Visteon Village Corporate Center

Van Buren Township, MI

Technical Assignment #1

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Structural Option
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Executive Summary

The purpose of this report is to analyze and assess the existing structural conditions of the Visteon Corporate Headquarters Village Center in Van Buren, MI designed by SmithGroup architects and engineers. The Village Center is a steel framed building with composite slab construction, and rises five stories above grade. The fifth story, or Penthouse level, was constructed using a pre-engineered steel structure that was attached onto the top of the building.

During my analysis of the gravity load resisting systems, I used Load and Resistance Factor Design (LRFD), but the original calculations completed by SmithGroup were using Allowable Stress Design (ASD). As it turns out, all of the spot checks I performed using the LRFD method of analysis corresponded to the same results as the original design.

While determining lateral loads due to wind and seismic forces, I used ASCE 7-05 and IBC 2006. These are the latest editions of the code to date, and are more recent than the codes used for the original design (ASCE 7-95 and IBC 2000). While my results matched the assessment that wind loading was the controlling lateral force, the numbers that I came up with during my wind and seismic calculations did not match the values SmithGroup obtained during their original analysis. I believe that this difference was due to the updated code, as wind and seismic analysis has changed slightly in the ten years between the code releases.

The following technical report provides an overview of the Visteon Corporate Headquarter Village Center's structural system, an analysis of the gravity and lateral loads, and a spot check of a typical bay of the structure. All calculations that were completed for this analysis can be found in the Appendix.

Structural System

Foundation:

All of the foundation systems for the Visteon Village Corporate Center were designed based upon the findings of a geo-technical investigation performed by Somat Engineering on October 14, 2002**.** There is a deep foundation system to support all building columns, walls, grade beams and other foundation elements. The deep foundation elements are comprised of friction steel H-piles in native medium compact to compact sand. All H-piles consist of 75 foot long HP12x84 sections with concrete pile caps and are of ASTM A992 steel (Fy = 50 ksi). The number of piles for each foundation element range from 1 to 7 providing capacities of 100 kips to 1050 kips respectively. The concrete pile caps are of reinforced concrete construction with their top elevation at a minimum depth of 3'-6" below finished grade as to prevent frost heave. The dimensions of the caps range from 3'x3' for a single H-pile element up to 13'x11'-8" for a 7 H-pile element. All concrete used in the foundation systems has a minimum compressive strength of 3000 psi.

Slab on Grade:

The typical slab on grade used in the building is an 8 inch thick two way slab system spanning bays of 20'x20'. Reinforcement typically consisted of 6x6xW2.9xW2.9 WWF.

Columns:

All of the columns of the building are composed of structural steel. The main column system is made up of ASTM A992 wide flange shapes ranging in size from W14x43 to W14x311. Typically, these columns rest upon the deep foundation system and span 72 feet to the penthouse level with a column splice at an elevation of 52 feet (falling within the third story). These multistory columns are also part of the moment frame system that resists lateral loading. In areas where the building is only one story, the columns are composed of W14x43 and W14x61 wide flange shapes as well as ASTM Grade A500 Grade B (Fy = 46 ksi) hollow structural steel shapes of HSS6x6x1/4 sections. These columns also sit on the deep foundation system but only span 14 feet to the top of the first story.

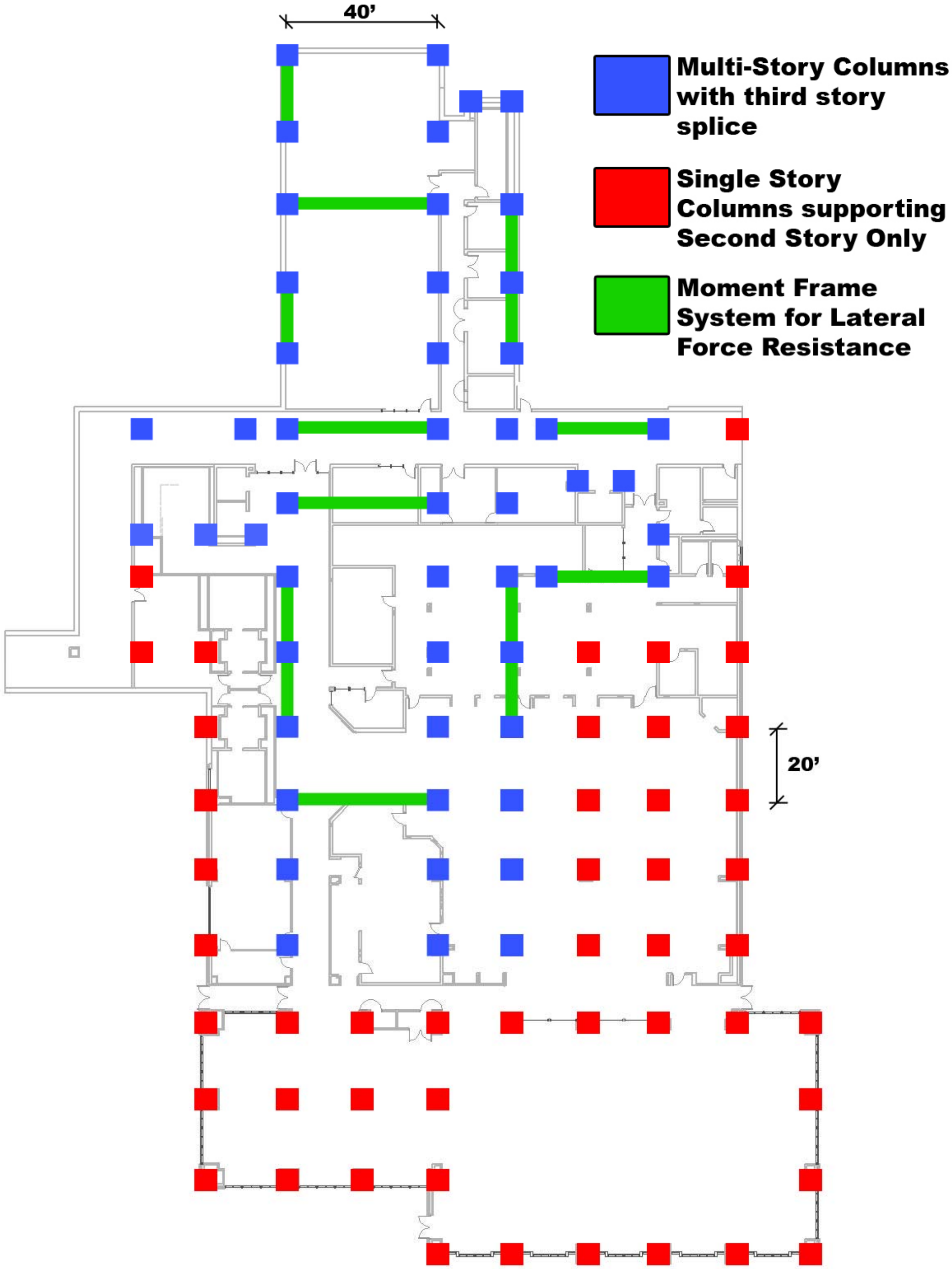
Floor and Roof Framing System:

The typical framing system for the Visteon Village Corporate Center is composed of structural steel composite beams and girders. The supported floors will consist of 40 foot long ASTM A992 wide flange shapes spanning a column free space. The typical bay for each floor is 40'x20' with wide flange beams spaced at 10' on center supporting 3" composite metal floor deck with 3-1/4" light weight concrete fill providing a total slab depth of 6-1/4". The typical bay for the flat roof is 20'x20' with wide flange beams spaced at 6'-8" on center supporting a 3" composite metal roof deck with 2-1/2" normal weight concrete to form a total slab depth of 5-1/2". The cafeteria area roof will be framed by long span steel trusses composed of hollow structural steel tubes supporting 3" composite metal floor deck with 3-1/2" light weight concrete and additional 6" (average) of light weight concrete topping to support the plaza above.

Lateral:

All lateral loads caused by wind and seismic forces are resisted by structural steel moment frames. There are five moment frames running in the North/South direction of analysis and six moment frames running in the East/West direction of analysis. Each moment frame consists of multistory wide flange columns and wide flange beams. The beam's flanges are bevel welded to the flange while the beam web is connected in a typical gravity resisting fashion. The moment connection also includes stiffener plates which are double fillet welded at the location of the beam flanges. The lateral forces for the entire penthouse level of the building are applied to a rigid bent that has been modeled above the penthouse floor level of the moment frames. The height of the rigid bent where the lateral forces are applied corresponds with the mean roof height of the building which is equal to the mid-height of the gabled penthouse roof.

Column Grid Diagram:



Code and Design Requirements

Building Codes:

-Michigan Building Code – Incorporating the 2000 edition of the International Building Code
-2000 International Building Code – IBC 2000

Structural Codes:

-Minimum Design Loads for Buildings and Other Structures, ASCE 7-95
-Building Code Requirements for Structural Concrete, ACI 318-99
-Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design, AISC 1989
-Structural Welding Code, AWS D1.1-1996
-Building Code Requirements for Masonry Structures, ACI 530-95/ASCE 5-95/TMS 402-95

Loads

Dead Loads	Total	Construction
Supported Floor w/ Composite Floor Deck	91 psf	65 psf
Penthouse Composite Floor Deck	89 psf	80 psf
Roof Deck; Low, Concrete Roofs	85 psf	65 psf
Roof Deck; High, Sloped, Light-Gage Roofs	29 psf	-
Attic Floor; Low, Enclosed, Floor Metal Deck	22 psf	-
Fire-Rated Composite Floor Deck	92 psf	56 psf
Fire-Rated Composite Floor Deck, Penthouse	94 psf	80 psf
Fire-Rated Composite Plaza Deck; Supported Conc. Flr	207 psf	65 psf
Basement Fire-Rated Composite Deck; Supported Conc. Flr	230 psf	92 psf
Mezzanine Floor w/ Composite Floor Deck	80 psf	32 psf
4” Brick and 8” Partially Grouted CMU Exterior Wall	100 psf	-
Insulated Glass Exterior Wall	10 psf	-

Live Loads (MBC, IBC 2000)

Office Building	100 psf
Penthouse Floor*	150 psf
Courtyard, non-vehicular	100 psf
CEP Chiller Floor	250 psf
CEP Mezzanine Floor*	150 psf

*Accounts for weight of mechanical equipment pads

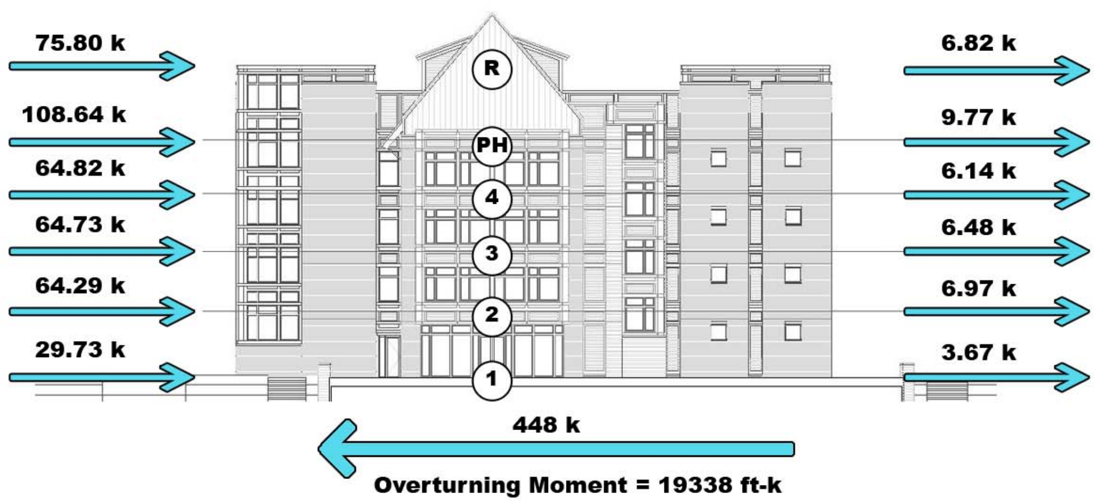
Roof Live Loads (MBC, IBC 2000)

Ground Snow Loads, P_g	25 psf
Snow Exposure Factor, C_e	0.9
Thermal Factor, C_t	1.0
Snow Load Importance Factor, I	1.0
Flat Roof Snow Load, $P_f = 0.7{*}C_eC_pI_SP_g$	15.75 psf
Code Minimum, Flat Roofs ($L_r = 20R_1R_2$ where R_1=1.0 and R_2=1.0)	20 psf

Wind Loads:

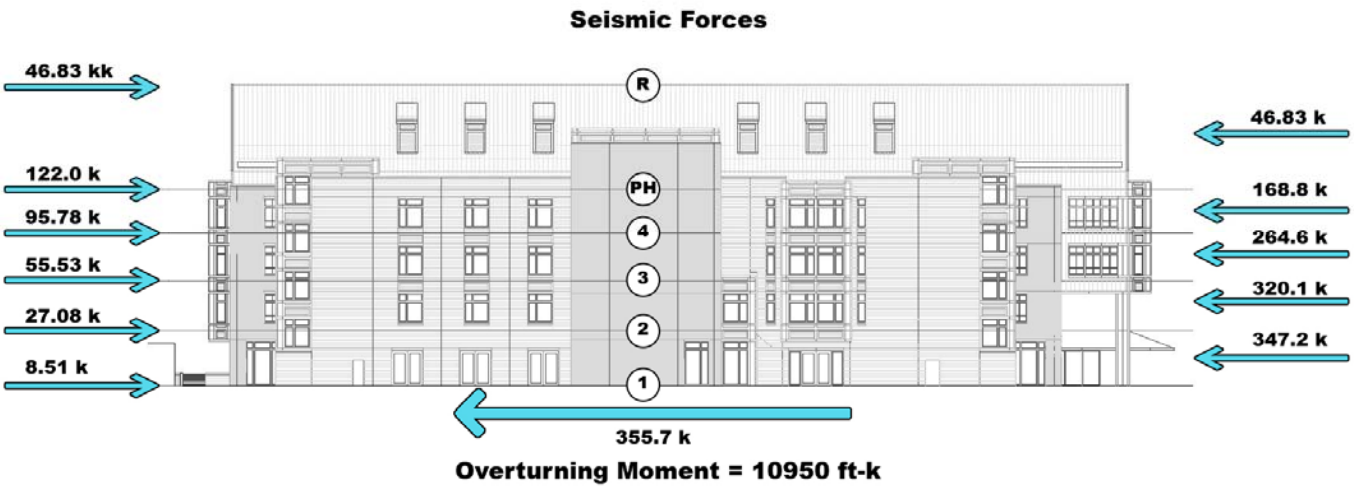
All wind loads were calculated in accordance with Chapter 6 of ASCE 7-05. I determined the loads on the North-South direction as well as the East-West orientation using the analytical method. I found that the building was rigid, and categorized as Exposure B as it is part of a complex of many similar buildings. The East-West dimension of 340’-0” is longer than the 177’-6” span of the North-South direction. This caused the wind forces in the East-West direction to be larger, thus controlling. All calculations and loading diagrams for each side of the building can be found in the Appendix.

East-West Wind Forces



Seismic Loads:

All seismic load calculations were done in accordance with Chapter 12 of ASCE 7-05. The data from the geotechnical report indicated that the site of the building was classified as Site Class B. All other factors, variables, and accelerations were obtained from figures and tables within ASCE 7-05. For the determination of effective seismic weight, I calculated the weight at each floor using values obtained from the construction documents. Summing these weights gave me total seismic weight (w) which was then used to calculated base shear and overturning moment. It was found that for this building, wind was the controlling lateral force. All calculations and determinations of values can be found in the Appendix.



Spot Checks

Spot checks were performed on a typical 40’x20’ bay to determine if the sizes provided were capable of handling the loading being applied.

First was the analysis of the 20’ composite beam. The design provided a W12x19 with 3” Metal Decking and a 2.5” LWC slab, with 16 shear studs. It was determined that the capacity was adequate and that the number of shear studs to reach full composite design was reached.

Next was the analysis of the 40’ composite girder. The girder was a W24x76 with 3” Metal Decking and a 2.5” LWC slab, and had 75 shear studs. Much like the beam, the analysis showed that the girder’s capacity was adequate and the number of shear studs provided was sufficient to achieve full composite action.

The last spot check performed was on a gravity column that spanned from the foundation to the penthouse floor. Since the column was spliced and changed sizes, the floor to floor heights were different per story, and the axial force changed per story, an independent analysis was done for each floor. It was determined that all of the columns were sufficient to carry the loading as designed.

All spot check hand calculations can be found in the Appendix.

Appendix

Wind Calculations

V=	90 mph	**Rigid Structure**	
k_d=	0.85	**G=**	0.85
I=	1.15	**$g_Q=g_v$=**	3.4
Exposure Category=	B	**z=(0.6)h=**	43.6
k_{zt}=	1	**$I_z=c(33/z)^{1/6}$=**	0.286
z_g=	1200 ft	**$L_z=l(z/33)^{\epsilon}$=**	351.1
α=	7		

$Q=\text{sqrt}(1/(1+0.63((B+h)/L_z)^{0.63}))$= 0.814 NS
0.768 EW

$G=0.925((1+1.7g_QI_zQ)/(1+1.7g_vI_z))$= 0.818 NS
0.791 EW

Gc_{pi}= -0.18 -0.18

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

$q_z=0.00256*k_z*k_{zt}*k_d*V^2*I$

$p_{ww}=q_z*G*C_p - q_h*Gc_{pi}$

$p_{LW}=q_h*G*C_p - q_h*Gc_{pi}$

C_p	Roof		Wall	
	WW	LW	WW	LW
NS	0.4	-0.6	0.8	-0.5
EW	0.33	-0.6	0.8	-0.32

Wind Pressures (psf)

					p_{NS}		p_{EW}	
Story	Start Ht (ft)	End Ht (ft)	K_z	q_z	WW	LW	WW	LW
PH	57.67	72.67	0.90	18.28	15.26	-4.19	14.86	-1.34
4	44.67	57.67	0.84	17.12	14.49	-4.19	14.12	-1.34
3	30.67	44.67	0.78	15.91	13.70	-4.19	13.36	-1.34
2	16.17	30.67	0.71	14.29	12.64	-4.19	12.33	-1.34
1	0	16.17	0.59	11.90	11.08	-4.19	10.82	-1.34

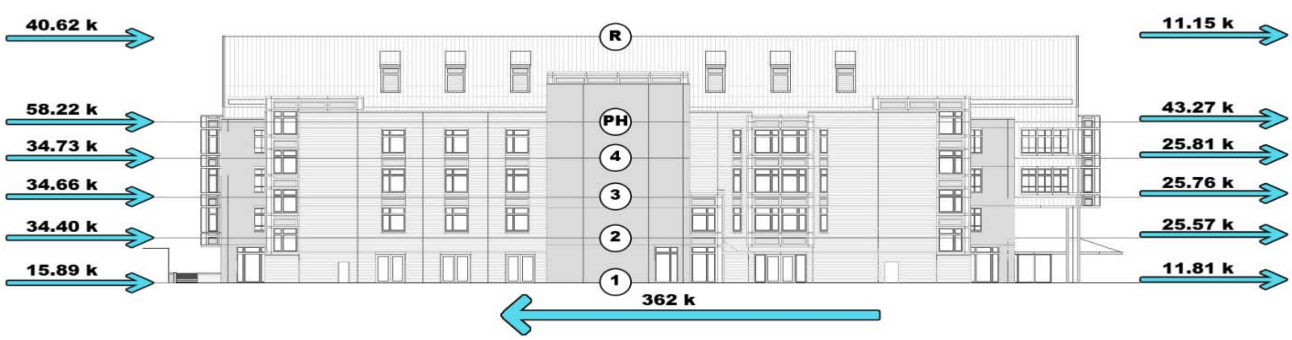
Wind Forces (k)

		NS			EW		
Story	Trib Ht (ft)	Width (ft)	WW	LW	Width (ft)	WW	LW
Roof	15	177.5	40.62	-11.15	340	75.80	-6.82
PH	21.5	177.5	58.22	-43.27	340	108.64	-9.77
4	13.5	177.5	34.73	-25.81	340	64.82	-6.14
3	14.25	177.5	34.66	-25.76	340	64.73	-6.48
2	15.33	177.5	34.40	-25.57	340	64.29	-6.97
1	8.08	177.5	15.89	-11.81	340	29.73	-3.67

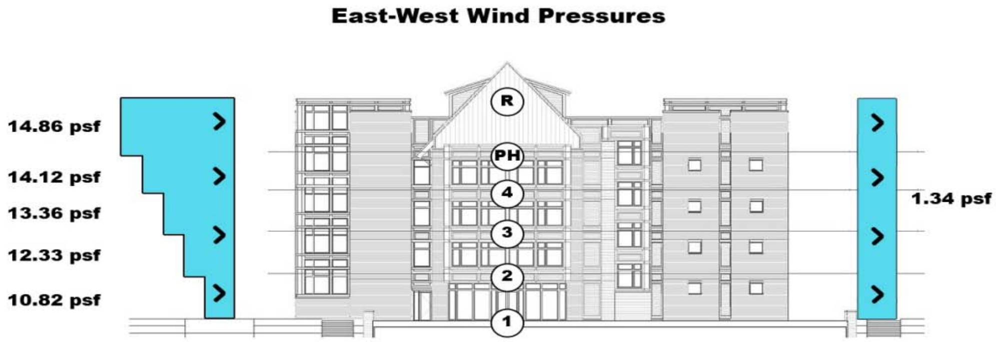
Overturning Moment (ft-k)	NS	EW
	15142	19338

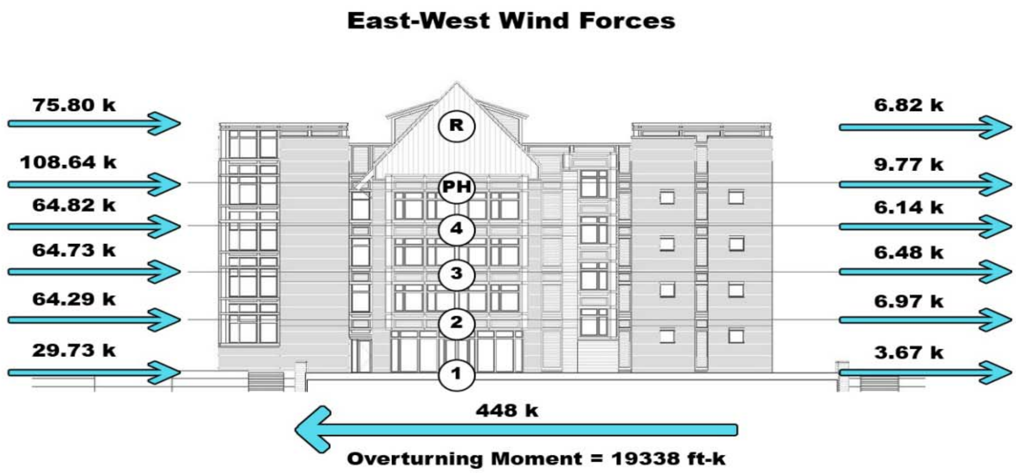
North - South Wind Pressures

North-South Wind Forces



Overturning Moment = 15142 ft-k


East-West Wind Pressures
14.86 psf
14.12 psf
13.36 psf
12.33 psf
10.82 psf
1.34 psf
R
PH
4
3
2
1


East-West Wind Forces
75.80 k
108.64 k
64.82 k
64.73 k
64.29 k
29.73 k
6.82 k
9.77 k
6.14 k
6.48 k
6.97 k
3.67 k
R
PH
4
3
2
1
448 k
Overturning Moment = 19338 ft-k

Seismic Load Analysis:

USGS Seismic Hazard Curves and Uniform Hazardous Reponse Spectra

Ss=	0.126		
S1=	0.046	**42.241 N**	**83.433 W**
Category=	III		
Site Class=	D		
Fa=	1.6		
Fv=	2.4		
Sms= Fv*Ss=	0.2016		
Sm1= Fv*S1	0.1104		
SDS= 2*Sms/3=	0.1344		
SD1=2*Sm1/3=	0.0736		
Ct=	0.028		
hn=	108		
x=	0.8		
Ta=Ct*hn^x=	1.186		
Ts=Sd1/SDs=	0.548		
0.8*Ts=	0.438		
SDC=	B		
R=	8		
I=	1.25		
TL=	12		
Cs=SD1/(T*R/I)<=SDS/(R/I)=	0.009<=0.021=0.021 (use 0.01)		
W=	16935 k (see next page)		
V = CsW=	169.4		
k=	2		

Level	Ht (ft)	wx	wx*Ht^k	Cvx	Fx	Vx
PH Roof	108.48	435	5119041	0.1316465	22.30092	22.30092
PH Floor	72.67	2525	13334345	0.3429198	58.09061	80.39153
4th	58.67	3042	10471078	0.269285	45.61688	126.0084
3rd	44.67	3042	6070034	0.1561032	26.44389	152.4523
2nd	30.67	3147	2960222	0.0761281	12.8961	165.3484
1st	14	4745	930020	0.0239174	4.051599	169.4
		Total=	38884740			

Overturning Moment= 10950.485

Seismic Weight:

First Story	Weight(k)
3"MD + 3.5" LWC	3945.6
Typical Roof	174
Masonry Walls	551.8
Metal Panelling/Glass	72
Total=	4743.4

Second Story	Weight(k)
3"MD + 2.5" LWC	1948.8
Masonry Walls	1197.8
Total=	3146.6

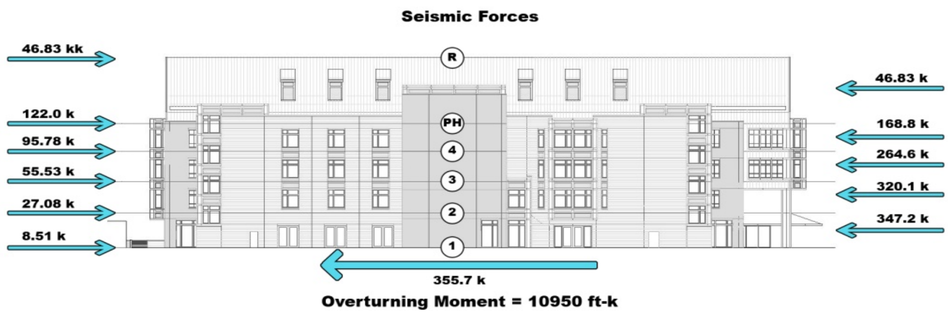
Third Story	Weight(k)
3"MD + 2.5" LWC	1948.8
Masonry Walls	1093.7
Total=	3042.5

Fourth Story	Weight(k)
3"MD + 2.5" LWC	1948.8
Masonry Walls	1093.7
Total=	3042.5

Penthouse Story	Weight(k)
3"MD + 3.5" LWC	1117.2
Typical Roof	116
Attic	110
Masonry Walls	1038.7
Metal Panelling/Glass	143.1
Total=	2525

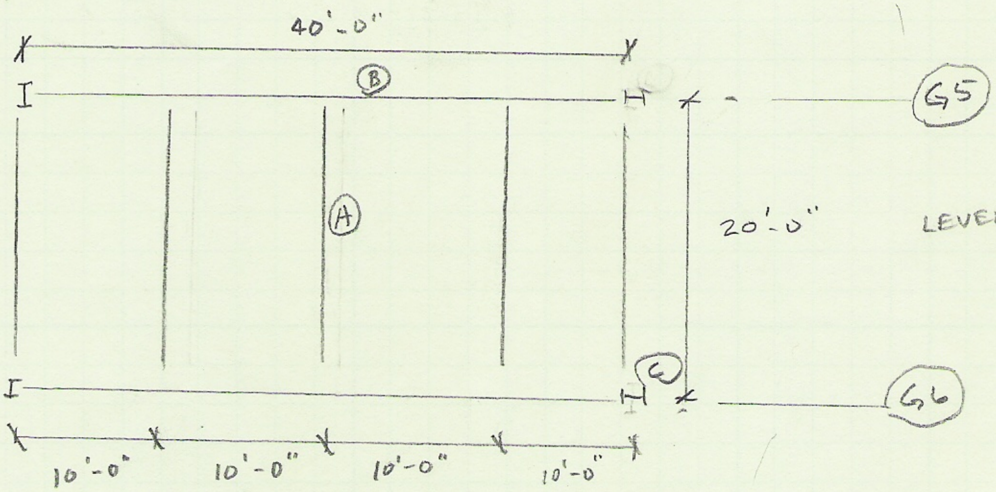
Penthouse Roof	Weight(k)
Typical Roof	435
Total=	435

Total Weight (W)= 16935


Seismic Forces
46.83 kk
122.0 k
95.78 k
55.53 k
27.08 k
8.51 k
R
PH
4
3
2
1
46.83 k
168.8 k
264.6 k
320.1 k
347.2 k
355.7 k
Overturning Moment = 10950 ft-k

FLOOR SYSTEM SPOT CHECK

TYPICAL BAY



LEVEL 3

BEAM "A"

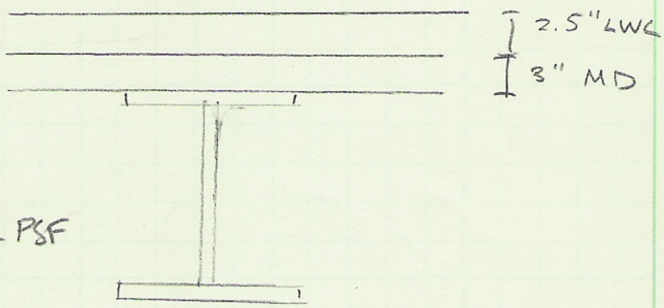
GIVEN: COMPOSITE W12x19 (FULL COMPOSITE ACTION) w/ 16 STUDS

SPAN = 20'-0"

SPACED @ 10'-0" O.C.

$A_s = 5.57 \text{ IN}^2$

$f'_c = 4000$ PSI



DL

FIRE RATED COMPOSITE FLR DECK: 92 PSF

LL

OFFICE = 100 PSF

b_{eff} $\begin{cases} 10' \text{ TRIB WIDTH} \\ 20'/4 = 5' \leftarrow \text{CONTROLS} \end{cases}$

COMPRESSION FORCES

$$V'_c = 0.85(4)\underbrace{(5')(12)}_{60"}(2.5) = 510 \text{ K}$$

$$V'_s = 5.57(50) = 278 \text{ K} = V'_q = \Sigma Q_n$$

STEEL CONTROLS SO PNA IS AT OR ABOVE FLANGE

DEPTH OF CONCRETE TO BALANCE V's

$$a = \frac{278}{(0.85)(4)(60)} = 1.36"$$

MOMENT ARM OF COMPRESSIVE FROM TOS

$$Y_2 = 2.5 - \frac{1.36}{2} = 1.82 \rightarrow 2" \text{ (MIN)}$$

TABLE 3-19

$\phi M_n = 169'\text{K}$ $\quad \Sigma Q_n = 279 \text{ K}$

MU

$w_{DEAD} = (10')(92\ PSF) = 920 \approx 0.92\ K/FT$

$w_{LIVE} = (10')(100\ PSF) = 1000 \approx 1\ K/FT$

$W_u = 1.2(0.92) + 1.6(1.0) = 2.70\ K/FT$

$M_u = \frac{wL^2}{8} = 135'K < 169'K$ ✓

\# OF STUDS

$Q_n = 21.2^K$ PER STUD (3-21)

$\frac{\Sigma Q_n = 279^K}{21.2} = 13.3 \rightarrow 14$ STUDS

[16] BY DESIGN > 14 ✓

GIRDER "B"

GIVEN:

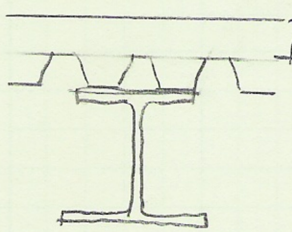
COMPOSITE 24 x 76 (FULL COMPOSITE ACTION) w/ 75 STUDS

SPAN: 40'

SPACING: 20' O.C.

$A_S = 22.4\ IN^2$

$f'_c = 4000$ PSI



DL:

FIRE RATED COMPOSITE ~~FLR~~ DECK: 92 PSF

LL:

OFFICE: 100 PSF

b_{eff} { 20' TRIB WIDTH ; 40'/4 = 10' ← CONTROLS

COMPRESSION FORCES

$V'_c = 0.85(4)(\underbrace{10')(12}_{120"})(2.5) = 1020^K$

$V'_s = 22.4(50) = 1120^K$

$V'_s > V'_c$ SO PNA IN STEEL

WHERE IS PNA

$T_f = 0.68''(8.99'')(50) = 305.7^k$

$T_w = 1120 - 2(305.7^k) = \underline{508.6^k}$

$814.3^k < 1020^k$

$\therefore$ PNA IN FLANGE

AREA OF STEEL IN COMPRESSION

$$A_{s-c} = \frac{1120 - 1020}{2(50)} = 1 \text{ IN}^2$$

$$x = 1/8.99 = 0.111''$$

$$M_n = T_s\left(\frac{d}{2}\right) + C_c\left(\frac{t}{2}\right) - 2A_{sc}F_y\left(\frac{x}{2}\right)$$

$$M_n = 1120\left(\frac{23.9}{2}\right) + 1020\left(5.5 - \frac{2.5}{2}\right) - 2(1)(50)\left(\frac{0.111}{2}\right)$$

$$M_n = \frac{17713.5}{12} = 1476.12'k$$

$$\phi M_n = 0.9(1476) = 1328'k$$

$w_D = (20')(92 \text{ PSF}) = 1840 = 1.84 \text{ K/FT}$

$w_L = (20')(100 \text{ PSF}) = 2000 = 2.0 \text{ K/FT}$

$1.2(1.84) + 1.6(2.0) = 5.41 \text{ K/FT}$

$$M_u = \frac{wl^2}{8} = \frac{5.41(40')^2}{8} = 1082'k < 1328'k \checkmark$$

	ΣQ_n
0	1120
0.111	1019
0.170	966

$\Sigma Q_n = 1019$

$Q_n = 17.2$

$$\frac{1019}{17.2} = 59.2 \rightarrow 60 \text{ STUDS}$$

[75] AS DESIGNED $\therefore \checkmark$

COLUMN E-7 "C"

FIFTH - PH FLOOR $h = 14'$

$D = 62.6\ k$
$L = 84.4\ k$

W14x43
$A_g = 12.6\ in^2$
$I_x = 428\ in^4$ $I_y = 45.2\ in^2$
$r_y = 5.82\ in$ $r_y: 1.89\ in$

$$\frac{KL}{r_x} = \frac{14(12)}{5.82} = 28.7 \qquad \frac{KL}{r_y} = \frac{14(12)}{1.89} = 88.9$$

↑ CONTROLS

$$\frac{KL}{r} \le 4.71\sqrt{E/F_y}$$

$88.9 < 113.4$ YES ∴ INELASTIC BEHAVIOR

$$F_{cr} = 0.658^{(F_y/F_e)} F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29000)}{88.9^2} = 36.22$$

$$F_{cr} = 0.658^{(50/36.22)}(50) = 28.05\ ksi$$

$$P_n = F_{cr} A_g = 28.05(12.6) = 353.4\ k$$

$$P_u = 1.2(62.6\ k) + 1.6(84.4\ k) = 210.2\ k$$

$$\phi P_n = 0.9(353.4) = 318.1\ k > 210.2\ k$$

✓

FOR ALL OTHER LEVELS, TABLE 4-1 WAS USED AS IT IS BASED ON THIS METHOD

FOURTH FLOOR: W14x61

$P_u = 1.2(118) + 1.6(117.8)$. $KL = 14'$
$P_u = 330\ k < \phi P_n$ of $572\ k$ ✓

THIRD: $P_u = 1.2(173.7) + 1.6(140.4) = 432.6 < 572\ k$ ✓
W14x61 $KL = 14$
$\phi P_n = 572\ k$

SECOND: $P_u = 1.2(229.4) + 1.6(161.3)$
W14x90 $P_u = 533.4\ k < 954\ k$ ✓
$KL = 17'$
$\phi P_n = 954\ k$

FIRST: $P_u = 1.2(286.7) + 1.6(181.2)$
W14x90 $P_u = 807.9\ k$
$KL = 14$ $\phi P_n > P_u$
$\phi P_n = 1020\ k$ ∴ ✓