Fall 2009

Hunter College school of Social Work

Structural Systems Overview

The Structural Concepts / Structural Existing Conditions Report consists of a requirement to describe the physical existing conditions of the structure of your building including information relative to design concepts and required loading.

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Technical Report 1 – Structural Analysis

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Executive Summary

The facility is approximately 148,000 square feet, has 8 stories and reaches a height of 134' above grade with a typical floor to floor height of approximately 14 feet. A typical interior bay of the steel frame structure is 30 feet by 28 . The lateral system utilizes steel braced frames and trusses. The building is supported by concrete columns in the cellar level and the whole structure sits on a mat foundation.

A mat foundation was recommended by the geotechnical report. Although mat foundations are typically more expensive, they are used to avoid deep excavation and when too many different thicknesses of spread footings are required through the foundation level. Varying building heights due to New York City's set-back laws led to different strength requirements at the basement level, thus a mat foundation was probably the best option. It also provides a greater ease of constructability.

Column sizes were found to be very large when checked for gravity load, specifically chose those engaging the lateral load resisting frame. This is because the columns are part of a moment connection on the braced frame. The lateral load resisting braced frame induces large moments on the columns resulting in larger columns than expected from gravity design alone.

Seismic was found to be not very important in this area and according to the geotechnical report, soil liquefaction is not a concern. The R-value obtained from my analysis was a 7; however the design was based on the New York City Building Code and stated an R-value of 8. The wind forces on the structure control over seismic forces. When calculating building weight, approximations were made on curtain wall weight and therefore the seismic base shear may be slightly lower were designed for a dead loaf than the actual base shear.

Beam sizes were not adequate for initial assumptions of 71psf dead load. It is more likely that the beams were designed for a dead load of 50psf and live load of 100psf. Design was controlled by flexure capacity. To account for a deflection of 0.08" over that which was allowable, a camber of 1 ¼" camber was applied to the beam.

Concrete slab on metal deck runs perpendicular to beams and parallel to girders. Shear studs allow for full composite action where extra capacity is needed by the beams. Number of shear studs varies with the beam size and required capacity, spacing of the studs are distributed evenly along the length of the member.

Unlike the beams, girder sizes were very close to what was expected and deflection limits were not an issue, actual design however shows a ¾" camber.

Introduction

The building's design responds to the School of Social Work's mission by providing an open and engaging face to the neighborhood and opportunities for community use of parts of the facility. The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. Classrooms and lecture halls occupy the lower levels with academic departments and offices on upper floors. An auditorium on the second floor is expressed on the facade, with a glazed wall allowing views of activity in and outside the building. A rear landscaped terrace will link the School to a planned CUNY Residential building adjacent to the site on 118th Street. The School of Social Work building will be LEED certified.

-Cooper Robertson & Associates

Keyplan

The structure of Hunter College School of Social Work is comprised of a composite steel floor system that utilizes steel braced frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation.

The purpose of Technical Report I is to gain an understanding of how gravity and lateral loads are resisted by the existing structural system. Upon completion of this report, conclusions will be drawn on the validity of member sizes based on gravity loads. Future technical reports will include lateral forces with member spot checks.

Structural Systems Overview

Foundations

There is one below-grade level in the Hunter College School of Social Work. This level known as the cellar contains a parking garage for the residential building adjacent, a library, computer labs, large kitchen areas, and mechanical rooms.

Slab thickness varies throughout the cellar level. It can be 30”, 33”, or 40”. Steel reinforcement varies according to the slab thickness. For a 30” slab #7@11 are required top and bottom (T&B) each way, for a 33” slab #8@13 top and bottom, and for a 40” slab #9@13 top and bottom each way. The mat foundation will have a 2” mud slab above 12” of ¾ crushed stone to facilitate installation of waterproofing membrane. The subgrade is composed of undisturbed soil or compacted back fill with a required bearing capacity of 1.5 tons.

The soil is not considered susceptible to liquefaction for a Magnitude 6 earthquake and a peak ground acceleration of 0.16g. It is expected to encounter ground water during erection of the cellar level. Excavation depths are anticipated to vary from about 12ft to 20ft below existing ground surface grades. Footings shall bear on sound rock with a bearing capacity of 20 ton per square foot or on decomposed rock with a bearing capacity of 8 ton per square foot or on sand with a bearing capacity of 3 ton per square foot.

Foundation walls are designed to resist lateral pressures resulting from static earth, groundwater, adjacent foundations, and sidewalk surcharge loads. These walls will extend 14ft below existing ground surface grades. Concrete for foundations and site work shall be air-entrained normal weight stone concrete with a minimum compressive strength of 4000psi at 28 days and a maximum water to cement ratio of 0.45 by weight.

In the western portion of the six story faculty housing building footprint, it is recommended to excavate rock 12” below bottom of foundation in order to limit differential settlement between sections of the mat foundation bearing on rock and that bearing on soil.

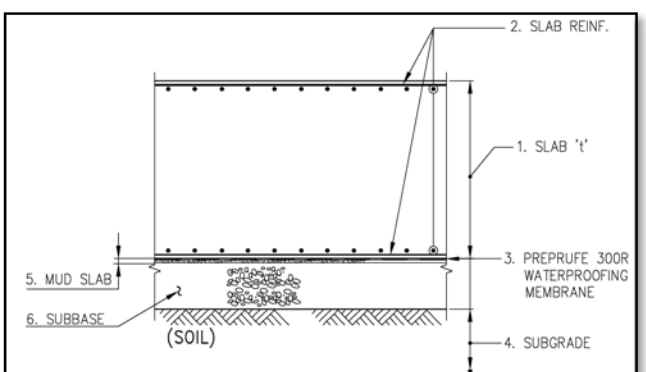


Figure 1: Mat Foudation Detail

Floor system

The slab thickness for all floors is 3 ¼" thick 3500psi lightweight concrete placed over 3" deep 18 gage composite galvanized metal deck reinforced with 6x6- W2.9xW2.9 welded-wire-fabric. Exceptions on the ground floor are on the outdoor court, entry vestibules, and loading area; here 3" lightweight concrete is placed over 16 gage metal deck is used and instead of WWF, reinforcement is #4@12" o.c. top bars each way and 1-#5 bottom bars each rib. The exception for the second floor is the roof terrace where there is 5" of lightweight concrete over 3"-16 gage metal deck. On the roof level, the floor slab for the electrical control room is 8" lightweight concrete formed slab reinforced with to#4@12"o.c. top and bottom each way.

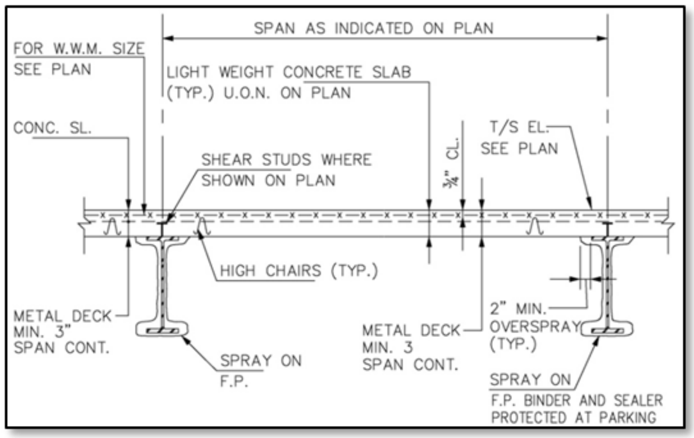


Figure 2. Typical Floor Construction , Metal Deck Perpendicular to Floor Beams on Girders

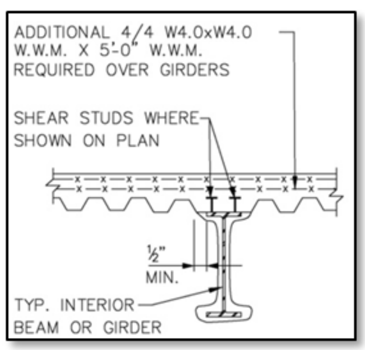


Figure 3. Typical Floor Construction, Metal Deck Parallel to Beams or Girders

Gravity System

Columns in the basement are 4000psi air-entrained concrete and vary in size from 32x48 to 36x60. The bay sizes vary from 30'x28', 30'x 28'2", 30'x31'5" and 30'x36' from north to south respectively.

All columns in the superstructure are W14s. Due to setbacks and varying story footprint, service loads carried by the columns at the ground level vary ranging from 137 to 1154kips. Because the service loads vary greatly throughout the floor, the column sizes vary as well; for example, on the ground floor column sizes range from w14x68 to w14x730. In the levels above the cellar, the bay sizes do not change.

There are non-composite beams as well as composite beams (with studs). Non-composite beams are found where beam to beam and beam to column connections are designed to transfer the reaction for a simply supported, uniformly loaded beam . For composite beams, connections are designed to have 160% capacity of the reaction for a simply supported, uniformly loaded beam of the same size, span, fy, and allowable unit stress. For framed beam connections, including single plate connections, the minimum number of horizontal bolt rows should be provided based on 3" center-to-center.

Lateral System

Trusses with vertical members attached using moment connections make up the lateral system. Locations of these trusses are represented on figure 4 in red; they run all the way up the building levels. The only exception to this is the frame truss represented on figure 4 as blue since it changes as you go up in elevation. An elevation view of this truss is shown as figure 5. Braced frames were chosen to resist lateral forces because they are more efficient than moment frames in both cost and erection time.

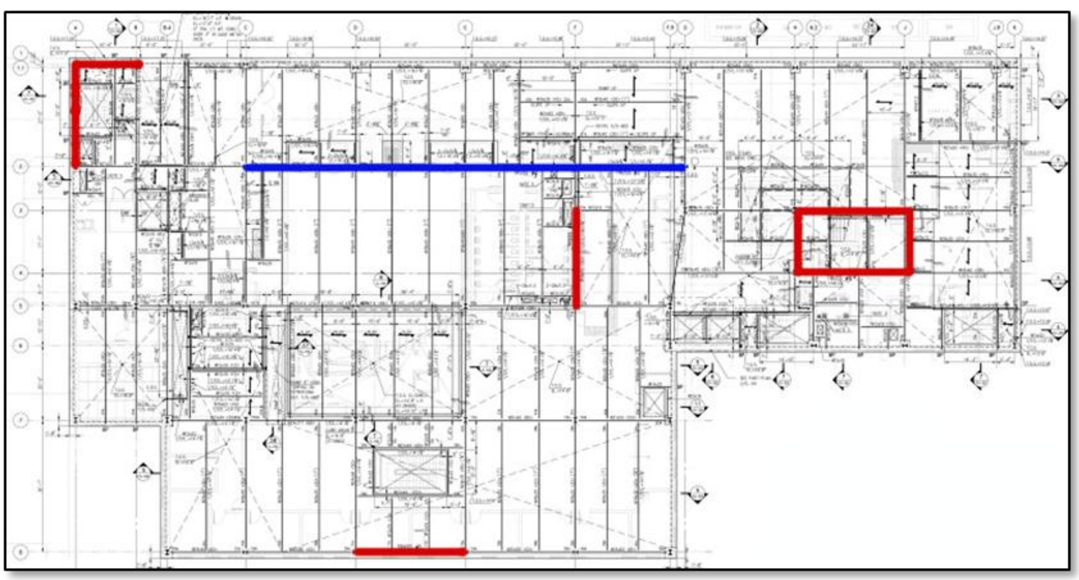


Figure 4. Location of Lateral Force Resisting Systems (Braced Frames)

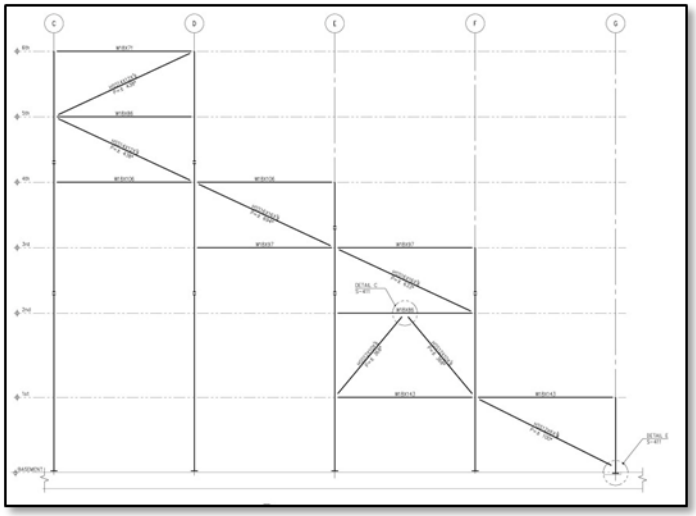


Figure 5. Truss Elevation at Grid 2

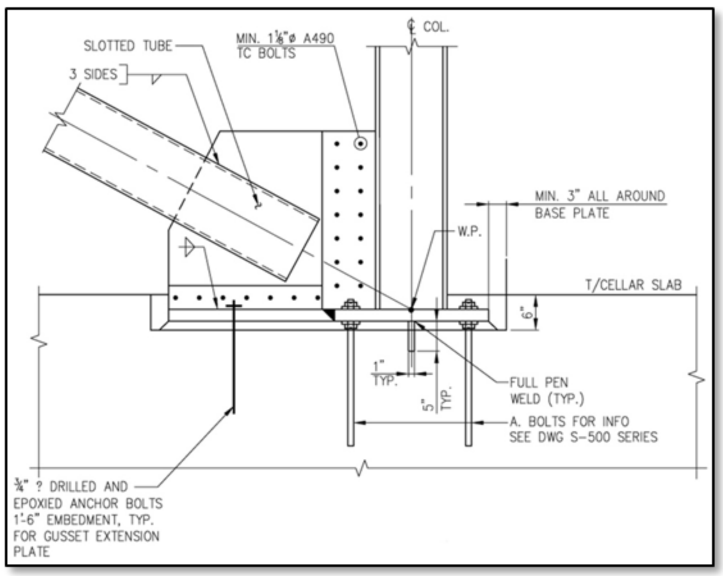


Figure 6. Lateral Load Resisting Detail

Roof System

The roof is typically composed of 3 1/2 " light weight concrete over 3"-18 gage metal deck reinforced with 6x6-2.9x2.9 WWF. In a 200 square foot section the slab is 8" lightweight concrete slab reinforced with #4@12 top and bottom E.W. Columns are placed where needed and don't necessarily follow a typical framing layout. To provide additional vibration control, 4" concrete pads are located below mechanical equipment.

Curbs on the roof are of CMU and concrete.

Codes and Design Standards

Applied to original Design

The Building Coded of the City of New York (most current) - Amended seismic design

AISC-LRFD, LRFD Specification for Structural Steel Buildings

AISC- ASD 1989, Specifications for Structural Steel Buildings- ASD and Plastic Design

ACI 318-89, Building Code Requirements for Structural Concrete

Substituted for thesis analysis

2006 International Building Code

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Material strength requirement summary

Structural Steel:

- All W Beams and Columns: ASTM A992, Fy=50ksi
- HSS Steel, Fy=46ksi
- Connection Material:Fy=36 ksi
- Base plates: ASTM 572 GR50, Fy=50ksi

Metal Decking:

- Units shall be 3" galvanized composite deck of 18 gage formed with integral locking lugs to provide a mechanical bond between concrete and deck

-Strength: Fy=40ksi

-Deflection of form due to dead load of concrete and deck does not exceed L/180 , but not more than ¾"

-Deflection of composite deck cannot exceed L/360 of deck span under superimposed live load.

Concrete:

-Caissons and Piers: 4000psi normal weight concrete

-Slabs on ground and footings: 4000psi normal weight concrete

-Retaining Walls: 4000 psi normal weight concrete

-Slab on deck: 3500psi lightweight concrete

- Foundations: 4000psi, air entrained, normal weight

-Walls, curbs, and parapets: 4000 psi

Reinforcement:

-Strength: 60ksi

Building Load Summary

Total building weight was found to be approximately 15,388kips. Detailed charts in Appendix A tabulate the columns and beams used in finding the total weight. Curtain wall weight was approximated to be 15 psf although curtain wall type varies as you go up in elevation. Glass curtain wall is used on the upper and lower sections of the building façade and precast masonry and stucco panels are used on the middle section of the building façade. Calculation of the building weight was tedious due to the varying bay sizes, column and beam sizes, and varying lengths of these members. In erection of the structure, careful coordination must be taken in order to correctly identify and place these frame elements.

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
Penthouse	134	80750	0	38245	0	118995
Roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Figure 7. Building Dead Load Summary

ID	location	Live Loads (psf)			Dead Loads (psf)
		Design Live Loads	ASCE 705-05	NYC BLDG CODE 08	Design Dead Loads
1	loading dock	600	-	-	150
2	1st floor	100	100	100	130
3	Podium	100	100	-	200
4	Archive	350	-	-	75
5	Offices	50	50	50	71
6	roof with garden	100	100	100	365
7	library stacks	100	100	100	71
8	Classrooms	40	40	60	71
9	Corridor	100	100	100	71
10	Auditorium	60	60	100	85
11	roof with pavers on 2	100	-	-	150
12	roof	45	20	30	90
13	roof with drift	60	45	-	85
14	Mechanical	100	125	100	120

Figure 8. Loading Schedule

Design Analysis & Conclusions

Wind Load Summary

Since the Hunter College School of Social Work is located in New York City, the NYC Building Code governed the structural design. For this analysis, however, ASCE-7-05 was used along with Fanella Wind Analysis flowcharts. For detailed calculations please refer to Appendix A. In the north/south direction the base shear due to lateral wind loads was found to be 559 kips, much larger than in the East/West direction; 162 kips. This difference in base shear is due to building's rectangular shape as opposed to a square footprint. Wind forces were found to be much higher than seismic forces (figure 14). Seismic base shear was found to be 154 kips, less than wind-caused shear in either direction; north/south or east/west.

Due to the building's setbacks, it has differing roof levels, creating a potential for snow drifts. The allowable snow drift calculations were found to be 46psf (refer to Appendix A for details). The allowable snow drift values, along with the wind or seismic analysis, were not checked against the values originally found by the structural designers. The information needed was not provided on the construction documents for verification.

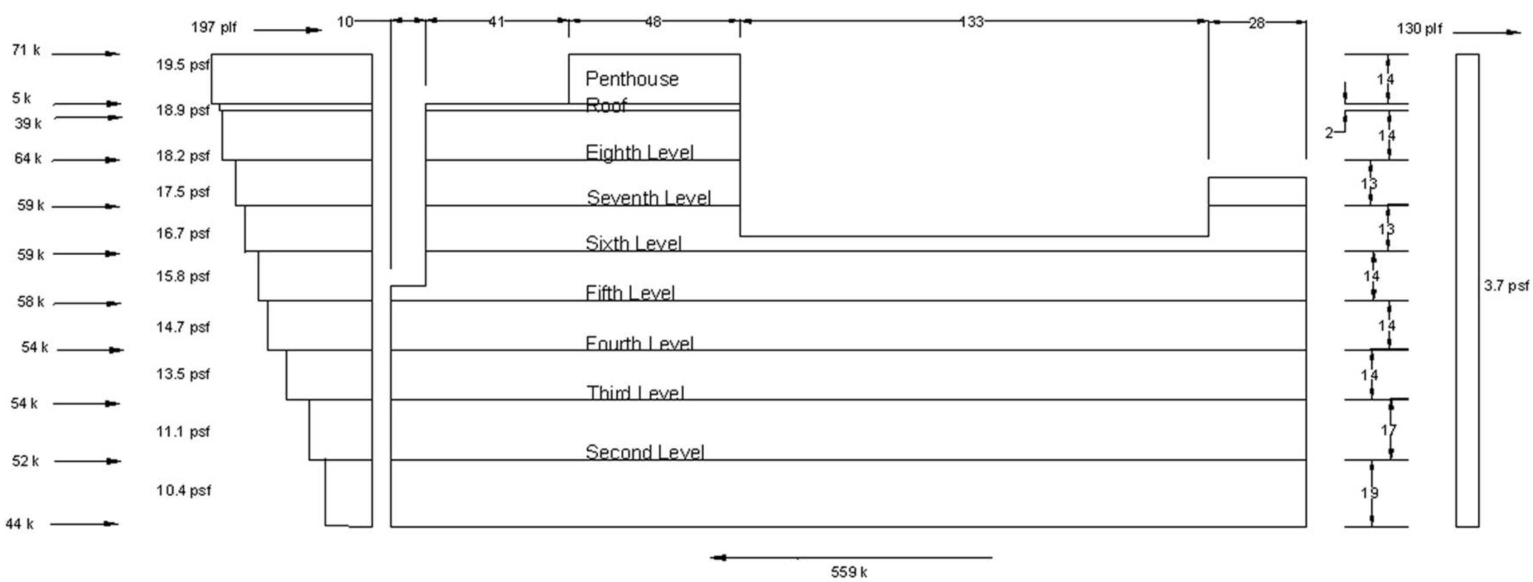


Figure 9. Wind Diagram using ASCE7 – In North/South wind direction

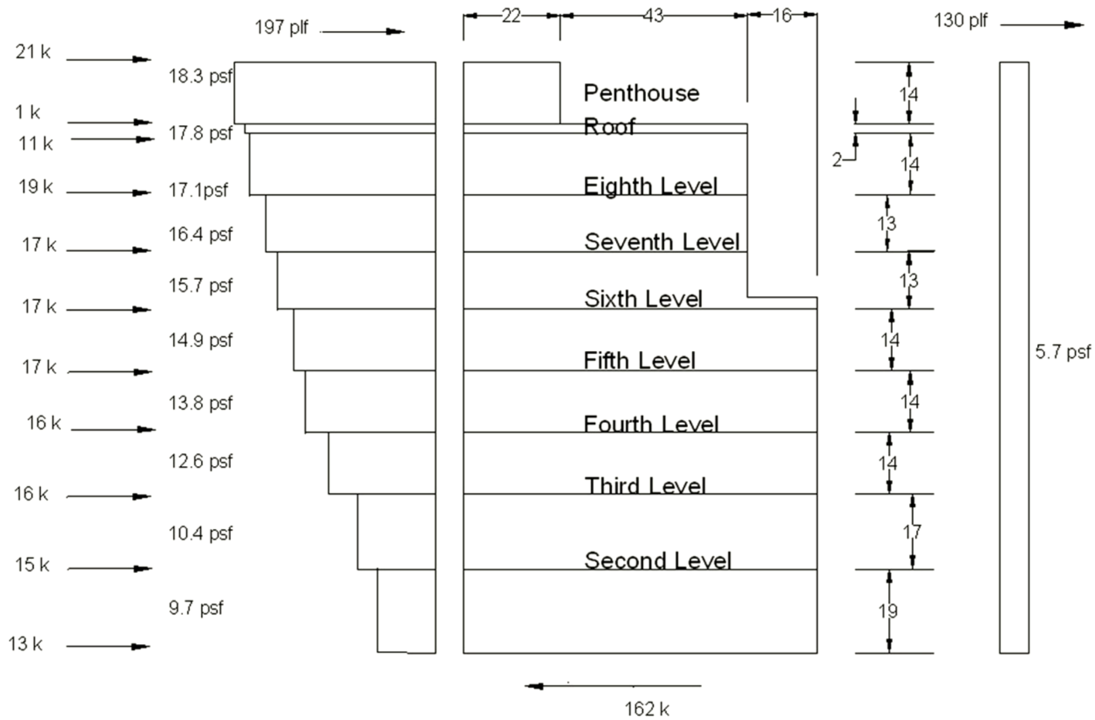


Figure 10. Wind Diagram using ASCE7 – In East/West wind direction

Refer to figures 11 through 13 for design forces, shears, moments, and assumptions for wind using ASCE 7. For detailed calculations, refer to the appendix.

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

Figure 11. Wind Design Forces and Shears

Design Category	III
V (mph)	90
K_d=	0.85
Importance Factor (I)	1.1
Exposure Category	B (urban areas)
K_{zt}=	1
n1=	0.75
Gf	1.173 (N-S)
	1.189 (E-W)
Qp	20.16
GCpn	+1.5 windward
	-1.0 leeward
GCpi	n/a
z_g=	1200 ft
α=	7

Figure 12. Wind Design Criteria

	Level	Height Above Ground (ft)	Floor Height (ft)	K_z	q_z
windward	Penthouse	134	14	1.07	20.75
	T.O. Parapet	120	0.25	1.04	20.16
	Roof	118	1.7	1.04	20.16
	8	104	14	1	19.39
	7	91	13	0.96	18.61
	6	78	13	0.92	17.84
	5	64	14	0.87	16.87
	4	50	14	0.81	15.70
	3	36	14	0.74	14.35
	2	19	17	0.61	11.83
	Ground	0	19	0.57	11.05
Leeward	All	All	All	1.04	20.16

Figure 13. Wind Design qz factors for different story levels

Seismic Summary

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Please refer to Appendix A for detailed calculations used to obtain building weight as well as base shear and overturning moment distribution for each floor as seen in figure 14 below. According to the construction documents, seismic analysis was not found to control this design. The site was declared not an issue for soil liquefaction.

Due to low approximations on the building weight the base shear may in actuality be higher than what is reported in figure 14. However it would not control because the shear cause by lateral wind loads is more than 3 times in magnitude.

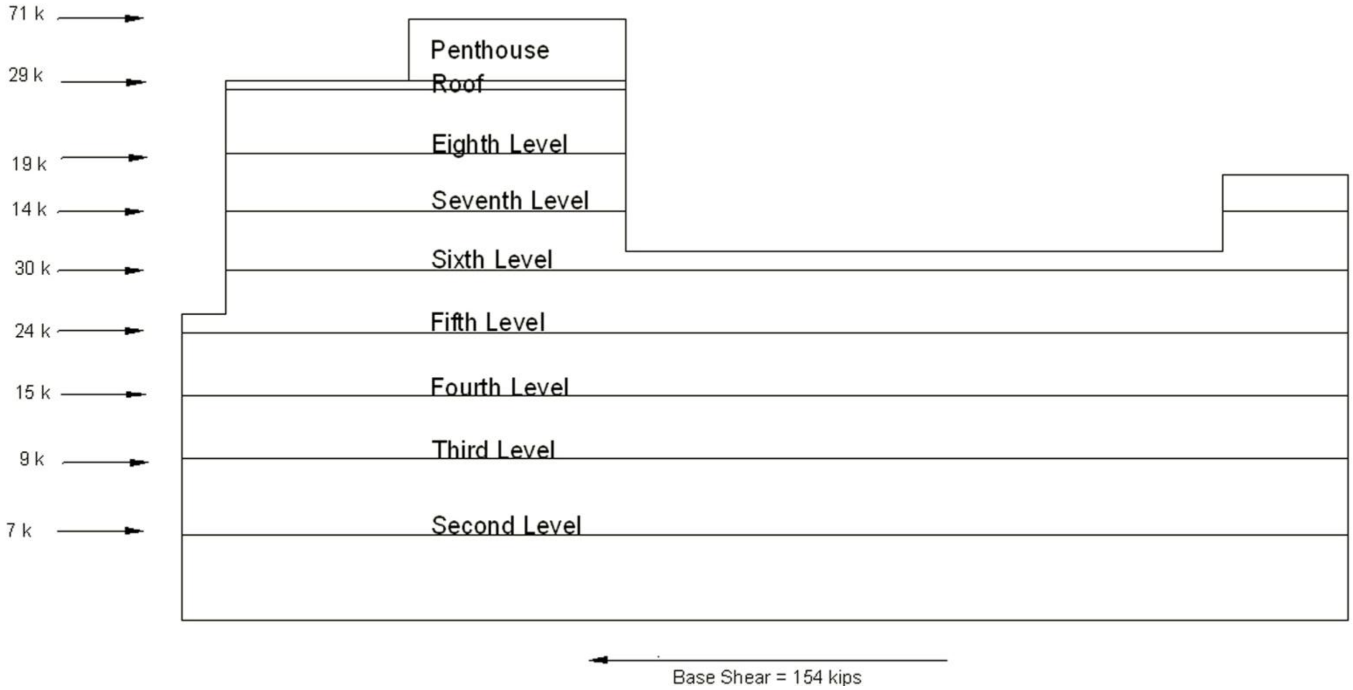


Figure 14. Seismic Force Diagram

Spot Checks

A typical bay on the second floor was analyzed in order to confirm the engineer of record's design methods. Please refer to appendix A for detailed calculations of the following descriptions.

Evaluation of a composite beam within an interior bay show that a typical W18x35 beam cannot carry the bending moment created by placing the concrete during construction. The number of shear studs required was different than that which was provided. This was probably due to an overestimation of the dead load on the beams. With the concrete and steel working together to distribute amongst each other compression and tension forces, the moment resisting capacity of the system increases. Although deflection limits was initially not satisfied, review with the construction documents show a 1-1/4" camber to offset the deflection.

A girder was examined next to ensure that the member can transfer the loads from the composite beam to the columns. It was determined that a W18x60; the typical member chosen, can carry the induced moment created by the beams framing on both sides. Deflection limits were also satisfied, however, the construction documents show a ¾" camber. This camber is probably due to a higher loading which the engineer designer for.

For the column spot check, the typical bay was not chosen due to the reduction in footprint at the roof level. Therefore, a column located at the roof level with columns below it at the same grid location was chosen. This column at the roof level was a W14x90. Dead loads applied to the columns were computed using the floor weights from the seismic calculations, taking into account influence area. A summary of the accumulated loads is found in Appendix A. Live loads were applied in accordance ASCE 7-05. The effective length of each column was taken as the actual length of the column, which meant that sometimes the same column run down two floor levels.

After performing the compression check for the column on the roof level using flexural buckling equations in Chapter E of the AISC Steel Manual, the Available Strength in Axial Compression Table, Table 4-1, was used for the remaining floor levels as it is based upon the same method. Upon completion of these calculations, it was concluded that the capacity of the structure was adequate for the dead and live load combinations applied.

Appendix A.- Calculations

Wind Loading

Figure A-1" Calculated Wind Pressures in North/South Direction

Distribution of Windward and Leeward Pressures								
			Wind Pressures (psf)					
Level	Height Above Ground (ft)	q (psf)	N-S windward	N-S leeward	N-S side wall	E-W windward	E-W leeward	E-W side wall
Penthouse	134	20.75	23.10	-7.29	-20.18	23.36	-9.36	-20.41
T.O. Parapet	120	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
Roof	118	20.16	22.55	-7.29	-20.18	22.81	-9.36	-20.41
8	104	19.39	21.82	-7.29	-20.18	22.07	-9.36	-20.41
7	91	18.61	21.09	-7.29	-20.18	21.33	-9.36	-20.41
6	78	17.84	20.37	-7.29	-20.18	20.60	-9.36	-20.41
5	64	16.87	19.46	-7.29	-20.18	19.67	-9.36	-20.41
4	50	15.70	18.37	-7.29	-20.18	18.57	-9.36	-20.41
3	36	14.35	17.09	-7.29	-20.18	17.28	-9.36	-20.41
2	19	11.83	14.73	-7.29	-20.18	14.88	-9.36	-20.41
Ground	0	11.05	14.00	-7.29	-20.18	14.14	-9.36	-20.41

Figures A2 & A3: Coefficients used to calculate Wind Loading and Gust Effect Factor Respectively

Design Category	III
V (mph)	90
K_d	0.85
Importance Factor (I)	1.1
Exposure Category	B (urban areas)
K_{zt}=	1
n_1=	0.75
Gf	1.173 (N-S)
	1.189 (E-W)
q_p	20.16
GC_{pn}	+1.5 windward
	-1.0 leeward
P_p	21.56 windward
	19.16 leeward
GC_{pi}	n/a
z_g=	1200 ft
α=	7

Cp Value	N-S	E-W
Windward wall	0.8	0.8
Leeward Wall	-0.155	-0.239
Side Wall	-0.7	-0.7

Gust Effect Factors

	N-S	E-W
B (ft)	260	80.5
L (ft)	80.5	260
h (ft)	134	134
n_1	0.75	0.75
Structure:	Flexible	Flexible
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.12	4.12
z bar	80.4	80.4
ε bar	0.33	0.33
L bar	320	320
b bar	0.45	0.45
α bar	0.25	0.25
Iz bar	0.259	0.259
Lz bar	430.6	430.6
Q	0.792	0.843
Vz bar	74.21	74.21
N_1	4.352	4.352
n_h	6.23	6.23
n_b	12.087	3.742
n_l	12.529	40.466
R_h	0.148	0.148
R_b	0.079	0.232
R_L	0.077	0.024
R_n	0.055	0.055
R	0.06	0.101
G_f	1.173	1.189

Figure A-4: Kz and q_z Factors

	Level	Height Above Ground (ft)	Floor Height (ft)	K_z	q_z
windward	Penthouse	134	14	1.07	20.75
	T.O. Parapet	120	0.25	1.04	20.16
	Roof	118	1.7	1.04	20.16
	8	104	14	1	19.39
	7	91	13	0.96	18.61
	6	78	13	0.92	17.84
	5	64	14	0.87	16.87
	4	50	14	0.81	15.70
	3	36	14	0.74	14.35
	2	19	17	0.61	11.83
	Ground	0	19	0.57	11.05
Leeward	All	All	All	1.04	20.16

Figure A-5: Wind Story Forces, Shears, and Moments

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
					N-S	E-W	N-S	E-W	N-S	E-W
Pent house	134	14	14	0.125	71	21	71	21	9580	2783
T.O. Parapet	120	0.25	0.125	0.9	5	1	77	22	605	176
Roof	118	1.7	0.9	7.0	39	11	115	33	4557	1324
8	104	14	7	6.5	64	19	179	52	6641	1930
7	91	13	6.5	6.5	59	17	238	69	5372	1561
6	78	13	6.5	7	59	17	297	86	4583	1331
5	64	14	7	7	58	17	354	103	3687	1071
4	50	14	7	7	54	16	408	119	2682	779
3	36	14	7	8.5	54	16	462	134	1953	568
2	19	17	8.5	9.5	52	15	514	149	987	287
Ground	0	19	9.5	7	44	13	559	162	0	0

FIND VELOCITY PRESSURES, q_z AND q_h:

DETERMINE BASIC WIND SPEED V FROM FIG. 6-1

$V = 90$ mph

DETERMINE WIND DIRECTIONALITY FACTOR K_d FROM TABLE 6-4 (ASCE 7-05)

$K_d = 0.85$

DETERMINE IMPORTANCE FACTOR I FROM TABLE 6-1 (ASCE 7-05)

CATEGORY III, $I = 1.1$

DETERMINE EXPOSURE CATEGORY FROM § 6.5.6 (ASCE 7-05)

CATEGORY B, URBAN AREA

ARE ALL 5 CONDITIONS OF §6.5.7.1 MET? NO

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

DETERMINE VELOCITY PRESSURE EXPOSURE COEFFICIENTS K_z AND K_h FROM TABLE 6-3 (ASCE 7-05)

$z_g = 1200$ ft
$\alpha = 7.0$
$z = 148$ ft ← NOTE: THIS IS THE MOST CRITICAL BUILDING HT.
EXPOSURE B, CASE 2
* REFER TO WIND ANALYSIS SPREADSHEET
$K_z = 1.07$ @ 134' (TOP OF PENTHOUSE)

DETERMINE VELOCITY PRESSURE AT HEIGHT z AND h

SAMPLE CALCULATION AT HT. = 134 ft (TOP OF PENTHOUSE)

$$K_z = 2.01\left(\frac{134}{1200}\right)^{(2/7)} = 1.07$$

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$
$$= 0.00256(1.07)(1.0)(0.85)(90^2)(1.1)$$
$$= 20.75$$

GUST EFFECT FACTORS, G & G_f :

DETERMINE B, L, and H

$B\,(N-S) = 260\,ft, \quad L\,(N-S) = 80.5\,ft$

$B\,(E-W) = 80.5\,ft, \quad L\,(E-W) = 260\,ft$

$H = 134\,ft$

DETERMINE n_1 & β

$n_1 = 100/H\ (ft)$ AVERAGE VALUE

$= 100/134 = 0.75\ Hz$

$\beta = 1.0$ PER ISO

IS $n_1 > 1\,Hz$? NO

STRUCTURE IS FLEXIBLE

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600\,n_1)} + \frac{0.577}{\sqrt{2\ln(3600\,n_1)}} \quad [EQ.\ 6-9]$$

$$g_R = \sqrt{2\ln(3600 \times 0.75)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.75)}} = 4.120$$

$\bar{z} = 0.6h \geq z_{min}$

$z_{min} = 30\ ft$ [TABLE 6-2 ASCE 7-05]

$\bar{z} = 0.6(134) = 80.4\,ft > 30\,ft \therefore OK$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} \quad [EQ.\ 6-5]$$

$c = 0.30$ [TABLE 6-2 ASCE 7-05]

$$I_{\bar{z}} = 0.30\left(\frac{33}{80.4}\right)^{1/6} = 0.259$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$\ell = 320\ ft, \quad \bar{\epsilon} = 1/3.0$ [TABLE 6-2 ASCE 7-05]

$$L_{\bar{z}} = 320\left(\frac{80.4}{33}\right)^{(1/3)} = 430.6\ ft$$

GUST EFFECT FACTORS, G & Gf CONTINUED:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad [EQ.\ 6\text{-}6]$$

$$Q_{(N-S)} = \sqrt{\frac{1}{1+0.63\left(\frac{260+134}{430.6}\right)^{0.63}}} = 0.792$$

$$Q_{(E-W)} = \sqrt{\frac{1}{1+0.63\left(\frac{80.5+134}{430.6}\right)^{0.63}}} = 0.843$$

DETERMINE BASIC WIND SPEED: $V = 90$ mph [FIG. 6-1 ASCE 7-05]

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \quad [EQ.\ 6\text{-}14]$$

$$\bar{b} = 0.45, \quad \bar{\alpha} = 1/4.0$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{80.4}{33}\right)^{(1/4)} 90\left(\frac{88}{60}\right) = 74.21$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.75)(430.6)}{74.21} = 4.352 \quad [EQ.\ 6\text{-}12]$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.352)}{(1+10.3(4.352))^{5/3}} = 0.055 \quad [EQ.\ 6\text{-}11]$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.75)(134)}{74.21} = 6.230$$

$$R_h = \frac{1}{6.230} - \frac{1}{2(6.230)^2}\left(1-e^{-2(6.230)}\right) = 0.148$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$\eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.6(0.75)(260)/74.21 = 12.087 \quad (N\text{-}S)$$

$$= 4.6(0.75)(80.5)/74.21 = 3.742 \quad (E\text{-}W)$$

$$R_B\ (N\text{-}S) = \frac{1}{12.087} - \frac{1}{2(12.087)^2}\left(1-e^{-(2\times 12.087)}\right) = 0.079 \quad (N\text{-}S)$$

$$R_B\ (E\text{-}W) = \frac{1}{3.742} - \frac{1}{2(3.742)^2}\left(1-e^{-(2\times 3.742)}\right) = 0.232 \quad (E\text{-}W)$$

GUST EFFECT FACTORS, G & G_F CONTINUED:

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \text{ for } \eta > 0$$

$$\eta = 15.4 n_1 L / \bar{V}_z = 15.4(0.75)(80.5)/74.21 = 12.529 \quad (N-S)$$

$$= 15.4(0.75)(260)/74.21 = 40.466 \quad (E-W)$$

$$R_L (N-S) = \frac{1}{12.529} - \frac{1}{2(12.529)^2}\left(1 - e^{-(2 \times 12.529)}\right) = 0.077 \quad (N-S)$$

$$R_L (E-W) = \frac{1}{40.466} - \frac{1}{2(40.466)^2}\left(1 - e^{-(2 \times 40.466)}\right) = 0.024 \quad (E-W)$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad \text{[EQ. 6-10]}$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.079)(0.53 + 0.47(0.077))} = 0.060 \quad (N-S)$$

$$= \sqrt{\frac{1}{1.0}(0.55)(0.148)(0.232)(0.53 + 0.47(0.024))} = 0.101 \quad (E-W)$$

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] \quad \text{[EQ. 6-8]}$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.792)^2 + (4.120)^2(0.060)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.173 \quad (N-S)$$

$$= 0.925\left[\frac{1 + 1.7(0.259)\sqrt{(3.4)^2(0.843)^2 + (4.120)^2(0.101)^2}}{1 + 1.7(3.4)(0.259)}\right] = 1.189 \quad (E-W)$$

BUILDINGS, MAIN WIND-FORCE RESISTING SYSTEMS

IS THE BUILDING ENCLOSED OR PARTIALLY ENCLOSED? YES

DOES THE BUILDING HAVE A PARAPET? YES

VELOCITY PRESSURE q_p = 20.16 mph

DETERMINE COMBINED NET PRESSURE COEFFICIENT GC_{pn}

$GC_{pn} = +1.5$ WINDWARD

$GC_{pn} = -1.0$ LEEWARD

DETERMINE COMBINED NET DESIGN PRESSURE ON THE PARAPET

$P_p = q_p GC_{pn}$ [EQ. 6-20]

$= (20.16) + 1.5 = 21.56$ (WINDWARD)

$= (20.16) - 1.0 = 19.16$ (LEEWARD)

IS THE BUILDING A LOW-RISE BUILDING AS DEFINED IN 6.2? NO

IS THE BUILDING RIGID? NO

DETERMINE VELOCITY PRESSURE q_z FOR WINDWARD WALLS ALONG THE HT. OF THE BUILDING AND q_h FOR LEEWARD WALLS, SIDE WALLS, & ROOF (SEE SPREADSHEETS)

DETERMINE PRESSURE COEFFICIENTS C_p FOR THE ROOF FROM FIG 6-6

$\frac{L}{B} = \frac{80.5}{260} = 0.310$ (N-S)

$\frac{L}{B} = \frac{260}{80.5} = 3.230$ (E-W)

	C_p VALUE	
	N-S	E-W
WINDWARD WALL	0.8	0.8
LEEWARD WALL	-0.155	-0.239
SIDE WALL	-0.7	-0.7

WINDWARD WALLS: $P_z = q_z G_f C_p$ ← SAMPLE CALCULATION (SEE SPREADSHEET)

(N-S) $P_z = (20.16)(1.173)(0.8)$

C_P VALUES	N-S	E-W
WINDWARD	0.8	0.8
LEEWARD	-0.155	-0.239
SIDE WALL	-0.700	-0.700

G_f (N-S) = 1.173

G_f (E-W) = 1.189

NOT INCLUDING UPLIFT ON ROOF SINCE ROOF FRAMING MADE UP OF W-SHAPES

$q_i = q_h = q_z$ FOR TOP OF BUILDING = 20.16 psf

INTERNAL PRESSURE COEFFICIENT : $GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES — $P_z + P_h$ (EQ 6-17)

WINDWARD WALLS: (psf)

$$P_z = q_z GC_p - q_h (GC_{pi})$$
$$= (1.173)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9384 q_z \pm 3.6288 \quad [N-S]$$

$$P_z = (1.189)(0.8) q_z \pm 20.16 (0.18)$$
$$= 0.9512 q_z \pm 3.6288 \quad [E-W]$$

LEEWARD WALLS & SIDE WALLS: (psf)

$$P_z = q_h GC_p - q_h (GC_{pi})$$
$$= (20.16)(1.173) C_p \pm 20.16 (0.18)$$
$$= 23.6477 C_p \pm 3.6288 \quad [N-S]$$

$$P_z = (20.16)(1.189) C_p \pm 20.16 (0.18)$$
$$= 23.9702 C_p \pm 3.6288 \quad [E-W]$$

Seismic

Figure A-6: Coefficients used for Seismic Analysis per ASCE 7-05

Seismic Analysis Coefficients	
Ss=	0.37
S1=	0.07
Occupancy Category=	III
Site Class=	C (very dense soil and soft rock)
Fa=	1.2
Fv=	1.7
Sms=	0.45
Sm1=	0.119
Sds=	0.3
Sd1=	0.079
Ta=	1.182
0.8Ts=	0.211
SDC=	B
Ts=	0.226
R=	7
I=	1.1
Ta=	1.182
Cu=	0.211
TL=	6 sec
Cs=	0.006
Cs=	0.01
k=	1.755
W=	15388 kips
V=	153.88 kips

Figure A-7: Equivalent Lateral Force Procedure

Lateral Seismic Force, Fx							
Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)	Fx (kips)
penthouse	134	80750	0	38245	0	118995	6.76
roof	120	492300	3440	50726	70560	617026	28.87
8	104	403570	15938	37130	61740	518378	18.87
7	91	374170	24463	42135	57330	498098	14.34
6	78	1108370	24463	116396	127335	1376564	30.24
5	64	1201959	16940	169389	144690	1532978	23.80
4	50	1201959	86174	90008.7	144690	1522831.7	15.33
3	36	1201959	76816.5	140824.5	144690	1564290	8.85
2	19	3223770.5	76816.5	220889.5	178755	3700231.5	6.82
1	0	3356119.75	236557.1637	177844	168240	3938760.916	0.00

Figure A-8: Distribution of Shear and Moment on Building

Base Shear and Overturning Moment Distribution							
Level	hx (ft)	Story Weight (k)	hxk Wx	Cvx	Fx=CvxV	Vx (k)	Mx (ft-k)
penthouse	134	119.0	643573	0.044	7	7	906
roof	120	617.0	2749581	0.188	29	36	4276
8	104	518.4	1796967	0.123	19	54	5668
7	91	498.1	1365943	0.093	14	69	6265
6	78	1376.6	2880199	0.197	30	99	7729
5	64	1533.0	2266636	0.155	24	123	7865
4	50	1522.8	1459971	0.100	15	138	6911
3	36	1564.3	842613	0.057	9	147	5294
2	19	3700.2	649294	0.044	7	154	2924
1	0	3938.8	0	0.000	0	154	0
Total	134	15388.2	14654776	1	**154**		**47835**
Base Shear=	154 kips						

SEISMIC GROUND MOTION VALUES & EQUIV. LAT. FORCE PROCEDURE

DETERMINE S_S AND S_1 FROM FIG. 22-1 THROUGH 22-14

$S_1 = 0.07$, $S_S = 0.350$

IS $S_S \leq 0.15$ & $S_1 \leq 0.04$? NO

IS THE STRUCTURE SEISMICALLY ISOLATED OR DOES IT HAVE DAMPING SYSTEMS ON SITE W/ $S_1 \geq 0.6$? NO

DETERMINE THE SITE CLASS IN ACCORDANCE W § 11.4.2 & Ch. 20

↳ SITE CLASS "C"

DETERMINE S_{MS} & S_{M1} by EQN 11.4-1 & 11.4-2

$F_a = 1.2$, $F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.370) = 0.45$

$S_{M1} = F_v S_1 = 1.7(0.07) = 0.119$

DETERMINE S_{DS} & S_{D1} BY EQN 11.4-3 & 11.4-4 RESPECTIVELY:

$S_{DS} = 2 S_{MS}/3 = 2(0.45)/3 = 0.30$

$S_{D1} = 2 S_{M1}/3 = 2(0.119)/3 = 0.079$

DETERMINE OCCUPANCY CATEGORY: III

IS $S_1 \geq 0.75$? NO

IS THE SIMPLIFIED DESIGN PROCEDURE OF 12.14 PERMITTED? NO

ARE ALL 4 CONDITIONS OF 11.6 SATISFIED? NO

$T_a = C_t h_n^x = 0.03(134)^{0.75} = 1.182$ (ECCEN. BRACED STEEL FRAMES)

$0.8T_S = 0.8 \frac{S_{D1}}{S_{DS}} = 0.8\left(\frac{0.079}{0.30}\right) = 0.211$

$T_a \nless 0.8 T_S$ $T = C_u T_a = 1.7(1.182) = 2.009$

DETERMINE SDC AS THE MORE SEVERE OF T. 11.6-1 & T. 11.6-2

SDC = B

DETERMINE R, RESPONSE COEFF. : 7 for truss frames

IMPORTANCE FACTOR : 1.1

DETERMINE T_L FROM FIG 22-15 THROUGH 22-20 : 6 sec.

DETERMINE C_s

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad C_s = \frac{0.079}{(2.009)\left(\frac{7}{1.1}\right)} \leq \frac{0.30}{\left(\frac{7}{1.1}\right)} = 0.006$$

Is $S_1 \geq 0.6$? NO

Is $C_s < 0.01$? YES

$C_s = 0.01$

DETERMINE EFFECTIVE SEISMIC WEIGHT W : 15388 KIPS

DETERMINE BASE SHEAR

$$V = C_s W = 0.01(15388 \text{ kips}) = 153.88 \text{ kips}$$

Is $T \leq 0.5$ sec? NO

Is $T \geq 2.5$ sec? NO

$$k = 0.75 + 0.5T = 0.75 + 0.5(2.009) = 1.755$$

Figure A-9: Building Weight Calculations

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
penthouse	134	80750	0	38245	0	118995
roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtainwall length (ft)	Curtainwall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
cellar level						
Ground						
loading dock	930	150	139500	701	16	168240
first floor level	14838	130	1928940			
podium	600	200	120000			
archive	900	75	67500			
Offices	1948	71	138308			
roof with garden	1330.84	365	485756.6			
library stacks	6705.847	71	476115.153			
second level						
roof with garden	4560	365	1664400	701	17	178755
classrooms	6784	71	481664			
corridors	7601.5	71	539706.5			
auditorium	2800	85	238000			
roof with pavers on 2	2000	150	300000			

Floor	Floor Area (sf)	Floor Dead Load (psf)	Floor Weight	Curtain wall length (ft)	Curtain wall height (ft)	Curtainwall weight (ft) (height*weight* 15 psf)
third level						
classrooms	11424	71	811104	689	14	144690
corridor	5505	71	390855			
fourth level						
offices	5712	71	405552	689	14	144690
classrooms	1200	71	85200			
corridors	10017	71	711207			
fifth level						
offices	7570.5	71	537505.5	689	14	144690
corridors	9358.5	71	664453.5			
sixth level						
offices	3050	71	216550	653	13	127335
corridors	2220	71	157620			
roof	4757.5	90	428175			
roof with drift	325	85	27625			
mechanical	2320	120	278400			
seventh level						
offices	2635	71	187085	294	13	57330
corridors	2635	71	187085			
eighth level						
offices	2335	71	165785	294	14	61740
corridors	2335	71	165785			
mechanical	600	120	72000			
roof level						
roof	4670	90	420300	294	16	70560
mechanical	600	120	72000			
penthouse level						
roof with drift	950	85	80750	248	0	0
		total:	12644927.3			1098030

Snow Loads

Figure A-10: Snow Drift Coefficients

Flat Roof Snow Loads	
Pg=	25 psf
Ce=	1 (Category B)
Ct=	1
I=	1.1
Pf=0.7CeCtIpg	19.25
Warm Roof Snow Loads	
Cs	1 (slope=0deg)
Ps=CsPf	19.25 psf

Snow Drifts	
y=0.13pg+14	17.25 pcf
hc cascade roof=	12.9 ft
hb=pf/y	1.1
hd=	1.6 ft
hc/hb cascade roof=	8.75
wcascade roof=	6.4 ft
DRIFT cascade roof:	Yes
Max Drift Load cascade roof=	46.6 psf

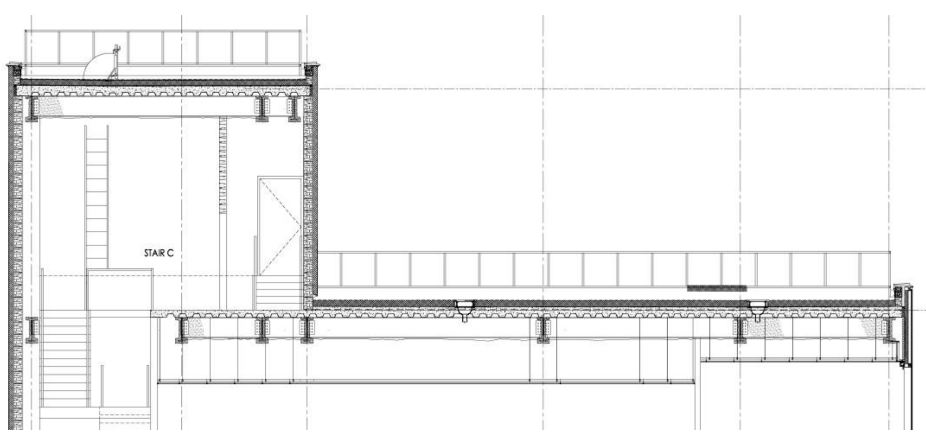


Figure A-11: Cascade Roof at Penthouse Level

Column Spot Check Calculations

Figure A-12: Accumulated Loads on Columns

LOCATION J3 : Accumulated Loads on Columns											
Level	tributary area	dead load (psf)	live load (psf)	influence area	LL red. Factor	live load (k)	dead load (k)	load comb.	load at floor (k)	accum. Load (k)	accum. load (k) by Turner
roof	525	90	45	2100	1.00	23.6	47.3	1.2D+0.5Lr	68.5	68.5	80
Eighth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	161.7	161
seventh	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	255.0	242
sixth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	348.2	337
fifth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	460.4	715
fourth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	572.6	852
third	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	684.8	997
second	675	85	100	3420	0.51	34.2	57.4	1.2D+1.6L	123.6	808.4	1123
Ground	675	130	100	3420	0.51	34.2	87.8	1.2D+1.6L	160.0	968.4	1349

At level 5 there is a large difference between the accumulated loads calculated by that which was provided by Turner Construction Company. This is due to the step- back of the floor levels above. Since the columns located at J1.6 at above levels don't continue to the fifth level, the fifth level is forced to carry the load from the J1.6 column at level 6. Below is a table depicting the adjusted accumulated loads and how they compare to values provided by Turner Construction Company.

Figure A-13: Adjustment of Accumulated Loads on Columns

Level	**accumulated load (k) by Turner for Loc. J1.6**	**LOCATION J3 : Accumulated Loads on Columns**		
		Adjusted accumulated load (k)	accumulated load (k) provided by Turner	percent Error = |adj-prov| /adj*100
roof	n/a	68.5	80	17
eighth	n/a	161.7	161	0
seventh	n/a	255.0	242	5
sixth	266	348.2	337	3
fifth	n/a	726.4	715	2
fourth	n/a	838.6	852	2
third	n/a	950.8	997	5
second	n/a	1074.4	1123	5
Ground	n/a	1234.4	1349	9

COLUMN SPOT CHECK

COLUMN AT ROOF LEVEL, @ J3 : W14×90

$P_u = 68.5^k$ (SEE SPREADSHEET)

$h = 14ft - 4ft = 10ft$ (4 ft: SPLICE HT)

$A_g = 26.5\ in^2$

$I_x = 999\ in^4$

$I_y = 362\ in^4$

$r_x = 6.14$

$r_y = 3.70$

$A_T = (24.96)(21.04) = 525\ ft^2$

$A_I = (49.92)(42.08) = 2100\ ft^2$

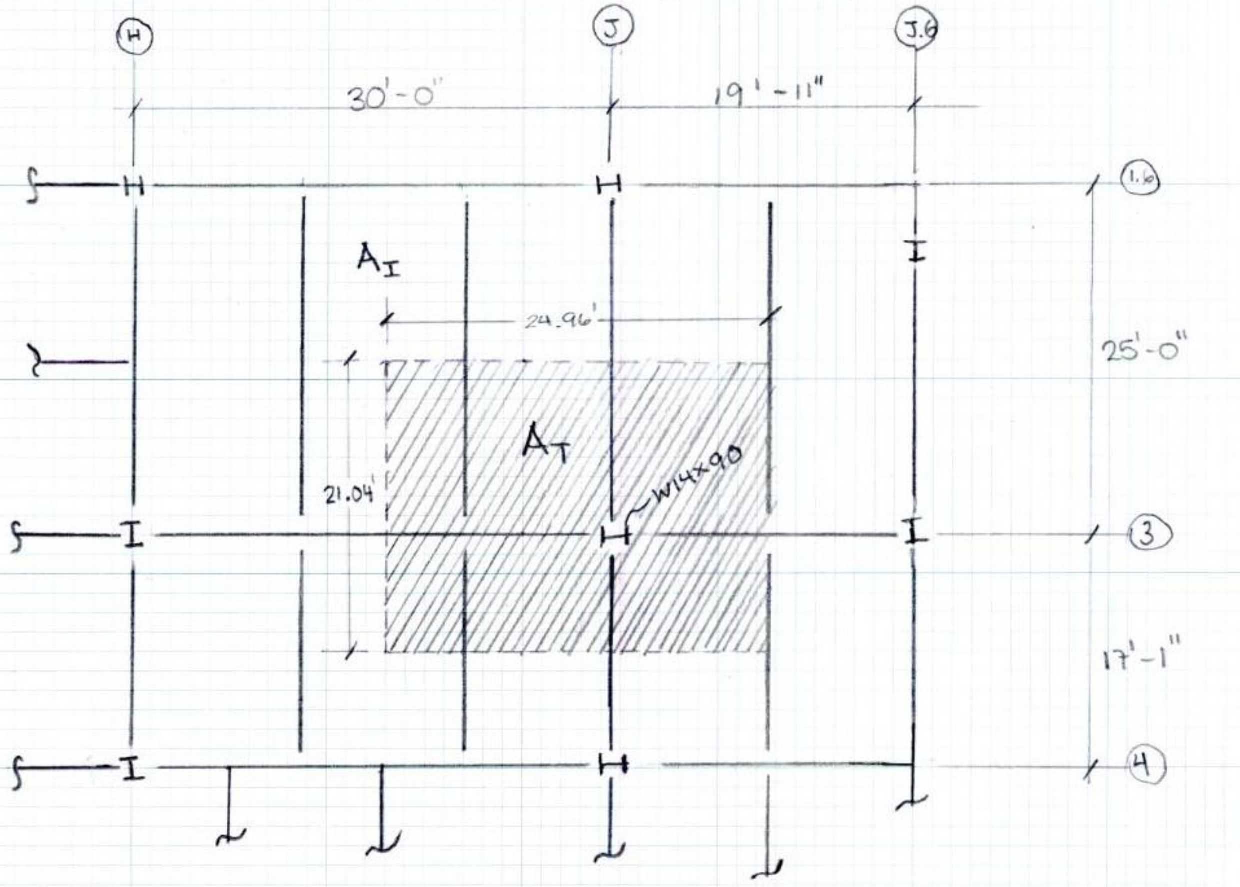
$$\frac{KL}{r_x} = \frac{10(12)}{6.14} = 19.5$$

* CONTROLS

$$\frac{KL}{r_y} = \frac{10(12)}{3.70} = 32.4$$

$$\frac{KL}{r} \le 4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000}{50}} = 113 > 32.4$$

∴ INELASTIC BEHAVIOR



ROOF LEVEL

$$f_{CR} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{50/273}\right](50) = 46.3 \text{ ksi}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29000)}{(32.4)^2} = 273 \text{ ksi}$$

$$\phi P_n = \phi F_{CR} A_g = (46.3)(0.9)(26.5) = 1104^k$$

$$P_u = 68.5^k << \phi P_n = 1104^k$$

CHECK W/ TABLE 4-22:

$\frac{KL}{r} = 32.4$ $\quad \phi F_{CR} = 41.7$ ksi

$F_{CR} = \frac{41.7}{\phi} = \frac{41.7}{0.9} = 46.3$ ✓ METHOD BY HAND CHECKS

CHECK W/TABLE 4-1:

KL = 10' W14×90

$\phi P_n = 1100^k \approx 1104^k$ ✓ METHOD BY HAND CHECKS

*NOTE: TABLE 4-1 SHALL BE USED FOR REMAINING COLUMN CHECKS AS IT IS BASED ON METHOD BY HAND SHOWN ABOVE

COMMENTS: COLUMN SIZES ARE VERY LARGE WHILE CONSIDERING GRAVITY LOADS ALONE, HOWEVER, IT IS PART OF A MOMENT CONNECTION WITH FRAME 7 & TRUSS 9 (see Appendix C for Braced Frames) HENCE IT RECEIVES LARGE INDUCED MOMENTS

LEVEL 8: $P_u = 161.7^k$
W14×90
h = 10 ft = KL
TABLE 4-1: $\phi P_n = 1100^k > P_u = 161.7$ ✓ OK

LEVEL 7: $P_u = 255^k$
W14×233
h = 26 ft = KL
TABLE 4-1: $\phi P_n = 2020^k > P_u = 255$ ✓ OK

LEVEL 6: $P_u = 348.2^k$
W14×233
h = 26 ft = KL
TABLE 4-1: $\phi P_n = 2020^k > 348^k = P_u$ ✓ OK

LEVEL 5: $P_u = 762.4^k$
W14×311
h = 28 ft = KL
TABLE 4-1: $\phi P_n = 2580^k > 762 = P_u$ ✓ OK

LEVEL 4: $P_u = 838.6^k$
W14×311
h = 28 ft = KL
TABLE 4-1: $\phi P_n = 2580^k > 838$ ✓ OK

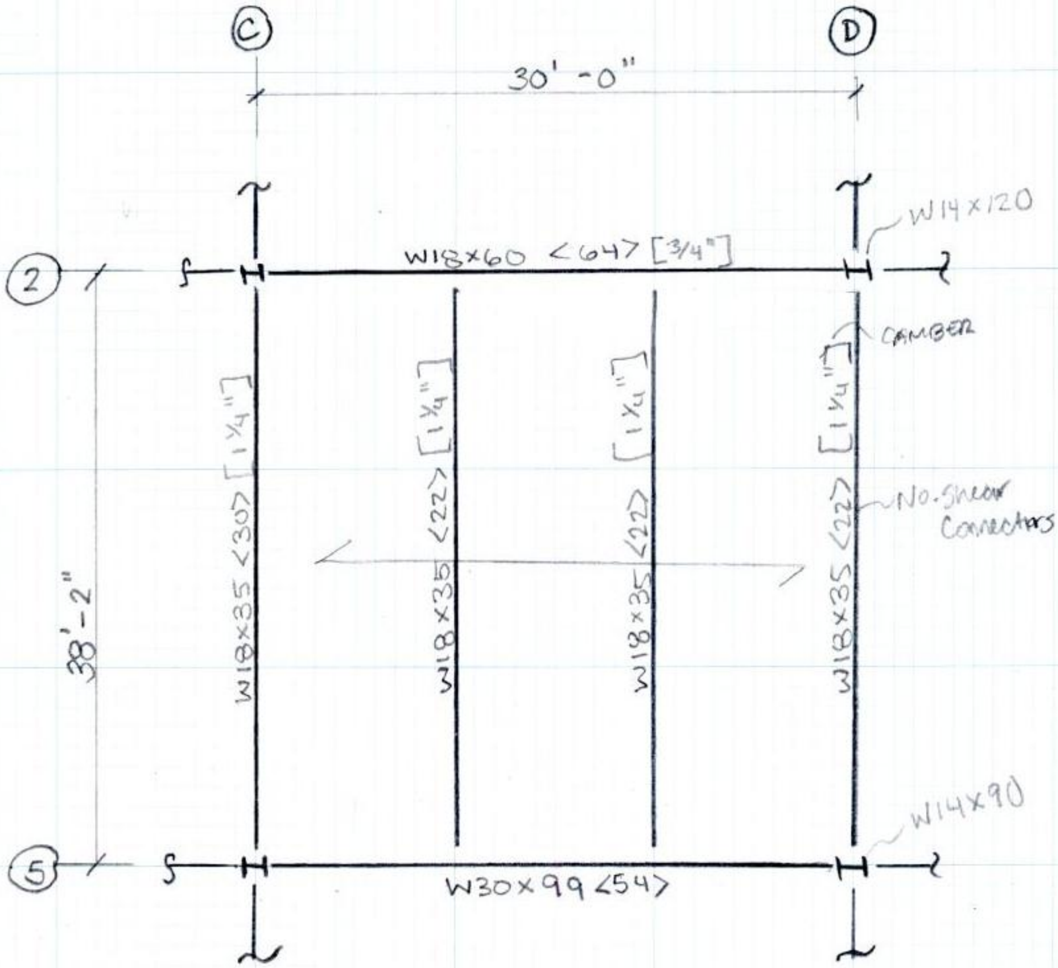
LEVEL 3: $P_u = 950.8^k$
W14×550
h = 31 ft = KL
TABLE 4-1: $\phi P_n = 4350^k > 950.8$ ✓ OK

LEVEL 2: $P_u = 1074.4^k$
W14×550
h = 31 ft = KL
TABLE 4-1: $\phi P_n = 4350^k > 1074$ ✓ OK

GROUND LEVEL: $P_u = 1234.4^k$
W14×730
h = 31 ft = KL
TABLE 4-1: $\phi P_n = 6150^k > 1234$ ✓ OK

Beam Spot Check Calculations

TYPICAL INTERIOR BAY : 2nd FLOOR



LOADS

LIVE LOADS : 40 psf & 100 psf
WILL GO w/ 100 psf ← MORE CRITICAL

DEAD LOADS : 71 psf
(LOADING AREAS 8 & 9)

FLOOR SYSTEM : 3 1/4" LT WT. CONC. OVER 3" - 18 GAGE METAL DECK

LOAD COMBINATION : 1.2D + 1.6L

SPOT CHECK BEAM

FACTORED LOAD : 1.2D + 1.6L

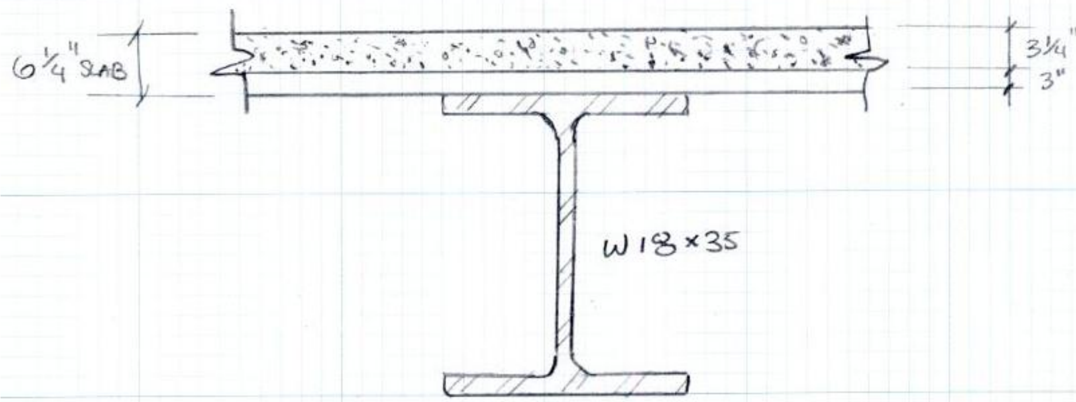
$w_u = 1.2(71.0) + 1.6(100) = 245$ psf

TRIB WIDTH = 10'

$w_u = 245 \text{ psf } (10')/1000 = 2.452$ klf

$$M_u = \frac{w_u l^2}{8} = \frac{(2.452 \text{ klf})(38'-2'')^2}{8} = 446'^k$$

$$b_{eff} = \min \begin{cases} \text{SPACING} = 10' \times 12 = 120'' \\ \frac{\text{SPAN}}{4} = \frac{(38.167')(12 \text{ in/ft})}{4} = 114.5'' \leftarrow \text{CONTROLS} \end{cases}$$



CHECK FOR DEFLECTION UNDER CONSTRUCTION LOADS :

$$\Delta_{CONSTR} = \frac{5 w_{conc} l^4}{384 \Delta_{constr} E}$$

$w_{conc} = 110 \text{ pcf } (3.26''/12) = 29.8$ psf

$w_{conc} = 29.8 \text{psf } (10') = 298 \text{plf} = 0.298$ klf

$$\Delta_{ALLOW} = l/360 = \frac{38.167'(12)}{360} = 1.27''$$

$$I_{req} = \frac{5 w_{conc} l^4}{384 \Delta_{CONSTR} E} = \frac{5(0.298)(38.167)^4(1728)}{384(1.27)(29000)} = 386 \text{ in}^4$$

$I_{W18\times35} = 510 \text{ in}^4 > 386 \text{ in}^4$ ∴ OK

LARGE DIFFERENCE B/C DESIGN IS NOT CONTROLLED BY DEFLECTION UNDER CONST. LOAD.

CHECK BENDING FOR CONSTRUCTION LOADING:

$W_{CONC} = 0.298$ klf

$W_{LIVE} = 20$ psf $(10') = 0.200$ k/f

$W_u = 1.2(0.298) + 1.6(0.200) = 0.678$ klf

$M_u = \frac{W_u l^2}{8} = \frac{0.678(38.167)^2}{8} = 123^{1k}$

$\phi M_{n\,W18\times35} = 249^{1k} > 123^{1k}$ ∴ OK

[COMPARE M_u WITH ϕM_n FOR W18×35 FROM Z_x TABLE B/C SYSTEM NOT COMPOSITE UNTIL CONSTR. IS COMPLETE]

FROM TABLE 3-19:

ASSUME $a = 1''$ ∴ $Y2 = 5.75''$ ROUND TO 5.5"

TRY W18×38 LOCATION: BFL

$\phi M_n = 445^{1k}$, $\Sigma Q_n = 260^k$

$b_{eff} = 114.5''$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{260}{0.85(3.5)(114.5)} = 0.763$

$Y2 = 6.25 - \frac{0.763}{2} = 5.87''$

$\phi M_n = 452^{1k} > 446^{1k} = M_u$ ∴ OK

CHECK NUMBER OF SHEAR STUDS: TABLE 3-21

STUD DIAM. = 3/4"
DECK PERPENDICULAR
LIGHT WT CONCRETE
$f'_c = 3$ ksi CONSERVATIVE
} FIND Q_n

$R_p = 0.6$

$\#STUDS_{REQ'D} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{260}{17.2} \times 2 = 30$ STUDS [1 STUD/RIB]

$\#STUDS_{PROVIDED} = 22$ [STUDS PLACED @ 20" O.C. OVER LENGTH OF 38'-2"]

$\#STUDS_{REQ} < \#STUDS_{PROV}$

∴ THE PNA USED IN THE DESIGN SHOULD HAVE BEEN AT LOCATION 6 INSTEAD OF BFL.

$\#STUD_{REQ'D\,LOC.6} = \frac{194}{17.2} \times 2 = 22.5$ STUDS [1 STUD/RIB]

$\approx \#STUDS_{PROVIDED}$

- MAY HAVE OVER-ESTIMATED THE FACTORED LOADS ON THE BEAM BY 35^{1k}

CHECK DEFLECTION : TABLE 3-20

$Y2 = 5.5" \Rightarrow I_{LB} = 1220 \text{ in}^4$

$$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(1.0)(38.167)^4(1728)}{384(29000)(1220)} = 1.35"$$

$w_u = 100\text{psf}(10')/1000 = 1.0 \text{ klf}$

$$\Delta_{allow} = \frac{l}{360} = \frac{38.167(12)}{360} = 1.27"$$

NOTE : THE BEAM HAS A 1¼" CAMBER

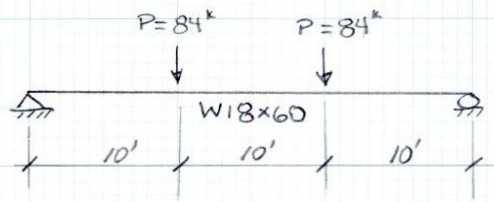
$\Delta = 1.35" - 1.25" = 0.10"$

$\Delta = 0.10" < \Delta_{allow} = 1.27" \quad \therefore \text{OK}$

Girder Spot Check Calculations

GIRDER SPOT CHECK

DL : 50psf
LL : 100psf



$w_{u_{DL}} = 1.2(50)(10')/1000 = 0.600$ klf

$P_{DL} = \frac{w_{u_{DL}}\, l}{2} = \frac{0.600(38.167)}{2} = 11.45^k$

$w_{u_{LL}} = 1.6(100)(10')/1000 = 1.6$ klf

$P_{LL} = \frac{w_{u_{LL}}\, l}{2} = \frac{1.6(38.167)}{2} = 30.53^k$

TOTAL P ON GIRDER = $(11.45 + 30.53) \times 2 = 84^k$ ← BEAMS FRAME IN ON EACH SIDE

$M_{max} = P(a) = 84^k(10') = 840'^k$

ASSUME $Y2 = 5.5''$ ⟶ REQUIRING PNA@TFL (TABLE 3-19)

$\Sigma Q_n = 882^k$

$b_{eff} = \min \begin{cases} \text{spacing} = 38.167' = 458'' \\ \text{span}/4 = \frac{30}{4} = 7.5' = 90'' \end{cases}$ ← CONTROLS

$a = \frac{\Sigma Q_n}{0.85 f'_c\, b_{eff}} = \frac{882}{0.85(3.5)(90)} = 3.29''$

$Y2 = 6.25 - 3.29/2 = 4.6''$ ROUND TO 4.5''

$\phi M_n = 901'^k > 840'^k = M_u$ ∴ OK

CHECK DEFLECTION :

$I_{LB} = 2620\ in^4$ (TABLE 3-20)

$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.298)(30)^4(1728)}{384(29000)(2620)} = 0.07''$

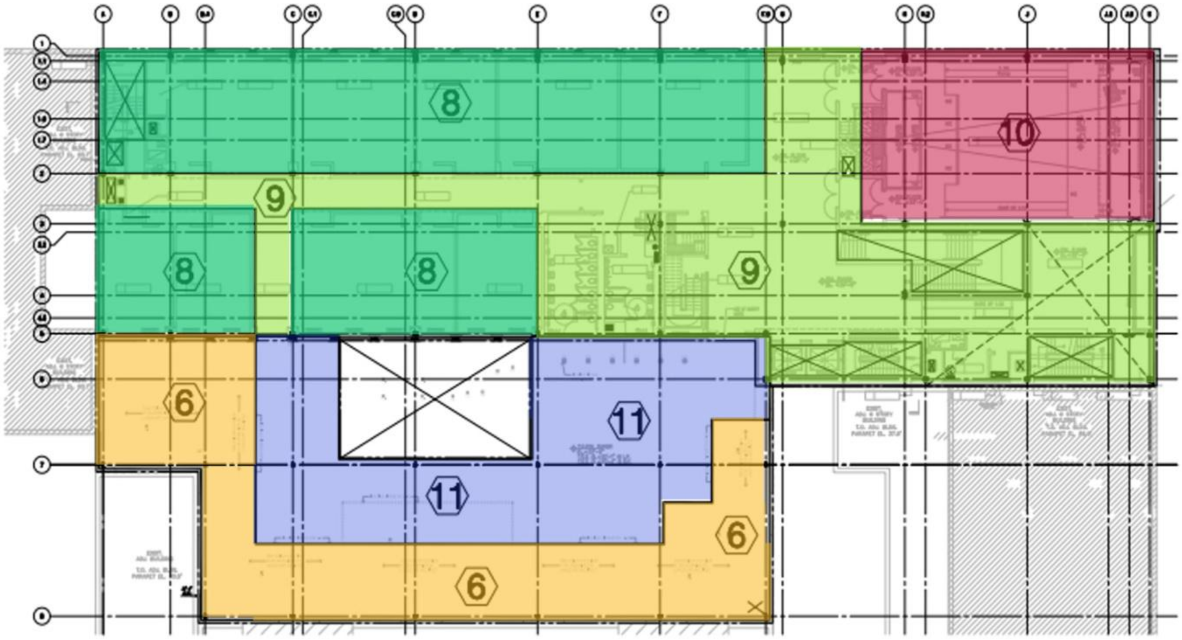
$w_u = 100psf(10')/1000 = 1.0$ klf

$\Delta_{allow} = l/360 = 30(12)/360 = 1'' > \Delta_{actual}$ ∴ OK

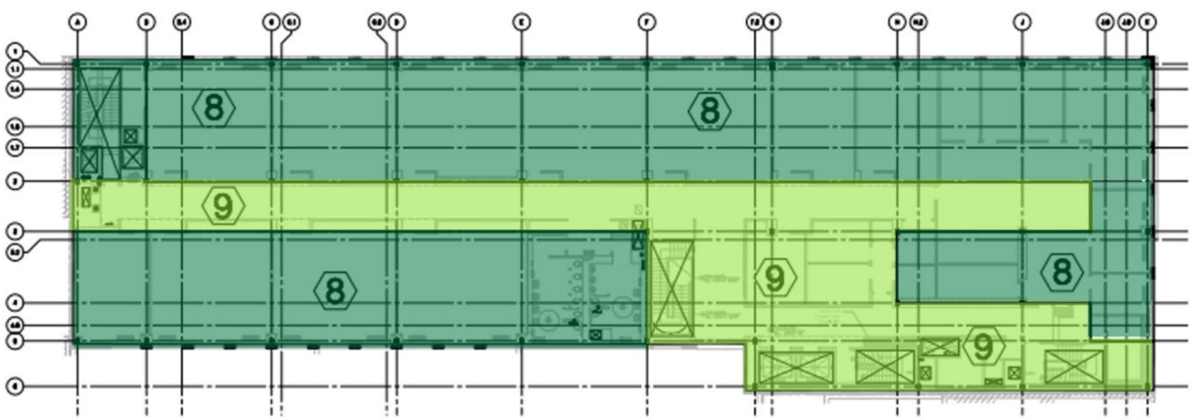
Appendix B. Loading Diagrams

LOADING SCHEDULE		
ID	DL psf	LL psf
1. LOADING DOCK	150.0	600.0
2. 1ST FLOOR	130.0	100.0
3. PODIUM	200.0	100.0
4. ARCHIVE	75.0	350.0
5. OFFICES	71.0	50.0
6. ROOF WITH GARDEN	365.0	100.0
7. LIBRARY STACKS	71.0	100.0
8. CLASSROOMS	71.0	40.0
9. CORRIDOR	71.0	100.0
10. AUDITORIUM	85.0	60.0
11. ROOF WITH PAVERS ON 2	150.0	100.0
12. ROOF	90.0	45.0
13. ROOF WITH DRIFT	85.0	60.0
14. MECHANICAL	120.0	100.0

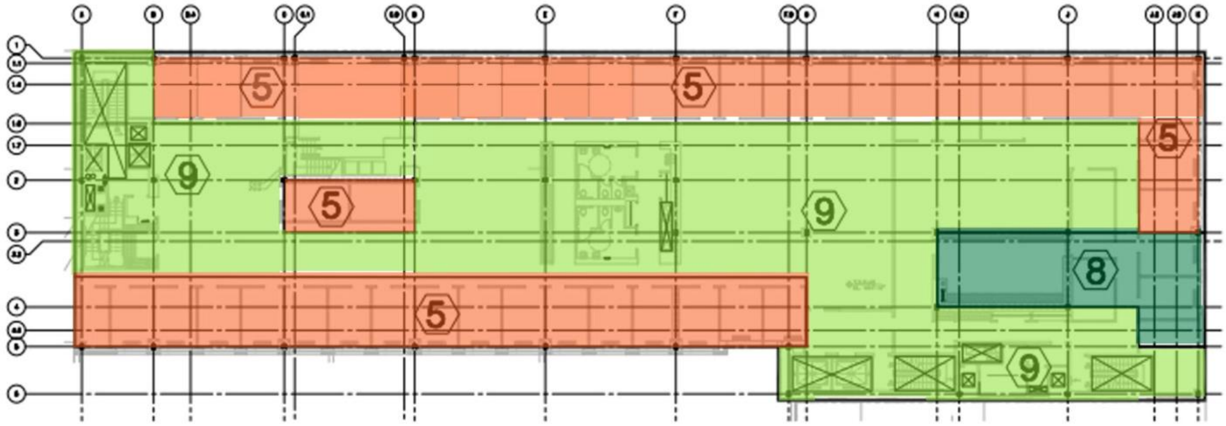
1 1ST FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



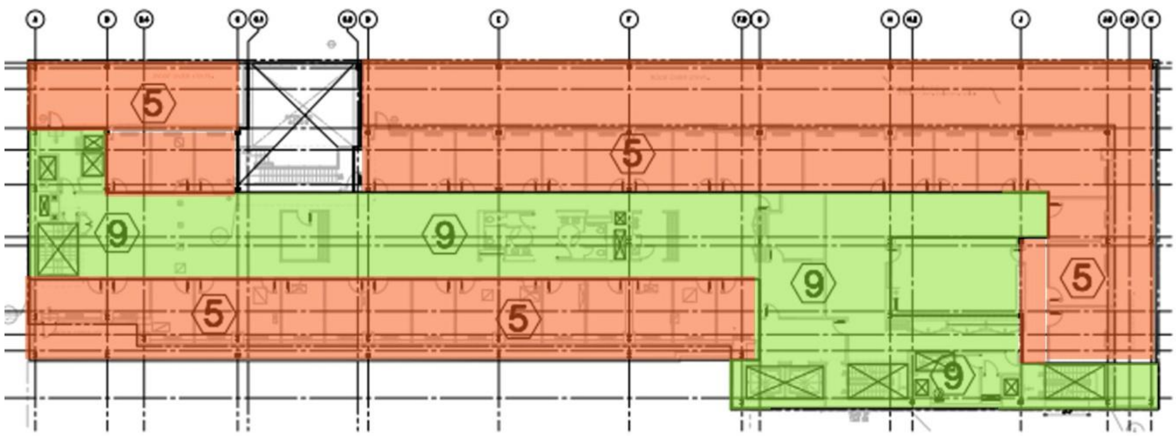
2 2ND FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



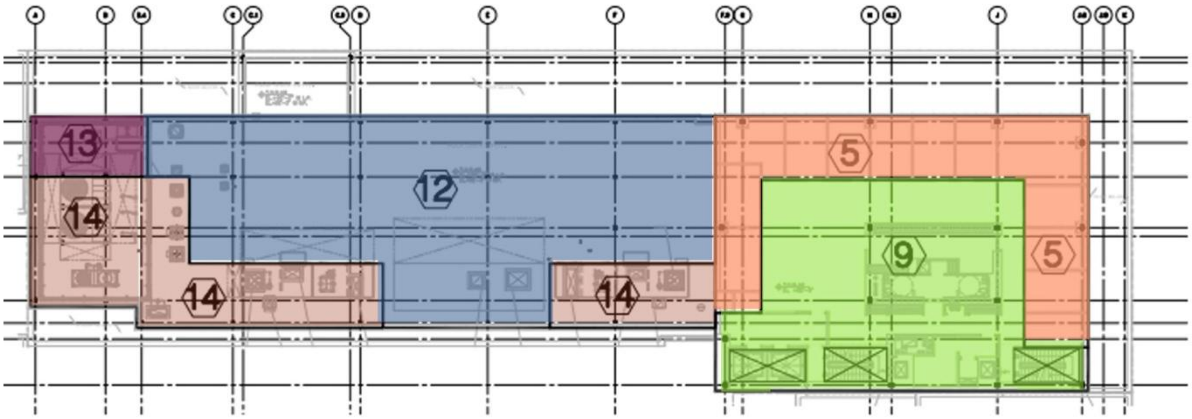
3 3RD FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



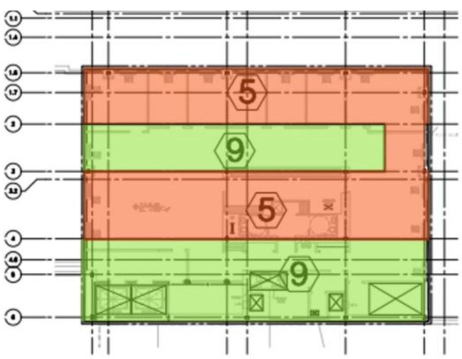
4 4TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



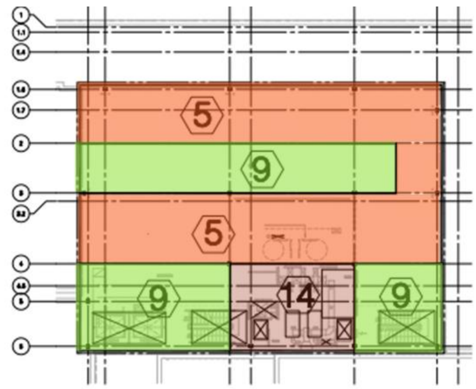
5 5TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



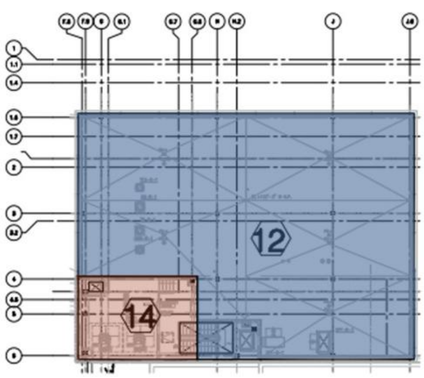
6 S-003 6TH FLOOR LOADING DIAGRAM
SCALE: 1/16"=1'-0"



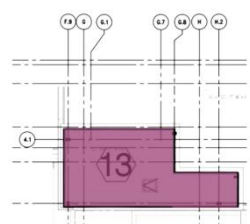
7 S-003 7TH FLOOR LOADING DIAGRAM
SCALE: 1/16"=1'-0"



8 S-003 8TH FLOOR LOADING DIAGRAM
SCALE: 1/16"=1'-0"

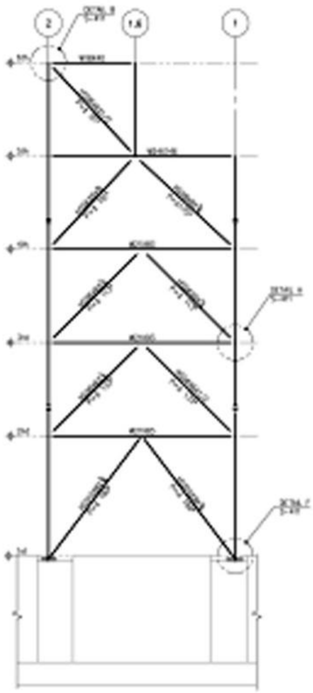


9 S-003 ROOF LOADING DIAGRAM
SCALE: 1/16"=1'-0"

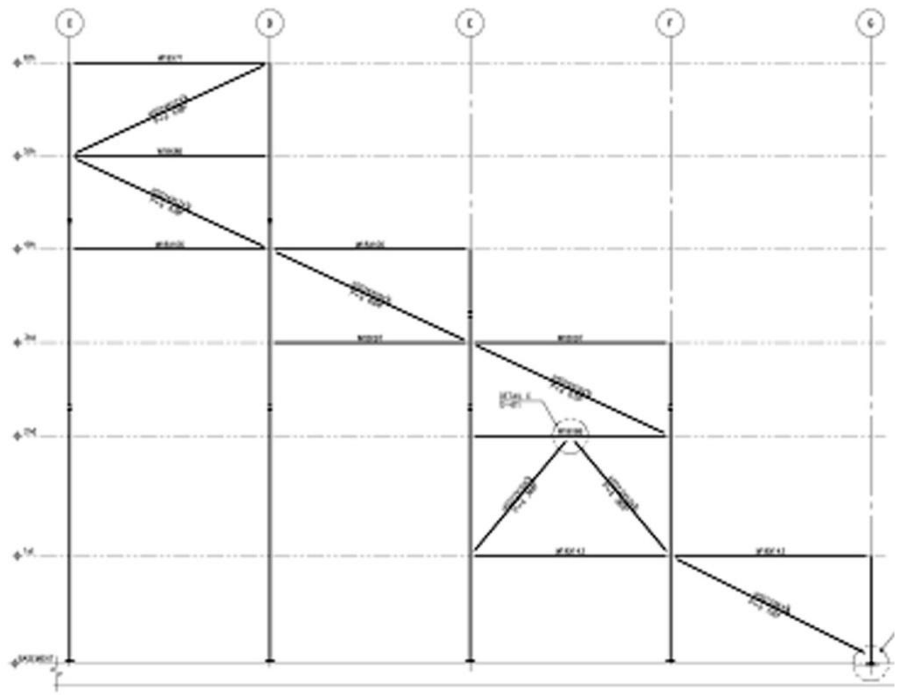


10 S-003 PENTHOUSE LOADING DIAGRAM
SCALE: 1/16"=1'-0"

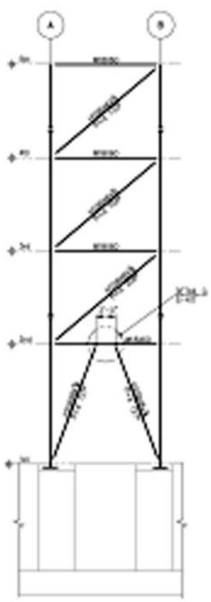
Appendix C. Braced Frames



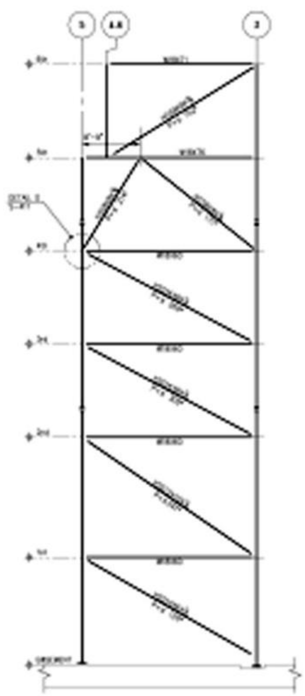
1 TRUSS @ GRID A



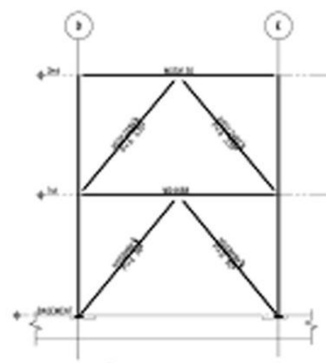
2 TRUSS @ GRID 2



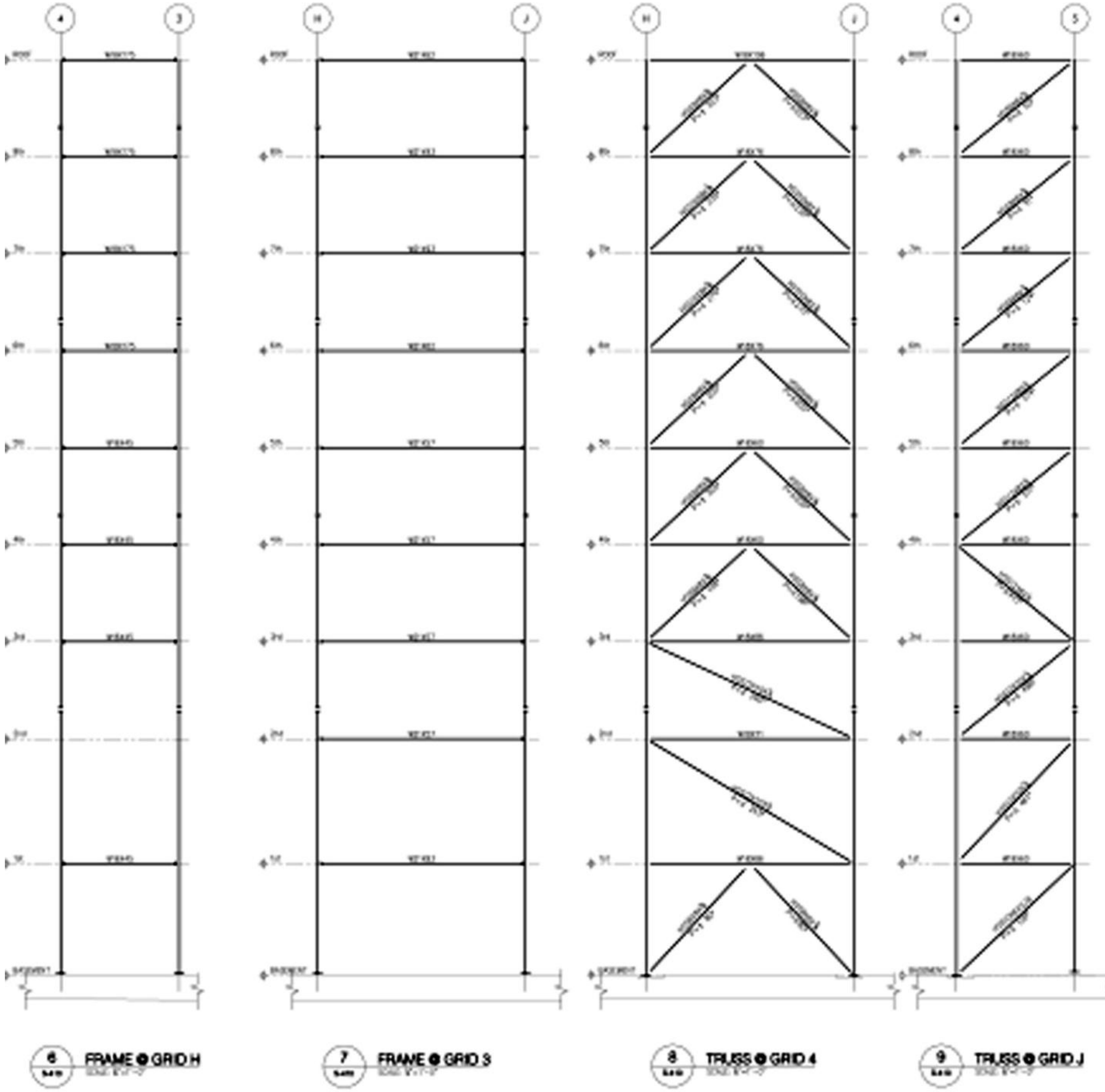
3 TRUSS @ GRID 1



4 TRUSS @ GRID F



5 TRUSS @ GRID B


6 FRAME @ GRID H
7 FRAME @ GRID 3
8 TRUSS @ GRID 4
9 TRUSS @ GRID J