: Building Drifts under North-South Wind Load

Drift Comparison East-West Wind Load					
Story	Height (in)	Δ_{actual} (in)	Δ_{allow} (in)	δ_{story} (in)	δ_{allow} (in)
roof	1356	0.702	3.390	0.090	0.435
lower roof	1182	0.612	2.955	0.042	0.510
6th	978	0.570	2.445	0.037	0.450
5th	798	0.533	1.995	0.054	0.450
4th	618	0.479	1.545	0.072	0.450
3rd	438	0.408	1.095	0.091	0.450
2nd	258	0.316	0.645	0.316	0.645

Table 12: Building Drifts under East-West Wind Load

Seismic drifts were checked in the same manner as the wind loading cases. The main difference between the two checks is the allowable inter-story drift for seismic loading is prescribed in the ASCE7-10 code. In section 12.12.1, it defines the allowable inter-story drift for concrete moment frames in risk category III to be $0.015h_{sx}$. This equation yields much higher allowable limits than H/400, a result of the potential mode shapes that the building can enter. Under seismic loading, the building can enter mode shapes where the floors above and below can enter positive deflection while the story of interest goes into negative deflection. These larger limits allow the designers to design the member to points after the yield stresses as the building tries to dissipate energy through hopefully minor failures. The results of the ETABS analysis under unfactored earthquake, dead and live loads can be seen in the following Tables 13 and 14. Once again, the frames have sufficient strength to meet drift and deflection requirements.

Drift Comparison North-South Seismic Load					
Story	Height (in)	Δ_{actual} (in)	Δ_{allow} (in)	δ_{story} (in)	δ_{allow} (in)
roof	1356	1.108	3.390	0.163	2.610
lower roof	1182	0.944	2.955	0.080	3.060
6th	978	0.865	2.445	0.060	2.700
5th	798	0.805	1.995	0.086	2.700
4th	618	0.719	1.545	0.111	2.700
3rd	438	0.608	1.095	0.138	2.700
2nd	258	0.470	0.645	0.470	3.870

Table 13: Building Drifts under North-South Seismic Load

Drift Comparison East-West Seismic Load					
Story	Height (in)	Δ_{actual} (in)	Δ_{allow} (in)	δ_{story} (in)	δ_{allow} (in)
roof	1356	0.930	3.390	0.146	2.610
lower roof	1182	0.784	2.955	0.063	3.060
6th	978	0.721	2.445	0.055	2.700
5th	798	0.666	1.995	0.074	2.700
4th	618	0.592	1.545	0.094	2.700
3rd	438	0.499	1.095	0.115	2.700
2nd	258	0.384	0.645	0.384	3.870

Table 14: Building Drifts under East-West Seismic Load

Uplift Forces

To check the potential for uplift, all load cases were checked in the ETABS model. From the analysis, none of the base reactions experienced an uplift force under any of the load combinations. This means that in every instance of loading, the building has sufficient weight on the base supports to resist any potential upward forces induced by lateral loads. To check the computer analysis, hand calculations were performed on the support that experienced the most upward force from computer analysis. The computer reported a minimum support reaction in the z-direction of 34.22 kips at the point called out in Figure 21 on the next page. To simplify the hand calculations, wind Case 1 was used and the value was compared to the computer output. The computer gave a reaction force of 69.60 kips and the hand calculation (which can be found in Appendix E) yielded a value of 59.45 kips. The disparity between the two values can be attributed to assumptions about distribution of forces to the columns. The hand calculations function under the assumption that each column receives the same amount of axial load, when in actuality, this is not the case. It does make for a simple and conservative assumption because it results in a value nearer to uplift controlling.

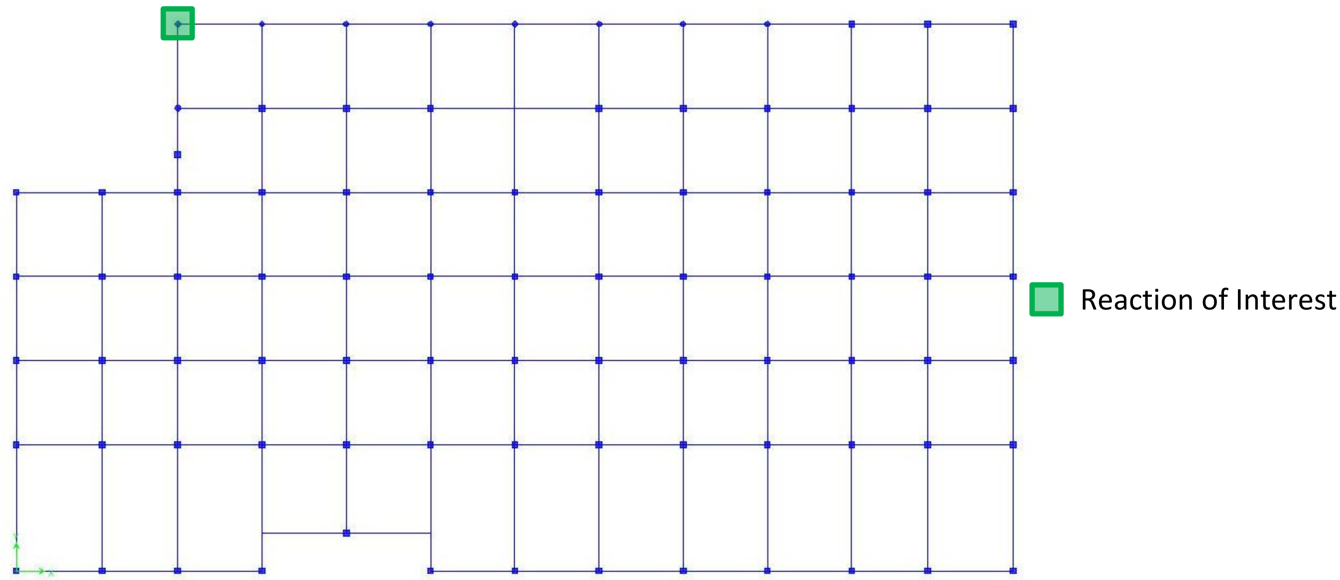

Figure 21: Reaction of Interest for Uplift Forces

Member Strength Checks

After determining the controlling load case and the distribution of loads to individual frames within the building, it was possible to check the adequacy of the members in strength resistance. Taking design values from the computer model, these numbers were compared to the members' axial, shear and moment capacities as calculated by hand.

The first member to be checked was a ground floor column that was subjected to the highest combined axial and moment load. The chosen column is a B7 column from the specifications and occurs at grid point D-6. Wind loading in the North-South direction was assessed to the building as per the controlling load combination and the values attained were a shear of V=12.48k, an axial load of P=334.67k, and a moment of M=243.43 ft-k. The hand calculations (which can be found in Appendix F) yielded a shear capacity of 60.0k and an interaction diagram that is sufficient for the axial and moment loads. The interaction diagram can be seen in Figure 22 on the next page.

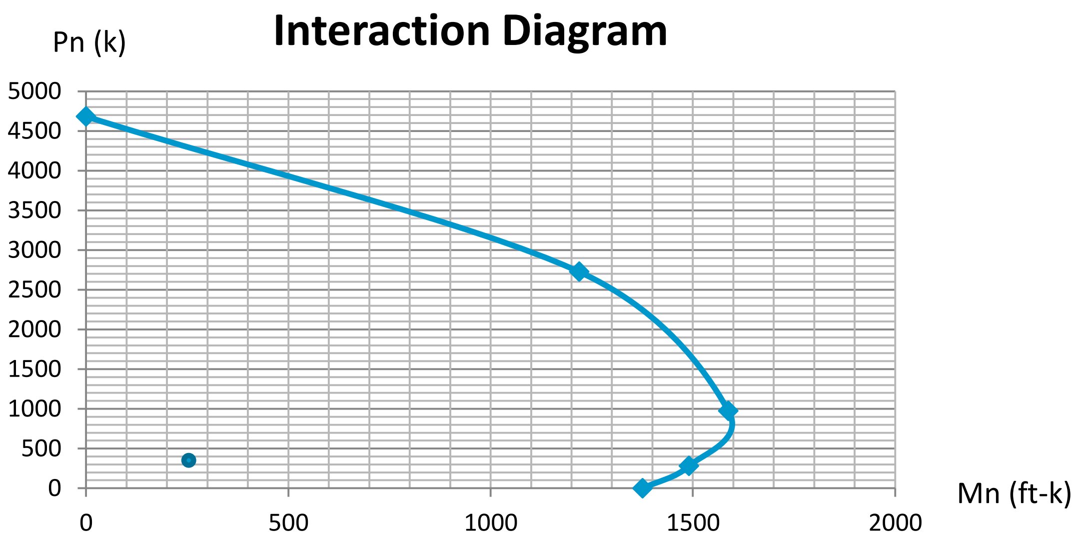

Figure 22: Column B7 Interaction Diagram

Utilizing the same technique as determining which column was a potential source of failure, a beam was checked for its shear and moment capacity. The beam chosen was in Frame N and was subjected to the controlling wind case as well. From the computer output, the following values in Figure 23 were checked by hand. After hand analysis, the beam was found to have a negative moment capacity of 1105.7 ft-k, a positive moment capacity of 959.3 ft-k, and a shear capacity of 112.7 ft-k, all of which are sufficient in strength. Detailed hand calculations can be found in Appendix F.

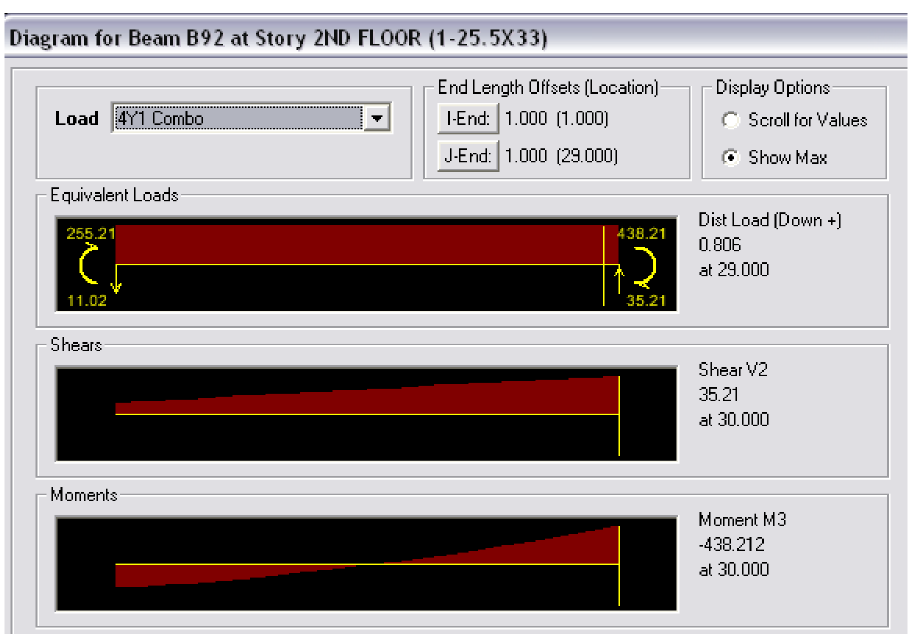

Figure 23: Beam Design Loads

Conclusions

From the analysis performed for this technical report, several conclusions can be drawn.

- Basic load combination 4 controls in the North-South direction
- Basic load combination 5 controls in the East-West direction
- The lateral frames are adequate in strength and deflection design

In determining the controlling load cases, two important facts arose. First, the controlling load combinations coincided with the controlling base shears from wind and seismic load analysis. This confirms that the load combinations are correctly computed. Secondly, overall building torsion does not affect the strength design of the building significantly. The controlling wind case was case 1, which is direct wind loading in one direction. Because the concrete moment frames can effectively resist moment throughout the entire building, even inherent torsion added to the accidental torsion cases did not result higher frame forces.

Distributing the lateral loads to the frames returned some interesting results as well. The exterior frames in both directions took the highest percentage of the load which can be attributed to the more rigid beams between the exterior columns. With the span lengths that are being considered, the axial deformation of beams is significant enough to affect the stiffness calculations of the frames. Another discovery from the load distribution is the lack of uplift on the base supports. Hand analysis confirmed that the weight of the building is large enough to exceed the upward force at the most susceptible support, further proving that the foundations do no need to consider uplift forces. A final result of the load distribution deals with the lateral drift checks. Under the controlling load cases, the overall building drift and the inter-story drift were acceptable compared to code and industry standards.

Checking critical members by hand showed that the lateral system is sufficiently designed for strength. The interaction diagram developed for the column checked proved that the member has plenty of capacity remaining should extreme loading situations occur. The beam checked also had more than adequate strength in bending and shear.

The one major area of concern going into the analysis turned out to be a non-entity, as the HSS steel beams were effective at transferring load to the diaphragm. The concrete moment frames ensure a rigid enough building to limit deflections, but are soft enough to allow a period of 1.41 seconds. Overall, the lateral system in Helios Plaza is adequate for all loading combinations in all aspects of design.

Appendix

Appendix A: General Building Layout

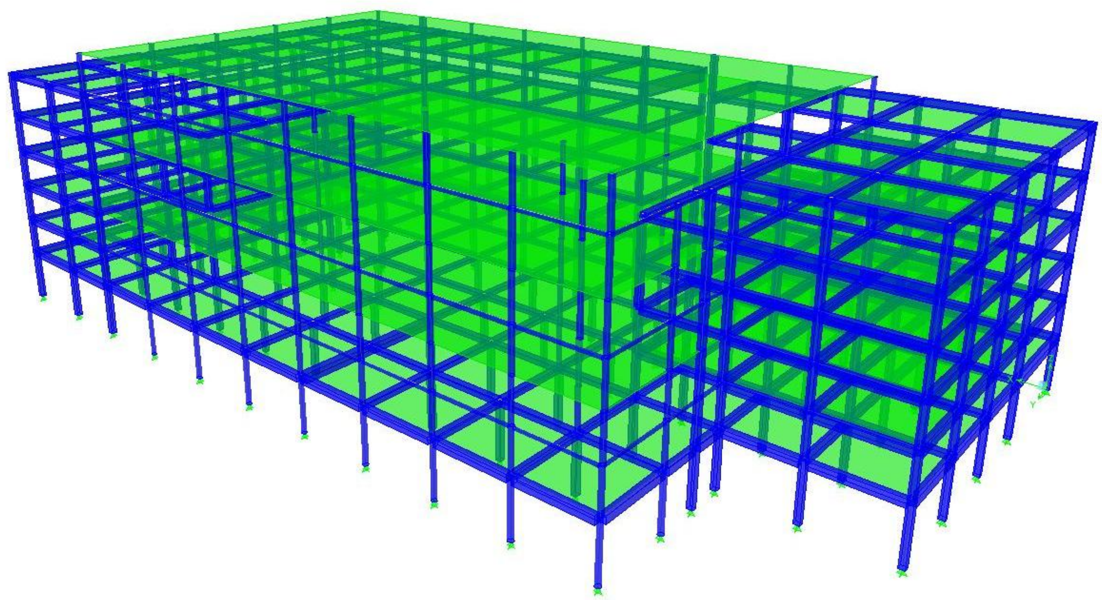

Figure 24: 3-D view from the Northwest Corner

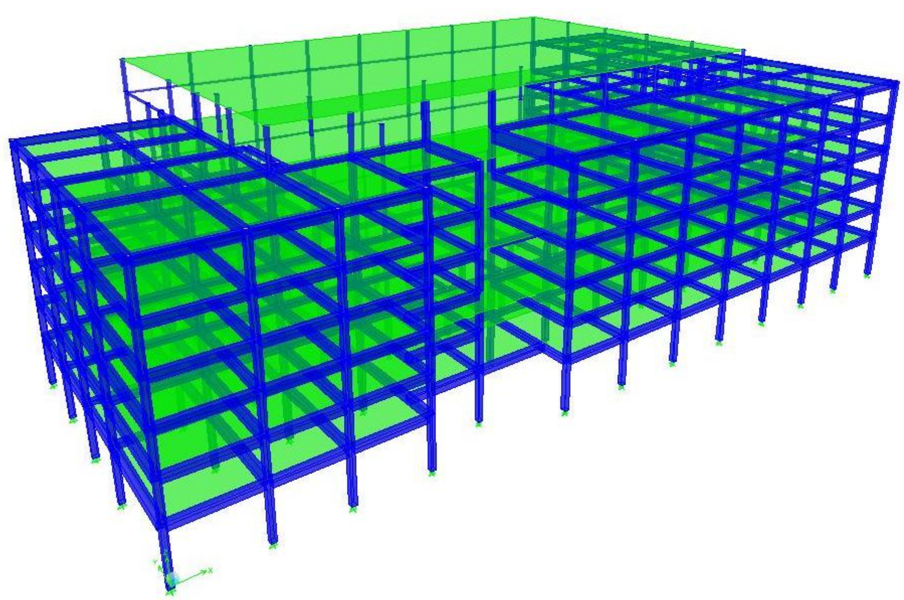

Figure 25: 3-D view from the Southwest Corner

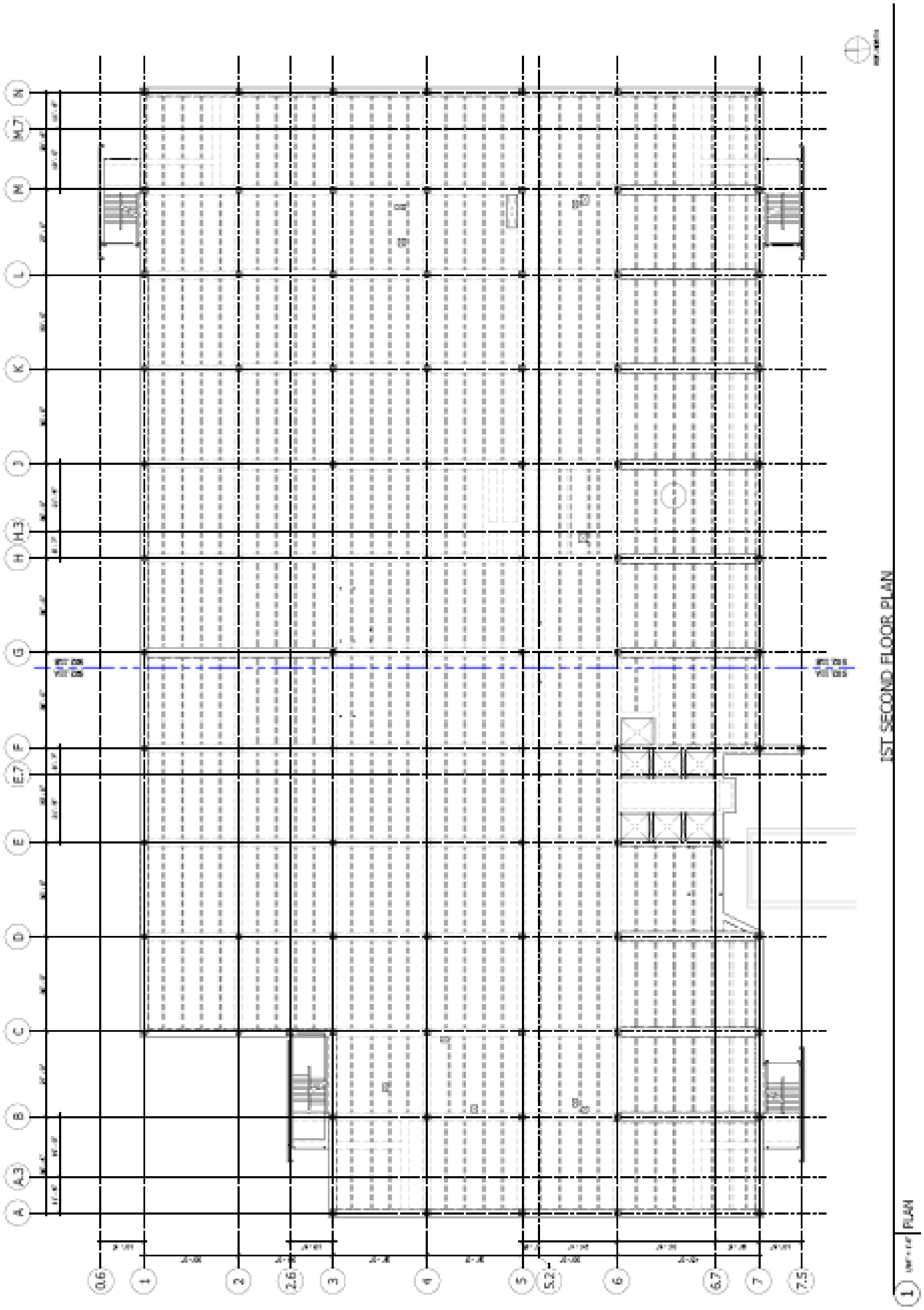

Figure 26: Second Floor Plan

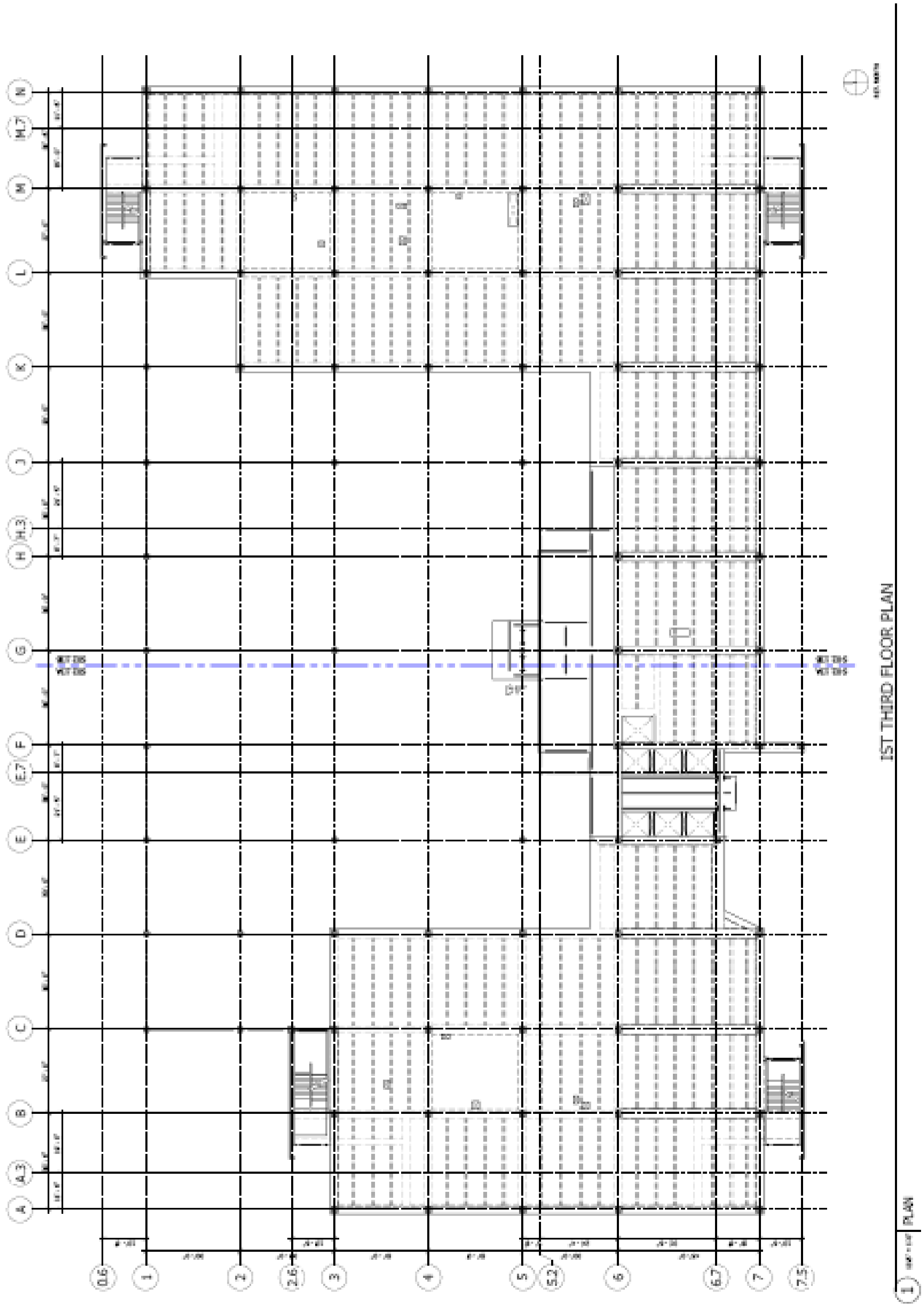

Figure 27: Third Floor Plan

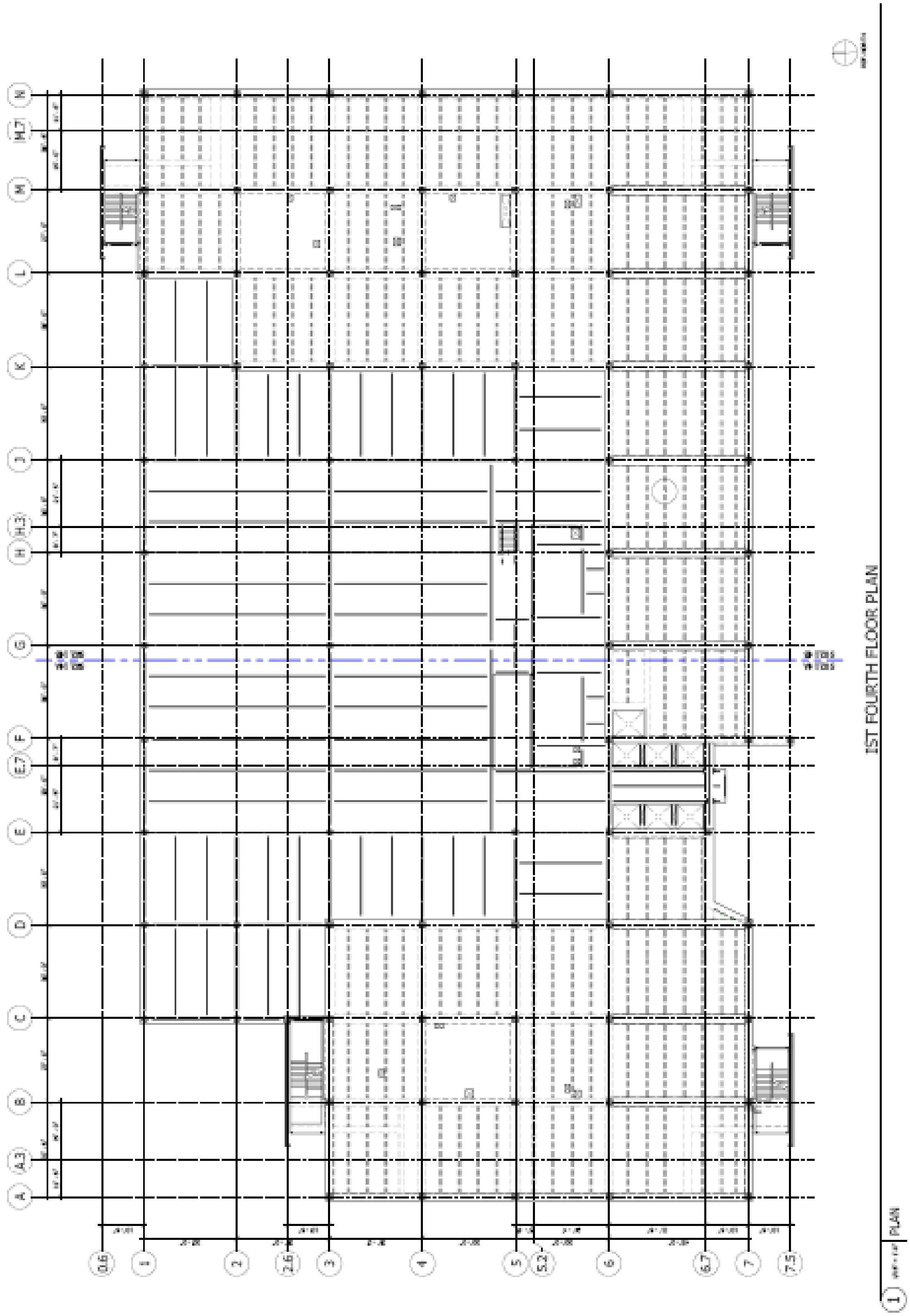

Figure 28: Fourth Floor Plan

Appendix B: Wind Analysis Calculations

The following table was used to tabulate the Gust Factor, G_f.

Parameter	N-S Direction	E-W Direction
n_1	0.618	0.618
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	4.073136697	4.073136697
N_1	2.272616358	2.272616358
L_{zbar}	406.8071365	406.8071365
l	320	320
ϵ_{bar}	0.333333333	0.333333333
z_{bar}	67.8	67.8
h	113	113
$Vbar_{zbar}$	110.6243953	110.6243953
β	0.015	0.015
b_{bar}	0.45	0.45
α_{bar}	0.25	0.25
V	140	140
R_n	0.085647553	0.085647553
η_{Rh}	2.903847738	2.903847738
η_{RB}	8.608752143	5.01106468
η_{RL}	16.77617306	28.820605
B	335	195
L	195	335
R_h	0.285253237	0.285253237
R_B	0.109414182	0.179647499
R_L	0.057831773	0.034095443
R	0.315109918	0.399709008
Q	0.773946132	0.808794669
I_{zbar}	0.266073708	0.266073708
c	0.3	0.3
G_f	0.847143709	0.891320818

Table 15: Gust Factor Calculations and Parameters

The following table was used to tabulate the wind pressures on the building.

Level	Height	K_z	q_z	p (psf)			
				N-S WWW	N-S LWW	E-W WWW	E-W LWW
roof	113	1.023	43.645	29.579	-18.487	31.122	-14.005
lower roof	98.5	0.984	41.966	28.441	-18.487	29.924	-14.005
6	81.5	0.932	39.755	26.942	-18.487	28.347	-14.005
5	66.5	0.880	37.510	25.421	-18.487	26.747	-14.005
4	51.5	0.818	34.868	23.631	-18.487	24.863	-14.005
3	36.5	0.741	31.602	21.417	-18.487	22.534	-14.005
2	21.5	0.637	27.167	18.411	-18.487	19.372	-14.005
1	0	0.570	24.310	16.475	-18.487	17.335	-14.005

Table 16: Wind Pressures Calculations

The next three pages contain the hand calculations, the assumptions made and the code references for the wind loads.

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Step 1: Category III building

Step 2: $V = 140$ mph

Step 3: $K_d = 0.85$
Exposure B
Topographic Factor, $K_{zt} = 1.0$
Gust Effect Factor, G_f

26.9.2.1 Limitations for Approximate Natural Frequency

1. $h = 113' < 300'$ ∴ okay

2. $L_{eff_{N\text{-}S}} = \dfrac{\sum h_i L_i}{\sum h_i} = \dfrac{113(195)}{113} = 195'$, $h_i \overset{?}{<} 4L_{eff_{N\text{-}S}} \Rightarrow 113' < 4(195) = 780'$ ∴ okay

$L_{eff_{E\text{-}W}} = \dfrac{\sum h_i L_i}{\sum h_i} = \dfrac{113(355)}{113} = 355'$, $h_i \overset{?}{<} 4L_{eff_{E\text{-}W}} \Rightarrow 113' < 4(355) = 1420'$ ∴ okay

26.9.3 Approximate Natural Frequency

$n_a = 43.5/h^{0.9} = 43.5/(113)^{0.9} = 0.618 < 1.0$ Hz ∴ Flexible

26.9.5 Flexible Buildings

$G_{f_{N\text{-}S}} = 0.925\left(\dfrac{1+1.7I_{\bar z}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_v I_{\bar z}}\right)$, where $g_Q = g_v = 3.4$

$g_R = \sqrt{2\ln(3600n_1)} + \dfrac{0.577}{\sqrt{2\ln(3600n_1)}} = 4.073$, $n_1 = n_a = 0.618$

$R = \sqrt{\frac{1}{\beta}R_n R_h R_B(0.53+0.47R_L)}$, where $\beta = 1.5\%$. From C26.9 for concrete buildings

$R_n = \dfrac{7.74N_1}{(1+10.3N_1)^{5/3}} = \dfrac{7.74(2.27)}{(1+10.3(2.27))^{5/3}} = 0.0857$

where $N_1 = \dfrac{n_1 L_{\bar z}}{\bar V_{\bar z}} = \dfrac{0.618(406.81)}{110.62} = 2.27$, $L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon} = 320\left(\frac{67.8}{33}\right)^{1/3.0} = 406.81$

$\bar z = 0.6h = 0.6(113) = 67.8'$, $\bar V_{\bar z} = \bar b\left(\frac{\bar z}{33}\right)^{\bar\alpha}\left(\frac{88}{60}\right)V = 0.45\left(\frac{67.8}{33}\right)^{1/4.0}\left(\frac{88}{60}\right)140 = 110.62$

R_h: set $\eta = 4.6n_1h/\bar V_{\bar z} = 4.6(0.618)113/110.62 = 2.90 > 0$ ∴ Use Eq. 26.9-15a

$R_\ell = R_h = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}(1-e^{-2\eta}) = \dfrac{1}{2.90} - \dfrac{1}{2(2.90)^2}(1-e^{-2(2.90)}) = 0.285$

R_B: set $\eta = 4.6n_1B/\bar V_{\bar z} = 4.6(0.618)335/110.62 = 8.61 > 0$ ∴ Use Eq. 26.9-15a

where $B_{N\text{-}S} = 335'$

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$R_B = R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{8.61} - \frac{1}{2(8.61)^2}(1-e^{-2(8.61)}) = 0.1094$

R_L: Set $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.618)195/110.62 = 16.78$

where $L_{N-S} = 195'$

$R_L = R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{16.78} - \frac{1}{2(16.78)^2}(1-e^{-2(16.78)}) = 0.0578$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = \sqrt{\frac{1}{0.015}(0.0857)0.285(0.1094)[0.53+0.47(0.0578)]} = 0.315$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{235+112.3}{406.81}\right)^{0.63}}} = 0.774$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{67.8}\right)^{1/6} = 0.266$

$G_{f_{N-S}} = 0.925 \left(\frac{1+1.7(0.266)\sqrt{3.4^2(0.774)^2 + 4.073^2(0.315^2)}}{1+1.7(3.4)0.266}\right) = \boxed{0.847}$

$G_{f_{E-W}} = \boxed{0.891}$ From Excel Spreadsheet

Enclosure Classification: Enclosed Building
Internal Pressure Coefficient, $GC_{pi} = \pm 0.18$

Step 4: $\alpha = 7.0$, $z_g = 1200'$ $\therefore$ $K_z = 2.01(z/z_g)^{2/\alpha}$

$K_{z4} = 2.01(51.5/1200)^{2/7} = 0.818$

Level	Height	K_z
1	0	0.570
2	21.5	0.637
3	36.5	0.741
4	51.5	0.818
5	66.5	0.880
6	81.5	0.932
r_1	99.25	0.986
r_2	113	1.023

Step 5: $q_{z4} = 0.00256 K_z K_{zt} K_d V^2 \text{ lb/ft}^2 = 0.00256(0.818)1.0(0.85)140^2 = 34.89$ psi

The rest of calculations are on Excel Spreadsheet

Step 6: N-S Wind:

Surface	L/B	Cp	Use
WWW	All	0.8	q_z
LWW	0.58	-0.5	q_h
SW	All	-0.7	q_h

E-W Wind:

Surface	L/B	Cp	Use
WWW	All	0.8	q_z
LWW	1.72	-0.36	q_h
SW	All	-0.7	q_h

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Step 7: 27.4.2 Enclosed and Partially Enclosed Flexible Buildings

N-S Wind

WWW: $p = q G_f C_p - q_i (GC_{pi})$ $p_4 = q_{z4} G_f C_p - q_i (GC_{pi}) = 34.89(0.847)0.8 - 43.65(\pm 0.18)$

$p_4 = 31.49$ psf

LWW: $p = 43.65(0.847)(-0.5) - 43.65(\pm 0.18) = -26.34$ psf

The rest of calculations are on Excel Spreadsheet

E-W Wind

WWW: $p_4 = 34.89(0.891)0.8 - 43.65(\pm 0.18) = 32.72$ psf

LWW: $p = 43.65(0.891)(-0.36) - 43.65(\pm 0.18) = -21.86$ psf

The rest of calculations are on Excel Spreadsheet

However, omit internal pressure for wind shear calcs

Appendix C: Seismic Analysis Calculations

The following two pages are the hand calculations, assumptions, and code references for seismic load calculations.

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From Fig 22-1 $S_s = 8.8\% g$

From Fig 22-2 $S_1 = 3.6\% g$

11.4.2 Site Class: E, From Table 20.3-1

11.4.3 Site Coefficients & Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

$S_{MS} = F_a S_s = 2.5(0.088) = 0.22$, $S_{M1} = F_v S_1 = 3.5(0.036) = 0.126$

$F_a = 2.5$ From Table 11.4-1, $F_v = 3.5$ From Table 11.4-2

11.4.4 Design Spectral Acceleration Parameter

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.22) = 0.147$, $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.126) = 0.084$

11.5 Occupancy Category: III, ∴ From Table 1.5-2 $I_e = 1.25$

11.6 Seismic Design Category (SDC)

From Table 11.6-1, SDC = A & From Table 11.6-2, SDC = B — More Conservative Controls

12.6 Analysis Procedures

From Table 12.6-1, Equivalent Lateral Force Analysis Permitted

12.8.1 Seismic Base Shear

$V = C_s W$, where $C_s = \frac{S_{DS}}{(R/I_e)} = \frac{0.147}{(3/1.25)} = 0.061$

12.8.2.1 Approximate Fundamental Period

$T_a = C_t h_n^x = 0.016(113)^{0.9} = 1.13\ s$ (Values From Table 12.8-2)

$\Rightarrow T = 1.13 < T_L = 12$ (From Table 22-12) $\therefore C_s \leq \frac{S_{D1}}{T(\frac{R}{I})} = \frac{0.084}{1.13(\frac{3}{1.25})} = 0.031 \Leftarrow$ controls

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$w_{RF} = 1089^k$, $w_{LRF} = 2672^k$, $w_{FL6} = 5701^k$, $w_{FL5} = 3957^k$, $w_{FL4} = 5701^k$

$w_{FL3} = 3957^k$, $w_{FL2} = 6496^k$ $\Rightarrow$ $w_T = 29,573^k$

$V = C_s w = 0.031(29573) = 915.6^k$

12.8.3 Vertical Distribution of seismic Forces

$F_x = C_{vx} V$, where $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1} w_i h_i^k}$, where $k = 1.315$ from linear interpolation

Using Eqn 12.8-12

$$C_{RF} = \frac{1089(113)^{1.315}}{1089(113)^{1.315} + 2672(98.5)^{1.315} + 5701(81.5)^{1.315} + 3957(66.5)^{1.315} + 5701(51.5)^{1.315} + 3957(36.5)^{1.315} + 6496(21.5)^{1.315}}$$

$= 0.0860$

$F_{RF} = 0.0860(915.6) = 78.8^k$

The rest of calculations are compiled in Excel Spreadsheet

Appendix D: Load Determination

The following excel tables are the input values for ASCE7-10 section 27.4 wind cases.

1			2			3		
fx	fy	mz	fx	fy	mz	fx	fy	mz
47.4	0	-4615.4	0	80.8	9601.9	35.5	0	9011.9
120.2	0	-25771.5	0	222.4	35200.3	90.1	0	12312.7
134.8	0	0.0	0	262.6	0.0	101.1	0	35473.5
121.5	0	-18597.0	0	237.9	30133.5	91.2	0	18047.5
116.4	0	-13657.8	0	229.0	23953.3	87.3	0	20410.5
110.3	0	-9068.8	0	218.4	17760.3	82.7	0	22230.2
123.4	0	0.0	0	247.1	0.0	92.6	0	32485.9

Table 17: Wind Cases 1-3 Forces

4			5			6		
fx	fy	mz	fx	fy	mz	fx	fy	mz
35.5	0	15935.1	0	60.6	45901.4	0	60.6	31498.6
90.1	0	50969.9	0	166.8	132964.5	0	166.8	80164.0
101.1	0	35473.5	0	196.9	125832.4	0	196.9	125832.4
91.2	0	45943.0	0	178.4	136595.4	0	178.4	91395.1
87.3	0	40897.2	0	171.8	127734.6	0	171.8	91804.7
82.7	0	35833.4	0	163.8	117980.5	0	163.8	91340.1
92.6	0	32485.9	0	185.3	118402.8	0	185.3	118402.8

Table 18: Wind Cases 4-6 Forces

7			8			9		
fx	fy	mz	fx	fy	mz	fx	fy	mz
35.5	60.6	3739.8	35.5	-60.6	-10663	26.7	45.5	41221.6
90.1	166.8	7071.7	90.1	-166.8	-45728.9	67.7	125.2	109054.8
101.1	196.9	0.0	101.1	-196.9	0	75.9	147.8	121087.0
91.2	178.4	8652.4	91.2	-178.4	-36547.9	68.4	133.9	116085.3
87.3	171.8	7721.6	87.3	-171.8	-28208.3	65.6	129.0	111207.6
82.7	163.8	6518.6	82.7	-163.8	-20121.8	62.1	122.9	105251.5
92.6	185.3	0.0	92.6	-185.3	0	69.5	139.1	113267.1

Table 19: Wind Cases 7-9 Forces

10			11			12		
fx	fy	mz	fx	fy	mz	fx	fy	mz
26.7	-45.5	-27691.7	26.7	45.5	-35606.8	26.7	-45.5	11683.0
67.7	-125.2	-90569.3	67.7	125.2	-98437.9	67.7	-125.2	21915.0
75.9	-147.8	-67829.4	75.9	147.8	-121087.0	75.9	-147.8	67829.4
68.4	-133.9	-88989.9	68.4	133.9	-103095.2	68.4	-133.9	34119.3
65.6	-129.0	-80564.6	65.6	129.0	-99614.9	65.6	-129.0	38214.6
62.1	-122.9	-71876.6	62.1	122.9	-95464.9	62.1	-122.9	41667.0
69.5	-139.1	-64494.9	69.5	139.1	-113267.1	69.5	-139.1	64494.9

Table 20: Wind Cases 10-12 Forces

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Model Assumption / Assignments

Floor 2: $w = 6496^k$, $A = 64875$; $\therefore \frac{w}{A} = 0.1001 \frac{k}{ft^2}$, $\frac{M}{A} = \frac{0.1001\ k/ft^2}{32.2\ ft\,k/s^2} = 0.00311\ s^2/ft^3$

$\frac{M}{A} = 0.00311 \frac{s^2}{ft^3} \cdot \frac{ft^3}{1728 in^3} = 1.7996\text{E-}6\ s^2/in^3$

Floor 3: $\frac{w}{A} = \frac{3957^k}{35175_{sf}} = 0.1125 \frac{k}{ft^2} \Rightarrow \frac{M}{A} = \frac{0.1125\ k/ft^2}{32.2\ ft\cdot k/s^2} = 0.0035 \frac{s^2}{ft^3} \cdot \frac{ft^3}{1728\ in^3} = 2.0218\text{E-}6\ s^2/in^3$

Floor 4: $\frac{w}{A} = \frac{5701}{64875} = 0.0878 \frac{k}{ft^2} \Rightarrow \frac{M}{A} = \frac{0.0878\ k/ft^2}{32.2\ ft\cdot k/s^2} = 0.0027 \frac{s^2}{ft^3} \cdot \frac{ft^3}{1728 in^3} = 1.5794\text{E-}6 \frac{s^2}{in^3}$

Floor 5: Same as Floor 3 $\therefore \frac{M}{A} = 2.0218\text{E-}6\ s^2/in^3$

Floor 6: Same as Floor 4 $\therefore \frac{M}{A} = 1.5794\text{E-}6\ s^2/in^3$

Lower Roof: $\frac{w}{A} = \frac{2672}{28875} = 0.0925 \frac{k}{ft^2} \Rightarrow \frac{M}{A} = \frac{0.0925\ k/ft^2}{32.2\ ft\cdot k/s^2} = 0.0029 \frac{s^2}{ft^3} \cdot \frac{ft^3}{1728 in^3} = 1.6633\text{E-}6\ s^2/in^3$

Roof: $\frac{w}{A} = \frac{1089}{36000} = 0.0302 \frac{k}{ft^2} \Rightarrow \frac{M}{A} = \frac{0.0302\ k/ft^2}{32.2\ ft\cdot k/s^2} = 0.0009 \frac{s^2}{ft^3} \cdot \frac{ft^3}{1728 in^3} = 5.4351\text{E-}7\ s^2/in^3$

Rigid Diaphram assumed, HSS sections, Shear connections only

Pinned Base, mode 1 T = 1.7345 sec y-translation

Fixed Base, mode 1 T = 1.2722 sec y-translation

Floor 2: DL = 2490 / 64875(144) = 2.665 E-4 k/in²

Floor 3: DL = 1470 / 35175(144) = 2.902 E-4 k/in²

Floor 4: DL = 2723 / 64875(144) = 2.915 E-4 k/in²

Floor 5: DL = 2.902 E-4 k/in²

Floor 6: DL = 2.915 E-4 k/in²

Lower Roof: DL = 1106 / 28875(144) = 2.660 E-4 k/in²

Roof: DL = 797 / 36000(144) = 1.537 E-4 k/in²

Appendix E: Load Distribution

The following figures are the frames as they were input into the ETABS computer model. The absence of beams is most often due to a rigid diaphragm restraining the horizontal movement.

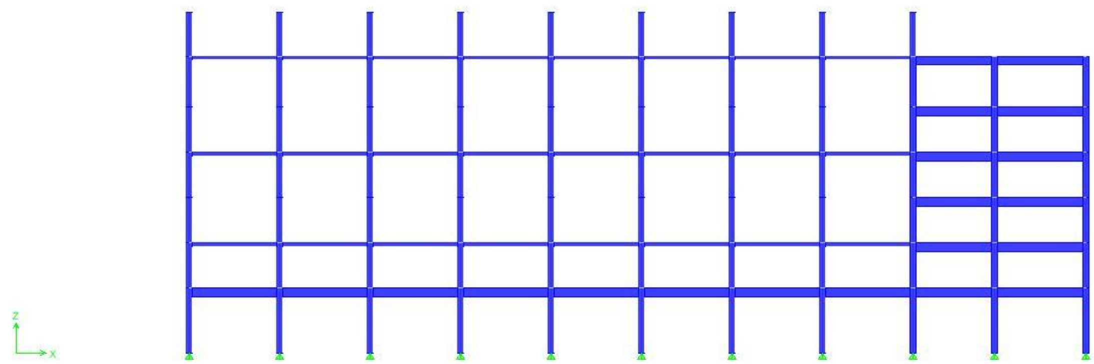


Figure 29: Frame 1

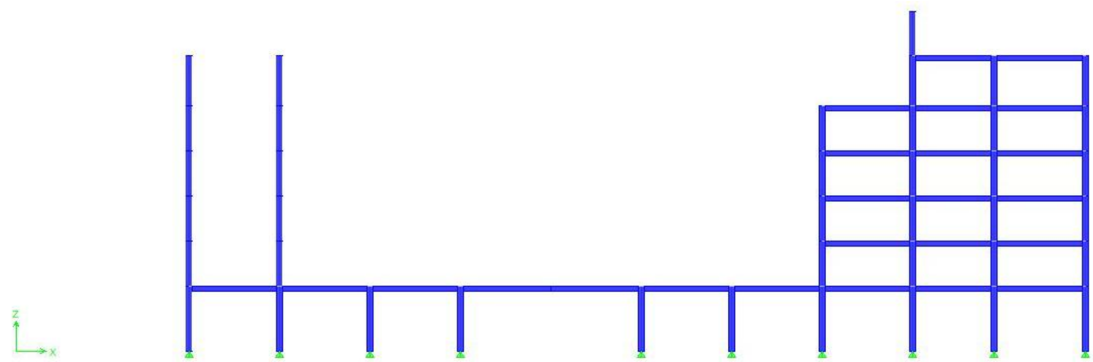


Figure 30: Frame 2

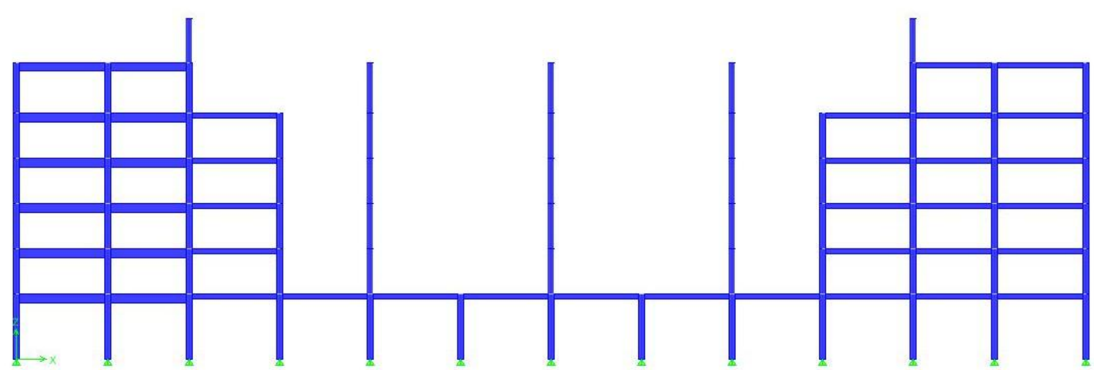


Figure 31: Frame 3

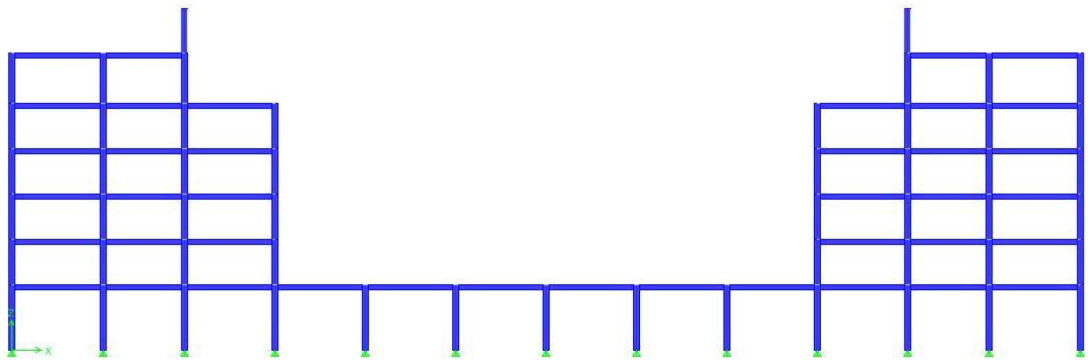


Figure 32: Frame4

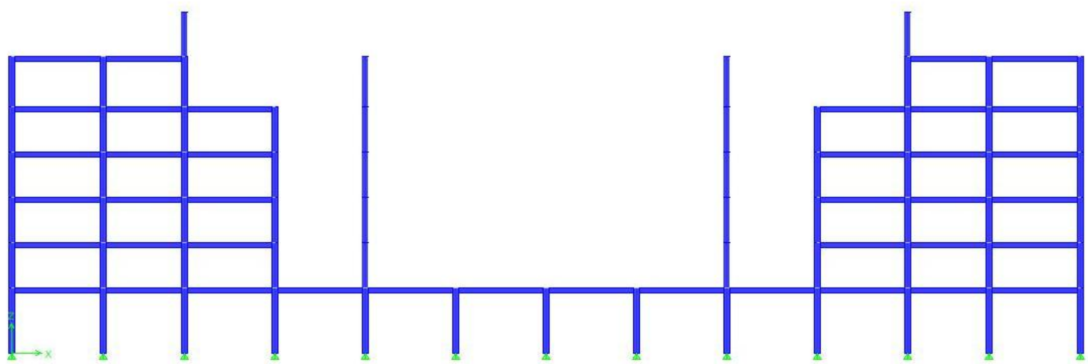


Figure 33: Frame 5

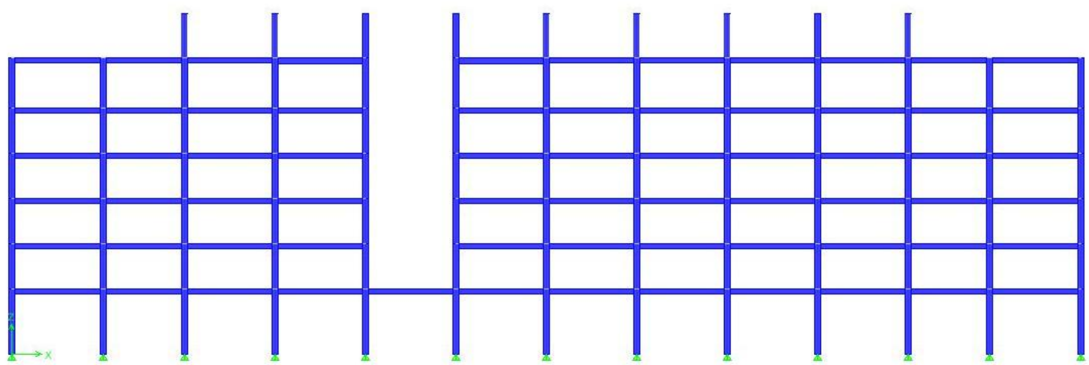


Figure 34: Frame 6

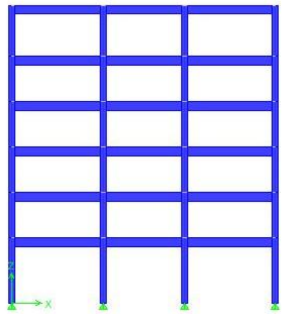


Figure 35: Frame 7

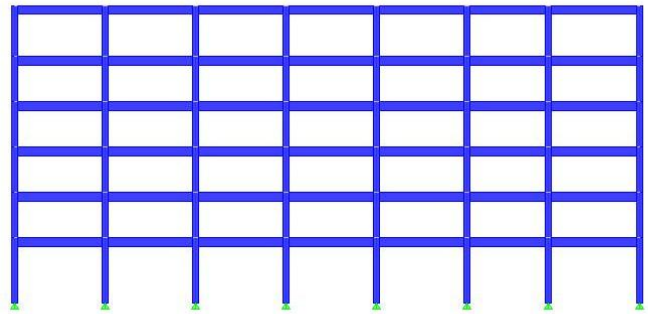


Figure 36: Frame 8

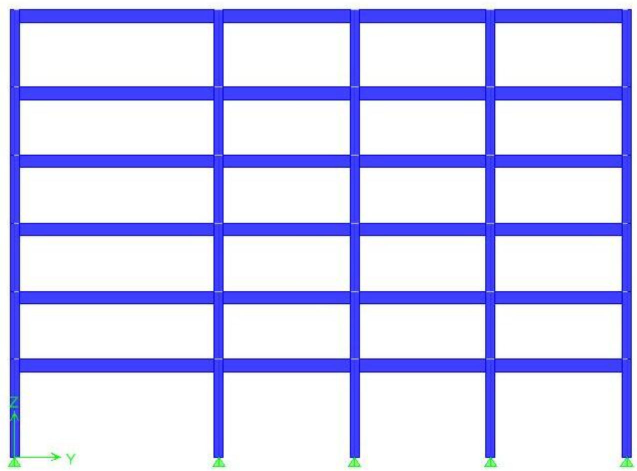


Figure 37: Frame A

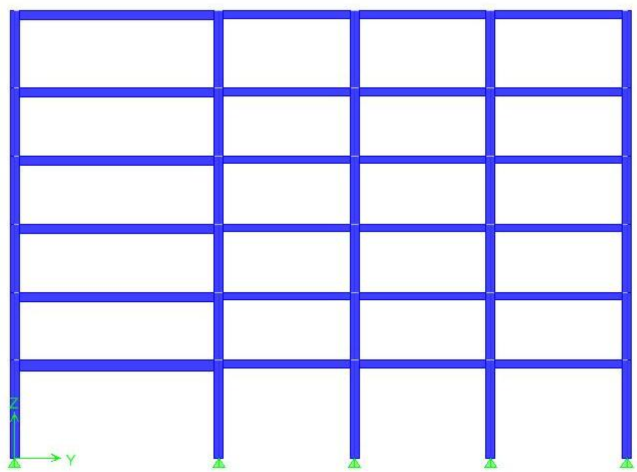


Figure 38: Frame B

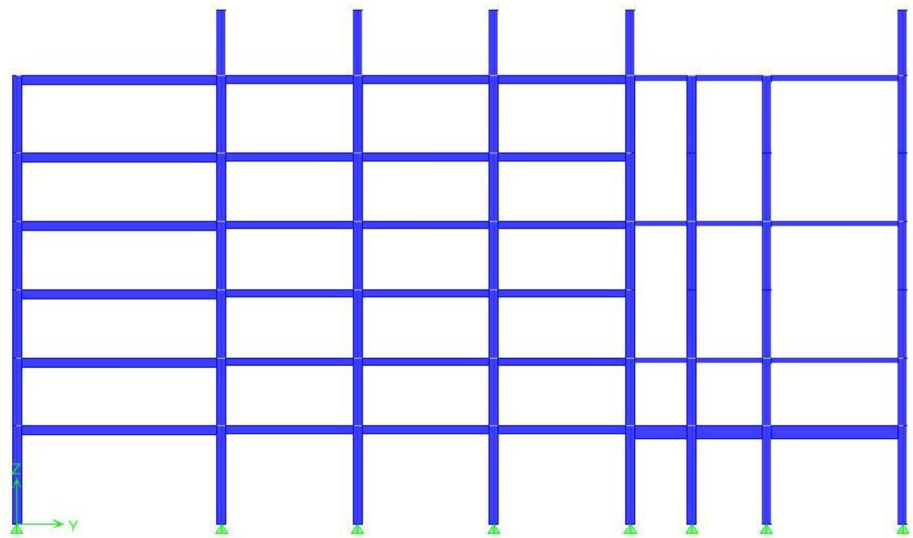


Figure 39: Frame C

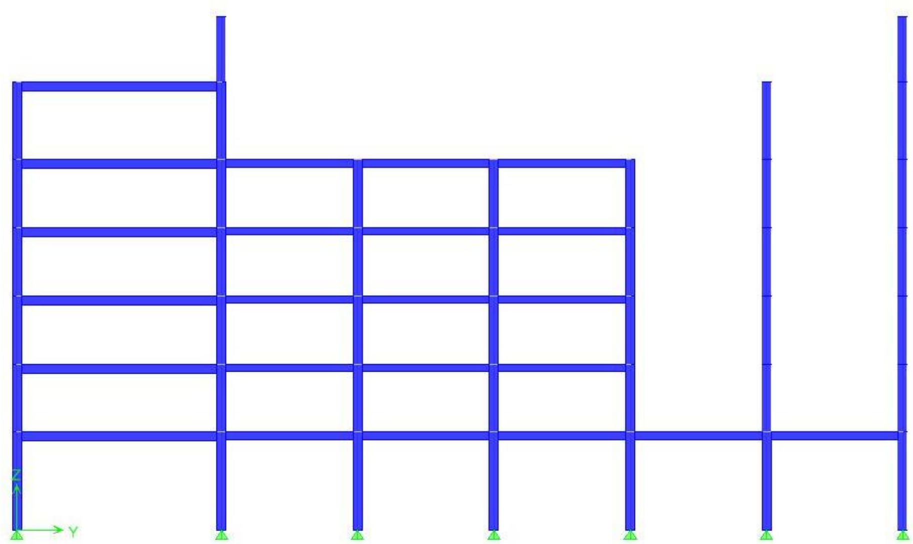


Figure 40: Frame D

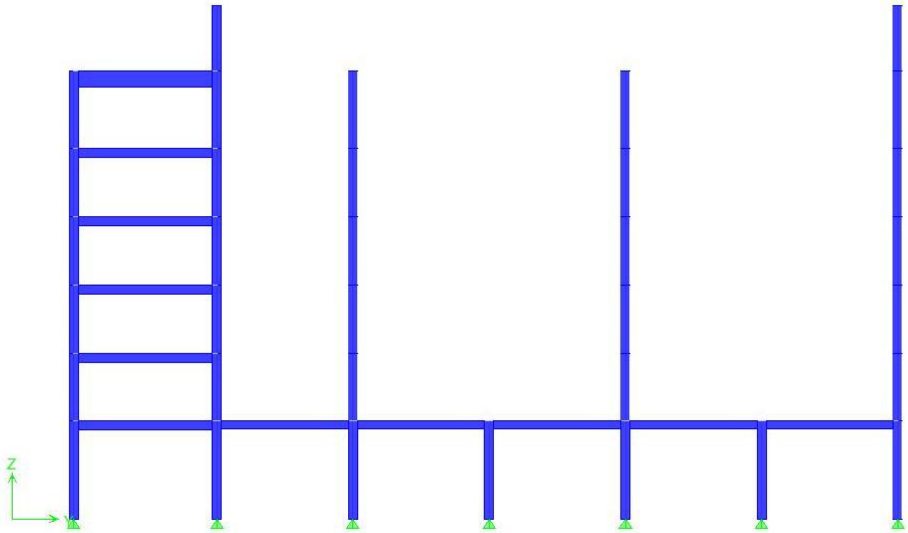


Figure 41: Frame E

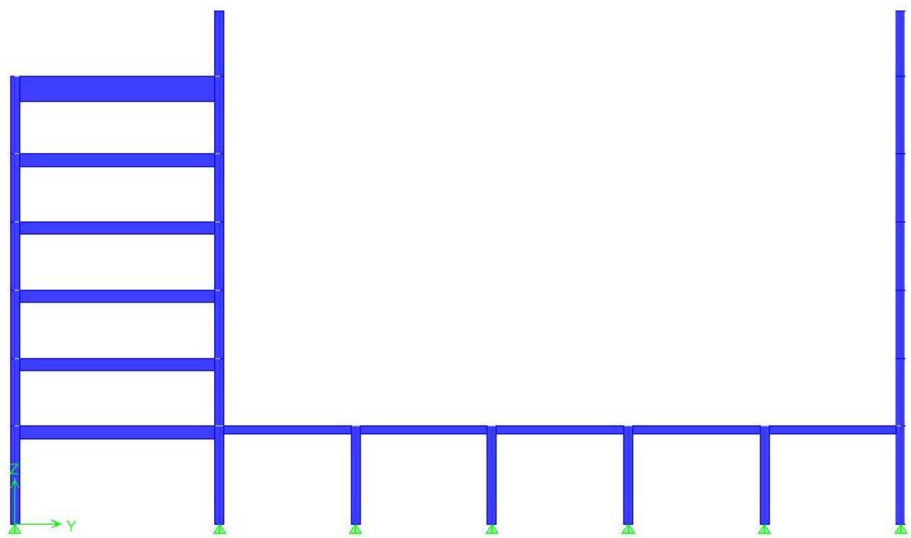


Figure 42: Frame F

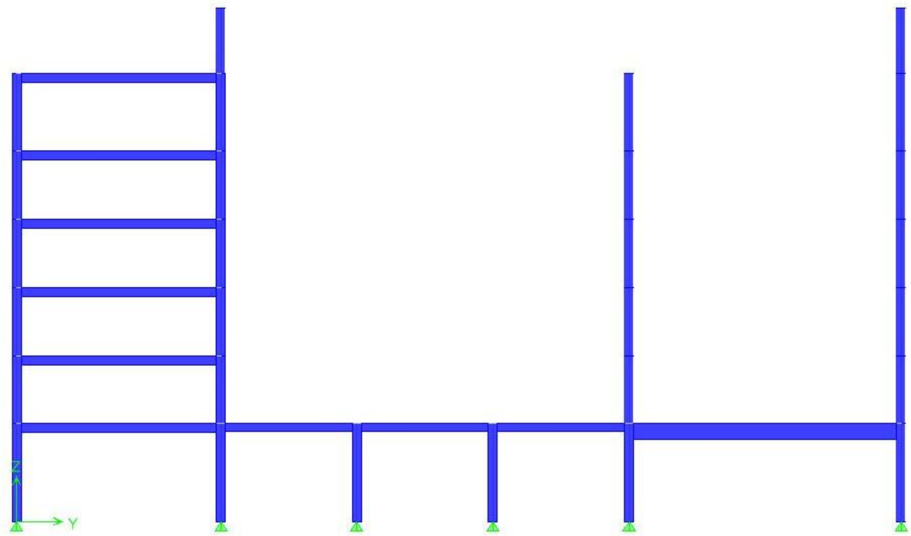


Figure 43: Frame G

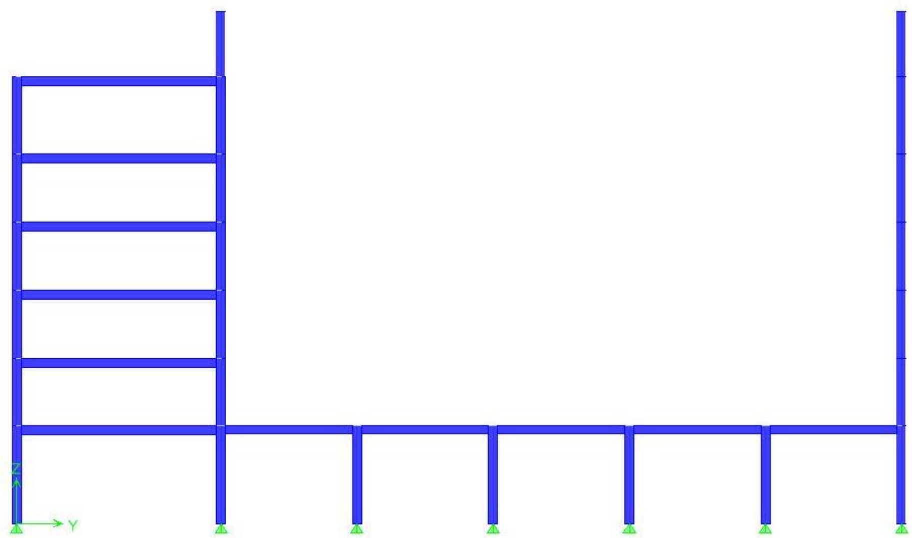


Figure 44: Frame H

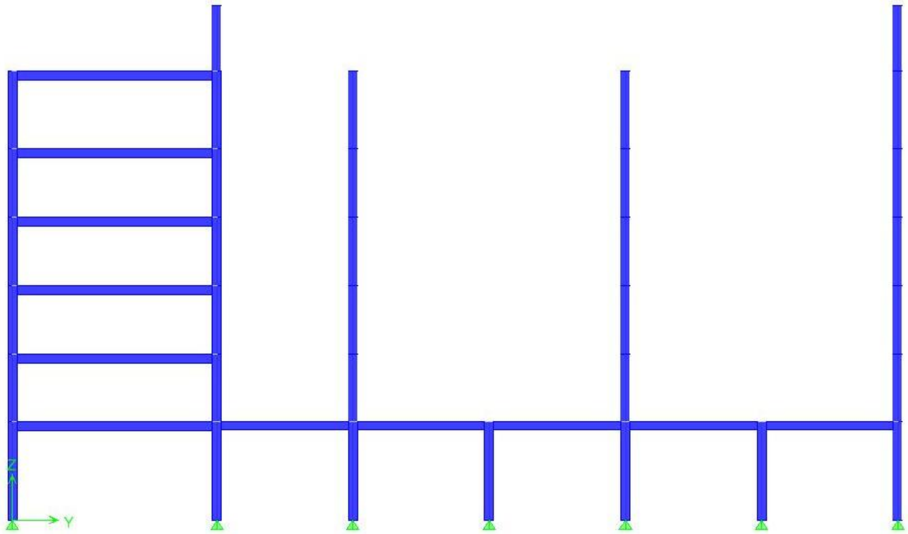


Figure 45: Frame J

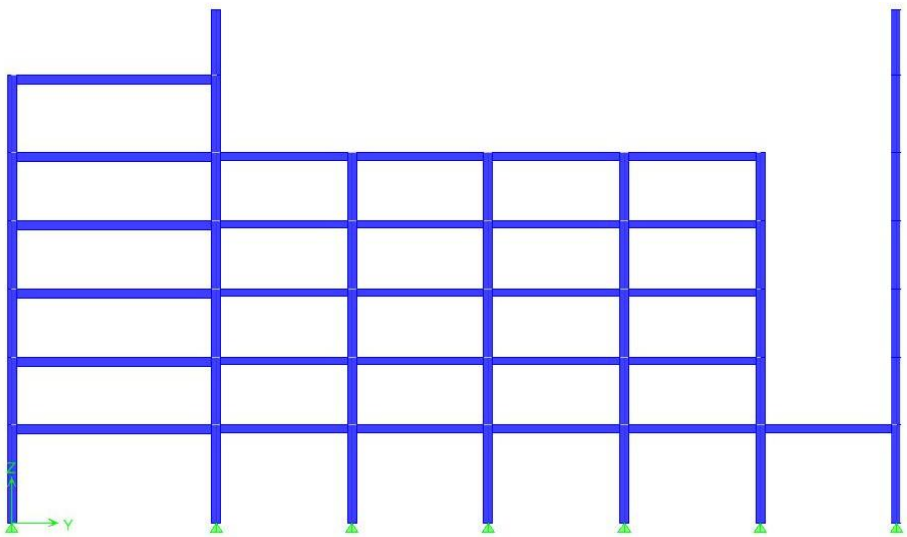


Figure 46: Frame K

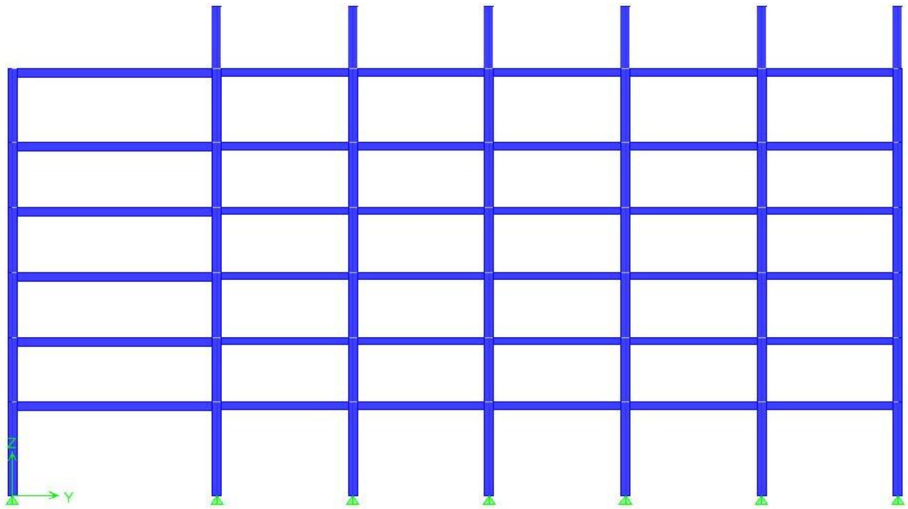


Figure 47: Frame L

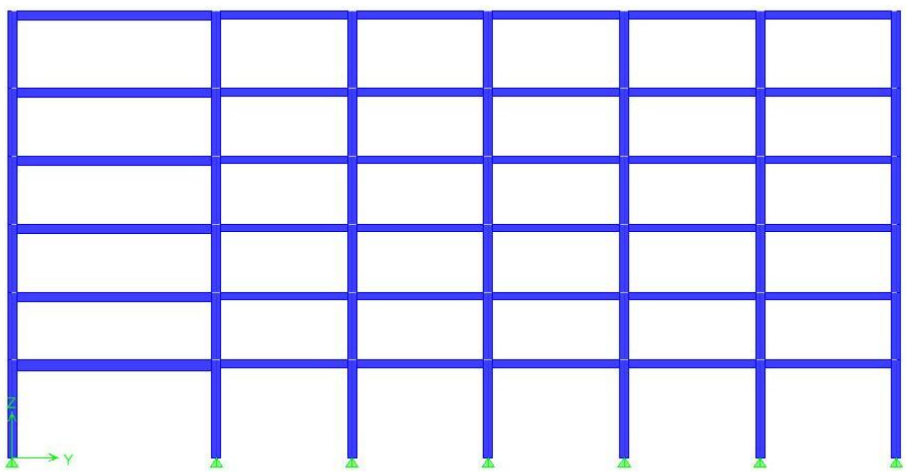


Figure 48: Frame M

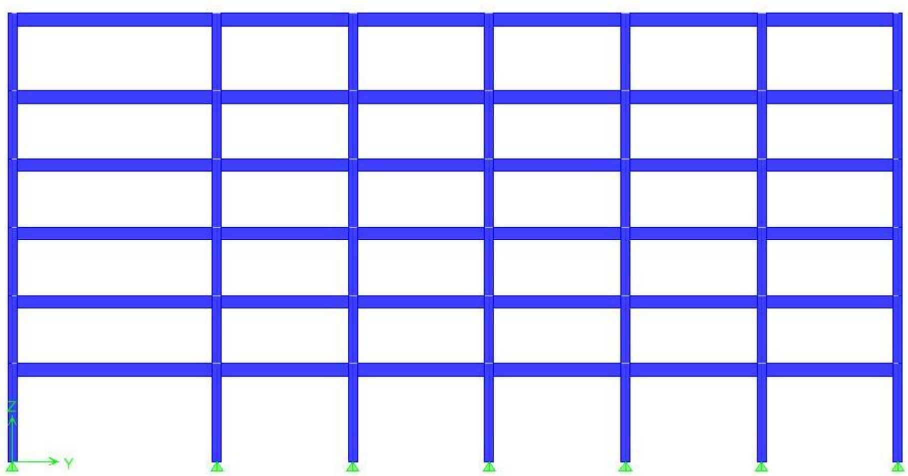
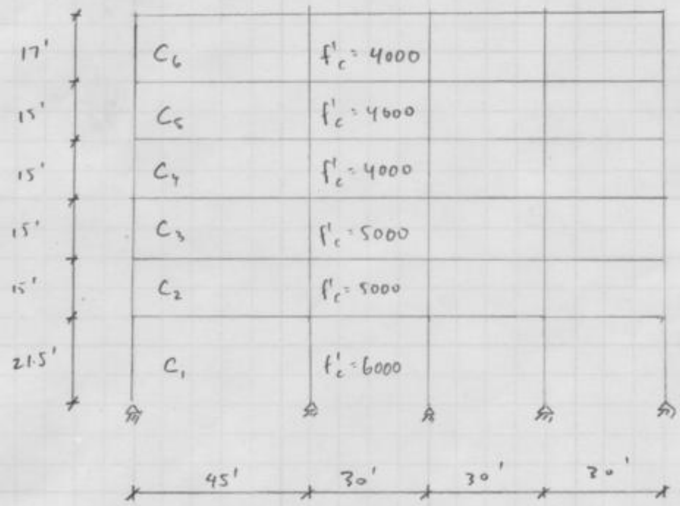


Figure 49: Frame N

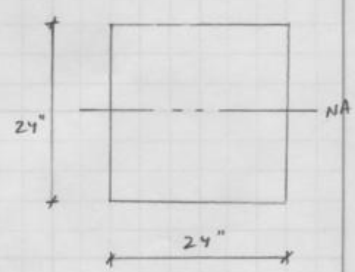
The following two pages are hand calculations of the Frame A and 8's stiffness.

Kevin Zinsmeister | Tech Report 3 | Stiffness Checks | ①

Frame A



All Columns 24"x24"



$I = \frac{bh^3}{12} = \frac{24(24)^3}{12} = 27648 \text{ in}^4$

$E_{f'c=6000} = 57000\sqrt{6000} = 4415 \text{ ksi}$

$E_{f'c=5000} = 57000\sqrt{5000} = 4031 \text{ ksi}$

$E_{f'c=4000} = 57000\sqrt{4000} = 3605 \text{ ksi}$

$K_{c1} = \frac{3EI}{h^3} = \frac{3(4415)27648}{(21.5(12))^3} = 21.323 \text{ k/in}$

$K_{c2} = K_{c3} = \frac{12EI}{h^3} = \frac{12(4031)27648}{(15(12))^3} = 229.319 \text{ k/in}$

$K_{c4} = K_{c5} = \frac{12EI}{h^3} = \frac{12(3605)27648}{(15(12))^3} = 205.084 \text{ k/in}$

$K_{c6} = \frac{12EI}{h^3} = \frac{12(3605)27648}{(17(12))^3} = 140.883 \text{ k/in}$

$K_{s1} = 5(21.323) = 106.615 \text{ k/in}, \quad K_{s2} = K_{s3} = 5(229.319) = 1146.595 \text{ k/in}$

$K_{c4} = K_{s5} = 5(205.084) = 1025.42 \text{ k/in}, \quad K_{s6} = 5(140.883) = 704.415 \text{ k/in}$

$\frac{1}{K_{FRAME\,A}} = \sum \frac{1}{K_i} = \frac{1}{106.615} + \frac{2}{1146.595} + \frac{2}{1025.42} + \frac{1}{704.415} \quad \therefore \boxed{K_{FRAME\,A} = 68.995 \text{ k/in}}$

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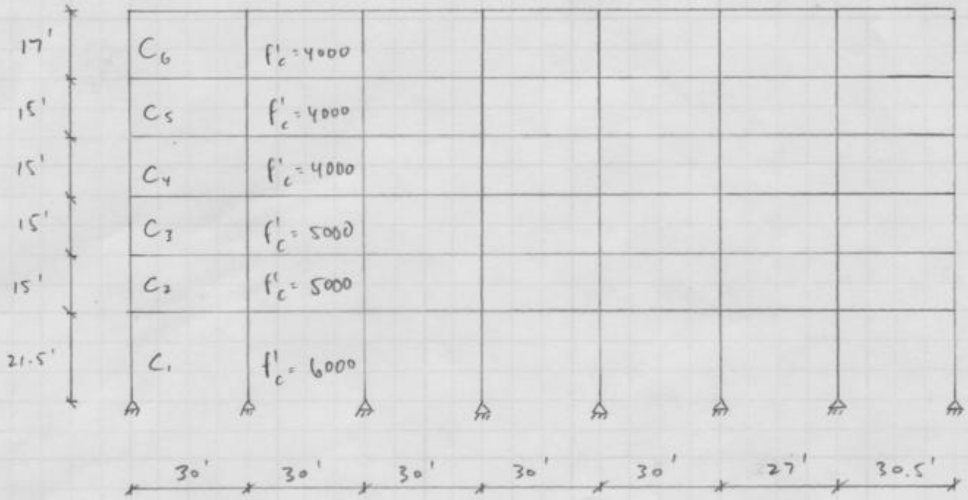
Frame 8



All Columns 24" x 24" $\therefore$ $I = 27648 \text{ in}^4$

$E_{f'_c=6000} = 4415$ ksi, $E_{f'_c=5000} = 4031$ ksi, $E_{f'_c=4000} = 3605$ ksi

$K_{c1} = 21.323 \text{ k/in}$, $K_{c2} = K_{c3} = 229.319 \text{ k/in}$

$K_{c4} = K_{c5} = 205.084 \text{ k/in}$ $K_{c6} = 140.883 \text{ k/in}$

$K_{s1} = 8(21.323) = 170.584 \text{ k/in}$

$K_{s2} = K_{s3} = 8(229.319) = 1834.552 \text{ k/in}$

$K_{s4} = K_{s5} = 8(205.084) = 1640.672 \text{ k/in}$

$K_{s6} = 8(140.883) = 1127.064 \text{ k/in}$

$$\frac{1}{K_{FRAME\,8}} = \sum \frac{1}{K_i} = \frac{1}{170.584} + \frac{2}{1834.552} + \frac{2}{1640.672} + \frac{1}{1127.064}$$

$\therefore \boxed{K_{FRAME\,8} = 110.391 \text{ k/in}}$

The following page is hand confirmation of uplift forces on the building

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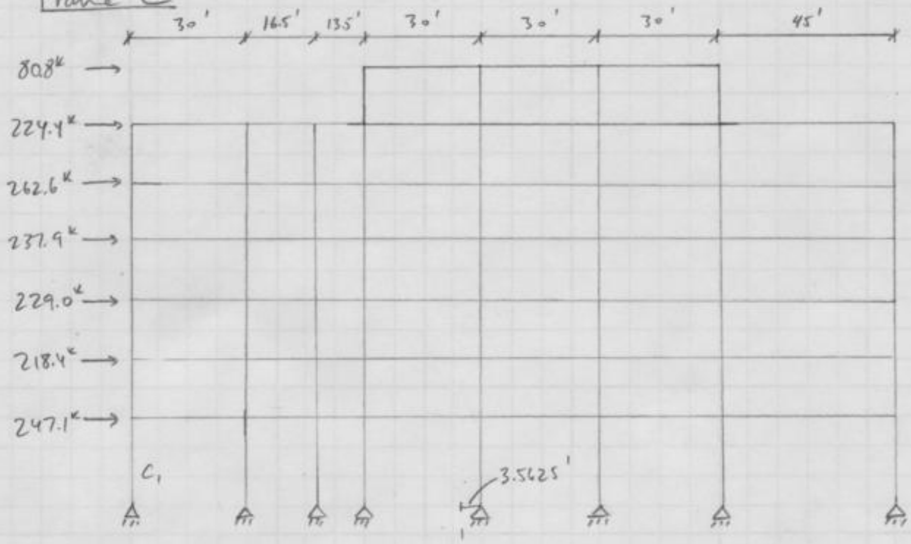
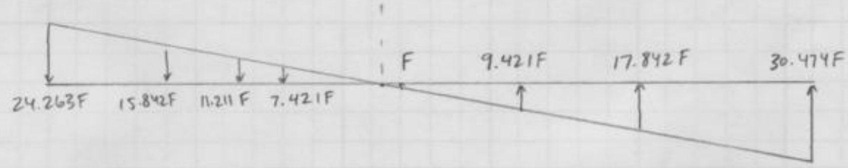
Frame C



$$\bar{x} = \frac{0+30+46.5+60+90+120+150+195}{8} = \frac{691.5}{8} = 86.4375'$$



All F values from similar triangles

$$\Sigma M_{centroid} = 0 = 24.263F(86.4375) + 15.842F(56.4375) + 11.211F(39.9375) + 7.421F(26.4375) + \ldots$$

$$\ldots + 3.5625F + 9.421F(33.5625) + 17.842F(63.5625) + 30.474F(108.5625) - 1498.0\left(\tfrac{21.5}{2}\right)$$

$\therefore$ $16103.5 = 8397.4185F \Rightarrow F = 1.9176^k$

$F_{C_1} = 24.263(1.9176) = 46.53^k \uparrow$

$$W = 15(15)\tfrac{4}{12}(115) + 3(30)54.62 + 2(15)55 + 2(30)35 + 44(15)15(2) + 30(14) + 15(44)$$

$$+ 15(68) + 30\left(\tfrac{25.5}{12}\right)\tfrac{29}{12}(115) + 30\left(\tfrac{6}{12}\right)\tfrac{16}{12}(115) + 30(15)15 + \pi(113)150$$

$= 117760 \text{ lb} = 117.76^k \downarrow$

$\therefore$ $F_{C_1} + 0.9W = 46.53 - 0.9(117.76) = -59.45^k$

$\therefore$ $F_{reaction} = 59.45^k$ $\therefore$ no uplift forces

Appendix F: Member Strength Checks

The following pages are the strength check for column B7 at grid line D-6.

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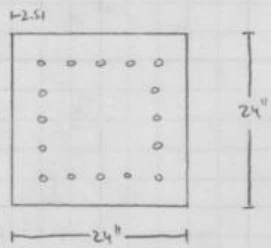
Column @ Grid B7



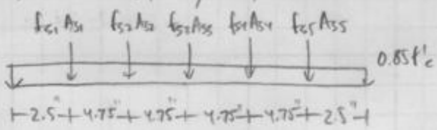
24"×24", (16) #11, #4 ties @12"

$f'_c = 6000$, $f_y = 75$ ksi

$A_s = 16(1.56) = 24.96\ in^2$

$\beta = 0.85 - \left[\frac{6000-4000}{4000}(0.2)\right] = 0.75$

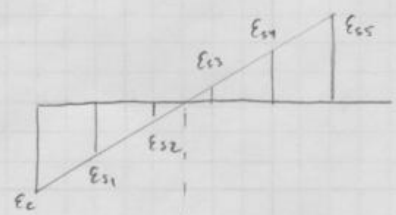
a) Axial Strength, P_o



$P_o = 0.85(6)[24^2 - 24.96] + 24.96(75) = 4682^k$

b) Balanced Strain Strength, P_bM_b

$\varepsilon_y = \frac{75}{29000} = 0.00259 \Rightarrow c = \frac{0.003}{0.003+0.00259}(24-2.5) = 11.54"$

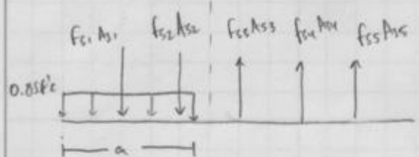


$\varepsilon_{s1} = \frac{0.003}{11.54}(11.54-2.5) = 0.00235$

$f_{s1} = 29000(0.00235) = 68.2$ ksi

$\varepsilon_{s2} = \frac{0.003}{11.54}(11.54-7.25) = 0.00111$

$f_{s2} = 29000(0.00111) = 32.3$ ksi



$\varepsilon_{s3} = \frac{0.003}{11.54}(11.54-12) = -0.00012$

$f_{s3} = 29000(-0.00012) = -3.5$ ksi

$\varepsilon_{s4} = \frac{0.003}{11.54}(11.54-16.75) = -0.00135$ $\quad f_{s4} = 29000(-0.00135) = -39.3$ ksi

$\varepsilon_{s5} = \frac{0.003}{11.54}(11.54-21.5) = -0.00259$ $\quad f_{s5} = 29000(-0.00259) = -75$ ksi

$P_b = 0.85 f'_c b \beta_1 c + \sum_{i=1}^{n} A_{si}f_{si} = 0.85(6)24(0.75)\,11.54 + 5(1.56)\,68.2 + 2(1.56)\,32.3 + \ldots$

$+ 2(1.56)(-3.5) + 2(1.56)(-39.3) + 5(1.56)(-75) = 973.6^k$

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$M_b = 0.85 f'_c b \beta_1 c \left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + \Sigma A_{si} f_{si} \left(\frac{h}{2} - d_i\right)$

$M_b = 0.85(6)24(0.75)\,11.54\left[\frac{24}{2} - \frac{0.75(11.54)}{2}\right] + 5(1.56)68.2\left[\frac{24}{2} - 2.5\right] + 2(1.56)32.3\left[\frac{24}{2} - 7.25\right] + \ldots$

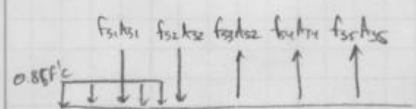
$+ 2(1.56)(-3.5)\left[\cancel{\frac{24}{2} - 12}\right] + 2(1.56)(-39.3)\left[\frac{24}{2} - 16.75\right] + 5(1.56)(-75)\left[\frac{24}{2} - 21.5\right]$

$= 8128 + 4312 + 477 + 582 + 5558 = 19057 \text{ in-k} = 1588^{'k}$

c) Pure Bending, $M_o \Rightarrow$ Assume only ϵ_{ss} yields

$f_{s1} = \frac{0.003}{c}(c-2.5)29000, \quad f_{s2} = \frac{0.003}{c}(c-7.25)29000$

$f_{s3} = \frac{0.003}{c}(c-12)29000, \quad f_{s4} = \frac{0.003}{c}(c-16.75)29000$



$\Sigma F_y = 0 = 0.85(6)24(0.75)c + 5(1.56)\frac{0.003}{c}(c-2.5)29000 + 2(1.56)\frac{0.003}{c}(c-7.25)29000 + \ldots$

$+ 2(1.56)\frac{0.003}{c}(c-12)29000 + 2(1.56)\frac{0.003}{c}(c-16.75)29000 + 5(1.56)(-75)$

$= 91.8c + 678.6 - \frac{1696.5}{c} + 271.44 - \frac{1967.94}{c} + 271.44 - \frac{3257.28}{c} + 271.44 - \frac{4546.62}{c} - 585$

$= 91.8c + 907.92 - \frac{11468.34}{c} \Rightarrow 91.8c^2 + 907.92c - 11468.34$

$\therefore c = 7.28'', \quad f_{s1} = \frac{0.003}{7.28}(7.28-2.5)29000 = 57.1 < 75 \therefore$ okay

$f_{s2} = \frac{0.003}{7.28}(7.28-7.25)29000 = 0.4 < 75 \therefore$ okay

$f_{s3} = \frac{0.003}{7.28}(7.28-12)29000 = -56.4 < 75 \therefore$ okay

$f_{s4} = \frac{0.003}{7.28}(7.28-16.75)29000 = -113.2 > 75 \therefore$ not okay

$\Sigma F_y = 0 \Rightarrow 91.8c + 402.48 - \frac{6921.72}{c} \Rightarrow 91.8c^2 + 402.48c - 6921.72$

$\therefore c = 6.76''$

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$f_{s1} = \frac{0.003}{6.76}(6.76-2.5)29000 = 54.8 \text{ ksi} < 75 \therefore$ okay

$f_{s2} = \frac{0.003}{6.76}(6.76-7.25)29000 = -6.3 \text{ ksi} < 75$ okay

$f_{s3} = \frac{0.003}{6.76}(6.76-12)29000 = -67.5 \text{ ksi} < 75$ okay

$\varepsilon_{s4} = \frac{0.003}{6.76}(6.76-16.75) = -0.00443 > 0.00259 \therefore$ okay

$M_o = 0.85(6)24(0.75)6.76\left(12-\frac{0.75(6.76)}{2}\right) + 5(1.56)54.8(12-2.5) + 2(1.56)(-6.3)(12-7.25) + \ldots$

$+ 2(1.56)(-67.5)(\cancel{12}-12) + 2(1.56)(-75)(12-16.75) + 5(1.56)(-75)(12-21.5)$

$= 5874 + 4061 - 93 + 1112 + 5558 = 16512 \text{ in-k} = 1376 \text{ 'k}$

d) A point between a) & b) ⇒ choose $c = 20"$

$\varepsilon_{s1} = \frac{0.003}{20}(20-2.5) = 0.00263 \Rightarrow f_{s1} = 75$ ksi

$\varepsilon_{s2} = \frac{0.003}{20}(20-7.25) = 0.00191 \Rightarrow f_{s2} = 55.5$ ksi

$\varepsilon_{s3} = \frac{0.003}{20}(20-12) = 0.00120 \Rightarrow f_{s3} = 34.8$ ksi

$\varepsilon_{s4} = \frac{0.003}{20}(20-16.75) = 0.00049 \Rightarrow f_{s4} = 14.1$ ksi

$\varepsilon_{s5} = \frac{0.003}{20}(20-21.5) = -0.00023 \Rightarrow f_{s5} = -6.5$ ksi

$P_n = 0.85(6)24(0.75)20 + 5(1.56)75 + 2(1.56)55.5 + 2(1.56)34.8 + 2(1.56)14.1 + 2(1.56)(-6.5)$

$P_n = 1836 + 585 + 173.16 + 108.576 + 43.992 - 20.28 = 2726^k$

$M_n = 1836\left(12 - \frac{0.75(20)}{2}\right) + 585(12-2.5) + 173.16(12-7.25) + 43.992(12-16.75) - 20.28(12-21.5)$

$M_n = 8262 + 5558 + 823 - 209 + 193 = 14627 \text{ in-k} = 1219 \text{ 'k}$

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e) A point where $\varepsilon_t = 0.005$

$$\varepsilon_t = \varepsilon_{s5} = 0.005 \Rightarrow c = \frac{0.003}{0.003 + 0.005}(21.5) = 8.06''$$

$$\varepsilon_{s1} = \frac{0.003}{8.06}(8.06 - 2.5) = 0.00207 \Rightarrow f_{s1} = 60.0 \text{ ksi}$$

$$\varepsilon_{s2} = \frac{0.003}{8.06}(8.06 - 7.25) = 0.00030 \Rightarrow f_{s2} = 8.7 \text{ ksi}$$

$$\varepsilon_{s3} = \frac{0.003}{8.06}(8.06 - 12) = -0.00147 \Rightarrow f_{s3} = -42.5 \text{ ksi}$$

$$\varepsilon_{s4} = \frac{0.003}{8.06}(8.06 - 16.75) = -0.00323 \Rightarrow f_{s4} = -75 \text{ ksi}$$

$$\varepsilon_{s5} = \frac{0.003}{8.06}(8.06 - 21.5) = -0.00500 \Rightarrow f_{s5} = -75 \text{ ksi}$$

$$P_n = 0.85(6)24(0.75)8.06 + 5(1.56)60 + 2(1.56)8.7 - 2(1.56)42.5 - 7(1.56)75$$

$$P_n = 739.908 + 468 + 27.144 - 132.6 - 234 - 585 = 283^k$$

$$M_n = 739.908\left(12 - \frac{8.06(0.75)}{2}\right) + 468(12 - 2.5) + 27.144(12 - 7.25) - 234(12 - 16.75) - 585(12 - 21.5)$$

$$M_n = 6643 + 4446 + 129 + 1112 + 5558 = 17888 \text{ in-k} = 1491'^k$$

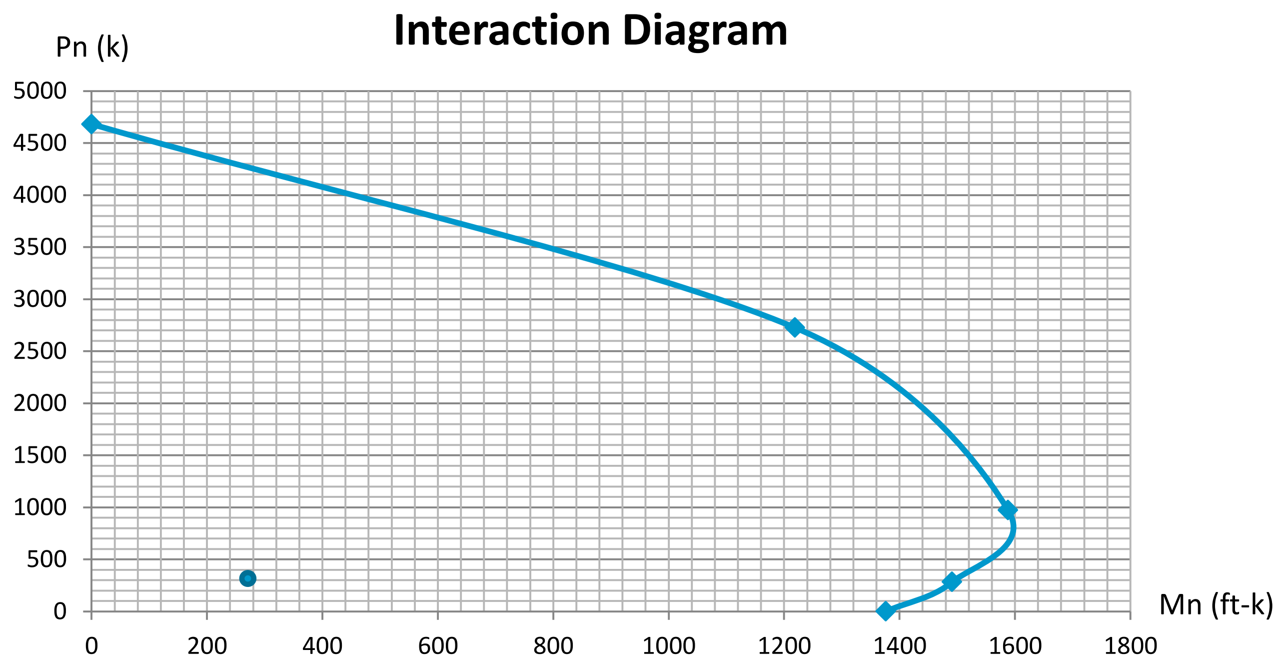
Shear Capacity

$$\phi(V_c + V_s) = 0.75\left[2\sqrt{6000}(24 - 2.5)24 + 0.2(60)\frac{21.5}{12}\right] = 0.75[79960]$$

$$= 59970 \text{ lbs} = 60.0^k > 12.48^k$$

$\therefore$ okay

Excel Graph on Next Page



Based upon the location of the actual forces on the interaction diagram, the column is adequate in strength design.

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Beam @ N1-N2

Top Reinforcing: $A_s = 7(1.00) = 7.00\ in^2$, $F_y = 75\ ksi$, $f'_c = 5000\ psi$

$d = 33 - 4 + 1.5 = 30.5''$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{7(75)}{0.85(5)25.5} = 4.84''$ $\therefore$ $c = \frac{4.84}{0.8} = 6.05''$

$\varepsilon_s = \frac{0.003}{6.05}(30.5 - 6.05) = 0.0121 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9 A_s F_y (d - \frac{a}{2}) = 0.9(7)75(30.5 - \frac{4.84}{2}) = 13267.8\ in \cdot k$

$\phi M_n = 1105.7'^k > 438.2'^k$ $\therefore$ okay

Bottom Reinforcing: $A_s = 6(1.00) = 6.00\ in^2$, $F_y = 75\ ksi$, $f'_c = 5000\ psi$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{6(75)}{0.85(5)25.5} = 4.15''$ $\therefore$ $c = \frac{4.15}{0.8} = 5.19''$

$\varepsilon_s = \frac{0.003}{5.19}(30.5 - 5.19) = 0.0146 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9 A_s F_y (d - \frac{a}{2}) = 0.9(6)75(30.5 - \frac{4.15}{2}) = 11512.1\ in \cdot k$

$\phi M_n = 959.3'^k > 255.2'^k$ $\therefore$ okay

Shear Reinforcing: $A_s = 0.11\ in^2$, $F_y = 60\ ksi$, $f'_c = 5000\ psi$, $s = 5''$

$\phi(V_c + V_s) = 0.75\left[2\sqrt{5000}(25.5)30.5 + 0.11(60)\frac{30.5}{5}\right]$

$= 0.75[110.0 + 40.3] = 112.7^k > 35.2^k$ $\therefore$ okay