WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS

CRANBERRY, PA

Technical Assignment 1

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Executive Summary

The purpose of the technical report is to analyze the existing conditions of the site of the Westinghouse Electric Company Corporate Headquarters Building One and design as was designed by LLI Engineering and IKM, Inc. One of the other goals of the report is to understand the structural design of the building so it can correctly be analyzed and adjusted in future reports.

The Westinghouse Electric Company Corporate Headquarters will be a three building campus with site features such as asphalt walking paths and volleyball courts on eighty-three acres in Cranberry, PA. For the purpose of the project, only Building One will be analyzed as the other two are considered a separate project by all parties involved. The truncated V-shape building has been given a look of importance with polished concrete block merging into brick stepped-out columns to accentuate the verticality of the five-story 74'-6" tall structure.

The report discusses the main structural framing, foundation types, and material types. The gravity load system is a composite metal deck and beam system using 2" 22 gage deck and 2-1/2" topping. Lateral load resisting systems are mentioned, but not discussed in detail as the system is an ordinary moment frame connection for wind moment connections. Spot checks of several members for gravity loading using Load and Resistance Factor Design (LRFD) are included in Appendix D of the report. All members looked at in the report have been determined to be acceptable.

All wind and seismic loads were determined using ACSE 7-05 chapters 6, 11, and 12. The wind loads were determined to be reasonable as well as the seismic. The seismic loads were calculated and then studied with more care. The design professionals used an R=3.5, which is certainly attainable for this building. However, due to the extra care buildings with an R>3 require, the buildings have a higher potential for incorrect installation. Therefore, the building was studied using an R=3 and an R=3.5 so a comparison could be made with the design professionals' values.

The appendices of the technical report include a typical floor plan with spot check columns labeled detailed wind calculations, seismic calculations, and spot checks.

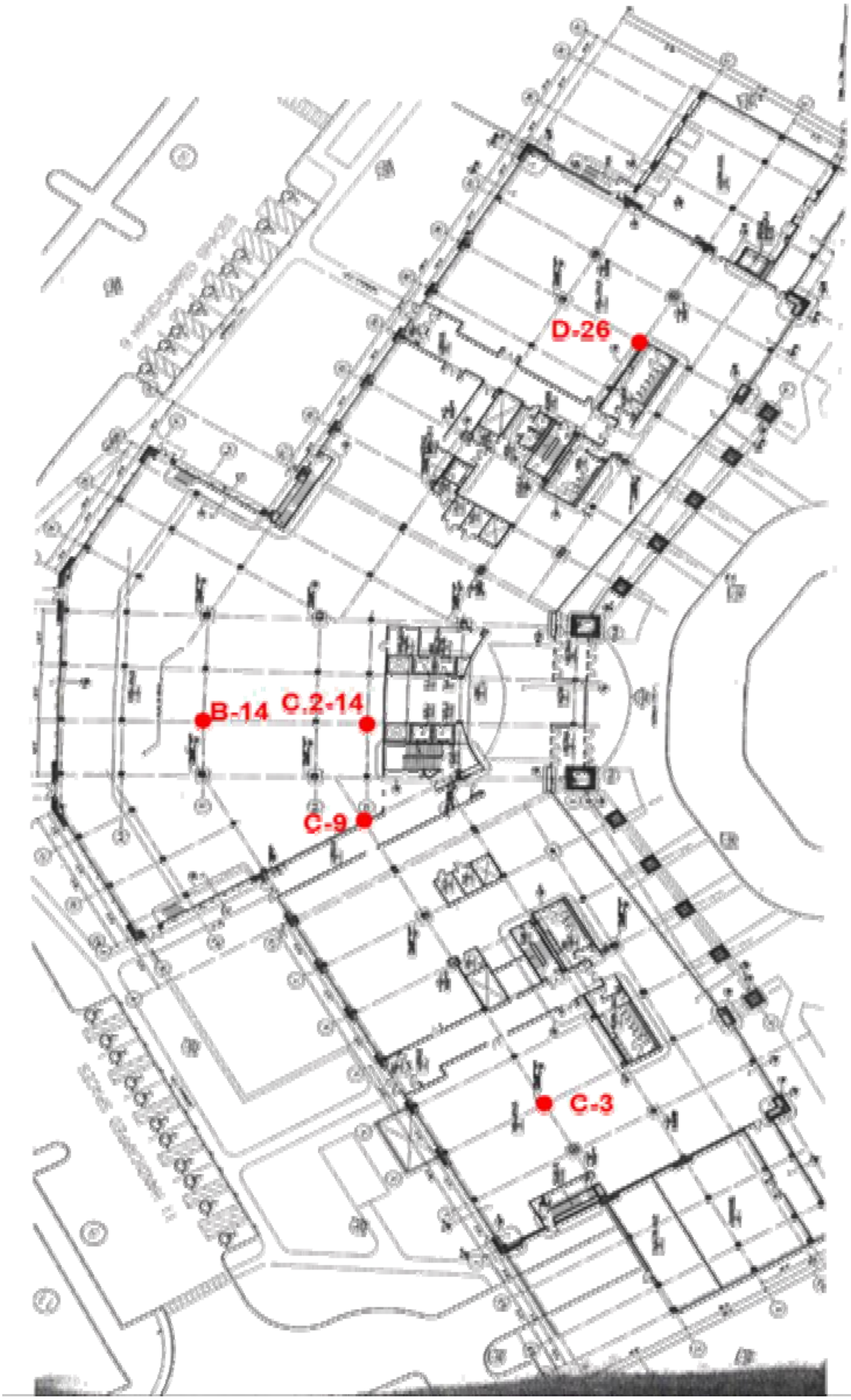
INTRODUCTION

WESTINGHOUSE ELECTRIC COMPANY CORPORATE HEADQUARTERS BUILDING ONE

The Corporate Headquarters building for the Westinghouse Electric Company is located in Cranberry, Pennsylvania. Just north of the city in Butler County, the site is on 83 acres in an office park easily accessible by I-79 and PA-228. With five above grade floors and a full 17' high basement, Building One will be the main building on this campus. Complete with cafeteria, gym, locker rooms, offices, and executive conference rooms, the flagship building comes well equipped. At 434,800 square feet, the building makes quite an architectural statement.

The main building utilizes a powerful entrance with a two-story atrium to express its importance. The first floor also has a height of 18'-0" to emphasize a larger space while floors two through four have floor-to-floor heights of 14'-0". The fifth floor has a height of 14'-6". Building one has a total height of 74'-6" above grade. Aluminum and glass curtain walls add light and make the building feel more open while polished concrete at the base of the brick façade accentuate the height. The foundation system consists of caissons in addition to some spread footings and grade beams. A typical bay is 45'-0" by 24'-0", and uses a steel system with composite beams and deck. In most of the building, the girders are not composite, but the beams framing into the girders have some composite action. The floor system is a 2" 22 gage steel deck with 2-1/2" of lightweight concrete topping. The Westinghouse Electric Company Corporate Headquarters Building One has two expansion joints present, thus creating essentially three structural buildings inside of one. The expansion joints create the East, Center, and West parts of the building. These joints can be seen along column lines 7.9 and 8 between the east and center portions, and column lines 21 and 21.1 between the center and west parts of the building.

Lateral loads were calculated in preliminary fashion and torsion was not considered. Wind and seismic loads were calculated using ASCE 7-05. Member checks were performed for a column on each floor, the deck, a beam, and a girder.



Typical floor plan with locations of spot checked members labeled

STRUCTURAL SYSTEMS

FOUNDATIONS

Caissons are the main element in the foundation system. Each was designed to carry 8,000 psf. The caissons range from 36" to 84" in diameter and from 8'-0" to 30'-8" in height. On top of each caisson, there is a 2'-6" cap with #6@8" each way on the top and the bottom and base plates for the columns. The 5" slab on grade in the basement bears directly on the soil and the thickened slabs under the non-load bearing walls. On the south side and the east portion of the building, where the sixty-seven caissons (67) are not present, there are spread footings or grade beams. The sub-grade walls in the basement (referred to in drawings as grade beams) range from 1'-4" to 1'-8" wide and are 14'-4" deep. The bottom reinforcement in the grade beams is mainly (3) #6, but varies from #6 to #9 and in number. Top reinforcement also varies from #6 to #9 and from two bars to four bars. All end reinforcing bars are #6, but vary from two bars to four bars.

FLOOR SYSTEM

The floor system for the corporate headquarters main building consists of 2" 22 gage metal deck with 2 ½" lightweight concrete topping, for a total slab depth of 4 ½". The typical bay size of this composite steel system is 24'-0" by 45'-0". W21 beams (W21x44 typ.) spaced 24'-0" on center and W18x35 beams spaced 8'-0" on center support the deck and transfer the load to the W24 girders (W24x55 typ.). The girders then continue to transfer the load to the columns. The 5" thick slab-on-grade in the basement of the headquarters is the exception to the typical floors. The roof uses a different system consisting of 1 ½" 20 gage roof deck, steel beams and steel K series joists. However, the roof system uses 2" 20 gage metal deck with a 2 ½" lightweight concrete topping at the penthouse. Where the penthouse is absent, roof uses a fully adhered EPDM roofing system including the membrane over ½" protection board over tapered insulation over 5/8" type X GWB over the roof decking.

LATERAL SYSTEM

The Westinghouse Corporate Headquarters Building One uses moment connections at every column to resist lateral loads.

COLUMNS

The columns used in the headquarters are typical for a mid-rise building. The large columns in the basement and first floor of the building are W36x230 at the largest, but typically are W14x90. The W36x230 columns are so large due to the entire front façade of the building bearing on a W36x230 beam and the two columns. On the roof, any columns that do not continue up from the fifth floor are W10x49 or W10x33. The rest of the building is generally the same size, of course with some smaller sizes of columns, such as W10's on the fifth and roof levels. The base plates have four possible layouts and range in thickness from 1 ¾" to 3".

CODE AND DESIGN REQUIREMENTS

These are the design standards, codes, and design criteria used by the design professional and in the calculations for this report.

APPLICABLE DESIGN STANDARDS

THE 2006 INTERNATIONAL BUILDING CODE

ACI 318-05 (REINFORCED CONCRETE DESIGN)

AISC STEEL CONSTRUCTION MANUAL, 13TH EDITION

ACI 530 (MASONRY STRUCTURES)

ASCE 7-05 (MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES)

DEFLECTION CRITERIA

FLOOR DEFLECTION CRITERIA

L/240 TOTAL LOAD

L/360 LIVE LOAD

L/600 CURTAIN WALL LOAD

LATERAL DEFLECTION CRITERIA

H/400 TOTAL ALLOWABLE WIND DRIFT

H/400 STORY WIND DRIFT

H/50 TOTAL ALLOWABLE SEISMIC DRIFT (Δ=0.02H_{SX} FROM TABLE 12.12-1 ASCE 7-05)

Materials

The materials used in the Westinghouse Electric Company Corporate Headquarters as listed on the general notes page of the structural drawing set are as follows and were used in design and analysis as appropriate.

Concrete

Freezing Temperature Exposure	Air entrained (6% ±1%)
Slab-on-grade	4,000 PSI
Slab-on-deck	4,000 PSI
Caissons	3,000 PSI
Footings and Caisson Caps	3,000 PSI
Walls and Piers	4,000 PSI
Over excavation fill	2,000 PSI

Reinforcing Steel

Reinforcing Bar	ASTM A-615
Welded Wire Fabric	ASTM A-185

Structural Steel

W-Shapes	ASTM A-992
C-Shapes	ASTM A-36
Steel Pipe	ASTM A-501
Tubes	ASTM A-500 Grade B

Metal Deck

Bolts	ASTM A-325, ¾" diameter
Deck	ASTM A611 Grade C or D
Studs	¾"x 3 ½" headed stud

Masonry

CMU	ASTM C-90
Concrete Brick	ASTM C-55 type N-1
Mortar	ASTM C-270
Grout	ASTM C-476

GRAVITY AND LATERAL LOADS

The loads on the building are applied as such based on the design professional's specification on the drawings. It is understood these values are conservative since the live load of 80 PSF is used everywhere on the upper floors and a partition load is also used. This is discussed further in other sections of the report. The load combinations from IBC 2006 were taken into consideration and the boxed ones were used for the lateral analysis of the frames in the building.

Dead Loads

Construction Dead Load

Concrete	115 PCF
Steel	490 PCF
Partitions	10 PSF
M.E.P.	5 PSF
Finishes	3 PSF

Live Loads

Public Areas	100 PSF
Lobbies	100 PSF
Corridors above 1st	80 PSF
Office	50 PSF
Mechanical	150 PSF
Stairs	100 PSF

From IBC 2006:
1605.2.1 Basic Load Combinations
(As applied to this Report)

1.4 D	Eq 16-1
1.2D + 1.6L	Eq 16-2
1.2D+1.0L	Eq 16-3
1.2D+0.8W	Eq 16-3
1.2D+1.0L+1.6W	Eq 16-4
1.2D+1.0E+1.0L	Eq 16-5
0.9D+1.6W	Eq 16-6
0.9D+1.0E	Eq 16-7

WIND DESIGN

Wind loads were analyzed using section 6.5 of ASCE 7-05. The loads were calculated as well as analyzed for the North-South direction in addition to the East-West direction. Despite the fact two other buildings will be in place on each side of Building One of the headquarters campus, the wind loads were calculated as if buildings two and three were non-existent. Appendix B contains a detailed analysis of this calculation. The footprint was considered to be rectangular with the one of the sides the maximum length of the exterior of the building for this analysis. Wind effects on the roof were also neglected. The fundamental period was determined using a simplified method. Since the building is much longer in the chosen N-S direction than the E-W direction, the wind analysis shows the N-S direction controlling the design.

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.908	13.924	11.09	-8.13	-10.38	11.86	-4.84	-10.69
14	5	60	0.85	13.034	10.54	-8.13	-10.38	11.67	-4.84	-10.69
14	4	46	0.79	12.114	9.97	-8.13	-10.38	11.26	-4.84	-10.69
14	3	32	0.712	10.918	9.24	-8.13	-10.38	10.85	-4.84	-10.69
18	2	18	0.59	9.201	7.89	-8.13	-10.38	10.64	-4.84	-10.69

Figure 1: Wind Pressure with respect to height

The wind story forces are summarized in these pictures of each side of the building. The story forces are on the left and the story shears are on the right side of the pictures.

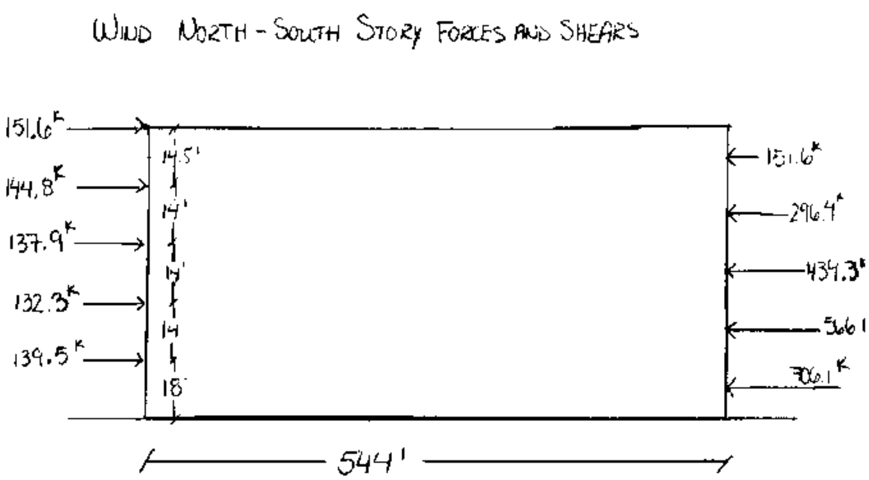


Figure 2: N-S Distribution of Wind Pressure

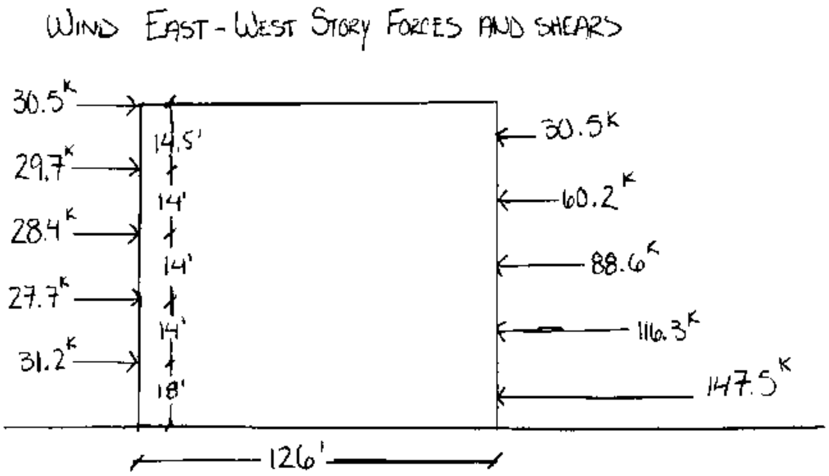


Figure 3: E-W Distribution of Wind Pressure

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	151.6	30.5	0	0	2198.6	442.4
5	144.8	29.7	151.6	30.5	2026.7	415.2
4	137.9	28.4	296.4	60.2	1930.7	397.7
3	132.3	27.7	434.3	88.6	1852.1	387.5
2	139.5	31.2	566.6	116.3	2511.1	562.0
Total	706.1	147.5	706.1	147.5	10519.2	2204.8

Note: Total Base Shear includes load from Windward and Leeward pressures

Seismic Design and Analysis

Seismic loads were calculated using ASCE 7-05, Chapters 11 and 12. Since the design professional used a response modification factor of 3.5, R= 3.5 was used. However, because not all structures are constructed to the higher standards used for this building, an R-value of 3 was also used for comparison. The site falls into site class D, seismic category B, and occupancy category II. Factors and accelerations were calculated using ASCE 7-05. After analysis, a response factor of R=3.0 would result in higher story forces and therefore higher shears and moments than the R=3.5 from the analysis performed and the design professionals' analysis.

The values calculated for R=3.0 and R=3.5 are not very different. More conclusive calculations are shown in the Appendix. The values used for all the seismic calculations, including the MCE Spectral Response Acceleration short and 1 second, the Design Spectral Acceleration short and for 1 second, and the Spectral Response Acceleration short and 1 second all are reasonably close to the United States Geological Survey website's calculations for the site. The weight of each of the floors of the building was calculated using slabs, exact weights of the columns, and approximate weights of all beams and connections. Some of the floor calculations are shown in the Appendix.

Seismic Design Values, ASCE 7-05		
Occupancy	II	Table 1-1
Importance Factor	I= 1	Table 11.5-1
Site Class	D	Table 20.3-1
Spectral Response Acceleration, short	S_S= 0.12	Figure 22-1
Spectral Response Acceleration, 1 sec	S_1= 0.046	Figure 22-2
Site Coefficient F_a	F_a= 1.6	Table 11.4-1
Site Coefficient F_V	F_V= 2.4	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{MS}= 0.192	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}= 0.1104	Eq. 11.4-2
Design Spectral Acceleration, short	S_{DS}= 0.128	Eq. 11.4-3
Design Spectral Acceleration, 1 sec	S_{D1}= 0.0736	Eq. 11.4-4
Seismic Design Category	B	Table 11.6-1

Figure 5: Seismic Design Values

Seismic Design Values, ASCE 7-05			
Response Modification Coefficient	R= 3	R= 3.5	Table 12.2-1
Coefficient	C_U= 1.7	C_U= 1.7	Table 12.8-1
Fundamental Period	T= 1.497	T= 1.497	Sec. 12.8.2
Seismic Response Coefficient	C_S= 0.016	C_S= 0.014	Eq. 12.8-3
Building Height (above grade)	h= 74.5	h= 74.5	

Figure 6: Seismic Design Values for R=3 and R=3.5

Figure 6: Story Shears, Forces and Moments for R=3.0

Figure 7: Story Shears, Forces, and Moments for R=3.5

Figure 8: Story Shears, Forces, and Moments, R=3.5 and Cs from design professional, conservative values

MEMBER CHECKS

Five members were chosen to determine the adequacy of their loading and size. All five members chosen were proven to be adequate. In actuality, for pure gravity loading, the columns are shown as oversized. This is due to the moment connections for the lateral system. The columns must take more lateral load in addition to the gravity loads. A composite girder was also checked for this report and found to be adequate. Appendix D contains these calculations. Aside from this column, the rest were calculated using a simplified method.

B-14 1ST FLOOR

COLUMN SIZE: W 14x211
HEIGHT: H=18'
$P_u = 734^k$
$\phi P_n = 2270^k$

$A_g = 62.0 \text{ in}^2$
$I_x = 2660 \text{ in}^4$ $r_x = 6.55$ IN
$I_y = 1030 \text{ in}^4$ $r_y = 4.07$ IN

AREA $= 38.125 \times 24 = 915 \text{ ft}^2$
DEAD $= 34$ PSF $\times 1.2 \times 5$
LIVE $= 80$ PSF $\times 1.6 \times 4$
ST $= 18$ PSF $\times 1.2 \times 4$
734^k

$$\frac{kL}{R_x} = \frac{18(12)}{6.55} = 33$$

$$\frac{kL}{R_y} = \frac{18(12)}{4.07} = 53.1$$ ← CONTROLS

$$\frac{kL}{r} \leq 4.71\sqrt{E/f_y} = 4.71\sqrt{29000/50} = 113.4$$

$53 < 113.4$ ∴ INELASTIC BEHAVIOR

$$F_{CR} = \left[0.658^{F_y/F_e}\right]F_y = 0.658^{50/101.6}\,50 = 40.7$$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2} = \frac{\pi^2(29000)}{(53.07)^2} = 101.6^k$$

$$P_n = F_{CR}A_g = 40.7(62) = 2523.4^k$$

$$\phi P_n = 0.9(2523.4) = 2271^k$$

TABLE 4-1 AISC 13TH ED

* WIND CONTROLLED MOMENT CONNECTION ON COLUMN ACCOUNTS FOR WHAT APPEARS TO BE OVERSIZING ERROR

CONCLUSIONS

The Westinghouse Electric Company Corporate Headquarters is a typical building with steel framing and composite floors. The spots checks chosen proved the building worked, but according to the loads used in this report, the columns were over-designed. The members appear to be over-designed due to the wind moment connections at every column and the fact that the moment connections were not taken into consideration. It is important to note wind and seismic loading were both ignored in the analysis of each member and the lateral system is a wind moment connection on every column. The wind moment connection is most likely the reason the columns appear to be over-designed, but the moment connection is the only lateral framing system which means the columns take more load than those with which they were checked. Also, second order effects and column and beam stiffness were not taken into consideration. In later reports and calculations, these effects will be addressed and accounted for in design. All these items taken into consideration, it is quite acceptable for a first analysis.

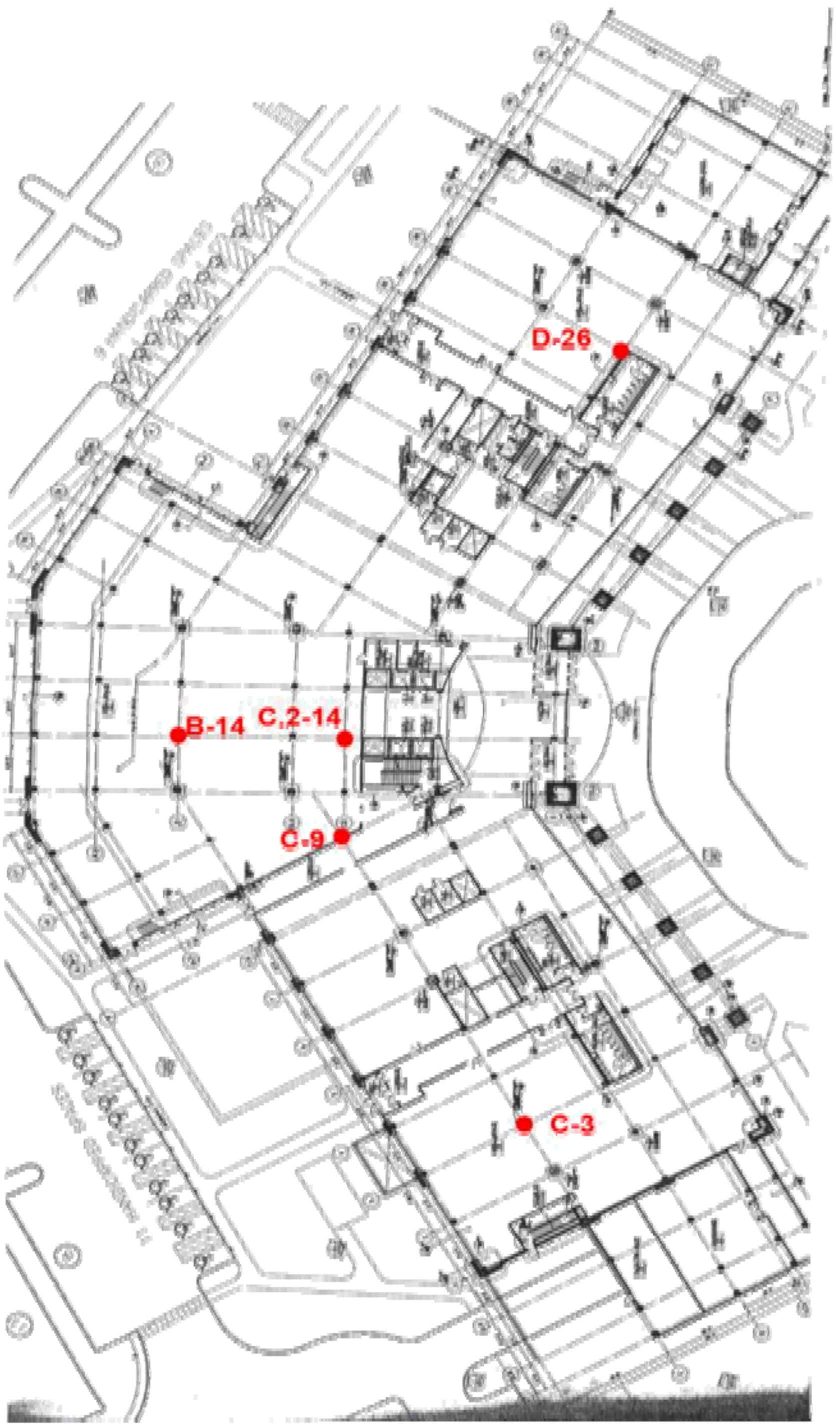
The lateral loads were only analyzed in a preliminary manner. The seismic loads have been are reasonably comparable to the design professionals' values for the same loads. Both R=3.5 values are within reason to each other, thus proving the accuracy of the calculations. An R=3 value was used since it is the highest value that does not require extra inspection and care in the design of the connections. The R values were analyzed with two different values for comparison since the design professional used R=3.5 and the typical value was used as a comparative value. The R=3.0 value gives the highest story shears and story forces. In the seismic loading, torsion was not considered, and the loads are therefore not as high as they have the potential to be. The wind design did not consider torsion either. The wind analysis was performed using ASCE 7-05 as a guide and making sure to meet all the code requirements.

The main difference in the spot checks stems back to the lateral system of the building. If there were different framing, braces or other types, the loads on the columns could be greatly reduced, and thus the size reduced. So if the size of each column could be reduced, money could potentially be saved, but the appearance and grandeur of the corporate headquarters building could also be greatly reduced. One of the main reasons for a separate headquarters building is to have it as a symbol of the building and the current design certainly does provide a symbol. The building currently has a composite metal deck and beam system. The deck and beams were found to be adequate in the member check portion of this report and the appendix.

Further calculations, tables, and notes are contained in the appendices.

Appendix A: Typical Floor Layout

First Floor Layout



APPENDIX B: WIND LOAD CALCULATIONS

MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1
Wind Directionality Factor (Kd)	0.85
Topographic Factor (Kzt)	1

WHOLE BUILDING L/B AND VALUES

		L/B	C_p
East-West Direction			
	Windward	4.317	0.8
	Leeward	4.317	-0.2
	Sidewall	4.317	-0.7
North-South Direction			
	Windward	0.232	0.8
	Leeward	0.232	-0.5
	Sidewall	0.232	-0.7

	Wind Direction	
Variable	N-S	E-W
Stiffness	Flex	Flex
B	544	126
L	126	544
h	74.5	74.5
z	30'	30'
ℓ	320	320
ϵ	0.333	0.333
α	0.25	0.25
β	0.05	0.05
V	90	90
V_z	64.082	64.082
L_z	354.06	354.06
n_1	0.706	0.706
N_1	3.90	3.90
R_n	0.059	0.059
R_h	0.230	0.230
R_b	0.036	0.144
R_L	0.046	0.011
b	0.45	0.45
R	0.073	0.145
I_z	0.285	0.285
g_R	4.106	4.106
q_p	13.924	13.924
g_V	3.4	3.4
Q	0.726	0.833
G_f	0.771	0.839

WIND CALCULATIONS CONTINUED:

q_p = $0.00256\ K_h K_{zt} K_d V^2 I$= 13.924

GCpn= 1.5 -1

Pp = q_pGCpn = 20.885 -13.924

$n_1 = \dfrac{22.2}{H^{0.8}} =$ 0.706

g_Q = g_V = 3.4

g_R = √(2ln(3600n_1)) +0.577/(√(2ln(3600n1))= 4.106

z= 0.6h = 44.7

z_{min} = 30'

$I_z = c(33/z)^{1/6}$= 0.285

$L_z = l(z/33)^{\epsilon}$ = 354.06

$Q_{N\text{-}S}$= $\sqrt{(1/(1+0.63(\ B+h/L_z)^{0.63}))}$ = 0.726

$Q_{E\text{-}W}$= $\sqrt{(1/(1+0.63(\ B+h/L_z)^{0.63}))}$ = 0.833

$V_z = b(z/33)^{\alpha}V(88/60)$= 64.082

N_1= $n_1 L_z/V_z$ = 3.90

R_n= $7.47N_1/(1+10.3N_1)^{5/3}$= 0.059

R_h= $1/h-1/(2h^2)(1-e^{-2h})$ = 0.230

h= $4.6n_1h/V_z$= 3.774

WIND CALCULATIONS CONTINUED:

N-S

$R_b = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.036

$h = 4.6n_1B/V_z =$ 27.558

$R_L = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.046

$h = 15.4n_1L/V_z$ 21.36901

$R = \sqrt{((1/b)(R_nR_hR_b(0.53+0.47R_L)} =$ 0.073

$G_f = 0.925 [(1+1.7I_z\sqrt{(g_Q{}^2Q^2+g_R{}^2R^2)})/(1+1.7g_vI_z)] =$ 0.771

E-W

$R_b = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.144

$h = 4.6n_1B/V_z =$ 6.383

$R_L = 1/h - 1/(2h^2)(1-e^{-2h}) =$ 0.011

$h = 15.4n_1L/V_z$ 92.25985

$R = \sqrt{((1/b)(R_nR_hR_b(0.53+0.47R_L)} =$ 0.145

$G_f = 0.925[(1+1.7I_z\sqrt{(g_Q{}^2Q^2+g_R{}^2R^2)})/(1+1.7g_vI_z)] =$ 0.839

$g_z = 0.00256K_zK_{zt}K_dV^2I =$ 15.33427 K_z

WIND CALCULATIONS CONTINUED:

Design Wind Pressures p in E-W Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	9.35	2.51	6.84	11.86
	70	13.65	9.17	2.51	6.66	11.67
	60	13.03	8.75	2.51	6.25	11.26
	50	12.42	8.34	2.51	5.84	10.85
	46	12.11	8.14	2.51	5.63	10.64
	40	11.65	7.83	2.51	5.32	10.33
	32	10.92	7.33	2.51	4.83	9.84
	30	10.73	7.21	2.51	4.70	9.72
	25	10.12	6.80	2.51	4.29	9.30
	20	9.51	6.39	2.51	3.88	8.89
	18	9.20	6.18	2.51	3.67	8.69
	15	8.74	5.87	2.51	3.36	8.38
Leeward	All	13.92	-2.34	2.51	-4.84	0.17
Side	All	13.92	-8.18	2.51	-10.69	-5.68

Design Wind Pressures p in N-S Direction (Table 5.41)						
Location	Height above Ground Level z (ft)	q(psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi}	Net Pressure p (psf)	
					+Gcpi	-Gcpi
Windward	74.5	13.92	8.58	2.51	6.08	11.09
	70	13.65	8.41	2.51	5.91	10.92
	60	13.03	8.03	2.51	5.53	10.54
	50	12.42	7.66	2.51	5.15	10.16
	46	12.11	7.47	2.51	4.96	9.97
	40	11.65	7.18	2.51	4.68	9.69
	32	10.92	6.73	2.51	4.22	9.24
	30	10.12	6.24	2.51	3.73	8.74
	25	9.51	5.86	2.51	3.35	8.37
	20	8.74	5.39	2.51	2.88	7.89
	18	8.74	5.39	2.51	2.88	7.89
	15	8.74	5.39	2.51	2.88	7.89
Leeward	All	14.61	-5.63	2.51	-8.13	-3.12
Side	All	14.61	-7.88	2.51	-10.38	-5.37

WIND CALCULATIONS CONTINUED:

Floor Heights	Level	Total Height	K_Z	q_Z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Sidewall
14.5	Roof	74.5	0.908	13.924	11.09	-8.13	-10.38	11.86	-4.84	-10.69
14	5	60	0.85	13.034	10.54	-8.13	-10.38	11.67	-4.84	-10.69
14	4	46	0.79	12.114	9.97	-8.13	-10.38	11.26	-4.84	-10.69
14	3	32	0.712	10.918	9.24	-8.13	-10.38	10.85	-4.84	-10.69
18	2	18	0.59	9.201	7.89	-8.13	-10.38	10.64	-4.84	-10.69

Level	Wind Design					
	Load (kips)		Shear (kips)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	151.6	30.5	0	0	2198.6	442.4
5	144.8	29.7	151.6	30.5	2026.7	415.2
4	137.9	28.4	296.4	60.2	1930.7	397.7
3	132.3	27.7	434.3	88.6	1852.1	387.5
2	139.5	31.2	566.6	116.3	2511.1	562.0
Total	706.1	147.5	706.1	147.5	10519.2	2204.8

Note: Total Base Shear includes load from Windward and Leeward pressures

APPENDIX C: SEISMIC LOAD CALCULATIONS

Seismic Design Values, ASCE 7-05			
Response Modification Coefficient	R= 3	R= 3.5	Table 12.2-1
Coefficient	C_U= 1.7	C_U= 1.7	Table 12.8-1
Fundamental Period	T= 1.497	T= 1.497	Sec. 12.8.2
Seismic Response Coefficient	C_S= 0.028	C_S= 0.014	Eq. 12.8-3
Building Height (above grade)	h= 74.5	h= 74.5	

F_a Values (Table 11.4-1 ASCE 7-05)					
	S_S≤0.25	S_S=0.5	S_S=0.75	S_S=1.0	S_S≥1.25
D	1.6	1.4	1.2	1.2	1

F_V Values (Table 11.4-2 ASCE 7-05)					
	S_1≤0.1	S_1=0.3	S_1=0.3	S_1=0.4	S_1≥0.5
D	2.4	2	1.8	1.6	1.5

Calculated Values

S_S= 0.12 (From Figure 22-1)

S_1= 0.046 (From Figure 22-2)

S_{MS}= F_a*S_S = 0.192

S_{M1}= F_V*S_1 = 0.1104

S_{DS}= $2S_{MS}/3$= 0.128 A (Table 11.6-1)

S_{D1}= $2S_{M1}/3$= 0.0736 B (Table 11.6-2)

USGS Website Values

S_S= 0.125

S_1= 0.048

S_{MS}= 0.2

S_{M1}= 0.116

S_{DS}= 0.133

S_{D1}= 0.077

SEISMIC CALCULATIONS CONTINUED:

The values for all the seismic coefficients were determined using ASCE 7-05 equations and tables. The building was first confirmed as Seismic design category B by using Table 11.6-2 of ASCE 7-05. Once the design category had been confirmed, the approximate period was calculated by using equation 12.8-7 and table 12.8-2. Since ASCE 7-05 section 11.6 requires where an S_1 value is less than 0.75 the Seismic Design Category can be determined solely on table11.6-1 and 11.6-2 when T_a> $0.8T_S$, the period used to calculate drift is less than T_S, equation 12.8-2 is used to find C_S, and rigid diaphragms are present.

Calculated Values

S_S=	0.12	(From Figure 22-1)	
S_1=	0.046	(From Figure 22-2)	
S_{MS}=	F_a*S_S =	0.192	
S_{M1}=	F_V*S_1 =	0.1104	
S_{DS}=	$2S_{MS}/3$=	0.128	A (Table 11.6-1)
S_{D1}=	$2S_{M1}/3$=	0.0736	B (Table 11.6-2)

USGS Website Values

S_S= 0.125
S_1= 0.048
S_{MS}= 0.2
S_{M1}= 0.116
S_{DS}= 0.133
S_{D1}= 0.077

C_T =	0.028	(From Table 12.8-2)
X =	0.8	(From Table 12.8-2)

T_a=	$C_t h_n^x$ =	0.8808116	
T_s=	S_{D1}/S_{DS} =	0.575	
$0.8T_s$=	0.46	$<T_a$ therefore must use Table 11.6-1,2	

T_L= 12 (From Fig. 22-15 p. 228 ASCE 7-05)

C_S values were calculated according to Section 12.8.1.1 equations 12.8-2, 12.8-3, and 12.8-4 and checked against the minimum requirement from EQ 12.8-5 of $C_S \geq 0.01$. Equation 12.8-3 is a maximum for this structure, and equation 12.8-4 does not apply since equation 12.8-3 does. The values were then compared based on R and what the professional calculated.

		R=3	R=3.5	R=3.5, consultant
	$S_{DS}/(R/I)$ =	0.0427	0.0366	0.0382
C_S= MAX	$S_{D1}/(T*R/I)$ =	0.0164	0.0140	0.0147
for $T>T_L$	$S_{D1}T_L/(T^2R/I)$ =	0.3795	0.3253	0.1181

T= C_U*T_a = 1.4973798

k= 1.499 W= 28502.4

V= C_s*W

466.99 R=3.0
400.28 R=3.5
419.85 R=3.5 from consultant

Floor # 3					
Approx. Area:	72932 ft^2		Floor to Floor Height:	14 ft	
Slab:					
22 GA Deck	2				
Thickness=	2.5				
unit weight=	34 PSF				
total weight=	2480 kips				
Columns:					
Shape	Quantity	Unit Weight (lb/ft)	Column Height (ft)	Total Weight	
W14x99	19	99	14	26.3 kips	
W14x132	6	132	14	11.1 kips	
W14x120	18	120	14	30.2 kips	
W14x90	8	90	14	10.1 kips	
W14x109	6	109			
W14x145	12	145	14	24.4 kips	
W14x159	6	159	14	13.4 kips	
W14x82	12	82	14	13.8 kips	
Total Weight=	129.2 kips				
Beams Connections, Bracing, etc.					
allowance=	11 psf				
Total Weight=	802.3 kips				
Super Imposed Dead:					
partitions=	10 psf				
MEP=	5 psf				
Finishes=	3 psf				
Total Weight=	1312.8 kips				
TOTAL FLOOR WEIGHT:			4724.0 kips	or	64.8 psf

Typical floor weight calculation.

The floor weights used for the seismic calculations were calculated using a 34 PSF deck weight from United Steel Deck over the entire area, added to the column weights. Also, the superimposed loads were added and a bracing allowance to account for beams as part of the floor system.

The different response modification coefficients yield different story forces, story shears, and moments as seen below.

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	167.42	0	12473.065
5	4713.6	60	462.27	2178985	0.288	134.55	167.42	8072.7394
4	4726.5	46	310.43	1467216	0.194	90.60	301.97	4167.4204
3	4724.0	32	180.20	851252	0.113	52.56	392.57	1681.9916
2	4653.4	18	76.08	354028	0.047	21.86	445.13	393.48265
1	5444.4						466.99	
Sum	28502.4	74.5	1668.39	7562930	1.000	466.99	466.99	26788.699

Story Shears, Forces and Moments for R=3.0

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	143.51	0	10691.199
5	4713.6	60	462.27	2178985	0.288	115.32	143.51	6919.4909
4	4726.5	46	310.43	1467216	0.194	77.65	258.83	3572.0746
3	4724.0	32	180.20	851252	0.113	45.05	336.48	1441.7071
2	4653.4	18	76.08	354028	0.047	18.74	381.54	337.27085
1	5444.4						400.28	
Sum	28502.4	74.5	1668.39	7562930	1.000	400.28	400.28	22961.742

Story Shears, Forces, and Moments for R=3.5

Floor	w_x (k)	h_x (ft)	h_x^k (ft)	$w_x h_x^k$	C_{vx}	Story Force F_x (k)	Story Shear V_x (k)	Moment at Floor (ft-k)
Roof	4240.5	74.5	639.41	2711449	0.359	150.53	0	11214.138
5	4713.6	60	462.27	2178985	0.288	120.97	150.53	7257.9442
4	4726.5	46	310.43	1467216	0.194	81.45	271.49	3746.7956
3	4724.0	32	180.20	851252	0.113	47.26	352.94	1512.2254
2	4653.4	18	76.08	354028	0.047	19.65	400.20	353.76779
1	5444.4						419.85	
Sum	28502.4	74.5	74.5	7562930	1.000	419.85	419.85	24084.871

Story Shears, Forces, and Moments, R=3.5 and C_s from design professional

Results: The story forces, shears, and moments are similar between all three different coefficients. The R=3.0 values are the most conservative, as was expected.

Appendix D: Spot Checks

First Floor Column B-14

B-14 1ST FLOOR

COLUMN SIZE: W 14x211
HEIGHT: H=18'
$P_u = 734^k$
$\phi P_n = 2270^k$

$A_g = 62.0 \text{ in}^2$
$I_x = 2660 \text{ in}^4$ $r_x = 6.55 \text{ in}$
$I_y = 1030 \text{ in}^4$ $r_y = 4.07 \text{ in}$

AREA $= 38.125 \times 24 = 915 \text{ ft}^2$
DEAD: 34 PSF x 1.2 x 5
LIVE = 80 PSF x 1.6 x 4
ST = 18 PSF x 1.2 x 4
734^k

$$\frac{KL}{R_x} = \frac{18(12)}{6.55} = 33$$

$$\frac{KL}{R_y} = \frac{18(12)}{4.07} = 53.1 \leftarrow \text{CONTROLS}$$

$$\frac{KL}{r} \leq 4.71\sqrt{E/F_y} = 4.71\sqrt{29000/50} = 113.4$$

$53.1 < 113.4$ ∴ INELASTIC BEHAVIOR

$$F_{CR} = \left[0.658^{F_y/F_e}\right]F_y = 0.658^{50/101.6}\,50 = 40.7$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(53.07)^2} = 101.6^k$$

$$P_n = F_{CR} A_g = 40.7(62) = 2523.4^k$$

$$\phi P_n = 0.9(2523.4) = 2271^k$$

TABLE 4-1 AISC 13TH ED

* WIND CONTROLLED MOMENT CONNECTION ON COLUMN ACCOUNTS FOR WHAT APPEARS TO BE OVERSIZING ERROR

SPOT CHECKS CONTINUED:

COLUMNS ON FLOORS 5 AND 4

D-26 5TH FLOOR
COLUMN SIZE: W14x90
HEIGHT = 14.5'

AREA = 24' x 29.25' = 702 ft²
DEAD : 34 x 1.2
LIVE : 20 x 1.6
Pu = 51.1 k

· FROM 4-1 AISC
NEED ATLEAST W14x48
∴ W14x90 w/ ϕPn = 1000 k OK ✓

* WIND MOMENT CONNECTION WAS NOT CONSIDERED IN THIS CHECK AND ACCOUNTS FOR APPARENT OVERSIZE

C-9 4TH FLOOR
COLUMN SIZE: W14x90
HEIGHT = 14'

AREA = 24 x 40.5 = 972 ft²
DEAD : 34 PSF x 1.2 x 2
LIVE = 80 PSF x 1.6
ROOF = 20 PSF x 1.6
Pu = 234.8 K

FROM 4-1 AISC - NEED AT LEAST W14x48
∴ W14x90 w/ ϕPn = 1020 K IS OK ✓

* WIND MOMENT CONNECTION WAS NOT CONSIDERED IN THIS CHECK AND ACCOUNTS FOR APPARENT OVERSIZE

SPOT CHECK CONTINUED:

Column on the third and second floor

C-2 -14 3RD FLOOR

COLUMN SIZE: W14X132
HEIGHT = 14'

AREA = 28.125x24 = 675 ft^2
DEAD: 34PSF x1.2 x 3
LIVE: 80PSF x1.6 x 2
ROOF: 20PSF x1.6 x 1
$P_u = 277^k$

· FROM TABLE 4-1 AISC 13TH ED, AT LEAST USE W14 x 48
∴ W14 x 132 w/ $\phi P_n = 1510$ IS OK ✓

* WIND MOMENT CONNECTION IS PRESENT, BUT WAS NOT TAKEN INTO CONSIDERATION IN THIS CHECK AND ACCOUNTS FOR OVERSIZING

C-3 2ND FLOOR

COLUMN SIZE: W14 X145
HEIGHT = 14'

AREA = 24 x 40.5 = 972 ft^2
DEAD: 34 PSF x 1.2 x 4
LIVE: 80PSF x 1.6 x 3
ROOF: 20 PSF x 1.6 x 1
$P_u = 516.3^k$

FROM TABLE 4-1 AISC W14X61 IS OK
∴ W14 X145 w/ $\phi P_n = 1690$ IS OK ✓

* WIND MOMENT CONNECTION IS PRESENT, BUT NOT TAKEN INTO CONSIDERATION IN THIS CHECK, WHICH ACCOUNTS FOR OVERSIZING

CHECK EXISTING BEAM

• FIND MAX ALLOWABLE MOMENT

$$M_n = V'(T_s - a/2) + A_s F_y (d/2)$$

$$M_n = 649^k(4.5'' - 1.988''/2) + (13\,in^2)(50\,ksi)(20.7''/2)$$

$$M_n = 9002.9\ k\text{-}in = 750.24\ k\text{-}ft$$

$$\phi M_n = (0.9)(750.24) = 675.2\ k\text{-}ft$$

$$\boxed{\phi M_n = 675.2\ k\text{-}ft}$$

• FIND STRENGTH OF ONE STUD

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} < R_g R_p A_{sc} F_u$$

$A_{sc} = 0.442\ in^2$
$E_c = w^{1.5}\sqrt{f'_c} = 115^{1.5}\sqrt{4} = 2466\ KSI$
$F_u = 65\ KSI$
$R_g = 1.0$
$R_p = 0.6$

$$0.5 A_{sc}\sqrt{f'_c E_c} = 0.5(0.442)\sqrt{4 \cdot 2466} = 21.9^k$$

$$R_g R_p A_{sc} F_u = (1.0)(0.6)(0.442)(65) = 17.238^k$$

$$\boxed{Q_n = 17.238^k}$$

• FIND STUDS REQUIRED FOR FULL COMPOSITE ACTION

$$\frac{V'}{Q_n} = \frac{649^k}{17.238^k} = 37.6\ \text{STUDS} \rightarrow 38\ \text{STUDS}$$ ∴ 76 STUDS FOR FULL COMPOSITE ACTION

CHECK EXISTING BEAM

• TRY PARTIAL COMPOSITE ACTION, 42 STUDS

$n = 21$

$V'c = 21(17.238^k) = 361.998\,k = 362^k$

$V'c < A_s f_y = 650$ ∴ PNA IN STEEL

IF $A_s f_y - 2A_{sf} f_y \leq V'c$, THEN PNA IN FLANGE

$(13)(50) - 2(2.925)(50) = 357.5^k$

$357.5^k < 362^k$ ∴ PNA IN FLANGE

• CHECK PARTIAL COMPOSITE ACTION > 25%

$\frac{\Sigma Q_n}{CF} > 0.25 = 25\%$

$CF = MIN \begin{cases} 0.85 f'c A_c = 816^k \\ A_s f_y = 650^k \end{cases}$

$\underline{CF = 650^k}$

$\frac{\Sigma Q_n}{CF} = \frac{362^k}{650^k} = 0.557 = 55.7\%$ $\underline{∴ OK}$

• FIND MAX ALLOWABLE MOMENT, M_n

$M_n = V'(T_s - a/2 + d_2) + A_s f_y (d/2 - d_2)$

$d_2 = \frac{A_s f_y - V'}{4 f_y b_f} = \frac{650 - 362}{4(50)(6.5)} = 0.222''$

$a = \frac{V'}{0.85 f'c\, b_{eff}} = \frac{362}{0.85(4)(96)} = 1.11''$

$M_n = 362(4.5 - 1.11/2 + 0.222) + 13(50)(20.7/2 - 0.222)$

$M_n = 8091.65\ K\text{-}in = 674.3\ K\text{-}ft$

$\phi M_n = 0.9(674.3) = 606.87\ k\text{-}ft$

$\boxed{\phi M_n = 606.87\ K\text{-}ft > M_u = 385.6\ k\text{-}ft}$

CHECK EXISTING BEAM

• CHECK DEFLECTION

• UNSHORED, BARE STEEL

$I = 843 in^4$

$w_a = \frac{(52+20)(8)}{1000} = 0.576 \text{ klf}$

$\Delta = \frac{5wl^4}{384EI} = \frac{5(0.576)(45)^4 1728}{384(29000)(843)} = 2.17''$

$\Delta_{MAX} = \frac{45(12)}{240} = 2.25''$

$\boxed{\Delta_{MAX} > \Delta \quad \therefore OK}$

• COMPOSITE DEFLECTION

$Y2 = T_s - a/2 = 4.5 - 1.11/2 = 3.945''$

LOCATION: $V'_c = A_s f_y - 2b_f Y1 f_y$

$362 = (13)(50) - 2(6.5)(Y1)(50)$

$Y1 = 0.443''$

Y1/Y2	3.5	3.945	4
0.338	1830	1901.2	1910
0.443	1726.9	1789.7	1797.5
0.450	1720	1782.3	1790

[TABLE 3-20]

$\boxed{I_{LB} = 1789.7}$

$\Delta = \frac{5wl^4}{384EI} = \frac{5(1.056)(45)^4 1728}{384(29000)(1789.7)} = 1.88''$

$\boxed{\Delta = 1.88'' < \Delta_{MAX} = 2.25'' \quad \therefore OK}$

W21x44 w/42 STUDS IS OK

CHECK EXISTING GIRDER

• CHECK W24X55 [14-2-14] GIRDER, $l = 24'$

LIVE = 80 PSF
DEAD = 52 PSF

FACTORED: 1.2D+1.6L = 190.4 PSF

UNFACTORED = D+L = 132 PSF

$$P = \frac{190.4\,PSF(8')\times 45'}{1000\,\#/K} = 68.544^K$$

$$M_u = 8'(68.544^K) = 548.352\ K\text{-}ft$$

UNFACTORED:

$$P = \frac{132\,PSF(8')(45')}{1000\,\#/K} = 47.52^K$$

$$M = 8'(47.52^K) = 380.16\ K\text{-}ft$$

• CHECK STRENGTH

$$\phi M_P = 503\ K\text{-}ft > M_{BARE\ STEEL} = 380.2\ K\text{-}ft \quad \checkmark OK$$

• ASSUME CONSTRUCTION LIVE LOAD = 20 PSF

$d = 23.6"$
$A_s = 16.2\ in^2$
$b_f = 7.01"$
$t_f = 0.505"$
$A_{SF} = (7.01")(0.505") = 3.54\ in^2$

$$B_{EFF} = MIN \begin{cases} l/8 = 24'/8 = 3' \\ S/2 = 45'/2 = 22.5' \end{cases} = 3' \times 2 = 6' = 72"$$

• FROM TABLE 3-2, $L_B = 8' > L_p = 4.73'$

• CHECK $\phi M_n < \phi M_{px}$

$$\phi M_n = C_b[\phi M_{px} - BF(L_B - L_p)] \leq \phi M_{px}$$

$$\phi M_n = 1.0[503 - 22.2(8 - 4.73)] = 430.4 \leq \phi M_{px} = 503$$

$$M_u = 380.2 < \phi M_n = 430.4\ K\text{-}ft < \phi M_{px} = 503\ K\text{-}ft\ \therefore OK$$

CHECK EXISTING GIRDER

• FIND V'

$V' = 0.85 f'_c A_c = 0.85(4)(3.5)(72) = 857^k$

$V' = f_y A_s = (16.2 in^2)(50 ksi) = 810^k$

$V' = \Sigma Q_n = 810^k$ [TABLE 3-19]

$\boxed{V' = 810^k}$

• DETERMINE LOCATION OF PNA

$a = \frac{V'}{0.85 f'_c B_{EFF}} = \frac{810^k}{0.85(4 ksi)(72'')} = 3.31'' < 3.5''$ ∴ PNA IN CONC.

• FIND MAX ALLOWABLE MOMENT

$M_n = V'(T_s - a/2) + A_s f_y (d/2)$

$M_n = 810(4.5 - 3.31/2) + 16.2(50)(23.6/2)$

$M_n = 11862.45$ k-IN $= 988.54$ k-ft

$\phi M_n = 0.9(988.54) = 889.6$ k-ft

$\boxed{\phi M_n = 889.6 \text{ k-ft}}$

• FIND STRENGTH OF ONE STUD [SECT. I3(3)]

$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} = R_g R_p A_{sc} F_u$ [EQ I3-3]

$A_{sc} = 0.442 in^2$
$E_c = w^{1.5} \sqrt{f'_c} = 115^{1.5} \sqrt{4} = 2466$ KSI
$F_u = 65$ ksi
$R_g = 1.0$ [$w_r/h_r = 12''/2'' = 6 \geq 1.5$ ∴ $R_g = 1.0$]
$R_p = 0.75$

$0.5 A_{sc} \sqrt{f'_c E_c} = 0.5(0.442)\sqrt{4 \cdot 2466} = 21.9^k$

$R_g R_p A_{sc} F_u = (1.0)(0.75)(0.442)(65) = 21.5^k$

$\boxed{Q_n = 21.5^k}$

• FIND STUDS REQUIRED FOR FULL COMPOSITE ACTION

$\frac{V'}{Q_n} = \frac{810^k}{21.5^k} = 37.7$ STUDS → 38 STUDS ∴ 76 STUDS FOR FULL COMPOSITE ACTION

CHECK EXISTING GIRDER

· CHECK PARTIAL COMPOSITE ACTION, 14-2-14 STUDS

$n = 14$

$V'_c = 14(21.5^k) = 301^k$

$V'_c = 301^k < A_s f_y = 810^k \therefore$ PNA IN STEEL

If $A_s f_y - 2A_{sf} f_y \leq V'_c$, THEN PNA IN FLANGE

$(16.2)(50) - 2(3.54)(50) = 465^k$

$\underline{465^k > 301^k \therefore \text{PNA IN WEB}}$

· CHECK PARTIAL ACTION > 25%

$\frac{\Sigma Q_n}{CF} > 0.25 = 25\%$

$CF = \text{MIN} \begin{cases} 0.85 f'_c A_c = 857^k \\ A_s f_y = 810^k \end{cases}$

$\underline{C_F = 810^k}$

$\frac{\Sigma Q_n}{CF} = \frac{301}{810} = 0.372 = 37.2\% > 25\% \therefore OK$

• FIND MAX ALLOWABLE MOMENT, M_n

$M_n = V'(T_s - a/2 + d_2) + A_s f_y (d/2 - d_2)$

$d_2 = \frac{A_s f_y - V'}{4 f_y b_f} = \frac{810 - 301}{4(50)(7.01)} = 0.363''$

$a = \frac{V'}{0.85 f'_c b_{eff}} = \frac{301}{0.85(4)(72)} = 1.23''$

$M_n = 301(4.5 - 1.23/2 + 0.363) + 16.2(50)(23.6/2 - 0.363)$

$M_n = 10542.6 \text{ k-in} = 878.6 \text{ k-ft}$

$\phi M_n = 0.9(878.6) = 790.7 \text{ k-ft}$

$\boxed{\phi M_n = 790.7 \text{ k-ft} > M_u = 548.4 \text{ k-ft} \therefore OK}$

CHECK EXISTING GIRDER

• CHECK DEFLECTION

· UNSHORED, BARE STEEL

$I = 1350 \text{ in}^4$

$P = \frac{(52+20)(8')(45')}{1000 \#/K} = 25.92^K$

$\Delta = \frac{Pl^3}{28EI} = \frac{25.92^K(24)^3(1728)}{28(29000)(1350\text{in}^4)} = 0.565"$

$\Delta_{MAX} = l/240 = 24\times12/240 = 1.2"$

$\boxed{\Delta = 0.565" < \Delta_{MAX} = 1.2" \quad \therefore OK}$

• COMPOSITE DEFLECTION

$Y_2 = T_S - a/2 = 4.5 - 1.23/2 = 3.885"$

LOCATION: $V'c = A_s f_y - 2 b_f Y1 f_y$

$301 = 16.2(50) - 2(7.01)(Y1)(50)$

$Y1 = 0.726"$

Y1/Y2	3.5	3.885	4
0.505	2710	2779.3	2800
0.726	2689.7	2758.5	2778.99
3.45	2440	2501.6	2520

[TABLE 3-20]

$I_{LB} = 2758.5 \text{ IN}^4$

$\Delta = \frac{Pl^3}{28EI} = \frac{68.544(24)^3(1728)}{28(29000)(2758.5)} = 0.731"$

$\boxed{\Delta = 0.731" < \Delta_{MAX} = 1.2" \quad \therefore OK}$

W24x55 [14-2-14] IS ADEQUATE