[Helios Plaza]

Houston, Texas

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[TECHNICAL REPORT I:]

Structural Concepts & Existing Conditions

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Executive Summary

The purpose of this report is to present the original design of Helios Plaza IST building. From this point forward, the IST building will simply be referred to as Helios Plaza. In this discussion, three main topics encompass the bulk of the report:

1. A summary of the overall structural system of Helios Plaza
2. Computations of all loads acting on the building, including wind and seismic forces
3. Spot-checks of typical floor framing elements

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

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. Due to the location of the project, there are no snow loads on the building.

Spot-checks were performed on representative typical framing elements. A column was checked at every level of the building and it was found to be adequate under gravity loadings at every floor. The other two checks were performed on a pan joist and a girder that frame into the column mentioned above. The pan was found to be adequate, but the girder failed in both flexure and shear under the calculated loads.

Based upon the load determinations and the spot checks performed, it can be concluded that the assumptions made in the analysis for this report were more conservative than the loads used by the designers. Despite finding that seismic loads control in the East-West direction, there is no mention in the structural commentary by the designer that seismic loads were taken into consideration. This finding is important to the progress of this thesis project.

Introduction

Helios Plaza is a corporate campus that comprises of three main structures. The first structure, which is the focus of this report, is a six-story IST building. In addition to the IST building, there is a 1,909 car capacity parking deck and a five megawatt combined heat and power plant housed in its own structure. The IST building will be referred to as Helios Plaza throughout the rest of this document.

Helios Plaza is 423,500 gross square feet with an overall building height of 113 feet, the typical floor to floor height being 15 feet. After the second level, the floors systems split between concrete and composite deck to allow for double-story trading floors. From story three upward, a u-shaped concrete floor repeats at every level until level six leaving a rectangular space open for the composite deck system. This rectangular composite deck only occurs at levels four and six to create a total of three double-story trading floors for the building occupant. Refer to Appendix A for additional floor plans and elevations.

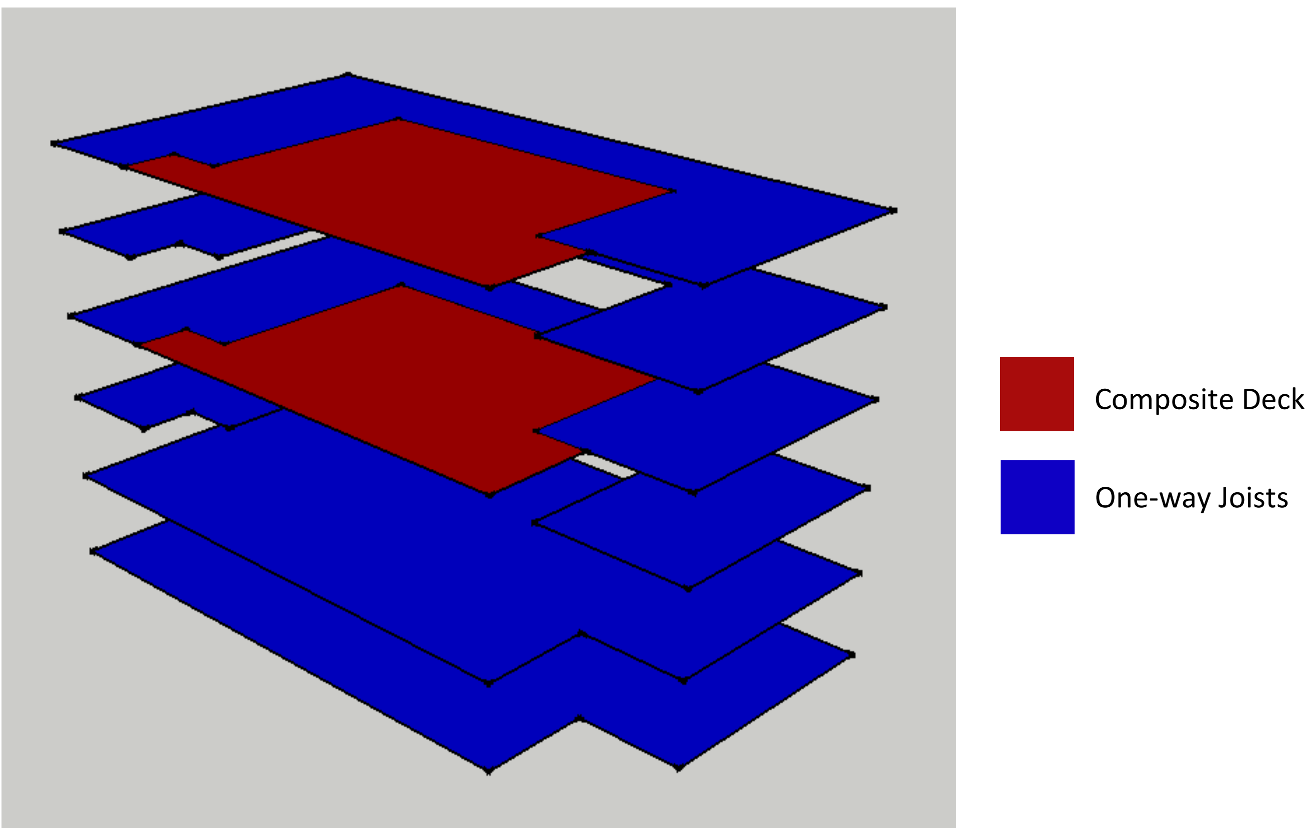

Figure 1: Simplified Floor Systems Diagram

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 larger spans of the magnitude 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 66/6 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. Large HSS members are used in the trading floors at the skip levels to transfer loads horizontally into the concrete adjacent and vertically to the floors above.

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.

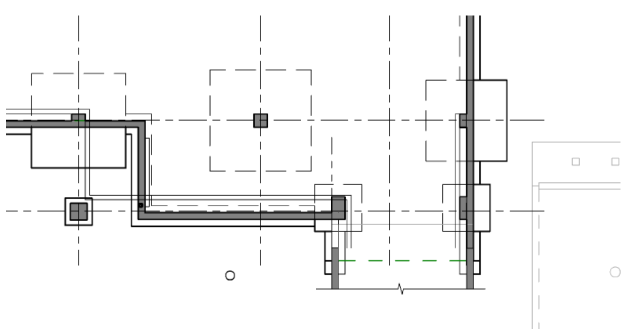

Figure 2: Basement Tunnel Entrance to Parking Structure

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.

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. The same concrete strengths apply to these HSS columns as the rectangular columns listed above.

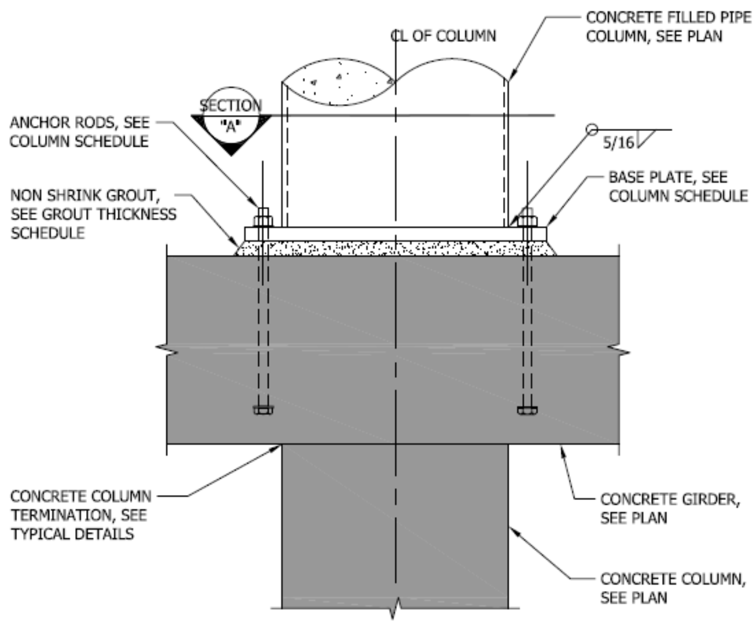

Figure 3: Typical Detail of Concrete Filled HSS Anchorage

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 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.

In the corner bays of the building, a large pan (typically 33” x 30”) is placed to transfer load from the exterior stairwells’ framing members. A large pan extends from the exterior stairwells' wall perpendicular to the enlarged pan from above and ties into it for load transfer. This is done to reduce torsion that would otherwise be placed on the edge girder of the main building.

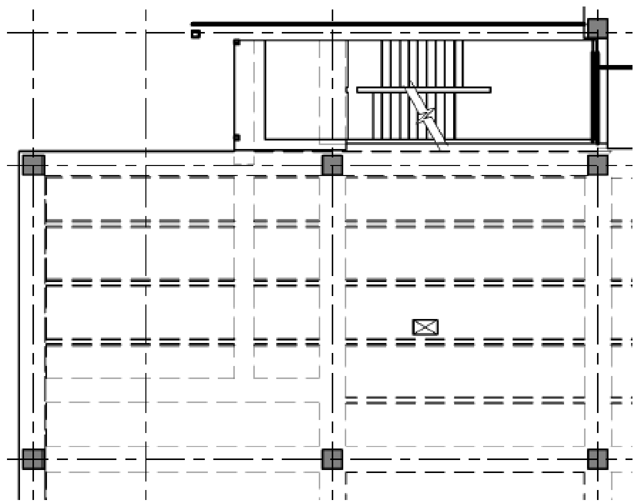

Figure 4: Plan of Enlarged Corner Pan Joists

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.

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 can reach 12" in thickness. These slabs are also used when bathrooms are placed over top.

The second 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.

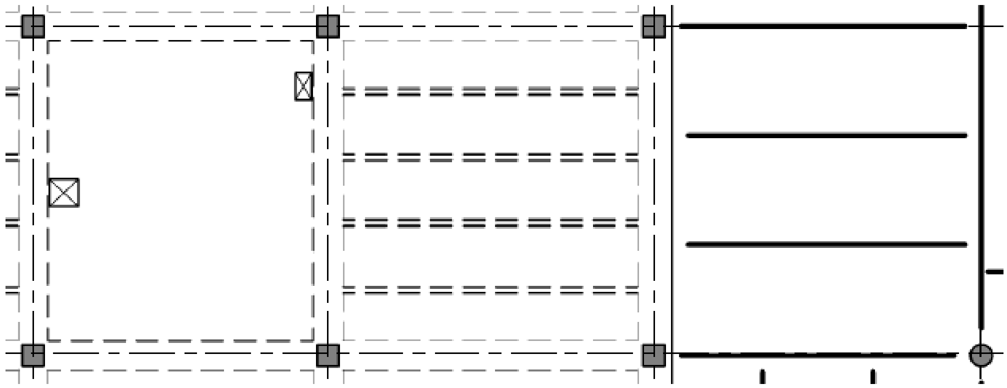

Figure 5: All Three Floor Systems in Adjacent Bays

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 frame to building width ratio is much smaller due to the double story 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.

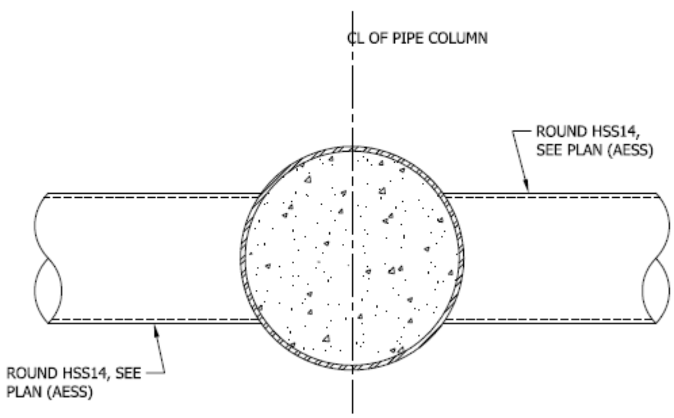

Figure 6: Round HSS Members Framing Into Each Other

Codes and References

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
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
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
Server Rooms	100 PSF	-
Mechanical Rooms	100 PSF	-
Roof	20 PSF	20 PSF

Table 3: Live Loads

Snow Loads

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

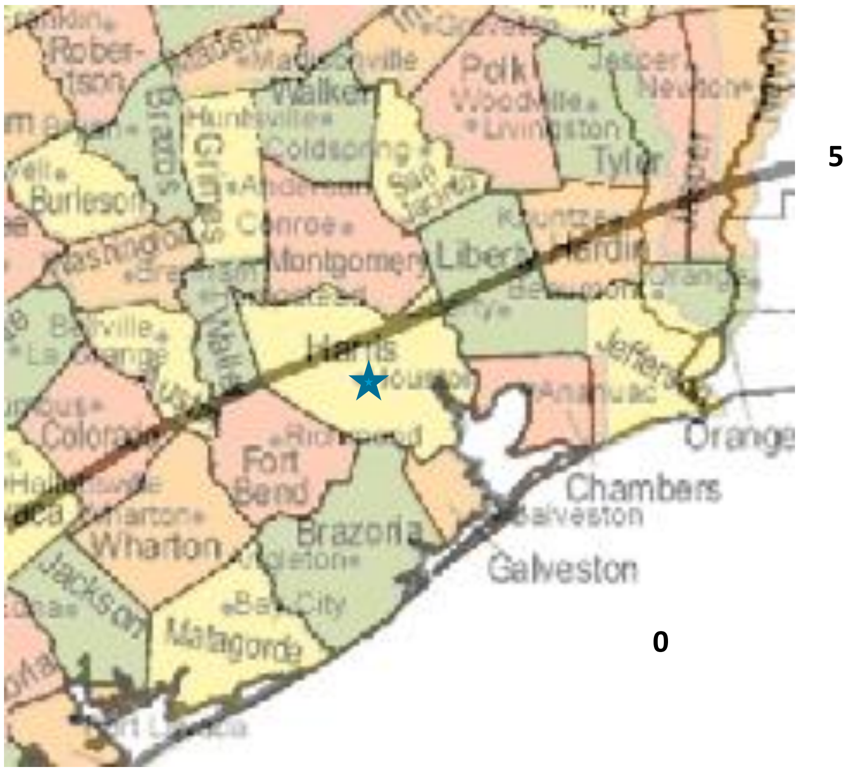

Figure 7: ASCE 7-10 Figure 7-1 Overlay on Texas County Map

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. 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)		Moment (ft-k)	
		N-S	E-W	N-S	E-W	N-S	E-W
r_2	113.0	61.8	40.2	61.8	40.2	0.0	0.0
r_1	99.25	176.3	105.7	238.1	145.9	849.3	552.7
6	81.5	207.2	118.4	445.3	264.3	5075.4	3142.2
5	66.5	181.3	103.6	626.5	367.9	11754.8	7107.2
4	51.5	172.4	98.5	799.0	466.3	21153.0	12625.5
3	36.5	161.8	92.3	960.8	558.6	33137.8	19620.7
2	21.5	178.2	101.5	1138.9	660.2	47549.1	28000.3
1	0.0	0.0	0.0	1138.9	660.2	72036.4	42193.9

Table 4: Wind Design Forces

From Table 4 above, it can be seen that the base shear is 1138.9 kips in the North-South direction and 660.2 kips in the East-West Direction. Although there are no values to compare these calculations to, it is almost certain that these values would 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.

Figure 8: North-South Wind Pressure Diagram

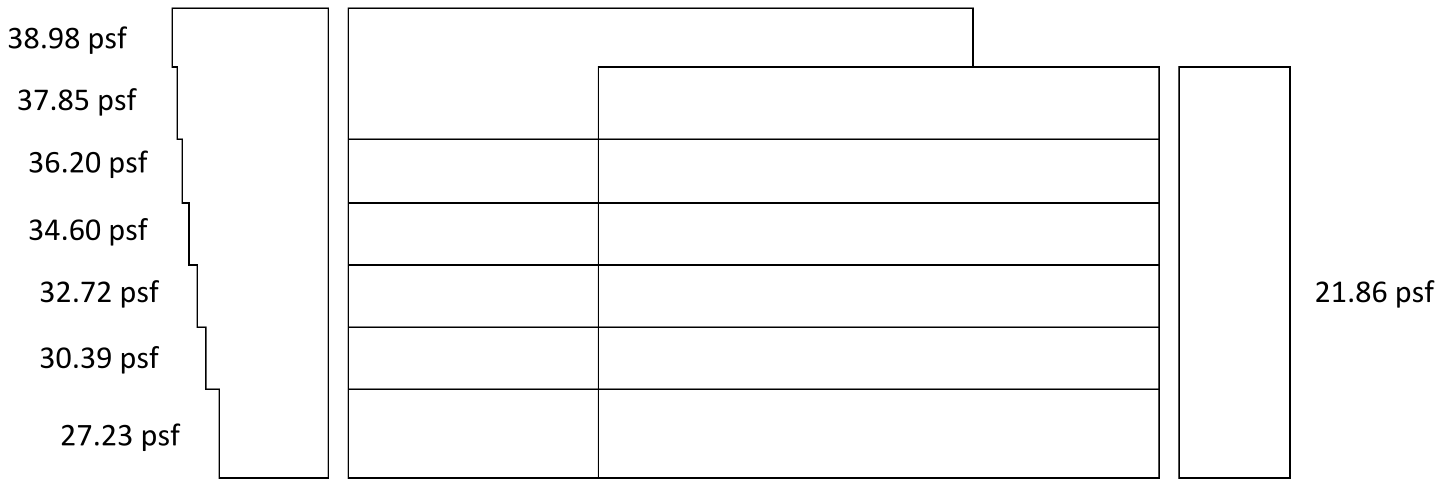

Figure 9: East-West Wind Pressure Diagram

In both Figure 8 and 9, 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' x 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 10 and 11.

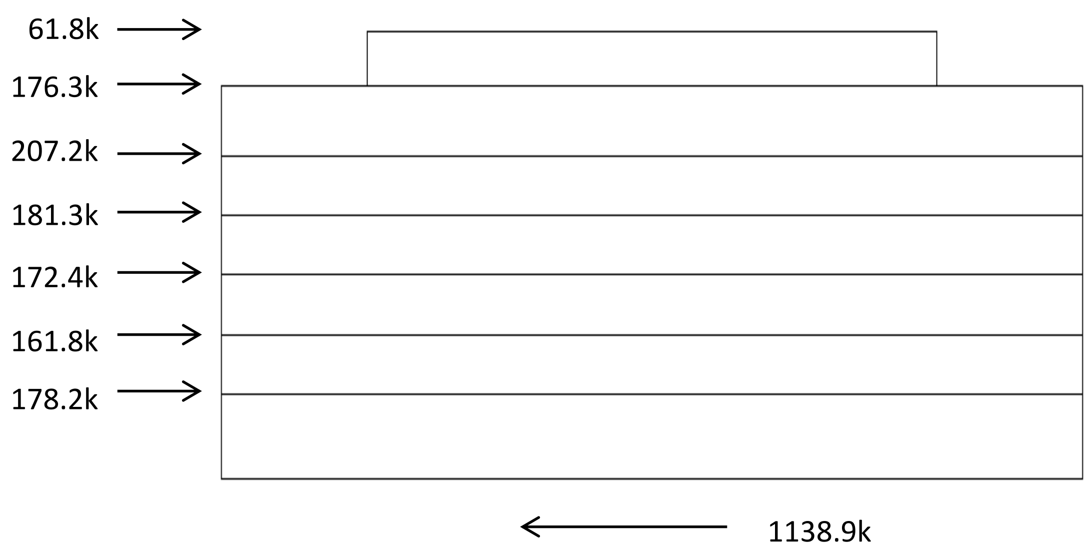

Figure 10: North-South Wind Story Forces

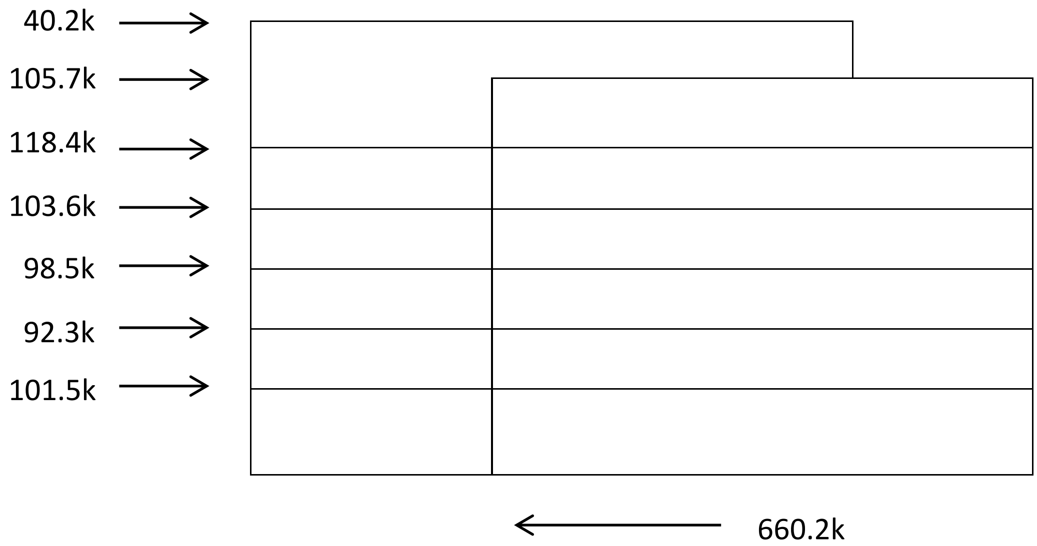

Figure 11: East-West Wind Story Forces

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 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 on 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)	C_{vx}	F_x (k)	Shear (k)	Moment (ft-k)
r_2	113	1089	0.0701	79.5	79.5	8980.4
r_1	99.25	3711	0.2013	228.2	307.7	22650.2
6	81.5	6777	0.2826	320.4	628.1	26111.3
5	66.5	4948	0.1585	179.7	807.8	11949.5
4	51.5	6777	0.1551	175.8	983.6	9055.6
3	36.5	4948	0.0720	81.6	1065.2	2979.4
2	21.5	8322	0.0604	68.5	1133.7	1472.2
Total		36572	1.0000	1133.7	-	83198.5

Table 5: Seismic Design Forces

The base shear due to seismic loading is slightly smaller than the base shear from wind loading in the North-South direction, but it is much larger than the base shear in the East-West direction. This is interesting to find because the structural specifications make no mention of taking seismic loads into account, despite it being a controlling case. Figure 12 is made by extracting the F_x values from Table 5.

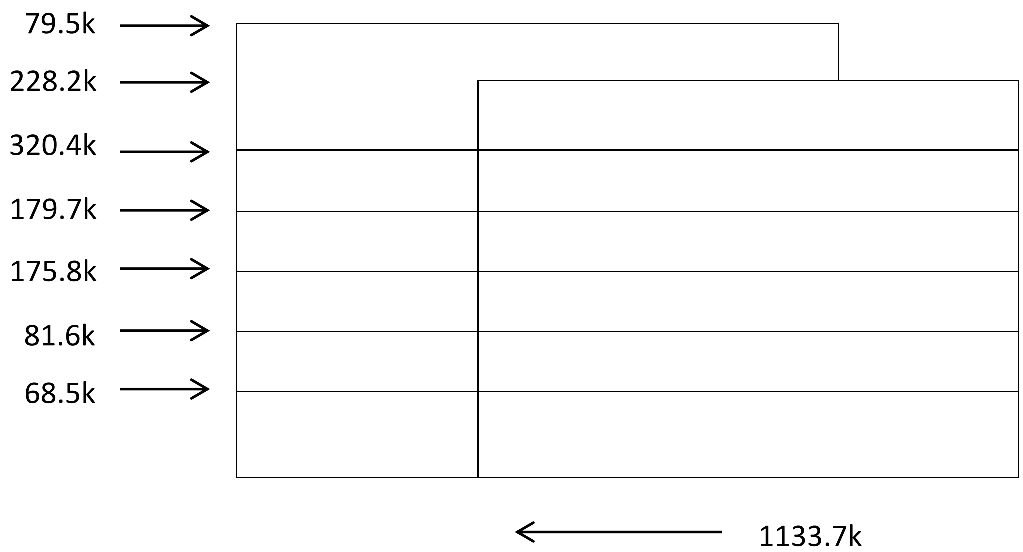

Figure 12: Seismic Force Diagram

Typical Element Spot-Checks

The area of interest for these spot-checks focuses on the column occurring at grid point B5. This area is chosen because it is a repeated section of the building that occurs frequently throughout the floor plans. The area around this column consists of one bay with a two-way slab and three bays with one-way pan joists. Column B5 can be seen highlighted in Figure 13.

Column Spot-Check

Assessing the gravity load based on the assumptions stated in the sections entitled Dead Loads and Live Loads, the forces that column B5 would need to support were found at each level. As the loads on the column increased as the level that it was supporting decreased, the designer added more steel and increased the concrete strength accordingly. The column was able to meet all of the loads and was found to be adequate. Calculations can be found in Appendix D.

Pan Joist Spot-Check

To assess the strength of the floor system, a pan joist was chosen from one of the bays surrounding column B5. The pan was specifically chosen because it was in the widest bay in the building and it is also an exterior bay, which would result in the highest possible loads. Its location can be seen in Figure 13. After gravity loads were tabulated for the pan, it was checked for strength requirements in flexure and shear. Using the coefficients from ACI 308-05 section R8.3.3, the maximum positive and negative moments as well as shear for an exterior beam were calculated. The pan joist was found to be adequate for all three maxima. Calculations can be found in Appendix D.

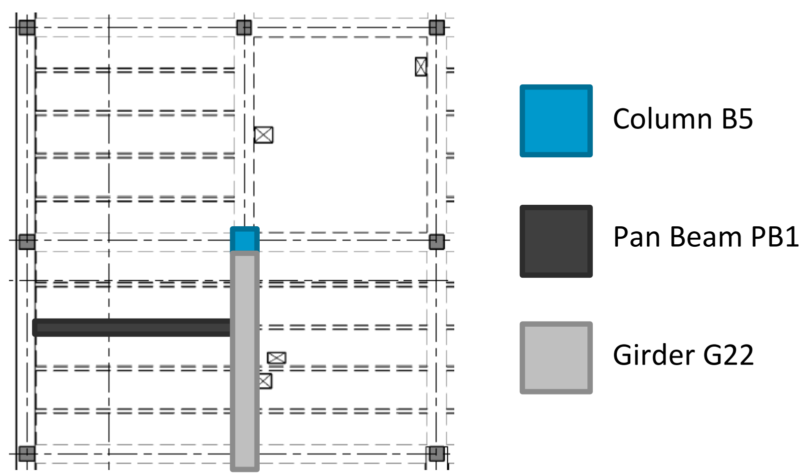

Figure 13: Spot-Check Members' Locations

Girder Spot-Check

Girder G22 was chosen because it a direct link between the other two member that had already been spot-checked. Its location can be seen above in Figure 13. Following along the same lines as the pan joist spot-check, the loads were assessed and the target strengths were found. The member initially failed upon checking its positive moment capacity when only one layer of steel was being used. With the introduction of the second row though, the girder was no longer tension controlled. This led to the factor of safety, ϕ, needing to be interpolated between its tension and compression controlled values as per ACI 308-05 Section R9.3.2. With the added steel, the girder was more than adequate for positive flexure. Checking the negative moments proved to be a failure again. This time, however, there was no more rebar in the beam to consider. Moving onto the shear capacity of the girder, the member failed once again. Calculations can be found in Appendix D.

Conclusions

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

- Wind loads control in the North-South direction
- Seismic loads control in the East-West direction
- The assumptions for gravity loads have resulted in member failure

It comes as no surprise that wind loads control the lateral design for Helios Plaza located in Houston. Being near the Gulf of Mexico, Houston is a hurricane prone region. Based on the assumptions made, the base shear that controls is equal to 1138.9 kips. This base shear is slightly larger than the seismic base shear that was calculated after the wind loads. There is only potential for the wind loads to become larger especially since the exterior passive shading devices will supply uplift on the structure.

The major revelation that came from this report is that seismic loads control in the East-West direction. With no mention made in the specifications in regards to seismic load checking, this finding was hardly expected. The base shear caused by seismic loading is 1133.7 kips, just shy of being the controlling loading case in both directions by roughly five kips. This base shear largely surpasses that of the wind loading in the East-West direction, which comes to a comparatively small 660.2 kips. The main reason for this may be from the assumption that the site class is E. Soils with markedly increased bearing capacity were brought and compacted to provide stronger support, but these measures still do not prevent the surrounding clayey soils around the site from being weak under shear loads. An earthquake could very well cause compaction of these surrounding soils below the imported soils and result in foundation settlement.

Spot-checks also returned another piece of information that was not expected. One of the regularly occurring members failed under the assumed loads. The probable cause for this failure is an overly conservative estimate of gravity loads on Helios Plaza. Despite trying to bring as much steel as possible into the member from adjacent framing elements, the reinforcement was simply not there. A potential change that could result in the member passing in flexure, but still failing in shear would be to implement moment distribution from the adjacent girder and post-tensioned girder.

Additional loads that need to be considered in the future are many and varied. As mentioned before, wind uplift will have a serious impact on Helios Plaza, especially on the roof where there is a 6’ cantilever shading element spanning the entire South elevation. A major source of potential problems comes from uplift. The water table of Houston is quite high due to the land being generally classified as swamp, marsh, and prairie.

The lateral system for Helios Plaza also needs to be analyzed further. The large loads on the south face of the building will have trouble transferring through the structure especially since this is the side of Helios Plaza that houses the double-story trading floors. The composite decking has details showing steel angles tying it to the HSS columns in the space, but these lateral ties seem few and far between. The skip levels are more concerning though when loads need to travel up to 120' horizontally before they meet with the concrete moment frames. These loads can travel up and down to the composite decking, but as stated before, there seems to be a lack of lateral bracing in the composite system.

Through all of this, it has been shown that Helios Plaza is a building that has a varied and complex structural system. There is an inherent dichotomy in the layout of the floor plans, and this dichotomy is carried over into the structural system. Unifying the two systems is a goal that needs to be accomplished to create a functional structure.

Appendix

Appendix A: Typical Floor Plans

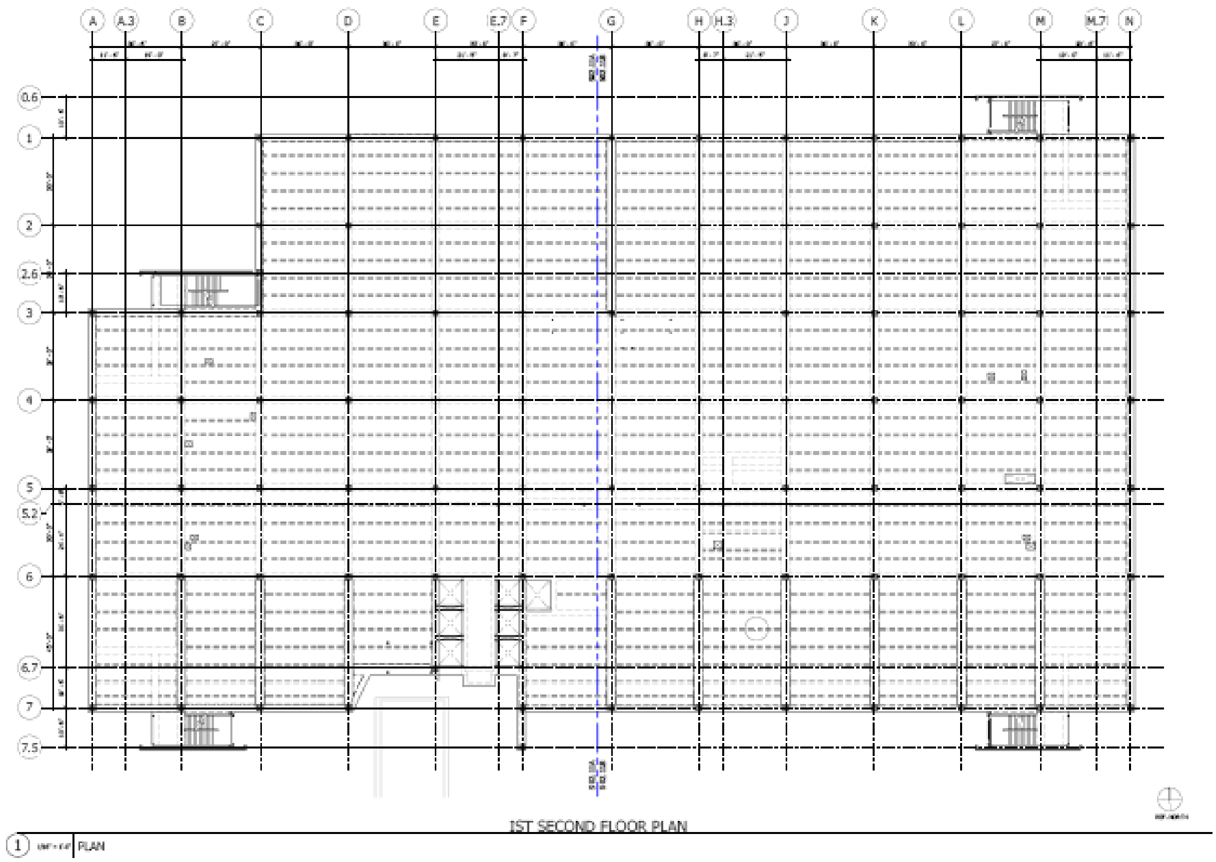

Figure 14: Second Floor Plan

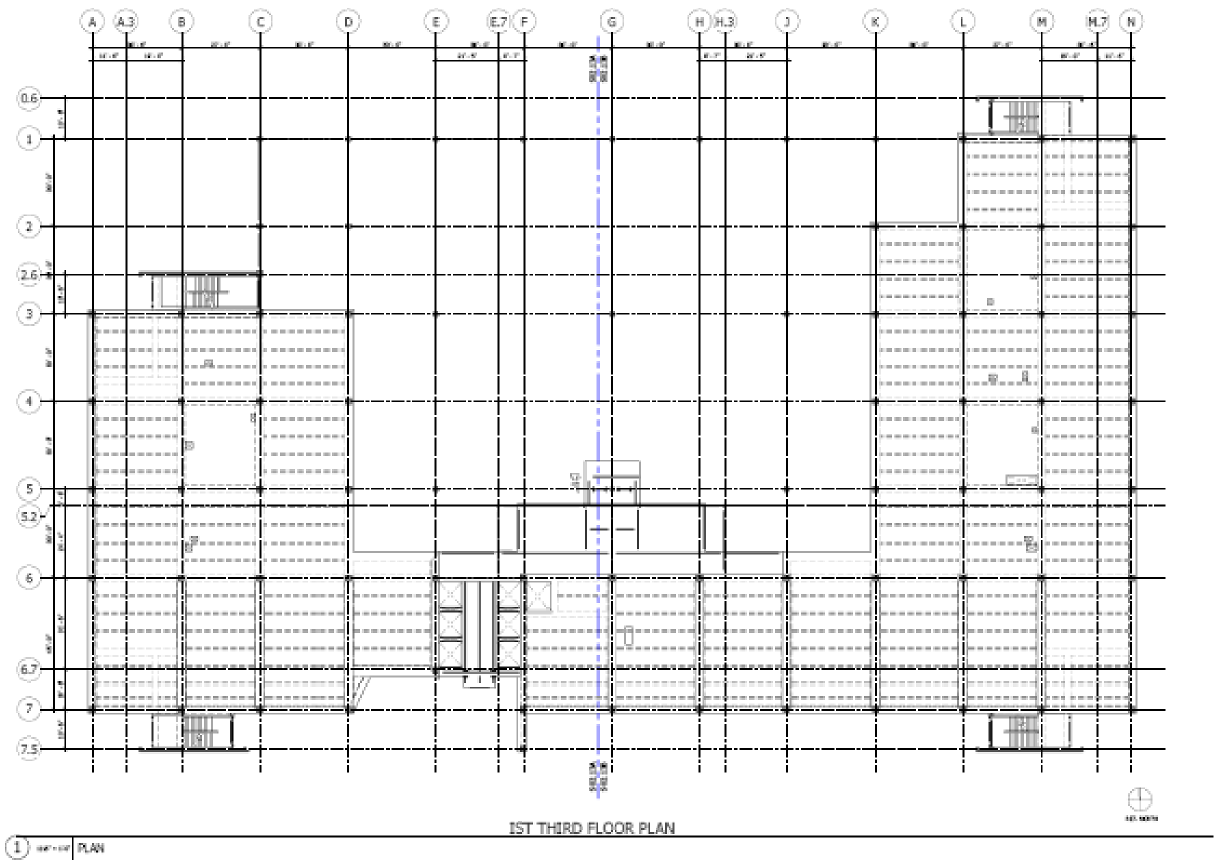

Figure 15: Third Floor Plan

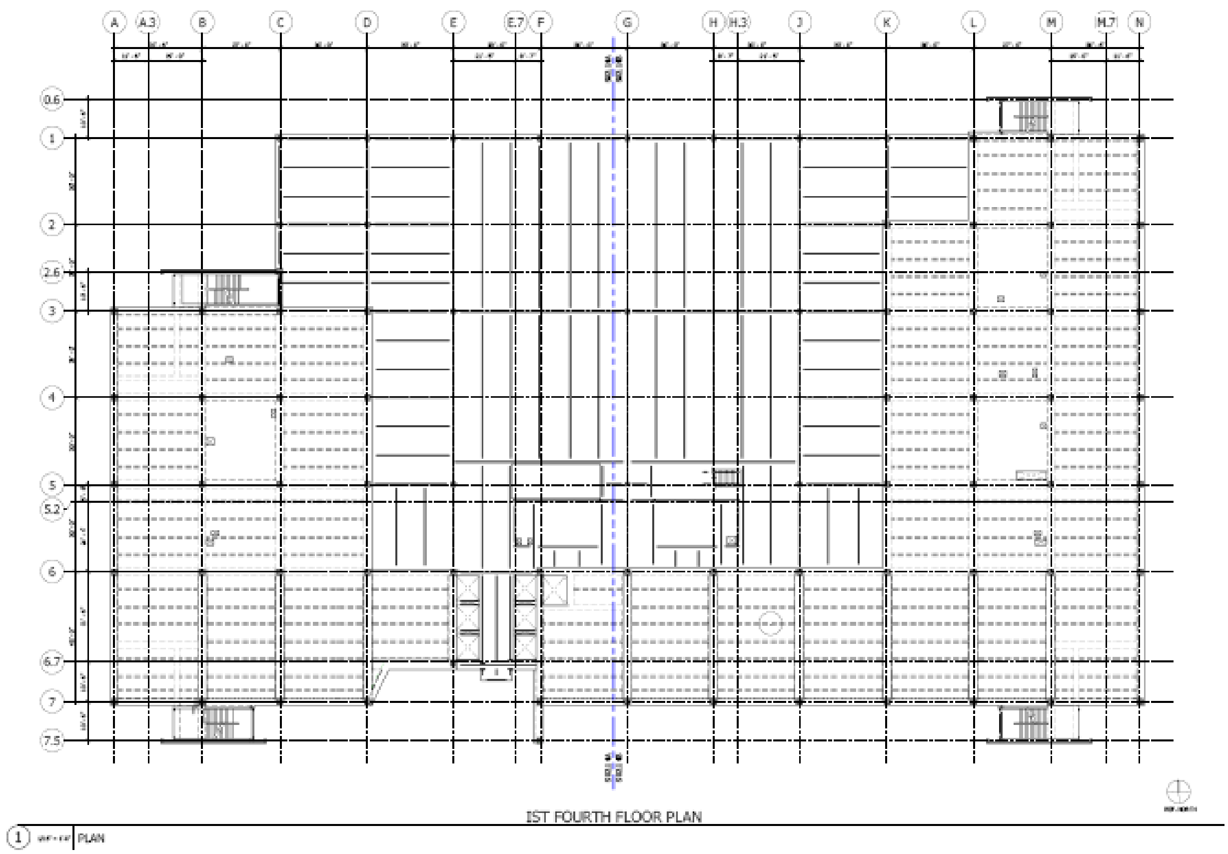

Figure 16: Fourth Floor Plan

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 6: 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
r_2	113	1.023	43.645	37.435	-26.343	38.978	-21.861
r_1	99.25	0.986	42.057	36.359	-26.343	37.845	-21.861
6	81.5	0.932	39.755	34.799	-26.343	36.204	-21.861
5	66.5	0.880	37.510	33.278	-26.343	34.603	-21.861
4	51.5	0.818	34.868	31.487	-26.343	32.719	-21.861
3	36.5	0.741	31.602	29.273	-26.343	30.390	-21.861
2	21.5	0.637	27.167	26.268	-26.343	27.228	-21.861
1	0	0.570	24.310	24.332	-26.343	25.191	-21.861

Table 7: Wind Pressures Calculations

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

Kevin Zinsmeister | Tech Report 1 | Wind Loads | 1

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. $h = 102.5' < 300'$ ∴ okay

2. $L_{eff_{N-S}} = \dfrac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \dfrac{113(195)}{113} = 195'$, $h_i \overset{?}{<} 4L_{eff_{N-S}} \Rightarrow 113' < 4(195) = 780'$ ∴ okay

$L_{eff_{E-W}} = \dfrac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \dfrac{113(355)}{113} = 355'$, $h_i \overset{?}{<} 4L_{eff_{E-W}} \Rightarrow 113' < 4(355) = 1420'$ ∴ 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

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

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

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

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

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

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

R_h: set $\eta = 4.6 n_1 h/\bar V_{\bar z} = 4.6(0.618)113/110.62 = 2.90 > 0$ ∴ Use Eq. 26.9-15a

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

R_B: set $\eta = 4.6 n_1 B/\bar V_{\bar z} = 4.6(0.618)335/110.62 = 8.61 > 0$ ∴ Use Eq. 26.9-15a

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

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

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

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

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

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

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{335+113}{406.81}\right)^{0.63}}} = 0.774$$

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

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

$G_{f_{E\text{-}W}} = \boxed{0.891}$ 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: $q_{z4} = 0.00256 K_z K_{zt} K_d V^2$ lb/ft² $= 0.00256(0.818)1.0(0.85)140^2 = 34.89$ psi

The rest of calculations are on Excel Spreadsheet

Step 6: N-S: Wind

Surface	L/B	C_p	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	C_p	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: $p = qG_fC_p - q_i(GC_{pi})$ $p_4 = q_{z4}G_fC_p - q_h(GC_{pi}) = 34.89(0.847)0.8 - 43.65(\pm 0.18)$

$p_4 = 31.49$ psf

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

Appendix C: Seismic Analysis Calculations

The following is a sample of the floor weight calculations.

third floor concrete weight				
Beam	Width (in)	Height (in)	Linear Length (ft)	ft^3
G11	32	20	30	106.666667
G12	32	20	60	213.333333
G129	36.5	20	30	121.666667
G19	36	20	30	120
G2	32	20	87	309.333333
G20	25.5	33	60	308.125
G21	25.5	33	30	154.0625
G22	33	20	120	440
G23	33	20	180	660
G24	33	20	30	110
G26	25.5	33	90	462.1875
G27	25.5	22	30	95.625
G28	33	20	60	220
G33	25.5	33	88	451.916667
G35	30	33	27	163.125
G36	25.5	33	30	154.0625
G36A	25.5	33	27	138.65625
G37	25.5	33	30	154.0625
G38	25.5	33	90	462.1875
G39	30	33	27	163.125
G4	32	20	27	96
G40	25.5	33	30.5	156.630208
G45	38	20	60	253.333333
G46	38	20	60	253.333333
G47	38	20	60	253.333333
G48	36	33	30	217.5
G49	32	24	31.5	140
G4A	32	20	162	576
G5	32	20	242.5	862.222222
G7	32	20	181.5	645.333333
G8	36	20	30	120
PB1	6	20	633.5	422.333333
PB11	6	20	30	20
PB14	31	20	122	420.222222
PB16	24	30	30	130
PB17	27	20	60	180
PB2	6	20	762	508
PB22	24	20	22	58.6666667
PB25	6	20	120	80
PB27	6	20	30	20
PB28	39	20	30	130
PB3	6	20	741	494

PB30	6	20	30	20
PB4	6	20	785.5	523.666667
PB5A	6	20	120	80
PB6	6	20	27	18
PB7	6	20	20	13.3333333
PB77	6	20	482	321.333333
PB9	6	20	108	72
TPG1	36	24	135	810
TPG10	30	33	90	618.75
TPG4	36	24	180	1080
TPG6	24	33	45	247.5
TPG7	36	24	45	270
TPG9	36	24	90	540
			Total=	15659.6267
			Weight (lb)=	2348944.01
			Weight (kips)=	2348.94401
slab	area (ft^2)	W (psf)	thickness (in)	ft^3
pan	31890		4	10630
composite	336	44	5.5	n/a
2-way	2430		10	2025
			Weight (lb)=	1913034
			Weight (kips)=	1913.034
columns	number	area (ft^2)	length (ft)	weight (lb)
below	72	4	7.625	329400
above	72	4	7.625	329400
			Total=	658800
			Total (kips)=	658.8
		Third Floor B Steel Weight		
		Linear Weight (lb)	Length (ft)	Weight
		19	51.66666667	981.666667
		22	19	418
		55	108.9166667	5990.41667
		84	22.91666667	1925
		103	36.16666667	3725.16667
		176	17.5	3080
			sum (lb)=	16120.25
			sum (kips)=	16.12025
		Third Floor A Steel Weight		
		Linear Weight (lb)	Length (ft)	Weight
		19	56.16666667	1067.16667
		40	60	2400
		55	109.0833333	5999.58333
		176	9.916666667	1745.33333
			sum (lb)=	11212.0833
			sum (kips)=	11.2120833
			Floor Weight (kips)=	4948.11034

Table 8: Sample Floor Weight Calculations

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

Kevin Zinsmeister	Tech Report 1	Seismic Loads	1

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) ∴ $C_s \leq \frac{S_{D1}}{T(\frac{R}{I})} = \frac{0.084}{1.13(\frac{3}{1.25})} = 0.031$ ⇐ controls

$W_{GF} = 1089^k$, $W_{FL2} = 8322^k$, $W_{FL3} = 4948^k$, $W_{FL4} = 6777^k$,

$W_{FL5} = 4948^k$, $W_{FL6} = 6777^k$, $W_{R1} = 3711^k$

$W_T = 36572^k$

$V = C_s W = 0.031(36572) = 1133.7^k$

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12.8.3 Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$, where $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$, $k = 1.315$ From linear interpolation using Eqn 12.8-12

$$C_{RF} = \frac{1089(113)^{1.315}}{1089(113)^{1.315} + 3711(99.25)^{1.315} + 6777(81.25)^{1.315} + 4948(66.5)^{1.315} + 6777(51.5)^{1.315} + 4948(36.5)^{1.315} + 8322(21.5)^{1.315}}$$

$C_{RF} = \dfrac{545539.2885}{7786651.111} = 0.0701,$ $\quad F_{RF} = C_{RF} V = 0.0701(1133.7) = 79.5^k$

$C_{R1} = \dfrac{3711(99.25)^{1.315}}{7786651.111} = 0.2013,$ $\quad F_{R1} = C_{R1} V = 0.2013(1133.7) = 228.2^k$

$C_{F6} = \dfrac{6777(81.25)^{1.315}}{7786651.111} = 0.2826,$ $\quad F_{F6} = C_{F6} V = 0.2826(1133.7) = 320.4^k$

$C_{F5} = \dfrac{4948(66.5)^{1.315}}{7786651.111} = 0.1585,$ $\quad F_{F5} = C_{F5} V = 0.1585(1133.7) = 179.7^k$

$C_{F4} = \dfrac{6777(51.5)^{1.315}}{7786651.111} = 0.1551,$ $\quad F_{F4} = C_{F4} V = 0.1551(1133.7) = 175.8^k$

$C_{F3} = \dfrac{4948(36.5)^{1.315}}{7786651.111} = 0.0720,$ $\quad F_{F3} = C_{F3} V = 0.072(1133.7) = 81.6^k$

$C_{F2} = \dfrac{8322(21.5)^{1.315}}{7786651.111} = 0.0604,$ $\quad F_{F2} = C_{F2} V = 0.0604(1133.7) = 68.5^k$

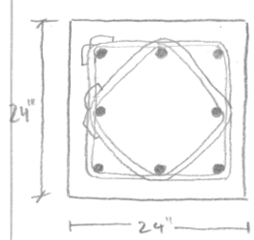
$1.0000 \Rightarrow$ good

Appendix D: Spot-Check Calculations

The following five pages are the hand calculations and assumptions for the spot-checks.

Kevin Zinsmeister	Tech Report 1	Column Spot Check	1

Column B5 @ Level 6



$f'_c = 4000$

$f_y = 75$ ksi

8 - #9

#3 ties @ 12"

$A_T = 30' \times 30' = 900\ ft^2$

pans / g1 & g13 / g5 3 & g72 / pan slab

$D_L = \left[30'(3)\frac{6''}{12}\left(\frac{16''}{12}\right) + 30'\left(\frac{32''}{12}\right)\frac{16''}{12} + 30'\left(\frac{33''}{12}\right)\frac{16''}{12} + \frac{4''}{12}\left(\frac{3}{4}\right)900\ ft^2 + \frac{10''}{12}\left(\frac{1}{4}\right)900\ ft^2\right] 150\ pcf + 20(900)$

2-way slab / misc. dead

$DL = 121375\ lb = 121.4^k$

mechanical rooms / office

$LL = 100\left(\frac{1}{4}\right)900 + 80\left(\frac{3}{4}\right)900 = 76500\ lb = 76.5^k$

$P_{u_{min}} \begin{cases} 1.4(121.4) = 170^k \\ 1.2(121.4) + 1.6(76.5) = 268^k \end{cases}$, $\phi P_n = 0.85\phi\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right]$ ← ACI Eqn (10-1)

$\phi P_n = 0.85(0.65)\left[0.85(4)(24^2 - 8(1.00)) + 75(8)1.00\right] = 1398.5^k > 268^k \therefore$ okay

Other Loads in Excel Spreadsheet

From Excel

Column B5 @ Level 5, $\phi P_{n5} = \phi P_{n6} = 1398.5^k > 546.9^k \therefore$ okay

Column B5 @ Level 4, $\phi P_{n4} = \phi P_{n6} = 1398.5^k > 836.6^k \therefore$ okay

Column B5 @ Level 3, f'_c increases to 5000

$\therefore \phi P_n = 0.85(0.65)\left[0.85(5)(24^2 - 8) + 75(8)\right] = 1665.2^k > 1137.0^k \therefore$ okay

Column B5 @ Level 2, A_s increases to 12 - #9

$\therefore \phi P_n = 0.85(0.65)\left[0.85(5)(24^2 - 12) + 75(12)\right] = 1821.6^k > 1448.2^k \therefore$ okay

Column B5 @ Level 1, $f'_c = 6000$, $A_s = 8(1.27) = 10.16\ in^2$

$\therefore \phi P_n = 0.85(0.65)\left[0.85(6)(24^2 - 10.16) + 75(10.16)\right] = 2015.4^k > 1742.8^k \therefore$ okay

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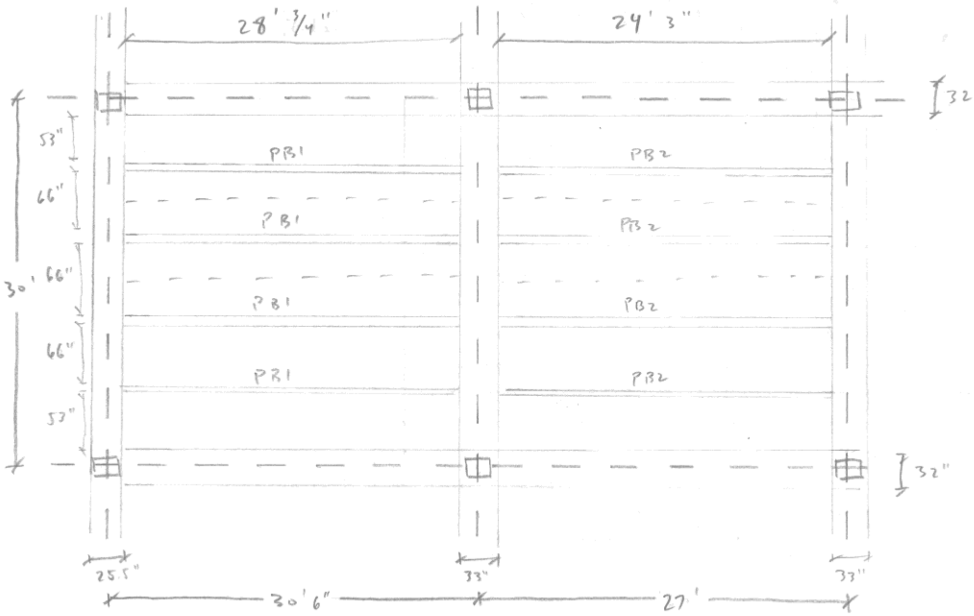

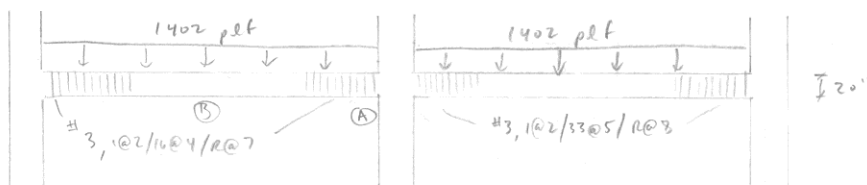

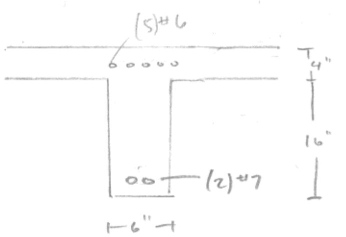

PB1 Detail @ interior suppor

$$DL = \frac{6(20)}{144}150 + \frac{4(66)}{144}150 + \overset{SDL}{25\left(\frac{70}{12}\right)}$$

$$DL = 125 + 275 + 145.83 = 546 \text{ plf}$$

$$LL = 80\left(\frac{70}{12}\right) = 467 \text{ plf}$$

$$L = 1.2(546) + 1.6(467) = 1402 \text{ plf}$$

$$\text{Max neg moment}_{A} = \frac{w_u l_n^2}{9} = \frac{1402(28.0625)^2}{9} = 122675.6 \text{ lb.ft} = 122.7'^{k}$$

$$\text{Max pos. moment}_{B} = \frac{w_u l_n^2}{14} = \frac{1402(28.0625)^2}{14} = 78862.9 \text{ lb.ft} = 78.9'^{k}$$

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Top Reinforcing: $F_y = 60$ ksi, $A_s = 5(0.44) = 2.2\ in^2$, $f'_c = 5000$ psi

$d = 16 + 1.5 = 17.5''$, $a = \frac{A_s F_y}{0.85 f'_c b} = \frac{2.2(60)}{0.85(5)6} = 5.18''$ $\therefore c = \frac{5.18}{0.8} = 6.475''$

$\varepsilon_s = \frac{0.003}{6.475}(17.5 - 6.475) = 0.0051 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9 A_s F_y (d - \frac{a}{2}) = 0.9(2.2)60(17.5 - \frac{5.18}{2}) = 1771.31$ in k $= 147.6^{'k} > 122.7^{'k}$ $\therefore$ okay

Bottom Reinforcing: $F_y = 60$ ksi $A_s = 0.6(2) = 1.2\ in^2$, $f'_c = 5{,}000$ psi

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{1.2(60)}{0.85(5)6} = 2.82$ $\therefore$ $c = \frac{2.82}{0.8} = 3.53''$

$\varepsilon_s = \frac{0.003}{3.53}(17.5 - 3.53) = 0.0118 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9(1.2)60(17.5 - \frac{2.82}{2}) = 1042.6$ in k $= 86.9^{'k} > 78.9^{'k}$ $\therefore$ okay

Joist Shear: $A_v = 0.11\ in^2$

$V_u = \frac{1.15(1.4)28.0625}{2} = 22.59^k$

$\phi(V_c + V_s) = 0.75\left[2\sqrt{5000}(6)17.5 + 0.11(60)\frac{17.5}{4}\right] = 0.75[14.85 + 28.88] = 32.8^k > 22.59^k$

$\therefore$ okay

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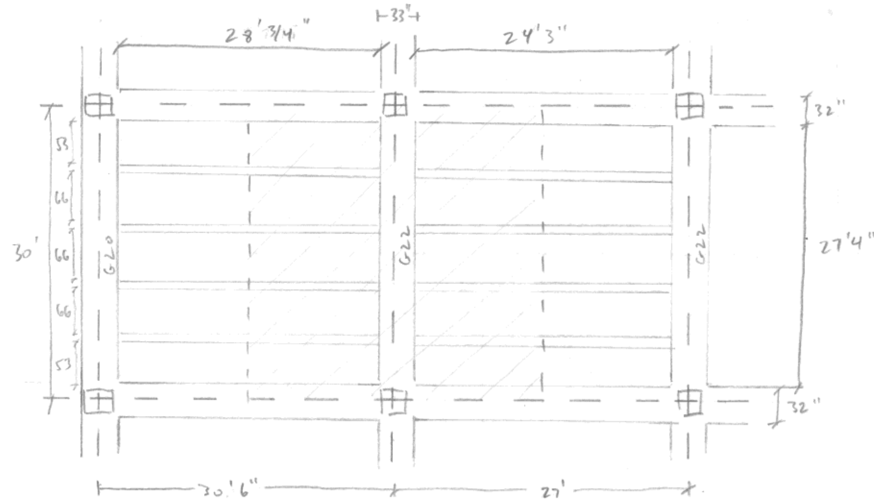
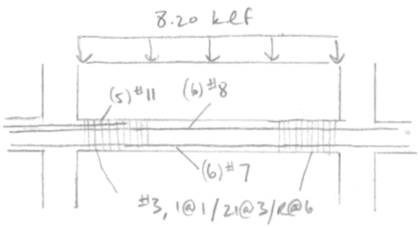




$$\text{Trib. width} = \frac{24.25 + 28.0625}{2} = 26.15625$$

$$w_{u\,floor} = 1.4\left(\frac{12}{66}\right) = 0.254\ k/ft^2$$

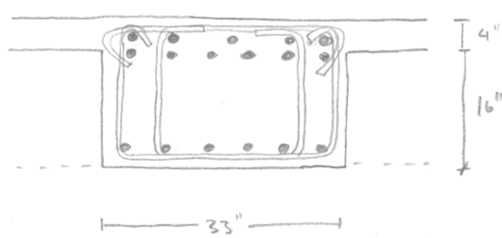
$$w_{u\,girder} = \frac{33}{12}\left(1.2(\underbrace{0.025}_{SDL}) + 1.6(\underbrace{0.08}_{LL})\right) = 0.435\ klf$$

$$\text{selfwt } w_u = 1.2\left[33(20)\frac{150}{144}\right] = 0.825\ klf$$

$$w_u = 0.254\left(26.2 - \frac{33}{12}\right) + 0.435 + 0.825 = 7.22\ klf$$

$$\text{Max Neg. Moment} = \frac{w_u l_n^2}{11} = \frac{7.22(27.3)^2}{11} = 490.2'k$$

$$\text{Max Pos. Moment} = \frac{w_u l_n^2}{14} = \frac{7.22(27.3)^2}{14} = 385.2'k$$



G22 Detail @ left support

Kevin Zinsmeister | Tech Report 1 | Girder Spot Girder | 2

Top Reinforcing: $F_y = 75$ ksi, $A_s = 6(0.79) = 4.74\ in^2$, $f'_c = 5000$ psi

$d = 16+1.5 = 17.5''$, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.74(75)}{0.85(5)33} = 2.53''$ $\therefore c = \frac{2.53}{0.8} = 3.16''$

$\varepsilon_s = \frac{0.003}{3.16}(17.5-3.16) = 0.0136 > 0.005$ ∴ girder is tension controlled

$\phi M_n = 0.9 A_s F_y (d - a/2) = 0.9(4.74)75(17.5 - \frac{2.53}{2}) = 5194.4$ in·k $= 432.9'^k < 490.2'^k$ ∴ not okay

∴ Add in additional rebar, $A_s = 6(0.79) + 5(1.56) = 12.54\ in^2$, $d = 17.5 - 11/8(1/2) = 16.81$

$a = \frac{12.54(75)}{0.85(5)33} = 6.71''$ ∴ $c = \frac{6.71}{0.8} = 8.38''$

$\varepsilon_s = \frac{0.003}{8.38}(16.81 - 8.38) = 0.0030 < 0.005$, $0.0030 > 0.002$ ∴ girder is in transition zone

$\phi = 0.65 + (\varepsilon_t - 0.002)\frac{250}{3} = 0.65 + (0.0030 - 0.002)\frac{250}{3} = 0.73$ — ACI R9.3.2

∴ $\phi M_n = 0.73(12.54)75(16.81 - \frac{6.71}{2}) = 9237.7$ in·k $= 769.8'^k > 490.2'^k$ ∴ okay

Bottom Reinforcing: $F_y = 75$ ksi, $A_s = 6(0.6) = 3.6\ in^2$, $f'_c = 5000$ psi

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.6(75)}{0.85(5)33} = 1.93''$, $c = \frac{1.93}{0.8} = 2.41''$

$\varepsilon_s = \frac{0.003}{2.41}(17.5 - 2.41) = 0.0188 > 0.005$ ∴ girder is tension controlled

$\phi M_n = 0.9(3.6)75(17.5 - \frac{1.93}{2}) = 4018$ in·k $= 334.8'^k < 385.2'^k$ ∴ not okay

Shear:

$V_u = \frac{w_u l_n}{2} = \frac{7.22(27.3)}{2} = 98.7^k$

$\phi(V_c + V_s) = 0.75[2\sqrt{5000}(33)17.5 + 0.11(60)\frac{17.5}{3}] = 0.75[81.67 + 38.5] = 90.13^k < 98.7^k$
∴ not okay