Massachusetts General Hospital - Building for the Third Century

Structural Concepts and Existing Conditions Report

55 Fruit Street
Boston, MA 02114

The Pennsylvania State University
Department of Architectural Engineering
Senior Thesis 2007-2008

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EXECUTIVE SUMMARY:

Purpose

This structural concepts and existing conditions report contains a description of the Massachusetts General Hospital project "The Building for the Third Century" (B3C) including design and loading considerations. This structural analysis of the B3C project provides in depth review of strength of components.

Building Description

The B3C hospital facility contains 530,000 square feet total including: 162,300 square feet of patient bed space, 45,900 square feet of mechanical, and 114,900 square feet of procedural space. The façade of the building is mostly glass. The main structural system consists of a steel moment frame with composite metal deck flooring. The columns transfer load through concrete load bearing elements to bedrock. The systems are being constructed in a manner which allows for fast track construction to ensure that the hospital will become operational in a timely manner.

Structural Analysis Results

The scope of this report includes analysis of the wind and seismic loading of the building to determine which governs the design of the lateral resisting component. These loads were analyzed utilizing ASCE 7-05. It was determined that the building design is controlled by the wind loading resisted in the North and South directions.

Typical elements within the building were also checked including a composite beam and girder. After performing the calculations it was determined that these members have been designed by the engineer to be the most efficient use of the materials employed.

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INTRODUCTION

This structural concepts and existing conditions report contains the description of the existing physical conditions of the Building for the Third Century (B3C) including information pertaining to design concepts and required loading governed by code. An overview of the structural components of the high-rise is included for review of the moment frame system, flooring systems, exterior envelope systems and foundations. Relevant design codes and confirmation through analysis of B3C's strength is also included herein.

BACKGROUND

The B3C project (Cover and Figure 1) is located at 55 Fruit Street in Boston, Massachusetts. The site being built on today once held three outdated hospital buildings. The Clinics, Tilton and Vincent Burnham Kennedy Buildings were demolished in order for this project to move forward. Being located within 1000ft of the Charles River (Figure 2) on the existing hospital campus there are considerations for the higher water table.

The building is being constructed with LEED criteria integrated throughout including several green roofs and a large atrium these spaces create unique design loading in portions of the building.

Being a large bedding facility and emergency facility supply and emergency vehicles require access to the building. Due to the small site these dock areas were incorporated into the plan of the building but the large loads and open spaces call for castellated beams and floors that are hung from tension members rather than attached to columns. The flooring system on the floors that are hung must also be adjusted to allow constructability, so there are concrete planks utilized in parts of the building. All of these systems will be discussed in further detail in this report and reports to follow.

Structural System Discussion

The structural overview section of this report will focus on all of the main structural features of the building. The features to be discussed include: general floor framing, structural slabs, the lateral force resisting system, foundation system, secondary structural systems, the exterior envelope, and expansion joints. An understanding of the interaction of these building components will allow for deeper study of specific components of the system.

General Floor Framing – The main framing type for this building is a steel frame building with beams transferring load to girders and girders to columns. The system is constructed of mostly W shapes whose strengths may be found in Appendix C. Most of the connections in the system are simple or shear connections however the main lateral force resisting system consists of a moment frame, which will be discussed later. Beams commonly have 30ft spans in the building but there are spans of up to 42ft. Floor heights vary, as seen in the Building Height Diagram in Appendix B, between 14ft and 30ft. Column splices commonly occur at 4ft above the floor level of the splice. This framing system necessarily holds up the structural slabs of the building which are discussed next.

Structural Slabs – Four levels of this hospital facility are subterranean on the site and play an interesting part in the construction process of the building. The structural slabs of the basement levels are flat slabs supported by the steel columns of the building and drop panels. The slab thickness is 14 inches in most areas and an additional 8 inches is employed for the drop panel areas of the slab. Material strengths of the concrete and the reinforcement utilized in these structural slabs has been documented in Appendix C. The construction of this hospital is fast tracked, due to its obvious importance, and these structural slabs play an important role in that process which will be talked about in the foundations discussion.

Main Lateral Force Resisting System – As discussed earlier in the general floor framing plan of the building the lateral force resisting system is based on a moment frame. This frame is constructed with moment connections as designated by AISC and the architect. The columns set approximately 10ft inside the perimeter of the slabs, on floors 1-10, makeup the moment frame. This system wraps the building around all sides of the building, as is portrayed in Figure 4. The strengths of this moment frame may also be found on Appendix C. A preliminary analysis of the lateral forces on the building was conducted for both wind and seismic loading. After calculating the lateral forces on the hospital it was determined that the wind loading in the North – South directions would present the largest lateral loads on the building. These calculations and results are discussed eventually in this report. Wind loads are first met by the curtain wall that covers a majority of the building façade. The load is transferred from the glass to the hangers directly into the floor slabs. The metal deck composite floor system aids the lateral force system by distributing the wind forces to the moment frame. The transmission of the lateral load can be seen in Figure 3.

Foundation System – The portion of this building that is buried underground is not to be forgotten. There are several important parts to the foundation system including: a slurry wall, load bearing elements, and caissons. Describing these components in order of construction will be beneficial to help describe the unique construction process being used on this fast track site.

The first element of the foundation system is a 30 inch thick slurry wall. The perimeter of the building as dug down to the bearing bead-rock and then reinforcing steel cages were lowered into the slurry filled holes. Concrete is then pumped into the hole while the slurry is removed. These walls will hold back the soil pressure while building. The holes for the Load Bearing Elements (LBEs) were also excavated to proper depths before any of the dirt was taken from the slurry wall surrounded site. These LBEs support the majority of the structural load of the building. Thus the columns were imbedded into the concrete of the LBE. Those columns reach from the lowest basement level floor to the first floor when they are placed. This column and slurry wall layout allows for "Up – Down" Construction to take place. This construction method calls for a crew to be working under ground to excavate under the floor slabs and the steel crew to be setting steel going up. This process is presented in Figure 6. Caissons also play an important role in the structural support of the building. The caissons carry the load of the massive shielding walls needed for the use of the Linear Accelerators used to create radiation for cancer treatment. All of the materials used in the foundations elements can be found in Appendix C.

Other structural considerations that will need to be made later on are the lateral soil loads that the slurry walls will have to withstand after the lower levels have been excavated. Also the water table is high in this area, due to its proximity to the river, which will necessitate consideration of uplift on the structure.

Secondary Structural Systems – In order to create a more connected atmosphere within the hospital campus bridges are being constructed to a few of the nearby buildings. The Yawlzey and Wang will be the buildings connected to. This requires creating a structure that will not transfer loads from the new building to the older buildings. These bridges are framed with large W shapes, have concrete on metal deck flooring, and glass facades.

There is also a canopy located at the entrance of the building which will need evaluated for wind and snow loads.

Exterior Envelope – The façade of the B3C project is designed to let in maximum amounts of natural light and thus is composed of mostly glass. The curtain wall system is hung from embedded mounts at each floor level. This allows the lateral loads to be transferred directly into the composite floor and eventually to the moment frame serving as the main lateral force resisting system. The curtain wall system also plays an important role in the environmental control in the building but its structural significance is lateral load transmission. Again this transmission is represented in Figure 4. This system is how the building meets the wind, how the building meets other buildings will be discussed next.

Expansion Joints – The building itself does not have any notable expansion joints causing the need for internal load separation but there are plenty of equally important expansion joints between the B3C and other adjacent buildings. Buildings close enough to require expansion joints are Ellison and White. The materials most commonly used in the expansion joints are large rubber gaskets and aluminum plates. These joints are commonly located where a floor, ceiling, or wall meets a similar feature of the joining buildings. The importance of these joints is providing transition from one building to another while not transmitting loads from one building to the other. Space is built into these joints to allow for movement of the buildings as well. It is appropriate to end our discussion of the structural system with a discussion about expansion joints because the B3C project is all about expanding into the third century of the hospital's existence.

Building Design Load Discussion

Wind Loading - Building load discussion will begin with wind loads because this tall building will experience high wind load pressures due to its location on the eastern sea boarder. In order to calculate the numbers in the following tables hand calculations found in Appendix A can be reviewed under the title "Wind Calculations". The major factors affecting the wind loads on the build include: location, façade dimensions, and surrounding area terrain. The façade dimensions are noted in Figure After the design factors were calculated the wind pressures for each direction and floor were determined. As seen by comparing Figures 7 and 8 there are greater pressures on the East and West Facades. These loads are transmitted to the main lateral force resisting system through the glass façade to the floor and finally through moment connections to the columns in the building. The overturning moment on the highest floor is 110,090 ft-kips as noted in the Wind (North South Direction) table. This loading condition must be compared to the loading condition presented by the seismic loads, discussed in the next section, on the building to determine the sizing of the columns in the building.

Velocity Pressures, q_h and q_z

α= 7 z_g= 1200

Floor	Floor Height (ft)	Height z (ft)	K_z	K_{zt}	K_d	K_h	V(mph)	I	q_h (lb/ft^2)	q_z (lb/ft^2)
2nd	12.50	12.50	0.57	1.00	0.85	1.15	105	1.15	31.65	15.73
3rd	12.50	25.00	0.67	1.00	0.85	1.15	105	1.15	31.65	18.35
4th	16.00	41.00	0.77	1.00	0.85	1.15	105	1.15	31.65	21.13
5th	16.00	57.00	0.84	1.00	0.85	1.15	105	1.15	31.65	23.22
6th	30.00	87.00	0.95	1.00	0.85	1.15	105	1.15	31.65	26.20
7th	14.00	101.00	0.99	1.00	0.85	1.15	105	1.15	31.65	27.34
8th	14.00	115.00	1.03	1.00	0.85	1.15	105	1.15	31.65	28.38
9th	14.00	129.00	1.06	1.00	0.85	1.15	105	1.15	31.65	29.32
10th	14.00	143.00	1.09	1.00	0.85	1.15	105	1.15	31.65	30.20
Roof	14.00	157.00	1.12	1.00	0.85	1.15	105	1.15	31.65	31.01
Penthouse	23.00	180.00	1.17	1.00	0.85	1.15	105	1.15	31.65	32.25

$K_z = 2.01(z/z_g)^{2/\alpha}$

K_h is case where z=mean roof height

Mean roof height= 168.50

$q_h=0.00256K_hK_{zt}K_dV_2I$

$q_z=0.00256K_zK_{zt}K_dV_2I$

Wind (North - South Direction)											
B(ft)	L(ft)	G_{N-S}= 0.8173			C_{p-W}= 0.80		C_{p-L}= -0.5				
208.00	192.00										
Floor	Floor Height (ft)	Height z (ft)	K_z	q_z(lb/ft2)	q_h(lb/ft2)	Windward lb/ft^2	Leeward lb/ft^2	Total lb/ft^2	Story Force (kips)	Story Sheer (kips)	Overturning Moment (ft-kips)
Ground	0	0	0	0	0	0	0	0	0.00	1124.87	110090
2nd	12.50	12.50	0.570	15.73	31.65	10.28	-12.93	23.21	60.36	1124.87	110090
3rd	12.50	25.00	0.665	18.35	31.65	12.00	-12.93	24.93	64.82	1064.51	96758
4th	16.00	41.00	0.766	21.13	31.65	13.82	-12.93	26.75	89.02	999.69	82115
5th	16.00	57.00	0.842	23.22	31.65	15.18	-12.93	28.11	93.56	910.67	66738
6th	30.00	87.00	0.950	26.20	31.65	17.13	-12.93	30.06	187.60	817.10	45789
7th	14.00	101.00	0.991	27.34	31.65	17.88	-12.93	30.81	89.72	629.51	30626
8th	14.00	115.00	1.028	28.38	31.65	18.55	-12.93	31.49	91.69	539.79	22441
9th	14.00	129.00	1.063	29.32	31.65	19.17	-12.93	32.10	93.49	448.10	18233
10th	14.00	143.00	1.095	30.20	31.65	19.74	-12.93	32.68	95.16	354.61	9858
Roof	14.00	157.00	1.124	31.01	31.65	20.28	-12.93	33.21	96.71	259.46	5608
Penthouse	23.00	180.00	1.169	32.25	31.65	21.09	-12.93	34.02	162.75	162.75	1896

Wind (East - West Direction)											
B(ft)	L(ft)	G_{E-W} = 0.8195			C_{p-W} = 0.80		C_{p-L} = -0.5				
192	208										
Floor	Floor Height (ft)	Height z (ft)	K_z	q_z(lb/ft2)	q_h(lb/ft2)	Windward lb/ft²	Leeward lb/ft²	Total lb/ft²	Story Force (kips)	Story Sheer (kips)	Overturning Moment (ft-kips)
Ground	0	0	0	0	0	0	0	0	0.00	1041.13	101937
2nd	12.50	12.50	0.570	15.73	31.65	10.31	-12.97	23.28	55.87	1041.13	101937
3rd	12.50	25.00	0.665	18.35	31.65	12.03	-12.97	25.00	59.99	985.27	89560
4th	16.00	41.00	0.766	21.13	31.65	13.85	-12.97	26.82	82.40	925.28	76000
5th	16.00	57.00	0.842	23.22	31.65	15.22	-12.97	28.19	86.60	842.88	61855
6th	30.00	87.00	0.950	26.20	31.65	17.18	-12.97	30.14	173.63	756.28	43768
7th	14.00	101.00	0.991	27.34	31.65	17.93	-12.97	30.89	83.04	582.65	28345
8th	14.00	115.00	1.028	28.38	31.65	18.60	-12.97	31.57	84.86	499.61	20770
9th	14.00	129.00	1.063	29.32	31.65	19.22	-12.97	32.19	86.53	414.75	14369
10th	14.00	143.00	1.095	30.20	31.65	19.80	-12.97	32.77	88.07	328.22	9123
Roof	14.00	157.00	1.124	31.01	31.65	20.33	-12.97	33.30	89.51	240.14	5190
Penthouse	23.00	180.00	1.169	32.25	31.65	21.14	-12.97	34.11	150.63	150.63	1755

Seismic Loading- The building seismic load must also be determined to properly size the main lateral force resisting system. After determining the load factors in the calculations presented in Appendix A the chart on the following page was developed to present the overturning moments of the structure. As can be noted from the table the highest overturning moment is only 60,955 ft –kips compared to the 110,090 ft-kips of the wind loading case. Thus the wind loading will be the controlling factor in the design of the main lateral force resisting system.

Structural Design Discussion

The structural design of the building begins in the foundations of the building and works all the way out to the façade's bearing on the floors of the building however for this report the structural design of a typical beam and girder will be the focus of discussion. These typical members can be seen in Figure 10. The design checks were performed by the methods of LRFD.

Assumptions and calculations are found in Appendix B.

After performing the calculations on both the typical beam and girder it was determined by the methods used that the composite beams were not adequate. The sizes were slightly too small to accommodate the loading. Assumptions may have skewed the results of the calculations but the main factor which would have affected the design strength is the positive camber put into the beams. At this time that factor has not been included in the design of the composite beam but would likely increase the strength of the member.

Future reports will contain checks of the lateral force resisting system in the columns of the building.

Level	Height h_i (ft)	W_i (K)	h_i^k (ft)	$W_x h_x^k$ (ft)	C_{vx}	Lateral Force F_x	Story Shear V_x	Overturning Moment (ft-kips)
Penthouse Roof	180.00	2486.19	4996.28	12421698.39	0.15606	95.62	95.62	1113.97
Roof	157.00	475.05	3992.78	1896766.84	0.02383	14.60	110.22	3000.00
Tenth Floor	143.00	4252.15	3425.71	14566662.66	0.18301	112.13	222.35	5400.00
Nineth Floor	129.00	4252.15	2893.12	12302010.30	0.15456	94.70	317.05	9100.00
Eighth Floor	115.00	4252.15	2396.32	10189512.81	0.12802	78.44	395.48	14091.00
Seventh Floor	101.00	4252.15	1936.81	8235614.59	0.10347	63.40	458.88	20071.00
Sixth Floor	87.00	7352.05	1516.39	11148590.88	0.14007	85.82	544.70	31454.00
Fifth Floor	57.00	6948.77	757.94	5266751.88	0.06617	40.54	585.24	44307.00
Fourth Floor	41.00	5093.77	441.54	2249078.82	0.02826	17.31	602.55	53810.00
Third Floor	25.00	5093.77	196.17	999219.19	0.01255	7.69	610.24	57370.00
Second Floor	12.50	5073.68	62.94	319341.55	0.00401	2.46	612.70	60955.00
First Floor	0.00	6149.53	0.00	0.00	0.00000	0.00	612.70	60955.00
	Total	55681.42	$\Sigma W_i h_i^k$ = 79595247.91					

Document and Code Review

Here is provided a list of the Documents and Codes utilized in analysis and discussion of the structural system.

- ASCE/SEI 7-05 Minimum Design Loads for Buildings and Other Structures published in 2006 by the American Society of Civil Engineers (ASCE 7)
- AISC *Steel Construction Manual* 13th Edition published December 2005 by the American Institute of Steel Construction, Inc. (AISC 13th ed.)
- ACI 318-08 *Building Code Requirements for Structural Concrete* published August 2008 by the American Concrete Institute (ACI 318)
- Construction Documents S100 – S602 Dated February 29 2008
- *Unified Design of Steel Structures* Published 2008 Louis F. Geschwindner

Professional Contacts

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McNamara/Salvia Inc. Consulting Engineers

Figure 1 – Birds Eye View of South East Corner

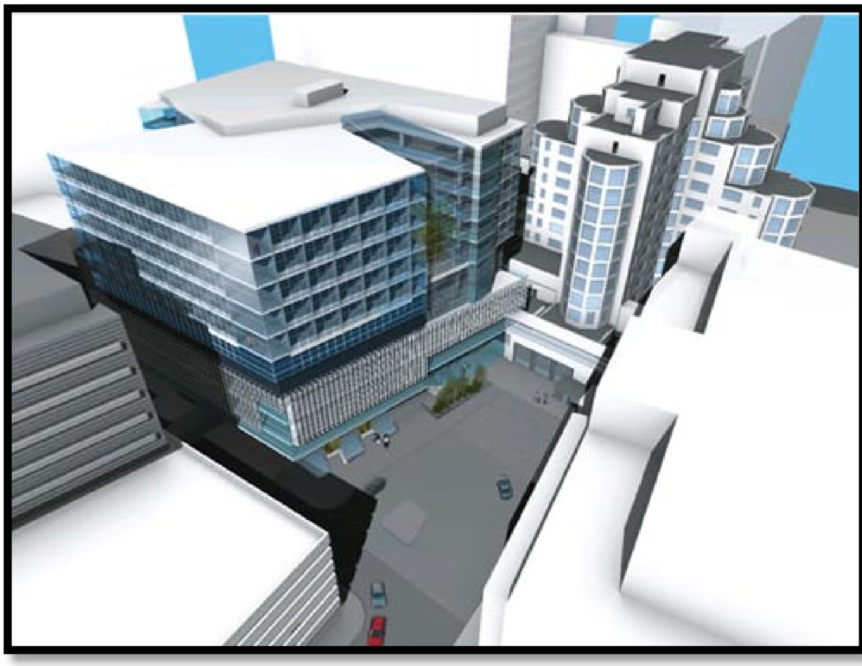


Figure 2 – Site Map

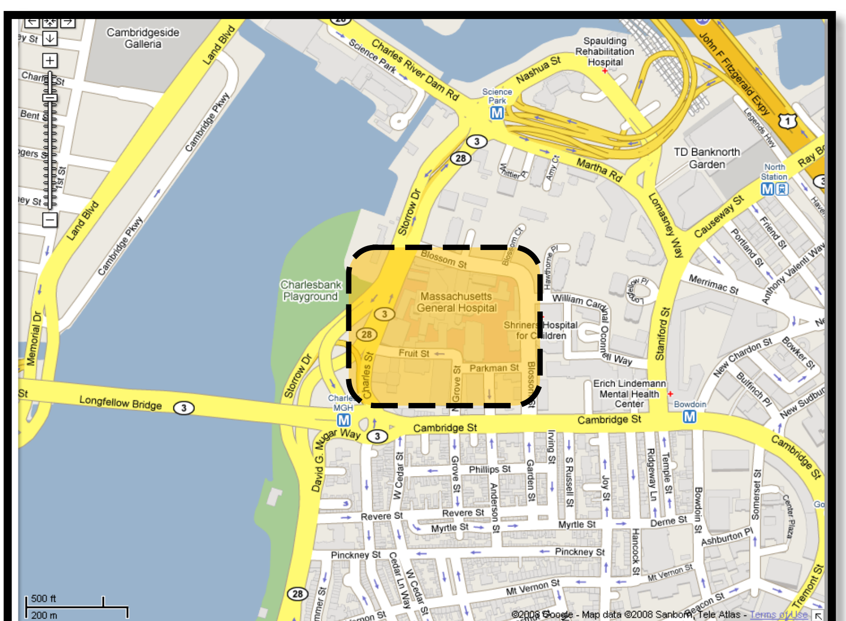


Figure 3: Lateral Load Transmission

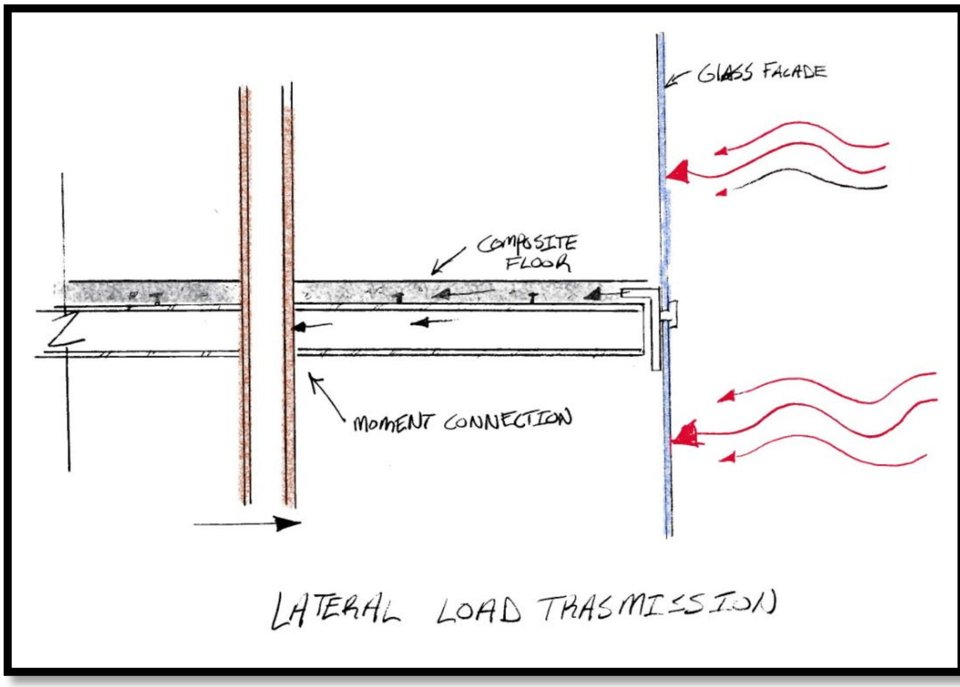


Figure 4: Moment Frame Diagram

Figure 5: Structural Perspective

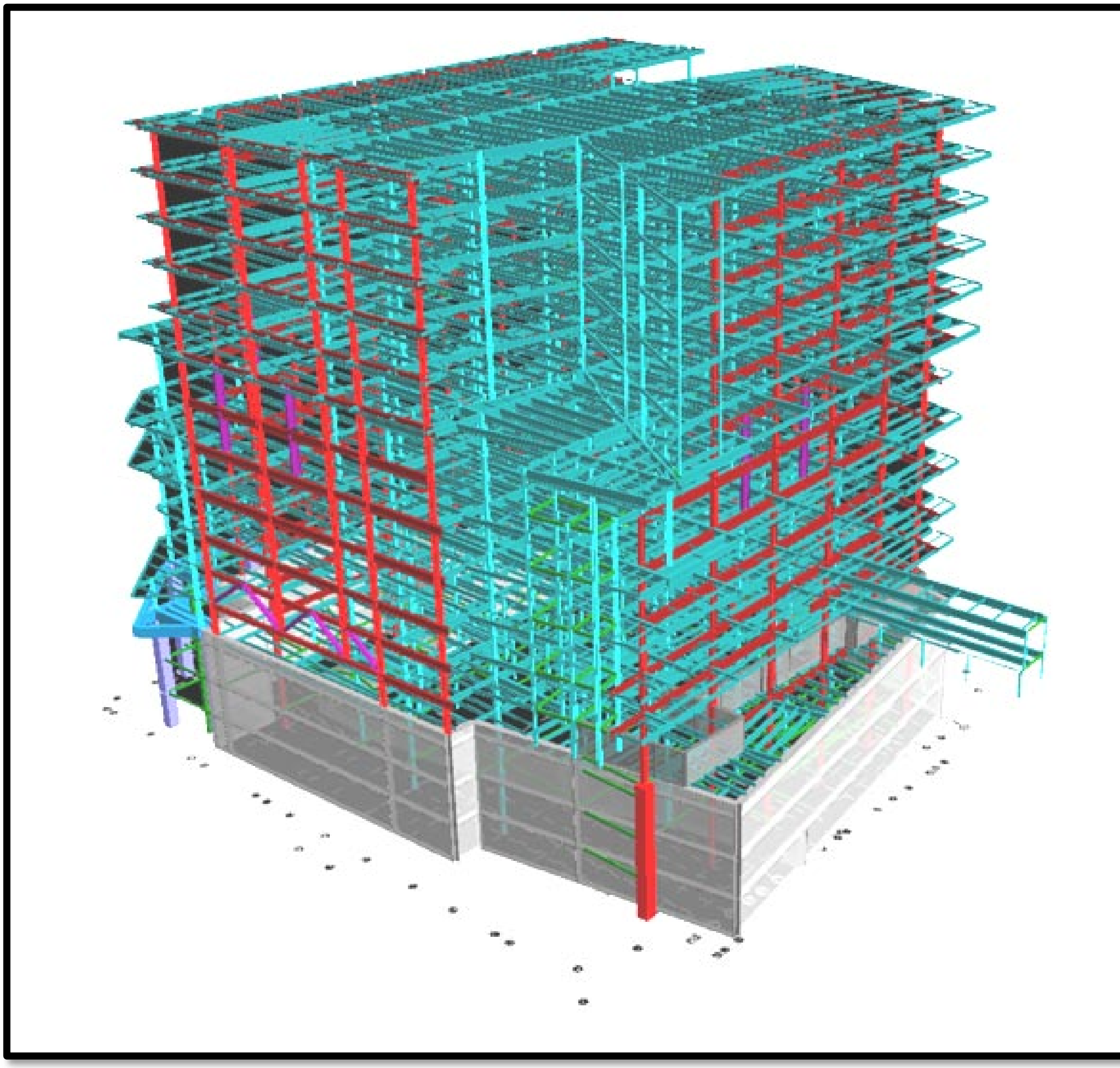


Figure 6: Up Down Construction

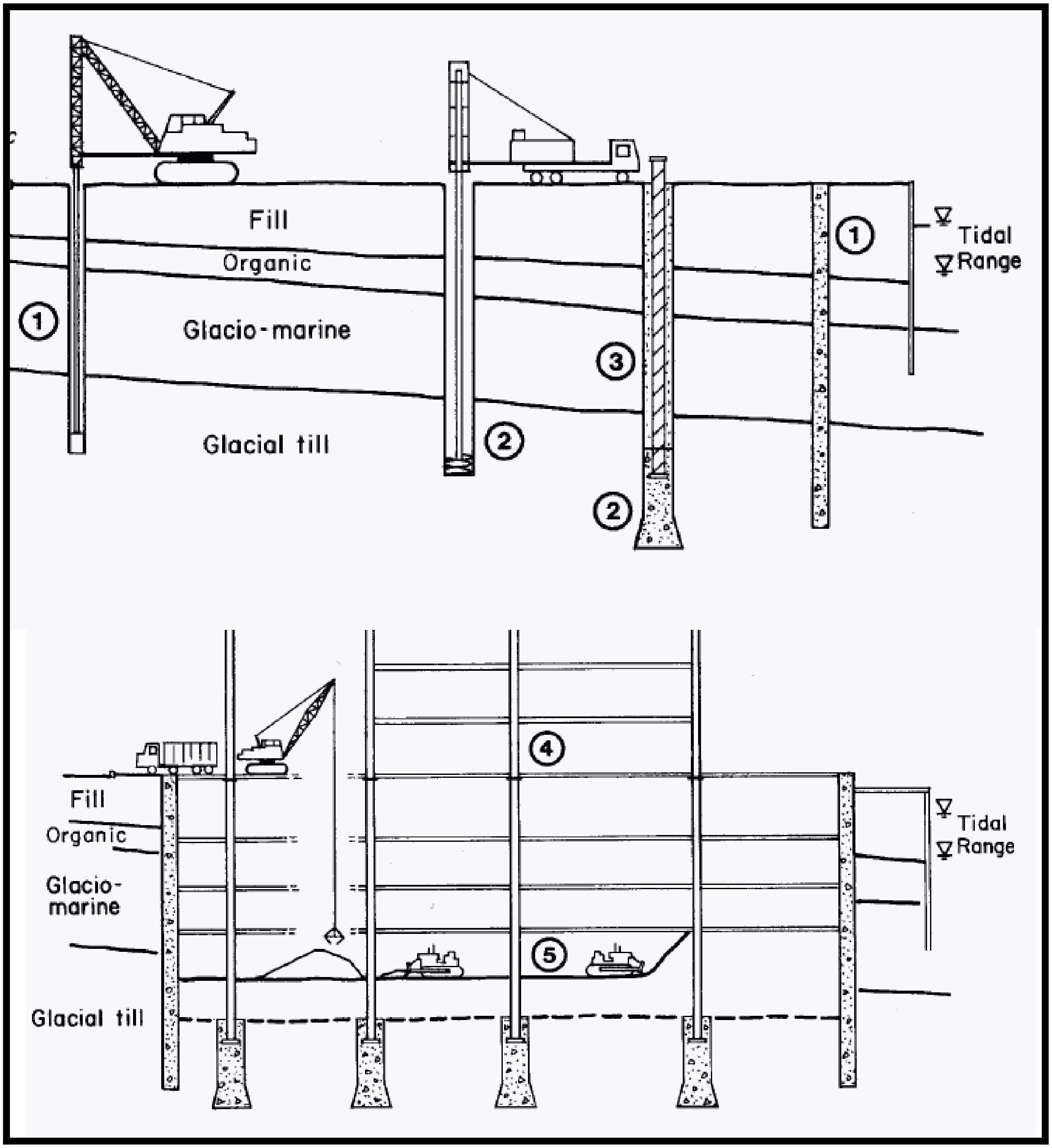


Figure 7: Wind Loading on North and South Faces of Building

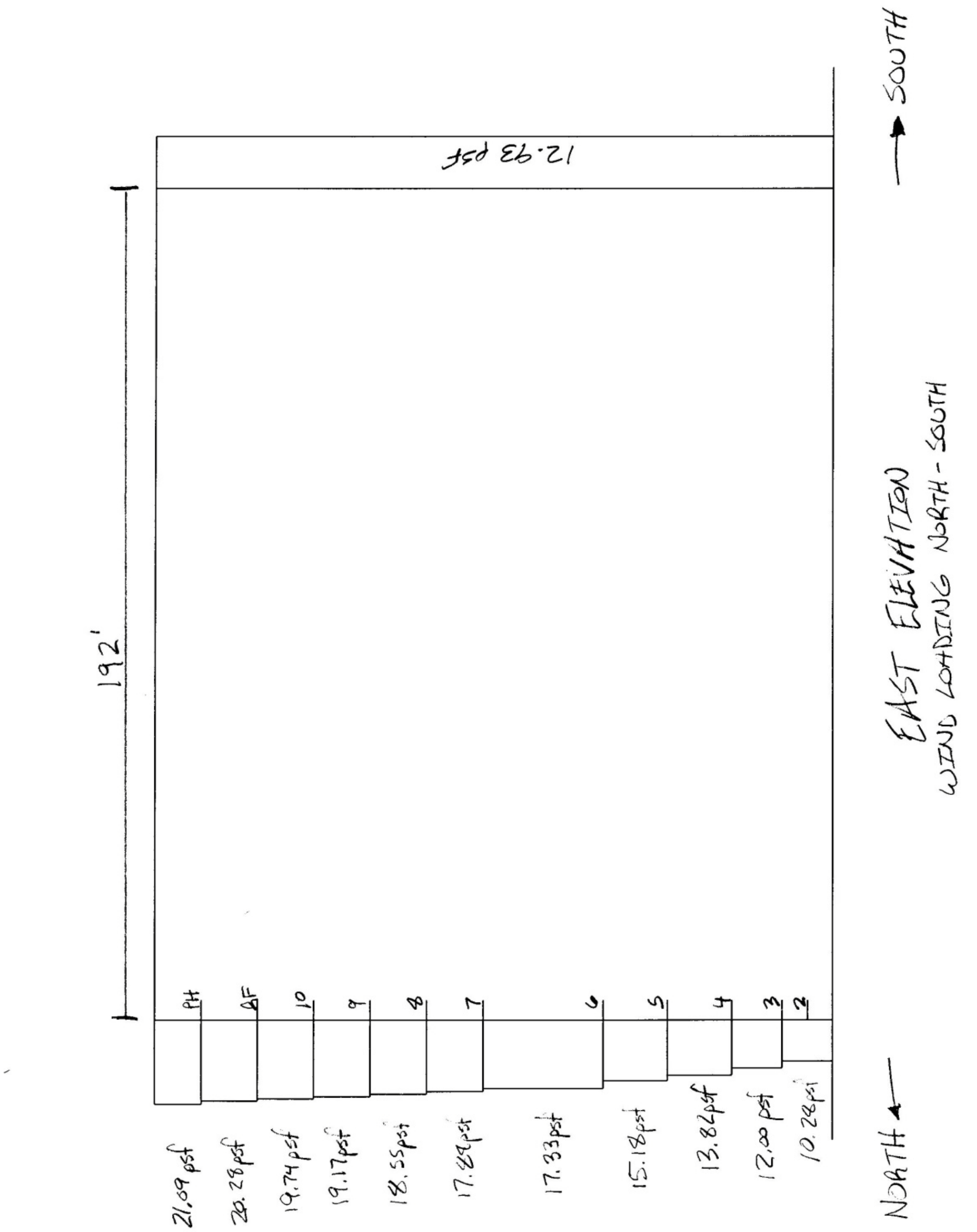


Figure 8: Wind Loading on East and West Faces of Building

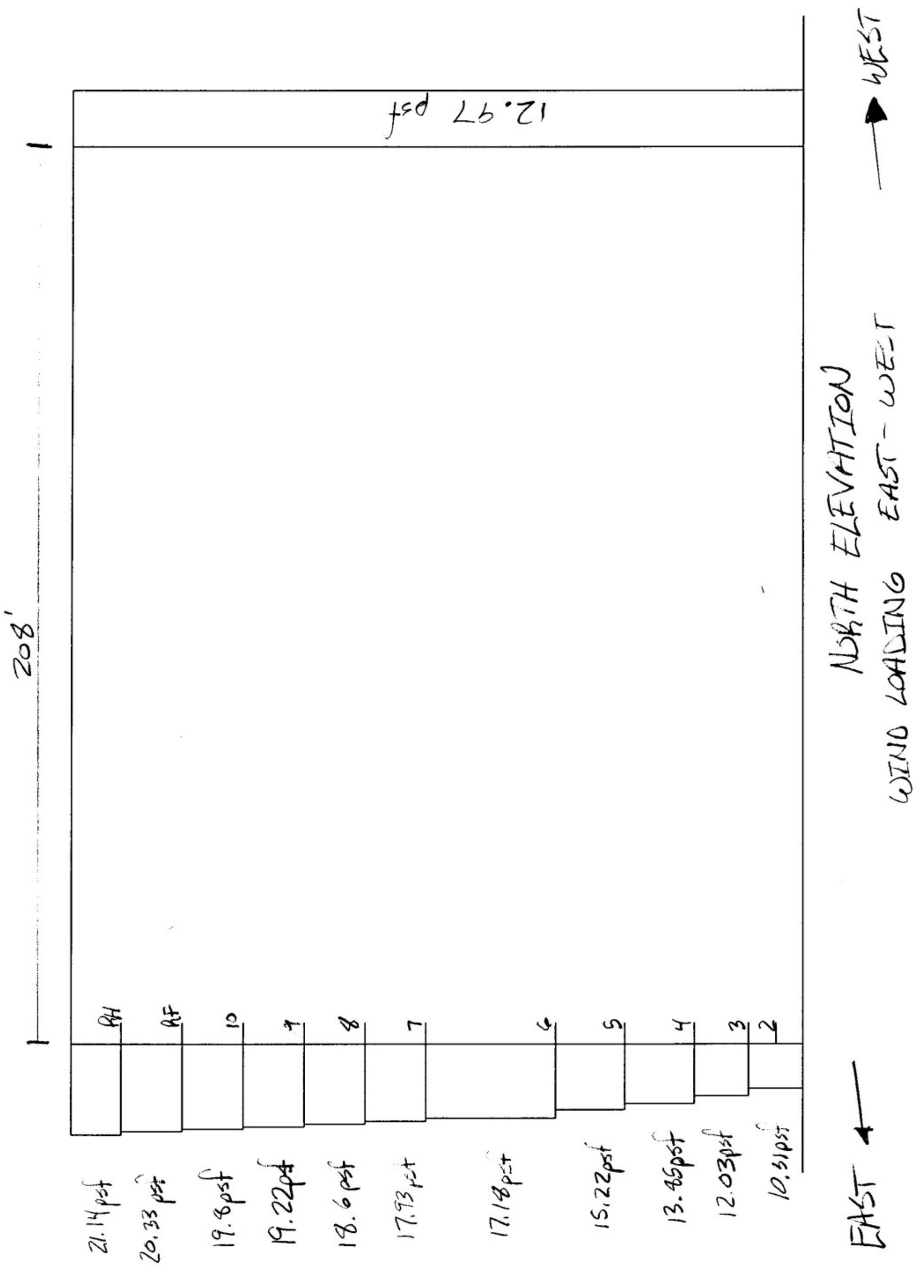


Figure 9: Wind Design Diagram

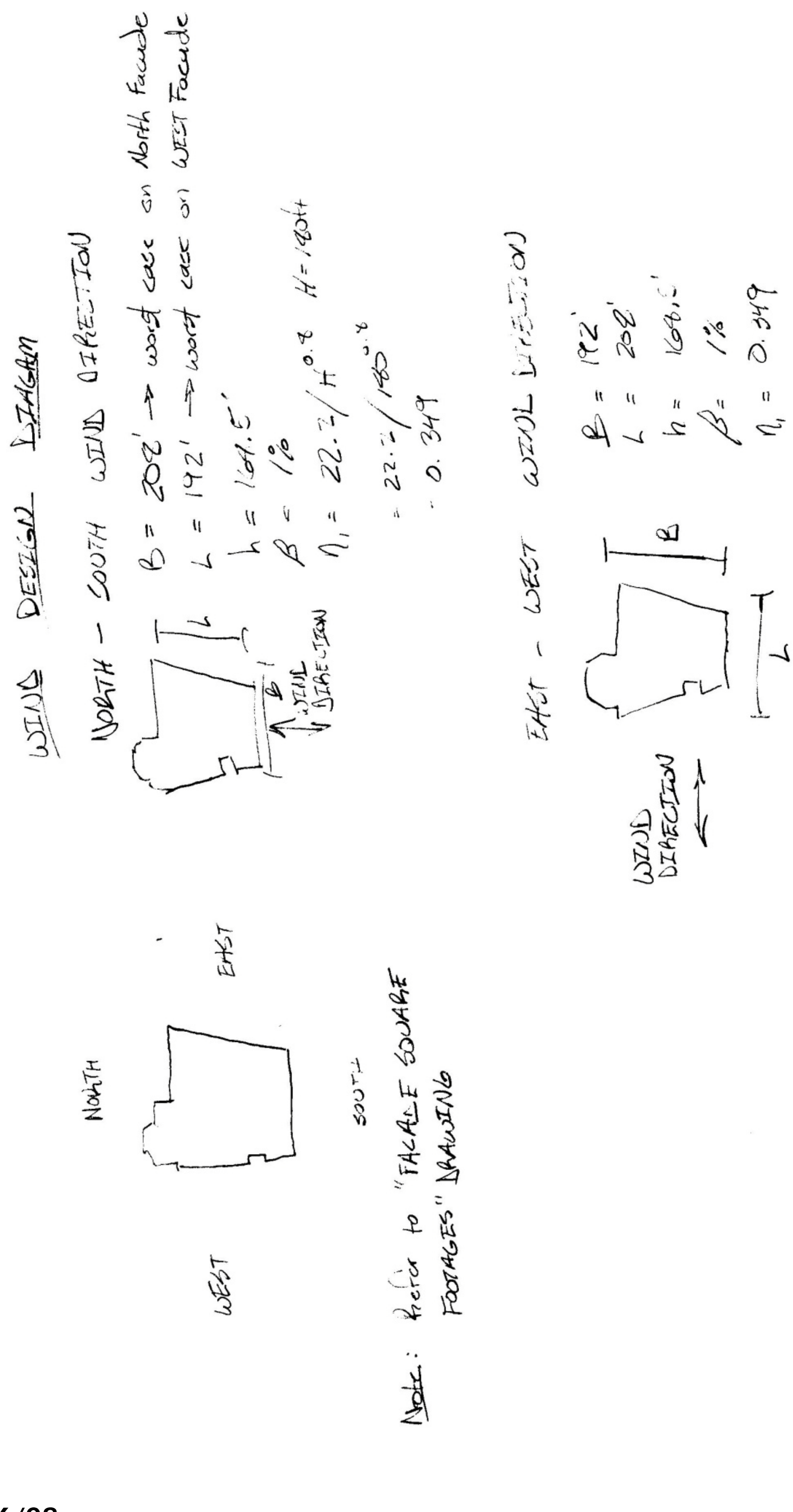
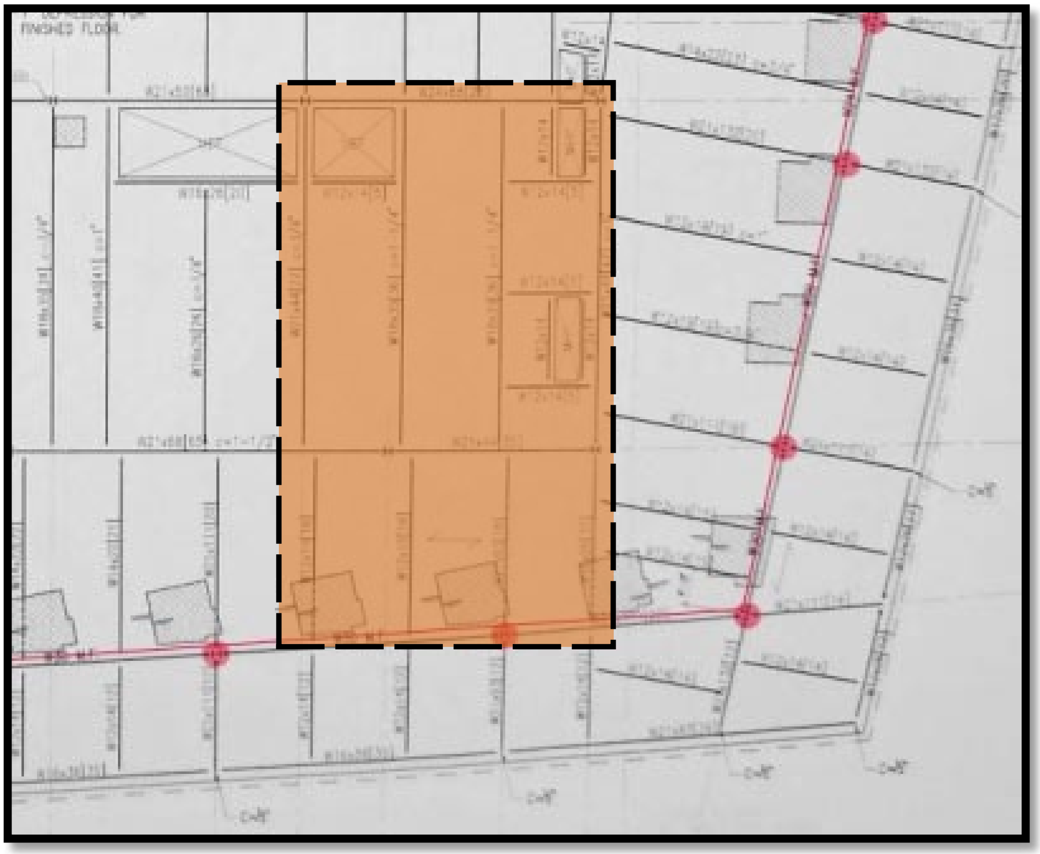


Figure 10: Spot Check Area



Wind Loading Calculations

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FLOWCHART 5.5 CONTINUED

⑤ ARE ALL 5 CONDITIONS OF 6.5.7.1 MET ?
① No

⑥ Topographic factor $K_{zt} = 1.0$

⑦ Determine velocity pressure exposure coefficients K_z and K_h from TABLE 6-3 (6.5.6.6)

See Spreadsheet

⑧ Determine velocity pressure at height z and h by Eq. 6-15

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$$

$$q_h = 0.00256\, K_h K_{zt} K_d V^2 I$$

FLOWCHART 5.6 → METHOD 2 GUST EFFECT FACTORS G and Gf

① Building Properties Directional
B → Horizontal dimension of building measured normal to wind direction, in ft
L → Horizontal dimension of building measured parallel to the wind direction, in ft
h → mean roof height of building
β → damping ration, percent critical for building
n_1 → building natural frequency, Hz

SEE FOLLOWING DIAGRAMS

② Assume $n_1 \geq 1$ Hz → Conservative Yes

③ STRUCTURE IS RIGID
④ $g_Q = g_v = 3.4$
⑤ $\bar{z} = 0.6h \geq z_{min}$ z_{min} given in table 6-2
$\bar{z} = 168.5(0.6) = 100.8$ ft

$z_{min} = 30$ ft

⑥ $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$
$= 0.30\left(\frac{33}{100.4\text{ft}}\right)^{1/6}$
$= 0.25$

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FLOWCHART 5.6 → METHOD 2 GUST EFFECT FACTORS G & Gf CONTINUED

⑦ $L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ Eq 6-7

ℓ → from TABLE 6-2 = 320 ft
$\bar{\epsilon}$ → from TABLE 6-2 = 1/3.0

$$L_{\bar{z}} = 320\text{ft}\left(\frac{100.8}{33}\right)^{(1/3.00)}$$
$$= 464.3$$

⑧ $Q = \sqrt{\dfrac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ Eq 6-6

B = 208'
h = 168.5'
REFER TO WIND DESIGN DIAGRAMS

$$Q_{N-S} = \sqrt{\dfrac{1}{1+0.63\left(\frac{208'+168.5'}{464.3}\right)^{0.63}}}$$

$Q_{N-S} = 0.803$

B = 192'
h = 168.5'

$$Q_{E-W} = \sqrt{\dfrac{1}{1+0.63\left(\frac{192'+168.5'}{464.3}\right)^{0.63}}}$$

$Q_{E-W} = 0.807$

⑨ $G = 0.925\left(\dfrac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right)$ Eq. 6-4

$$G_{N-S} = 0.925\left(\frac{1+1.7(3.4)(0.25)(0.803)}{1+1.7(3.4)(0.25)}\right)$$

$G_{N-S} = \underline{0.8173}$

$$G_{E-W} = 0.925\left(\frac{1+1.7(3.4)(0.25)(0.807)}{1+1.7(3.4)(0.25)}\right)$$

$G_{E-W} = \underline{0.8195}$

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FLOWCHART 5.7 METHOD 2 → BUILDINGS, MAIN WIND-FORCE RESISTING SYSTEMS

① Is the Building enclosed or partially enclosed
Yes

② Does the building have a parapet?
Assumption for simplification → No

③ Is the building a low-rise building as defined in 6.2?

B

No

④ Determine wheather the building is rigid or flexible and the corresponding factor G or Gf from Flow Chart 5.6

$G_{N-S} = 0.8173$

$G_{E-W} = 0.8195$

⑤ Is the building rigid?
Conservative assumption → Yes

⑥ Determine velocity pressure for windward walls along the height of the building and q_h for leeward walls, side walls, and roofing using Flowchart 5.5

SEE SPREADSHEET

⑦ DETERMINE the Pressure coefficients C_p for the walls and roof from Fig 6-6 or 6-8

C_p WALLS

NORTH - SOUTH DIRECTION		EAST-WEST DIRECTION	
Windward	0.8	Windward	0.8
Leeward	-0.5	Leeward	-0.5
Side	-0.7	Side	-0.7

$L/B = 192'/208'$ $\qquad L/B = 208/192$

$L/B = 0.923$ $\qquad L/B = 1.08$

ROOF

North-South Direction		EAST WEST DIRECTION	
WINDWARD	NOT CONSIDERED	WINDWARD	NOT CONSIDERED
LEEWARD	NOT CONSIDERED	LEEWARD	NOT CONSIDERED

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(8) DETERMINE q_i for the walls and roof using Flowchart 5.5 ??

(9) Determine Internal pressure coefficients (GC_{pi}) from Fig 6-5 based on enclousre classification

BEING NEGLECTED FOR SIMPLIFICATION DUE TO BALANCE OF INTERNAL PRESURE

(10) DETERMINE WIND PRESSURES Eq 6-17
WINDWARD WALLS

$$p_z = q_z GC_p - q_h \underbrace{(GC_{pi})}_{\to 0}$$

LEEWARD WALLS

$$p_h = q_h GC_p - q_h \underbrace{(GC_{pi})}_{\to 0}$$

SEE SPREADSHEET

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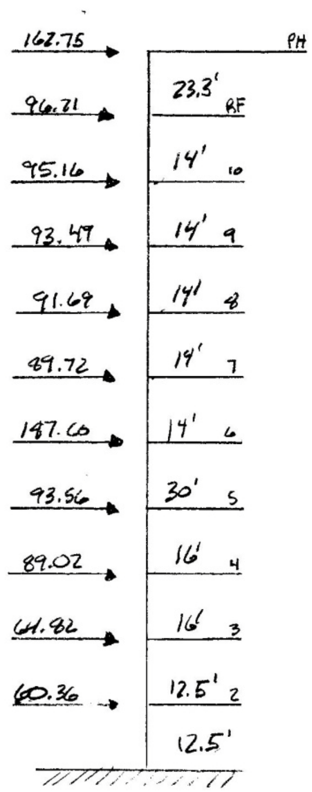
OVERTURNING MOMENT CALCULATIONS
WIND LOADING
NORTH SOUTH



Penthouse om @ 168.35' = (162.75k)(11.65)
= 1896 k ft

Roof om @ 150' = (162.75k)(30.3') + (96.71k)(7')
= 5608 k ft

FLOOR 10 om @ 136' = (162.75k)(44') + (96.71k)(21)
+ (95.16k)(7')
= 9858 k ft

Floor 9 om @ 122' = (162.75)(58.3') + (96.71k)(35')
+ (95.16k)(21) + (93.49k)(7')
= 18 233 k ft

Floor 8 om @ 108' = (162.75k)(72.3') + (96.71k)(49')
+ (95.16k)(35') + (93.49k)(21')
+ (91.69k)(7')
= 22 441 k ft

FLOOR 7 Om @ 94' = (162.75k)(86.3') + (96.71k)(63')
+ (95.16k)(49') + (93.49k)(35')
+ (91.69k)(21) + (89.72)(7')
= 30 626 k ft

FLOOR 6 OM @ 72' = (162.75k)(108.3') + (96.71k)(85')
+ (95.16k)(71') + (93.49k)(57')
+ (91.69k)(43') + (89.72k)(29')
+ (187.60)(15')
= 45 789 k ft

Floor 5 oM @ 49' = (162.75k)(131.3') + (96.71k)(108')
+ (95.16k)(94') + (93.49k)(80')
+ (91.69k)(66') + (89.72k)(52')
+ (187.60k)(38') + (93.56k)(8')
= 66734 k ft

FLOOR 4 OM @ 33' = (162.75k)(147.3) + (96.71k)(124')
+ (95.16k)(110') + (93.49k)(96')
+ (91.69k)(82') + (89.72k)(68')
+ (187.60k)(54') + (93.56k)(24')
+ (89.02k)(8')
= 82115 k ft

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OVERTURNING MOMENT CALCS
WIND LOADING
NORTH - SOUTH

FLOOR 3 OM @ 18.75' = (162.75k)(161.55') + (96.71k)(138.25')
+ (95.16k)(124.25') + (93.49k)(110.26')
+ (91.69k)(96.25') + (89.72k)(82.26')
+ (187.60k)(68.25') + (93.56k)(38.25')
+ (89.02k)(22.25') + (64.82k)(6.25')
= 96 758 k ft

FLOOR 2 OM @ 6.25' = (162.75k)(173.75') + (96.71k)(150.45')
+ (95.16k)(136.45) + (93.49k)(122.45')
+ (91.69k)(108.45) + (89.72k)(94.45')
+ (187.6k)(80.45) + (93.56k)(50.54)
+ (89.02k)(34.48) + (64.82k)(18.45')
+ (60.36k)(6.25')

= 110 090 k ft

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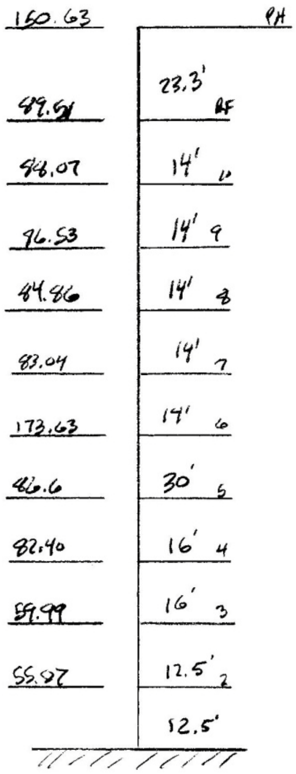
OVERTURNING MOMENT CALCULATIONS
WIND LOADING
EAST - WEST



PENTHOUSE @ 168.36' = (150.75k)(11.65')
= 1755

ROOF OM @ 150' = (150.63k)(30.3') + (89.51k)(7')
= 5190

FLOOR 10 OM @ 136' = (150.63k)(44') + (89.51k)(21')
+ (88.07k)(7')
= 9123.72

FLOOR 9 OM @ 122' = (150.63k)(58.3') + (89.51k)(35')
+ (88.07)(21') + (86.53)(7')
= 14 369

FLOOR 8 OM @ 108' = (150.63k)(72.3') + (89.51k)(49')
+ (88.07k)(35') + (86.53k)(21')
+ (84.86k)(7')
= 20 770

FLOOR 7 OM @ 94' = (150.63k)(86.3') + (89.51)(63')
+ (88.07k)(49') + (86.53k)(35')
+ (84.86k)(21') + (83.04k)(7')
= 28 345

FLOOR 6 OM @ 72' = (150.63k)(108.3') + (89.51k)(85')
+ (88.07k)(71') + (86.53k)(57')
+ (84.86k)(43') + (83.04k)(29')
+ (173.63)(15')
= 43 768

FLOOR 5 OM @ 49' = (150.63k)(131.3') + (89.51)(108')
+ (88.07k)(94') + (86.53k)(80')
+ (84.86k)(66') + (83.04k)(52')
+ (173.63)(38') + (86.6k)(8')
= 61 855

FLOOR 4 OM @ 33' = (150.63k)(147.3') + (89.51k)(124')
+ (88.07k)(110') + (86.53k)(96')
+ (84.86k)(82') + (83.04k)(68')
+ (173.63)(54') + (86.6k)(24')
+ (82.40k)(8')
= 76 000

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OVERTURNING MOMENT CALCS CONT.
WIND LOADING
EAST - WEST

FLOOR 3 om @ 18.75' = (150.63k)(161.55') + (89.51k)(138.25')
+ (88.07k)(124.25') + (86.53k)(110.25')
+ (84.86k)(96.25') + (83.04k)(82.25')
+ (173.63k)(68.25') + (86.6k)(38.25')
+ (82.40k)(22.25') + (59.99k)(6.25')
= 89 560

FLOOR 2 om @ 6.25' = (150.63)(173.78') + (89.51k)(150.45')
+ (88.07k)(136.45') + (86.53k)(122.45')
+ (84.86k)(108.45') + (83.04k)(94.45')
+ (173.63k)(80.45') + (86.6k)(50.54)
+ (82.40k)(34.45') + (59.99)(18.45')
+ (55.87)(6.25'
= 101 937

AE 481

Flowchart 6.8 Cont.
EQUIVALENT LATERAL FORCE PROCEDURE

(11) Determine the base shear V by

Eq. 12.8-1 $V = C_s W$

$W = 55{,}700$ K

$C_s = 0.011$

$V = 612.7$ K

(12) Is $T \le 0.5$ seconds?

$T = 1.78$ No

(13) Is $T \ge 2.5$ sec

No

(14) Exponent related to structure period K

$K = 0.75 + 0.5T$

$= 0.75 + 0.5(1.78)$

$= 1.64$

(15) Determine lateral seismic force F_x at level x by

Eqs 12.8-11 $F_x = C_{vx} V$

Eqs 12.8-12 $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

C_{vx} → vertical distribution factor

V → total design lateral force or shear at the base of the structure

w_i and w_x → the portion of the total effective seismic weight of the structure (w) located or assigned to level i or x

h_i and h_x → the height (ft) from the base to Level i or x

See spreadsheet for calcs

$\boxed{F}$

(16) Determine seismic design story shear V_x by Eq 12.8-13

$V_x = \sum_{i=x}^{n} F_i$

(17) Is the diaphragm flexible in accordance with 12.3

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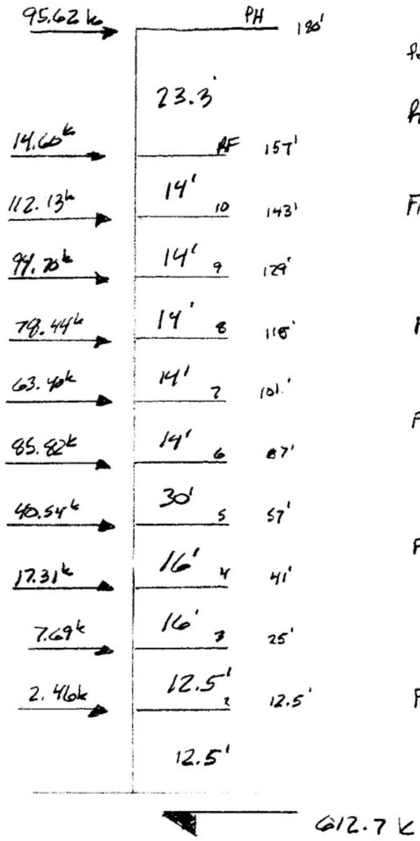
Overturning Moment Calcs → Seismic



Penthouse OM @ 168.35' = (95.62k)(23.3/2') = 1113.97 k-ft

Roof OM @ 150' = (95.62k)(30.3'') + (14.66)(7)
= 3000 k-ft

Floor 10 OM @ 136' = (95.62k)(44') + (14.6)(21)
+ (112.13k)(7')
= 5400 k-ft

Floor 9 OM @ 122' = (95.62k)(58.3) + (14.6k)(35)
+ (112.13k)(21) + (94.70k)(7)
= 9100 k-ft

Floor 8 OM @ 108 = (95.62k)(72.3') + (14.66k)(49')
+ (112.13k)(35) + (94.7k)(21)
+ (78.44k)(7)
= 14091 k-ft

Floor 7 OM @ 94' = (95.62k)(86.3') + (14.66k)(63')
+ (112.13k)(49') + (94.7k)(35)
+ (78.44)(21) + (63.46)(7')
= 20071 k-ft

Floor 6 OM @ 72' = (95.62k)(108.3) + (14.66k)(85')
+ (112.13k)(71') + (94.76)(57')
+ (78.44)(43') + (63.46)(29')
+ (85.82k)(15')
= 31454 k-ft

Floor 5 OM @ 49' = (95.62k)(131.3') + (14.6k)(108)
+ (112.13k)(94') + (94.7k)(80')
+ (78.44k)(66) + (63.46)(52')
+ (85.82k)(38) + (40.54)(8')
= 44307 k-ft

Floor 4 OM @ 33' = (95.62k)(147.3) + (14.6k)(124')
+ (112.13k)(110) + (94.7k)(96)
+ (78.44)(82) + (63.46k)(68)
+ (85.82k)(54) + (40.54)(24)
+ (17.31k)(8)
= 53 810

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OVERTURNING MOMENT CALCS CONTINUED

FLOOR 3 OM @ 18.75' = (95.62 k)(161.55') + (14.66)(138.25)
+ (112.13k)(124.25) + (94.76)(110.25)
+ (78.44k)(96.25) + (63.46)(82.25)
+ (85.82k)(68.25) + (40.54)(34.25)
+ (17.31k)(22.25) + (7.69k)(6.25)
= 57 370

FLOOR 2 OM @ 6.25' = (95.62k)(173.75') + (14.66)(150.45)
+ (112.13k)(136.45) + (94.76)(122.45')
+ (78.44k)(108.45) + (63.46)(94.45')
+ (85.82k)(80.45') + (40.54)(50.54')
+ (17.31k)(34.45) + (7.69k)(18.45')
+ (2.46k)(6.25)

= 60 955

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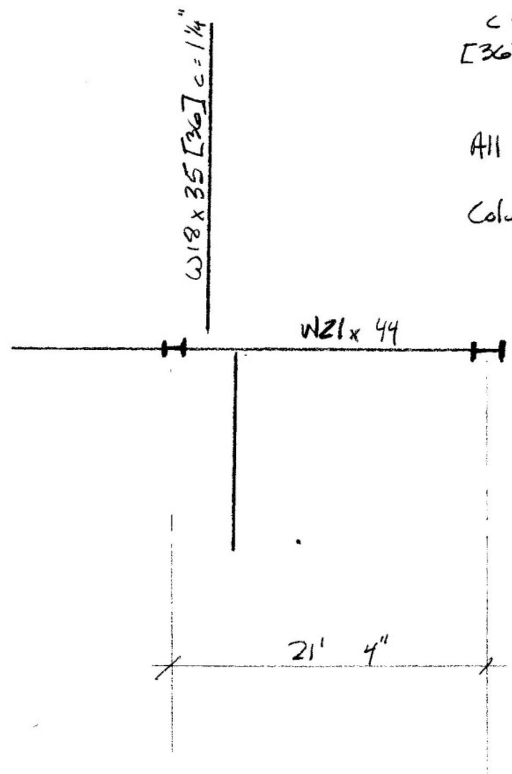
Spot Check typical floor framing Elements with gravity loads

Choose Area to Analyze
Tenth Floor
Column G 5.2 in South East Corner
Girder connected to left of G 5.2 (EAST)
Beam North of G 5.2

Diagram of area being analyzed



c → camber
[36] → amount of ¾"ϕ x 4½" long headed shear connectors

All W sizes are A 992 steel

Column G-5.2 is a W14x90 per CD S300

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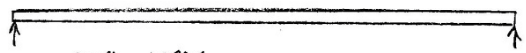
I. BEAM → COMPOSITE

BEAM SPACING
10FT



DEAD LOADS
Self Weight → 35 lb/ft = 1.75 lb/ft²

Concrete (Light weight) 6-10 lb/ft² in AISC Table 17-13 10(4.25in)
→ 42.5 lb/ft

Metal Deck → 2.25 lb/ft²

Partition Load → 10 lb/ft²

Ceiling Load → 20 lb/ft²

Total → 76.5 lb/ft²

Live Loads
80 lb/ft² from CD S100

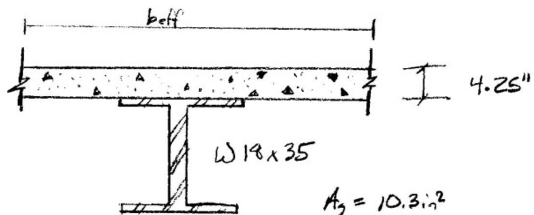
F_y = 55 ksi from ASTM

F_u = 65 ksi from ASTM

f'_c = 5,000psi CD S100

Shear studs must have strength of 21.9 kips

Trib width 20ft length 36ft
LOAD AREA = (10ft)(36ft) = 360 ft²



A_s = 10.3in²
d = 17.7in
bf = 6.00in
tf = 0.425in
beff = 108in

① $b_{eff} = b_1' + b_2'$

b_1' is least of
span/8 = 36ft/8 = 4.5
or
½(dist adj beam)
= ½(10ft) = 5'

$b_2' = b_1'$

b_{eff} = 2(4.5') = 9'

② Determine the controlling compression force

$V'_c = 0.85(f'_c)\,b_{eff}\,t$
$V'_c = 0.85(5)(108in)(4.25")$
V'_c = 1950 kip

$V'_s = A_s F_y$
V'_s = 10.3in²(55ksi)
V'_s = 566.5 kip

V'_s is less than V'_c, the PNA is in the concrete

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③ Determine PNA location using a

$$a = \frac{V'_s}{0.85 f'_c b_{eff}} = \frac{A_s F_y}{0.85 f'_c b_{eff}}$$

$$a = \frac{566.5 \text{ kip}}{0.85(5 \text{ ksi})(108 \text{ in})} = 1.234$$

④ Determine the nominal moment strength using Eq. 9.5

$$M_n = T_s(d/2) + C_c(t - a/2)$$

$$M_n = 566.5(17.7\text{in}/2) + 566.5\text{kip}(4.25 - \tfrac{1.23}{2})$$

$$M_n = 7074 \text{inkips}$$

$$M_n = \frac{7074 \text{ in kips}}{12} = 589.5 \text{ ft kips}$$

$$\phi M_n = 0.9(590 \text{ ft kips}) = 531 \text{ ft kip}$$

Table 3-19 W18x35
$\phi M_p = 249$ kip

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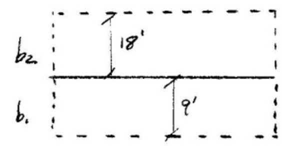
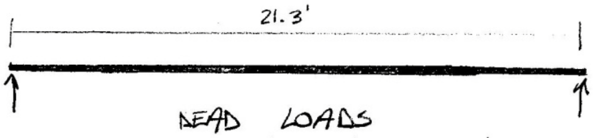
II. GIRDER

W21x44 [35]

NOTE METAL DECKING RUNS Parallel TO THE STEEL W SHAPE

Girder Spacing





DEAD LOADS

Self Weight → 44 lb/ft

44(21.3') = 937.2 lb

Concrete Light Weight

AISC Table 17-13 10 lb (4.25 in)

42.5 lb/ft²

F_y = 55 ksi

F_u = 65 ksi

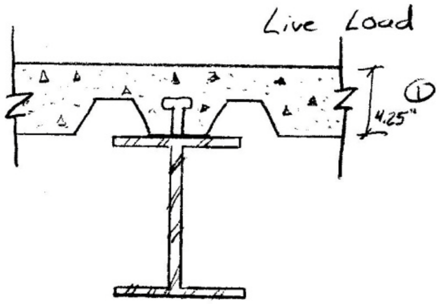
F'_c = 5000 psi

Metal Deck → 2.25 lb/ft²

Partition Load → 10 lb/ft²

Ceiling Load → 20 lb/ft²

Live Load 80 lb/ft²



① $b_{eff} = b_1' + b_2'$

$b_1' = \frac{span}{8} = \frac{21.3}{8} = 2.66'$ ← governs

$= \frac{1}{2}$ dis adj bm $= \frac{1}{2}(18') = 9'$

$b_2' = \frac{span}{8} = \frac{21.3}{8} = 2.66'$ ← governs

$\frac{1}{2}$ dis adj bm $= \frac{1}{2}(36') = 18'$

$b_{eff} = 5.325$ ft

$= 63.9$

② Determine the compression force using the full concrete and full steel areas

$V'_c = 0.85(F'c)(b_{eff})(t)$

$V'_c = 0.85(5ksi)(63.9in)(4.25")$

$V'_c = 1154.2$ kip

$V'_s = A_s F_y$

$V'_s = 13.0 in^2 (55ksi)$

$V'_s = 715$ kip

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II. GIRDER SPOT CHECK CONT.

③ $V'g = 715$ kips

④ Calculate the effective depth of concrete

$$a = \frac{715\text{kips}}{0.85(5\text{ksi})(63.9)}$$

$$a = 2.63\text{ in}$$

Because $a < 4.25"$ available in the concrete above the deck, the procedures for a flat soffit beam can be used.

⑤ Determine the nominal moment strength

$$M_n = T_s\ (d/2) + C_c\ (t - a/2)$$

$$M_n = 715\text{kips}\left(\frac{20.7\text{in}}{2}\right) + 715\text{kips}\left(4.25\text{in} - \frac{2.63\text{in}}{2}\right)$$

$$M_n = 9498.8 \text{ kip in}$$

$$= 791.6 \text{ ft kip}$$

TABLE 3-19 W21x44
AT $y2 = 4.5$

Available Flexure Strength = 722 kip ft

Structural Steel Strengths				
ASTM Designation	Governed Elements	F_y Min. Yield Stress (ksi)	F_u Min. Tensile Stress (ksi)	Reference Location
ASTM A-992	All W Shapes	50-65[a]	65[a]	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-36	All other rolled shapes, plates, and bars unless otherwise noted	36	58-80[b]	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-500 Grade B	HSS Sections (Square, Rectangular)	46	58	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-500 Grade C	HSS Sections (Round)	46	62	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-53 Grade B	Pipe	35	60	Vol. III Structural General Notes S100 & AISC Table 2-3
ASTM A-325 Type SC or N	All Bolts for connecting structural members	--	105	Vol. III Structural General Notes S100 & AISC Table 2-5
ASTM F1554 Grade 36	All anchor rods unless otherwise noted	36	58-80	Vol. III Structural General Notes S100 & AISC Table 2-5

Notes: a - A maximum yeild-to-strength ratio of 0.85 and carbon eqivalnet formula are included as mandatory in ASTM- 955

b- For shapes over the 426 lb/ft only the minimum of 5858ksi applies

Concrete Strengths		
Governed Elemets	Minimum Compressive Strenght (f'c) psi	Reference Location
Caissons, LEBs	5,000	Vol. III Structural General Notes S100
Slurry Wall Concrete Diaphram	5,000	Vol. III Structural General Notes S100
Cap Walls	5,000	Vol. III Structural General Notes S100
Two-Way Concrete Slabs	5,000	Vol. III Structural General Notes S100
Formed Walls	4,000	Vol. III Structural General Notes S100
Topping Slabs	4,000	Vol. III Structural General Notes S100
Slabs on Grade	4,000	Vol. III Structural General Notes S100
Fill Concrete Mud Slabs	2,000	Vol. III Structural General Notes S100
LinAcc Sielding	5,000	Vol. III Structural General Notes S100

Reinforcing			
ASTM Designation	Bars	Minimum Yield Strength (psi)	Minimum Tensile Strength (psi)
A 615 Grade 60	Less than #11	60,000	90,000
A 615 Grade 75	#11 and greater	75,000	100,000
A 706	To be welded	60,000	80,000