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Structural Option
Eight Tower Bridge
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Structural Technical Report #1
Structural Concepts/Existing Conditions Report

Introduction

The following report is an in-depth summary and preliminary analysis of the structural system for Eight Tower Bridge, a 16-story high-rise office building located in Conshohocken, Pennsylvania. Completed in April of 2002, Eight Tower Bridge sits on the shore of the Schuylkill River, next to the Fayette Street Bridge, leading to both interstates I-476 and I-76. This prime location less than 15 minutes outside of Centre City Philadelphia makes it a great location for a multi-tenant office building. The building was designed by the high profile architecture firm of Skidmore, Owings and Merrill, who have been responsible for such structures as the Sears Tower in Chicago, and are currently designing the new Freedom Tower in New York City. Eight Tower Bridge is the most recent office building to be constructed in the Conshohocken area by the real estate development company Oliver Tyrone Pulver Corporation. The company has built nearly $400 million worth of new office, commercial and retail space in the area over the past 10 years, adding nearly 1.2 million square feet of rentable space. The 315,000 square foot Eight Tower Bridge was the largest single function structure of the Tower Bridge buildings to be constructed, falling second in overall size to the mixed-use One Tower Bridge.

The scope of this report is limited to construction documents issued for construction on March 25th, 2001 and in some cases, revision bulletins one through seven. This report is intended to introduce the structural system of the building, including information relative to design concepts and required loadings, as well as design assumptions. This report includes an overview of the building's structural components including the general floor framing, structural slabs, and lateral load resisting system, foundation system, and bracing system. Spot checks have been completed for a typical floor beam, girder, as well as column. Additionally, both wind and seismic load analysis have been conducted on the structure in order to further analyze the effectiveness of the lateral reinforcing system. Copies of these calculations can be found in Appendices A, B and C. All loads for analysis have been developed through use of proper codes or else otherwise noted in the construction documents.

Code and Code Requirements

Both the gravity and lateral structural systems of Eight Tower Bridge were designed in accordance with requirements set forth by the BOCA National Building Code, 1996 edition. Structural steel members were designed using AISC "Load and Resistance Factor Design Specification for Structural Steel." For the development of lateral load analysis for this report, load development procedures were taken from ASCE7-02, chapters 6 and 9.

Gravity and Lateral Loads

As mentioned in "Code and Code Requirements" above, the gravity and lateral loads for this report were developed using methods and standards set forth by ASCE7-02. Additional loading cases and requirements were obtained from the structural documents. A combination of loads from both sources provided the necessary loadings analyze the structural system. Loads from both ASCE7-02 and the structural drawings are listed below:

Gravity Loads:

Loads: (PSF)

- Typical Offices: 50 + 20 for partitions
- Lobbies: 100 psf
- Stairs: 100 psf
- Parking: 50 psf
- Roof: 30 psf
- Mechanical: 125 psf
- Ceiling, Mechanical, Electrical, Plumbing: 5 psf

Additional Loads:

Terrace at Level 15

-Superimposed Dead Load: 75 psf
-Live Load: 50 psf

Roof/Mechanical Penthouse Level-

- Roof
  -Live Load: 30 psf
  -Superimposed Dead Load: 12 psf
- Mechanical
  -Live Load: 300 psf
  -CMEP: 8 psf
- Elevator Machine Room
  -Live Load: 150 psf
  -CMEP: 8 psf
- Cooling Tower
  -Live Load: 212 psf (150 psf + 62 psf snow load)
- Roof Drift Snow
  -Live Load: 62 psf
  -Superimposed Dead Load: 12 psf

The loadings listed above under "additional loads" apply strictly to the areas specified. Areas to which to loads apply are hatched on the structural drawings and is included in Appendix A.

Lateral Loads:

Wind and seismic lateral loads for the structure were derived from the methods set forth in ASCE7-02, chapters 6 and 9. A table summarizing the results of the both wind and seismic analysis in the E-W direction can be found below. For a complete table of all factors, assumptions and derivations, as well as the results of N-S seismic analysis, please refer to Appendix A.

Wind Loads (ASCE7-02)

Location: Conshohocken, PA

Dimension			**Penthouse**		
N-S	**118**	ft	N-S	**114**	ft
E-W	**196**	ft	E-W	**44**	ft
Total Height, h	**214**	ft	Height	**22**	ft
Inter-story Height, h_s	**12.08**	ft			

Velocity Pressu			
K_{zt}	**1.0**		(Table 6-4) Assume area is flat
K_d	**0.85**		(Table 6-4)
V	**90**	mph	(Figure 6-1)
Group	**II**		Office Building
Importance Factor, I	**1.0**		(Table 6-1)
Exposure	**B**		Assume area is flat
$q_z/K_z = 0.00256K_{zt}K_dV^2I$ =	17.6256		

ASCE7-02 Chapter 6: Wind Analysis
Method 2- Analytical Procedure

Story Forces		Storey Shear		Moment	
N-S	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
(kips)	(kips)	(kips)	(kips)	(ft-kips)	(ft-kips)
11.28	25.83			2,414.13	5,525.38
30.02	27.04	11.28	25.83	5,769.32	5,197.03
32.36	47.33	41.30	52.87	5,828.56	8,525.39
31.96	46.67	73.66	100.20	5,369.90	7,842.59
31.55	46.01	105.62	146.87	4,920.83	7,175.50
31.15	45.35	137.17	192.88	4,481.53	6,524.41
30.67	44.57	168.32	238.22	4,042.95	5,874.50
30.17	43.74	198.99	282.79	3,611.91	5,236.95
29.66	42.92	229.16	326.54	3,193.07	4,619.41
29.12	42.03	258.82	369.45	2,782.89	4,016.05
28.52	41.03	287.95	411.48	2,380.48	3,425.42
27.77	39.80	316.46	452.51	1,982.45	2,841.97
26.96	38.48	344.23	492.32	1,599.09	2,282.55
26.10	37.08	371.18	530.79	1,233.08	1,751.57
25.01	35.29	397.29	567.87	879.38	1,240.78
23.57	32.92	422.30	603.16	543.92	759.86
22.32	30.88	445.86	636.08	245.50	339.64
	Total Shear	**468.18**	**666.96**		
			Total Moment	**51,278.99**	**73,179.01**

Table 1.1- Results of a wind load analysis

(Not to Scale)

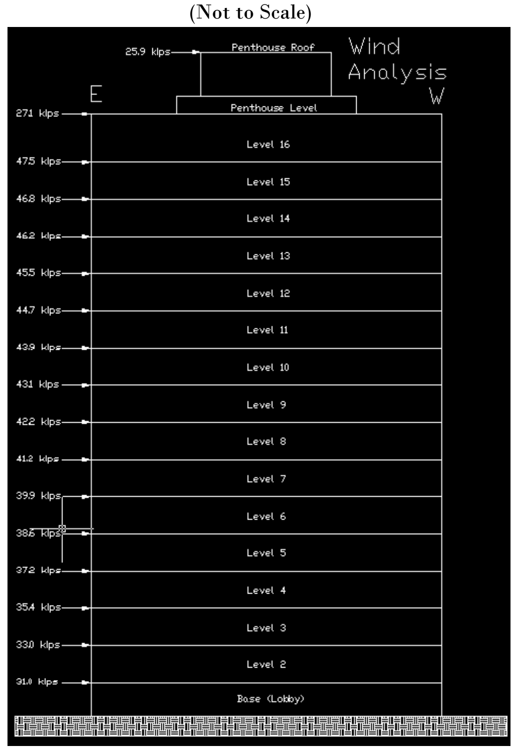


Figure 1.1- Floor Distribution of wind loads (E-W)

(Not to Scale)

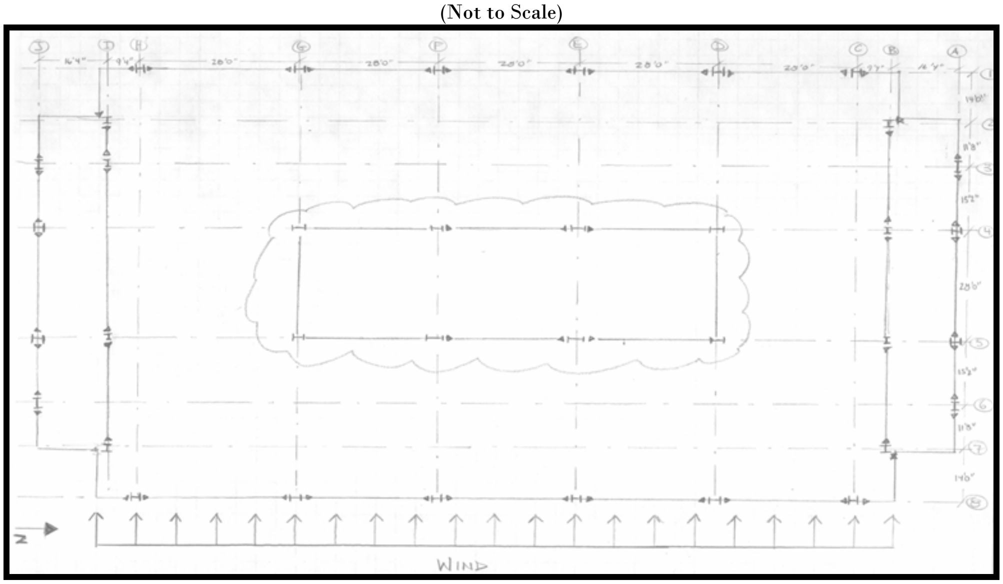


Figure 1.2- Wind force on E-W side of structure

Seismic Loads (ASCE7-02)

Table 2 : Vertical Distribution of Seismic Forces (E-W)

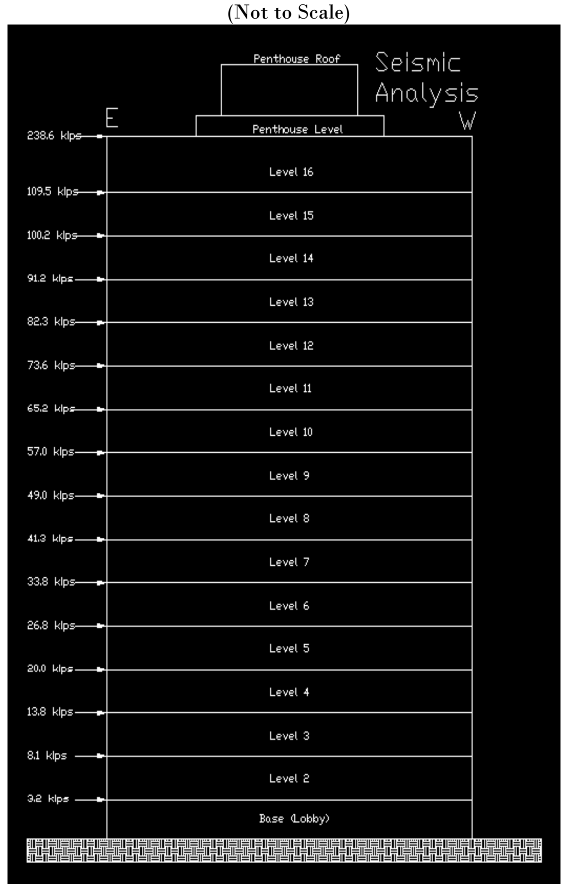
Seismic Base Shear, $V_{E\text{-}W} = C_{s,E\text{-}W}W =$ 1013 kips

Exponent $k_{E\text{-}W} = 1 + (T_{E\text{-}W} - 0.5)/(2.5 - 0.5) =$ 1.27

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	3100	192	2,443,848	0.235	238.6		45,853
16	1545	180	1,121,544	0.108	109.5	238.6	19,720
15	1545	168	1,026,681	0.099	100.2	348.1	16,842
14	1545	156	933,987	0.090	91.2	448.3	14,220
13	1545	144	843,202	0.081	82.3	539.5	11,843
12	1545	132	754,442	0.073	73.6	621.8	9,707
11	1545	120	667,841	0.064	65.2	695.4	7,805
10	1545	108	583,557	0.056	57.0	760.6	6,132
9	1545	96	501,778	0.048	49.0	817.6	4,681
8	1545	83	422,733	0.041	41.3	866.6	3,445
7	1545	71	346,708	0.033	33.8	907.8	2,417
6	1545	59	274,071	0.026	26.8	941.7	1,587
5	1545	47	205,321	0.020	20.0	968.4	947
4	1545	35	141,173	0.014	13.8	988.5	485
3	1545	23	82,771	0.008	8.1	1002.2	186
2	1545	11	32,334	0.003	3.2	1010.3	35
BASE						1013.5	
	Σ =		Σ =	Σ =	Σ =		Σ =
	26268		10381990	1.000	1013.5		145903

<u>Table 1.3</u> E-W vertical distribution of seismic forces

(Not to Scale)



<u>Figure 1.3</u> E-W vertical distribution of seismic forces

Discussion of Wind Analysis

As mentioned above, the wind analysis of Eight Tower Bridge was conducted in accordance with the provisions set forth in ASCE7-02, Method 2. Several assumptions and interpolations were made from during the wind analysis. To obtain the topographic factor (K_{zt}) and exposure category, the area surrounding the structure was assumed to be flat. Eight Tower Bridge was determined to be a use group of II (office building), and the lateral load resisting system was classified as "other structural system" in table 9.5.5.3.2.

To resolve the total wind pressures to forces on the 16th floor and mechanical penthouse roof, half of the 16th story height and half of the mechanical penthouse height were multiplied by the corresponding reduction in area of the mechanical penthouse in both directions. The penthouse roof forces were obtained by multiplying half the penthouse story height by its corresponding width, and multiplying by the directional wind pressure. This procedure resulted in an adjusted and more accurate wind forces for these levels.

Discussion of Seismic Analysis

The seismic analysis of Eight Tower Bridge also required several making several assumptions. Due to the combination of both braced frames and moment resisting frames, a C_t and an x value of 0.02 and 0.75 respectively from table 9.5.5.3.2. These values determine that the approximate period in both directions was sufficient to classify the building as a rigid structure.

In order to simplify the seismic analysis, the structure was analyzed as a 16-story structure, with a height totaling only 193 feet and neglecting the 22 foot mechanical penthouse. Instead, the mechanical penthouse was calculated as a dead load on the 16^{th} story of the building added to the total floor weight for that story. The analysis was then conducted as normal.

Description of Structural System

The structural system of Eight Tower Bridge supports 16 stories stretching 192' into the air. The superstructure also supports a mechanical penthouse level that rises 22' above the lower roof, topping the building out at 214'. The mechanical penthouse protects two massive cooling towers, a fan room, and an elevator machine room that controls the six general access elevators. The structural framing of Eight Tower Bridge provides strong lateral support, as well as opening the floor plan of the building in order to maximize rentable space to nearly 19,800 square feet per floor. In addition to mechanical roof loads, gravity floor loads, and lateral forces, the perimeter of the building must support a façade of pre-cast concrete panels and glazed windows.

Foundation

The building foundation system of Eight Tower Bridge consists of reinforced normal weight concrete pile caps ranging from 36" to 54" in depth. The pile caps range in dimension from approximately 7'x7' to 11'x10'. These pile caps are supported by four to eight 16" diameter auger–cast piles driven to an average bearing depth of thirteen feet below grade. The piles are made of normal weight concrete with a compressive strength of 4,000psi, and have been designed to a capacity of 100 tons.

The core of the building is supported by a 4'3" reinforced concrete mat foundation, supported by additional auger-cast piles. The entire building is supported by a total of 328 piles. Reinforced concrete grade beams connect all of the pile caps, as well as the interior core mat foundation.

Slab at the lobby level consists of a 5" concrete slab-on-grade reinforced with one layer of 4x4 welded wire fabric. The slab sits over a loose granular fill, which sits over compacted sub-grade soil. The inner core slab-on-grade is similar, but is cast 8" thick and has two layers of welded wire fabric as reinforcement. The lobby level also functions as a parking garage, thus eliminating the space for HVAC equipment underneath the building, thus forcing placement on the roof. The mechanical equipment loading creates an additional dead load on the structure, as well as adding to the complexity of wind and seismic calculations.

Superstructure Frame

Eight Tower Bridge is a steel framed system. The framing in this system is fairly straight forward in design. The simple design has allowed for 13 of the 16 stories to be designed with a typical framing plan. Beam sizes for this system are most commonly W 18x40 and typically spanning 44'4" and spaced at 9'4". Variations in this framing system occur at the extreme north and south end of the building, as well as in the buildings core due to mechanical system loads, and the insertion of six elevator towers through the height of the building. Exterior girders have been sized to W21x44 with spans ranging from 28' to 12', while interior girders are sized at W18x86. Beam-to-column and beam-to-girder connections are typically simple shear connections. Beam-to-column connections in the moment resisting frames are fully welded moment connections, or as an alternate, have bolted end-plate moment resisting connections. All structural steel beams spanning over 35' are designed with an upward camber and have been specified to ASTM A992 grade.

Lateral System

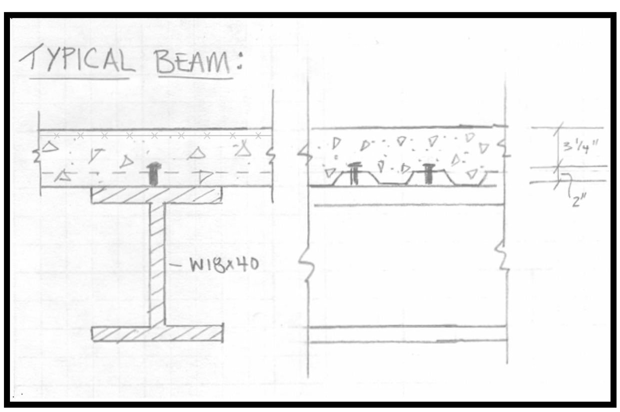
The lateral system of Eight Tower Bridge is actually two separate concentric frame systems. The inner framing structure is an 18-story core tower comprised of a combination of moment and braced frames. The braced frames span 28' along column lines D, E, F and G in the east-west dimension of the building. Additional braced frames span 56' along column lines 4.1 and 4.9 in the north-south direction between column lines D and F. Refer to the typical framing plan layout located in Appendix B. The typical frame is made of W 14x90 through W 14x550 columns, W18x50 beam members, and braced diagonally with two 8 x 6 x ¾ welded angles. The core tower supports the elevator machine room, as well as the mechanical fan room.

The outer frame is comprised of structural steel moment resisting frames located around the building perimeter. All structural steel is specified as ASTM A992 grade.

Structural Slab

Eight Tower Bridge employs the use of reinforced concrete slab poured over metal deck for their flooring system. The typical floor slab is 5-1/4" thick with 3-1/4" light-weight concrete over non-cellular 2" deep metal deck. The system uses 6x6-W1.4xW1.4 welded-wire mesh and shear studs spaced along the span of them beam to develop a full composite structural

<u>Figure 1.4</u>- **Typical beam section and elevation**



slab. Designing the slab to act with full composite strength allows the W-shape floor beams to develop a larger moment capacity, thus capable of spanning longer distances. The ability of a beam to span a longer distance eliminates the need for a overly repetitive column structure, increasing the openness of the floor plan which is a desirable trait in office building design. The concrete slab described above is shown in Figure 1.4 and is used as primary flooring system in all office spaces.

Special Design Cases and Concerns

Additional slab systems were designed for the mechanical penthouse and mechanical fan room. The mechanical fan room located on each level of Eight Tower Bridge requires and 8" thick normal weight concrete slab with 2" deep metal deck. Reinforcing for this slab is specified as #5 bars spaced at 12" for both top and bottom reinforcing. The slab also acts in composite with W-shape floor beams, and also required shoring during construction.

A similar slab system was used for the mechanical penthouse. Differences include the increase in slab depth to 9" total, and reinforcing of #4 bars spaced at 12". Shoring during construction was not specified.

Structural Member Check

Beam Check

A check of a typical floor beam found within a typical bay was conducted. The beam chosen was taken from a typical bay on the sixth level along column line G. Copies of hand calculations can be found in Appendix C.

Appendix A

Table A.1- Basic wind analysis factors and pressure distribution

Location: **Conshohocken, PA**

					Penthouse	
Dimension	N-S	**196** ft		N-S	**114**	ft
	E-W	**118** ft		E-W	**44**	ft
	Total Height, h	**214** ft		Height	**22**	ft
	Inter-story Height, h_s	**12.08** ft				

Velocity Pressure	K_{zt}	**1.0**		(Table 6-4) Assumed area is flat
	K_d	**0.85**		(Table 6-4)
	V	**90**	mph	(Figure 6-1)
	Group	**II**		Office Building
	Importance Factor, I	**1.0**		(Table 6-1)
	Exposure	**B**		Assumed area is flat
	$q_z/K_z = 0.00256K_{zt}K_dV^2I$ =	17.6256		

	N-S	E-W
Gust Factor Effect, G	**0.832**	**0.821**

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					Resultant pressure N-S	Resultant pressure E-W
External Pressure Coefficients, C_p				Windward	**0.8**	**0.8**
				Leeward	**-0.5**	**-0.36**
Story No.	z (ft)	K_z	q_z (lb/ft²)	q_h (lb/ft²)	$q_zC_pG - q_hC_pG$ (lb/ft²)	$q_zC_pG - q_hC_pG$ (lb/ft²)
Penthouse Roof	214	1.222	21.544	21.544	23.313	20.595
Penthouse Level	192	1.188	20.945	21.544	22.914	20.201
16	180	1.170	20.625	21.544	22.701	19.991
15	168	1.146	20.200	21.544	22.418	19.712
14	156	1.122	19.775	21.544	22.135	19.432
13	144	1.098	19.349	21.544	21.851	19.152
12	132	1.070	18.851	21.544	21.520	18.825
11	120	1.039	18.318	21.544	21.165	18.475
10	108	1.009	17.786	21.544	20.811	18.125
9	95.6	0.977	17.215	21.544	20.430	17.750
8	83.5	0.940	16.576	21.544	20.005	17.330
7	71.4	0.896	15.785	21.544	19.478	16.811
6	59.3	0.847	14.934	21.544	18.911	16.252
5	47.2	0.796	14.034	21.544	18.312	15.660
4	35.2	0.731	12.884	21.544	17.546	14.905
3	23.1	0.645	11.362	21.544	16.533	13.905
2	11	0.570	10.047	21.544	15.657	13.041
Base	0	0.570	10.047	21.544	15.657	13.041

Table A.2- Wind analysis results and force distribution, shear and moment

Location:		**Conshohocken, PA**			
				Penthouse	
Dimension	N-S	**118** ft	N-S	**114** ft	
	E-W	**196** ft	E-W	**44** ft	
	Total Height, h	**214** ft	Height	**22** ft	
	Inter-story Height, h_s	**12.08** ft			
Velocity Pressu	K_{zt}	**1.0**		(Table 6-4) Assume area is flat	
	K_d	**0.85**		(Table 6-4)	
	V	**90** mph		(Figure 6-1)	
	Group	**II**		Office Building	
	Importance Factor, I	**1.0**		(Table 6-1)	
	Exposure	**B**		Assume area is flat	
	$q_z/K_z = 0.00256K_{zt}K_dV^2I =$	17.6256			

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Method 2- Analytical Procedure

Story Forces		Storey Shear		Moment	
N-S	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
(kips)	(kips)	(kips)	(kips)	(ft-kips)	(ft-kips)
11.28	25.83			2,414.13	5,525.38
30.02	27.04	11.28	25.83	5,769.32	5,197.03
32.36	47.33	41.30	52.87	5,828.56	8,525.39
31.96	46.67	73.66	100.20	5,369.90	7,842.59
31.55	46.01	105.62	146.87	4,920.83	7,175.50
31.15	45.35	137.17	192.88	4,481.53	6,524.41
30.67	44.57	168.32	238.22	4,042.95	5,874.50
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28.52	41.03	287.95	411.48	2,380.48	3,425.42
27.77	39.80	316.46	452.51	1,982.45	2,841.97
26.96	38.48	344.23	492.32	1,599.09	2,282.55
26.10	37.08	371.18	530.79	1,233.08	1,751.57
25.01	35.29	397.29	567.87	879.38	1,240.78
23.57	32.92	422.30	603.16	543.92	759.86
22.32	30.88	445.86	636.08	245.50	339.64
	Total Shear	**468.18**	**666.96**		
			Total Moment	**51,278.99**	**73,179.01**

Table A.3- **N-S Wind Gust Factor Effect**

Gust Factor Effect Calculation			
1. N-S Direction			
B =	118 ft	L/B = 1.661	
L =	196 ft		
h =	214 ft		
C_t =	**0.02**	(Table 9.5.5.3.2) Steel Moment Frames/Braced Frame Core	
x =	**0.75**		
Estimated Frequency		$f = 1/C_t h^x$ = 0.894	
From Table 6-2	**Exposure B, Case 2**		
z_{min}	**30**		
c	**0.30**		
l	**320**		
ε	**0.33**		
b	**0.45**		
α	**0.25**		
g_Q	**3.40**	(Given)	
g_V	**3.40**	(Given)	
β	**0.05**		
0.6h =	128.40	$>z_{min}$ (OK!)	
z =	128.40		
$L_z = l(z/33)^{\varepsilon}$ =			503.3
$I_z = c(33/z)^{1/6}$ =			0.239
$Q = (1+0.63[(B+h)/L_z]^{0.63})^{-0.5}$ =			0.821
$V_z = b(z/33)^{\alpha}V(88/60)$ =			83.426
$n_1 = f$ =			0.894
$\eta_h = 4.6n_1 h/V_z$ =			10.545
$\eta_B = 4.6n_1 B/V_z$ =			5.814
$\eta_L = 15.4n_1 L/V_z$ =			32.332
$R_h = 1/\eta_h - (1-e^{-2\eta_h})/2\eta_h^2$ =			0.090
$R_B = 1/\eta_B - (1-e^{-2\eta_B})/2\eta_B^2$ =			0.157
$R_L = 1/\eta_L - (1-e^{-2\eta_L})/2\eta_L^2$ =			0.030
$N_1 = \eta_1 L_z/V_z$ =			5.391
$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ =			0.048
$R = [(R_n R_h R_B(0.53+0.47R_L)/\beta]^{0.5}$			0.086
			4.163
$g_R R$ =			0.360
			0.832

Table A.4- **E-W Wind Gust Factor Effect**

Gust Factor Effect Calculation			
2. E-W Direction			
B =	196 ft	L/B = 0.602	
L =	118 ft		
h =	214 ft		
C_t =	**0.02**	(Table 9.5.5.3.2) Steel Moment Frames/Braced Frame Core	
x =	**0.75**		
Estimated Frequency		$f = 1/C_t h^x$ = 0.894	
From Table 6-2	**Exposure B, Case 2**		
z_{min}	**30**		
c	**0.30**		
l	**320**		
ε	**0.33**		
b	**0.45**		
α	**0.25**		
g_Q	**3.40**	(Given)	
g_V	**3.40**	(Given)	
β	**0.05**		
0.6h =	128.40	$>z_{min}$ (OK!)	
z =	128.40		
$L_z = l(z/33)^{\varepsilon}$ =			503.3
$I_z = c(33/z)^{1/6}$ =			0.239
$Q = (1+0.63[(B+h)/L_z]^{0.63})^{-0.5}$ =			0.802
$V_z = b(z/33)^{\alpha}V(88/60)$ =			83.426
$n_1 = f$ =			0.894
$\eta_h = 4.6n_1 h/V_z$ =			10.545
$\eta_B = 4.6n_1 B/V_z$ =			9.658
$\eta_L = 15.4n_1 L/V_z$ =			19.465
$R_h = 1/\eta_h - (1-e^{-2\eta_h})/2\eta_h^2$ =			0.090
$R_B = 1/\eta_B - (1-e^{-2\eta_B})/2\eta_B^2$ =			0.098
$R_L = 1/\eta_L - (1-e^{-2\eta_L})/2\eta_L^2$ =			0.050
$N_1 = \eta_1 L_z/V_z$ =			5.391
$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ =			0.048
$R = [(R_n R_h R_B(0.53+0.47R_L)/\beta]^{0.5}$ =			0.069
$g_R = (2\ln(3600n_1))^{0.5}+0.577/(2\ln(3600n_1))^{0.5}$ =			4.163
$g_R R$ =			0.287
$[1+1.7I_z((g_Q Q)^2+(g_R R)^2)^{0.5}]/(1+1.7g_v I_z)$ =			**0.821**

Table A.5- Seismic analysis factor calculations

ASCE7-02 Chapter 9- Seismic Analysis

Reference					
	Building Location :			Conshohocken, Pennsylvania	
	Number of Stories :	N		16	
	Inter-story Height	h_s		12.08	
	Building Height :	h_n		193	ft
Table 1.1	Seismic Use Group :	I		I	
Table 9.1.4	Occupany Importance Factor :			1.00	
	Site Classification :			D	
Figure 9.4.1.1a	0.2s Acceleration :	S_S		0.31	g-s
Figure 9.4.1.1b	1s Acceleration :	S_1		0.08	g-s
Table 9.4.1.2.4a	Site Class Factor :	F_a		1.55	
Table 9.4.1.2.4b	Site Class Factor :	F_v		2.40	
	Adjusted Accelerations :	S_{MS}	$= F_a S_S$	0.481	g-s
		S_{M1}	$= F_v S_1$	0.180	g-s
	Design Spectral Response Accelerati	S_{DS}	$= (2/3)S_{MS}$	0.320	g-s
		S_{D1}	$= (2/3)S_{M1}$	0.120	g-s
Table 9.4.2.1a	Seismic Design Category :			**B**	
Table 9.4.2.1b	Both design category B				
	N-S Direction				
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		3	
	Seismic Response Coefficient :	$C_{s, N-S}$	$= S_{DS}/(R_{N-S}/I)$	**0.107**	
Table 9.5.5.3.2		$C_{T, N-S}$		0.028	
Table 9.5.5.3.2	(moment frames only)	x		0.80	
	Approximate Period of Structure :	T_{N-S}	$= C_{T, N-S} h_n^x$	1.89	
	Seismic Response Coefficient need				
	not be greater than	$C_{s\,max, N-S}$	$S_{D1}/T(R_{N-S}/I)$	**0.021**	
	and	$C_{s\,min}$	$= 0.044 I S_{DS}$	**0.0141**	
			Seismic Response Coefficient ($C_{s, N-S}$)	**0.021**	
	E-W Direction				
Table 9.5.2.2	Response Modification Factor :	R_{N-S}		3	
	Seismic Response Coefficient :	$C_{s, EW}$	$= S_{DS}/(R_{E-W}/I)$	**0.107**	
Table 9.5.5.3.2		$C_{T, EW}$		0.02	
Table 9.5.5.3.2	(moment and braced frame)	x		0.75	
	Approximate Period of Structure :	T_{N-S}	$= C_{T, E-W} h_n^x$	1.04	
	Seismic Response Coefficient need				
	not be greater than	$C_{s\,max, N-S}$	$S_{D1}/T(R_{E-W}/I)$	**0.039**	
	and	$C_{s\,min}$	$= 0.044 I S_{DS}$	**0.0141**	
			Seismic Response Coefficient ($C_{s, E-W}$)	**0.039**	

Table A.6- **Building Weight Calculations**

ASCE7-02 Chapter 9- Seismic Analysis

Load Summary for Building Weight

i. Roof :

Dead

Membrane	1.0	psf
Rigid Insulation	2.0	psf
Metal Roof Deck	2.0	psf
Roof Framing	12.0	psf
M&E Services	5.0	psf
TOTAL q_{roof} :	**22**	psf of roof area

ii. All other Floors :

Dead

Flooring	1.0	psf
Concrete Slab	2.0	psf
3" Metal Deck	2.0	psf
Structural Steel	5.0	psf
0.5" Drywall Ceiling	5.0	psf
M&E Services	5.0	psf
Live		
Moveable Partition	20.0	psf
Office Live Load	30.0	
TOTAL q_{floor} :	**70.0**	psf of floor area

iii. Perimeter Wall:

Dead

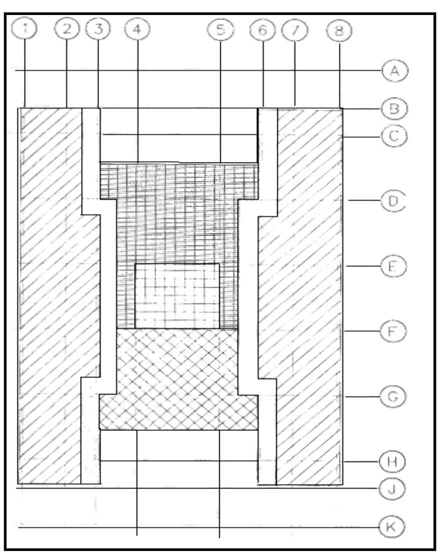
Exterior Wall, q_{wall} :	**180.0**	psf

iv. Weight of Roof

	(PSF)	Area	
Factored Roof Load	68	7500 ft^2	510000
Mechanical Load	330	2500 ft^2	825000
Elevator	250	790 ft^2	197500
Cooling Tower	350	2050 ft^2	717500
Snow	120	4000 ft^2	480000
Additional Dead Load	22	16840 ft^2	370480
		Total Roof Load (kips)	3100

Building Dimensions:

Length	196	ft
Width	118	ft
Gross Floor Area	23128	ft^2
Area of Voids	1065	ft^2
Total Floor Area	22063	ft^2
Weight of each floor	1545	kips
$w_{floors} = (N-1)w_{per\ floor} =$	23168	kips
Total Building Weight	26268	kips



(Not to Scale)

- Roof Load

-Mechanical Load

-Elevator Room Load

-Cooling Towers

-Snow Drift

Figure A.1- Shows the load distribution over the penthouse level of the structure. This distribution was used to determine the weight of the roof for the seismic analysis.

Appendix B

(Drawings not to scale)

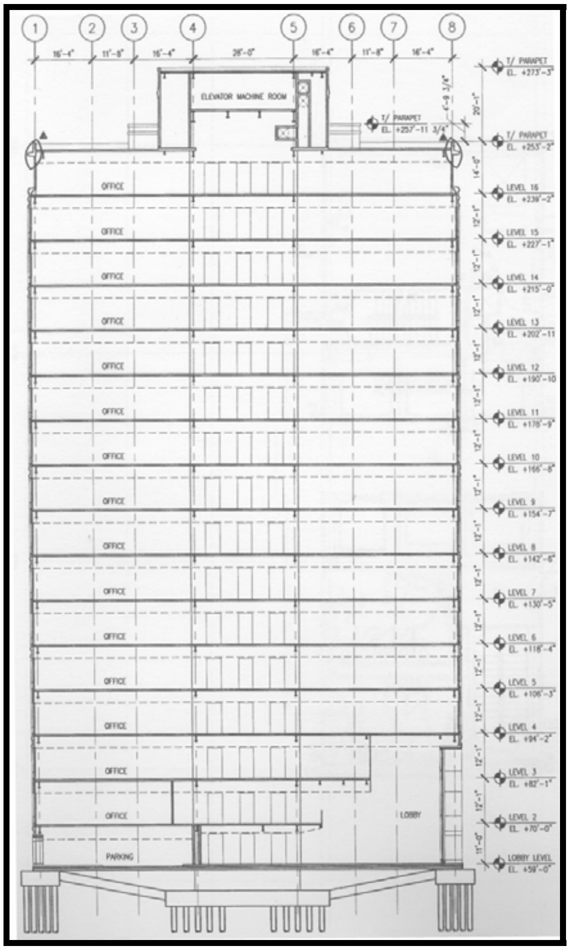


Figure B.1- East/West Section

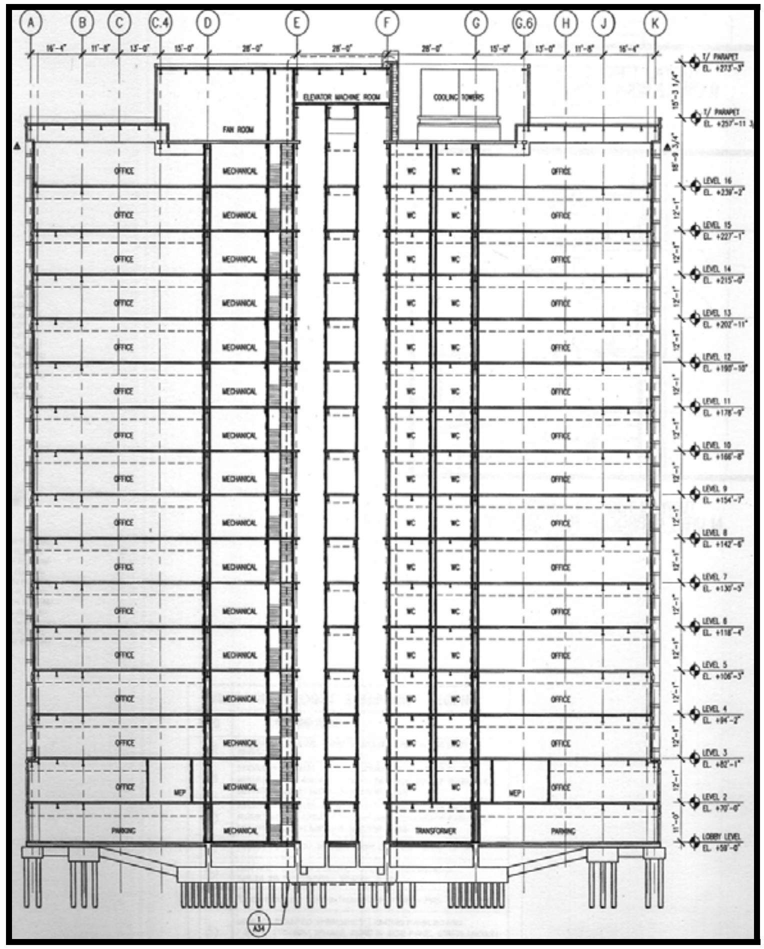


Figure B.2- North/South Section

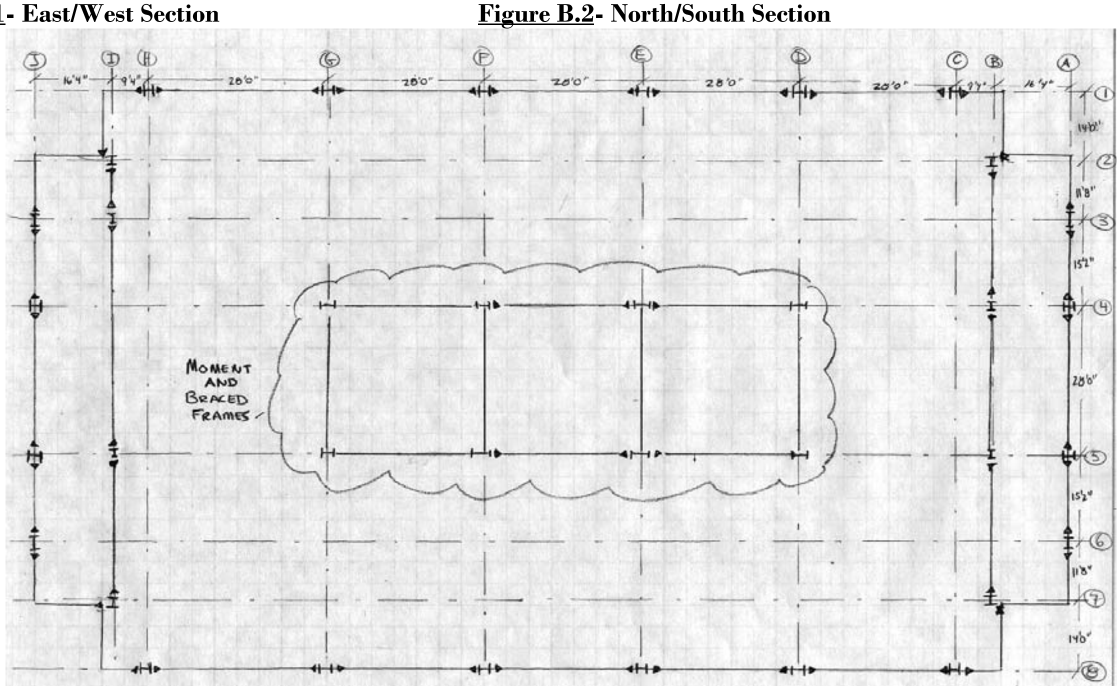
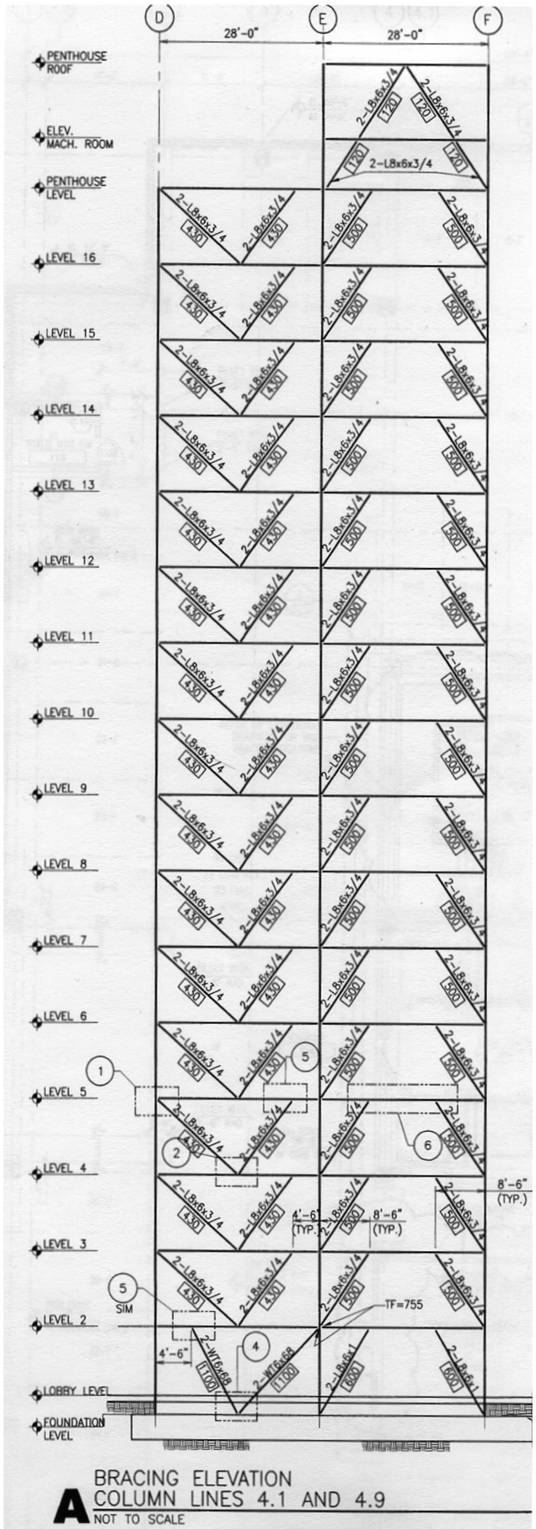
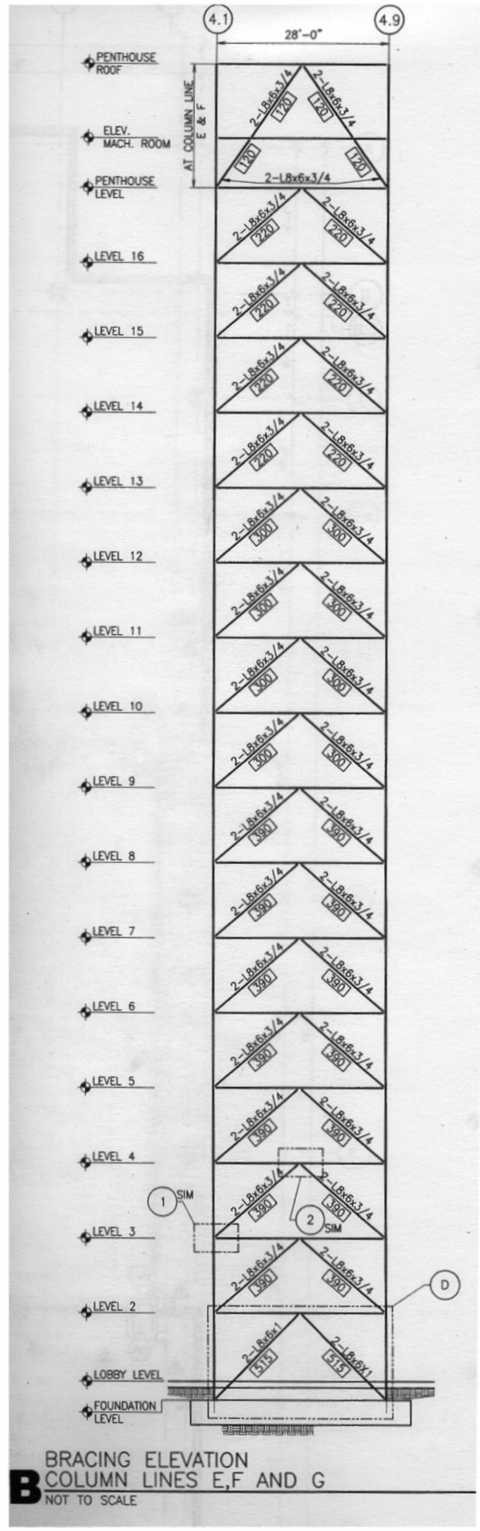


Figure B.3- Typical Framing Plan (Floors 3-15)

<u>Figures B.4 and B.5</u>- Typical Braced Frame elevations





Appendix C

BEAM CHECK:

TYPICAL FLOOR LOADING (LEVEL 6):

LIVE LOAD: 50 psf OFFICE
20 psf PARTITIONS
TOTAL 70 psf

DEAD LOAD: 20 psf SUPERIMPOSED D.L.
5 psf CEILING, MECH, ELEC., PLUMB.
TOTAL 25 psf

CONC. SLAB: 115 psf $(3\frac{1}{4}" + 2"/2)/12 = 40.7$ psf

LIVE LOAD REDUCTION:

$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$ $A_I = 2A_{TRIB}$

$L = L_o\left(.25 + \frac{15}{\sqrt{827}}\right)$ $A_I = 2(9.33')(44.33')$

$L = .77L_o > .60$ $A_I = 827\ ft^2$

$\therefore$ OK TO USE REDUCTION OF $.77L_o$

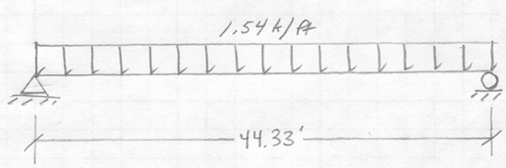
TOTAL FACTORED LOAD

$W_u = 1.6L + 1.2D$

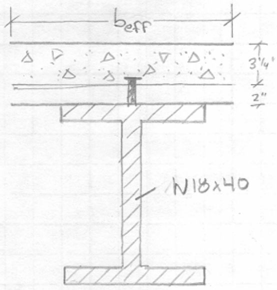
$W_u = \left[1.6(.77(70\ psf)) + 1.2(40.7 + 25\ psf)\right] 9.33'/1000$

$W_u = 1.54\ k/ft$

SIMPLE BEAM:



BEAM CHECK (CONT.)



$W_u = 1.54\ k/ft$

$f'_c = 4000\ psi$
$F_y = 60\ ksi$

$M_u = \dfrac{w\ell^2}{8}$

$M_u = \dfrac{(1.54\ k/ft)(44.33^2)}{8}$

$M_u = 378.3\ ft.k$

$b_{eff} = \begin{cases} 1/4\,(44.33 \cdot 12) = 133\ in \\ 9.33 \cdot 12 = 112\ in \end{cases}$

$b_{eff} = 112\ in$

ASSUME: $a = 1''$

$Y_2 = 5.25'' - 1''/2 = 4.75''$

TABLE 5-14 W18x40 $\phi_b M_p = 294\ ft.k < 378.3\ ft.k$

∴ CHECK COMPOSITE ACTION

$Y_2 = 4.75''$

TABLE 5-14 (p 5-143) LRFD
- IN W18x40, PNA CAN BE ANYWHERE IN SECTION AND STILL MEET $M_u < \phi_b M_n$
→ TRY LOCATION 4

Y_2	$\phi_b M_n$
4.5"	483
5"	496

$\phi_b M_n = 483 + \dfrac{496-483}{2} = 489.5\ ft.k$

@ LOCATION 4 $\Sigma Q_n = 353\ k$

CHECK $a = 1''$

$C_c = .85 f'_c\, b a = \Sigma Q_n$

$a = \dfrac{\Sigma Q_n}{.85 f'_c\, b} = \dfrac{353^k}{.85(4ksi)(112in)} = 0.93 in < 1''$ ∴ OK

$Y_2 = 5\tfrac{1}{4}'' - \dfrac{.93''}{2} = 4.78\ in \rightarrow Y_2 = 5\ in$

TABLE 5-14 @ LOCATION 4, $Y_2 = 5\ in$ $M_n = 496'k > 378.3'k$

∴ OK

BEAM CHECK (CONT):

$\phi_b M_n = 496'^k$ $\Sigma Q_n = 353\,k$

SHEAR STUD CAPACITY: 21^k/STUD

OF STUDS $= \frac{353^k}{21^k} = 16.8$ OR 17 STUDS

DEFLECTION CHECK:

$\Delta = \frac{5 w l^4}{384 EI}$ $1.54\,k/ft \cdot \frac{1ft}{12in} = .128\,k/in$

$\Delta = \frac{5(.128^k/in)(532 in^4)}{384(29000)(612 in^4)} = 2.5\,in$

$\frac{l}{360} = 1.5"$ ∴ MUST CAMBER!

COMMENTS:

- ALL LOADS USED WERE SPECIFIED IN STRUCTURAL DRAWINGS
- BEAMS WERE DESIGNED AS FULL COMPOSITE
- THE COMPOSITE DECK WAS DESIGNED TO OBTAIN A MINIMUM SAFETY FACTOR OF TWO.