CODES AND REQUIREMENTS

The structural design of Northbrook Corporate Center was based on the International Building Code 2003, which incorporates many of the provisions of ASCE 7. The lateral load design in this report is based on the procedures outlined in ASCE 7-02.

GRAVITY LOADS

All gravity and lateral loads are based on the provisions of ASCE 7-02. In all possible situations live load reductions are applied. The design check is based on loads list provided below.

Dead Loads (psf):

- Concrete Floor Slab on Metal Deck — 45 psf
- Mechanical and Ceiling — 7 psf
- Miscellaneous — 5 psf
- Exterior Wall — 80 psf

Live Loads (psf):

- Typical Floors (office) — 80 psf
- Roof — 30 psf

LATERAL LOADS

Wind and seismic design is based on provisions given in ASCE7-02. Most calculations of lateral design are presented in the tabular format. In order to check the accuracy of the calculation, the tables provide the value of variable, and the reference was used to obtain the value. All in-depth calculations are provided in the appendix, at the end of this report.

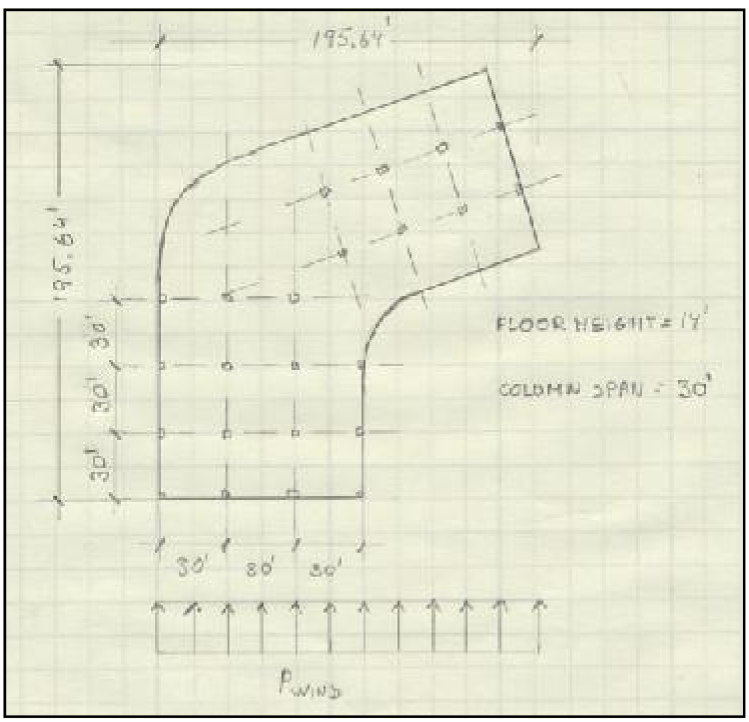
ASCE7-02 WIND LOAD CALCUALATION

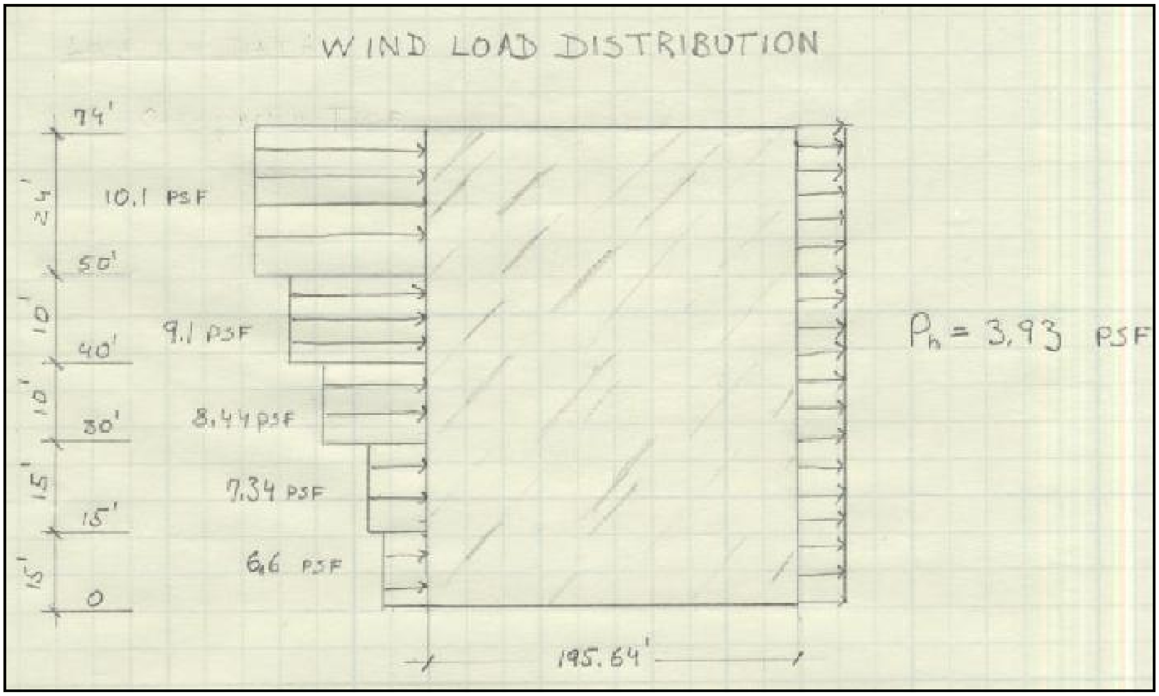
Gust Effect Factor	Value	Reference	Ch/Fg/Tb
Gust Coefficient	3.4	6.5.8.2	Eq 6-8
Gust Coefficient	3.4	6.5.8.2	Eq 6-8
Gust Coefficient	4.347	6.5.8.2	Eq 6-9
Background response	0.80821	6.5.8.1	Eq 6-6
Gust Factor	0.819462	6.5.8.2	Eq 6-8

Wind Pressure	**Value**	**Reference**	**Ch/Fg/Tb**
Velocity Pressure	17.6256	6.5.10	Eq 6-15
Volocity Pressure at z	15.99	6.5.12.2	T 6-3

External Pressure Coefficient	**Value**	**Reference**	**Ch/Fg/Tb**
Windward Side	0.8	6.5.11.2	F 6-6
Leeward Side	-0.3	6.5.11.2	F 6-6
Final Pressure (psf)	15.48577	P = qGCp - qGCp	

z (ft)	**Kz (T 6-3)**	**q**	**P**	**P total**
0-15	0.57	10.04	6.58624	10.512878
20	0.62	10.93	7.17008	11.0963349
25	0.66	11.64	7.63584	11.5617894
30	0.7	12.34	8.09504	12.0206881
40	0.76	13.4	8.7904	12.7155919
50	0.81	14.28	9.36768	13.2924931
60	0.85	14.99	9.83344	13.7579475
70	0.89	15.69	10.29264	14.2168462
80	0.93	16.39	10.75184	14.675745





#.STORY / HEIGHT	POINT LOAD (K)
ROOF / 74'	3.6
4th / 56'	5.9
3RD / 42'	5.4
2ND / 28'	4.9
1st / 14'	4.6

ASCE7-02 SEISMIC LOAD CALCUALATION

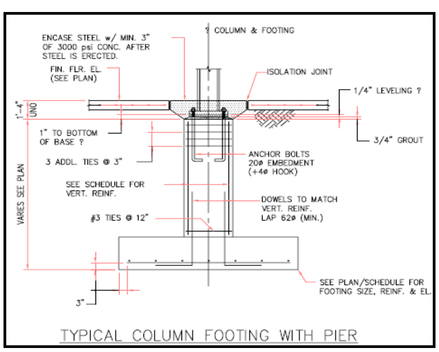
k=1.005	Level	Weight (w_x)	h_x	$w_x h_x^k$	C_{vx}	V	F_x(k)
Roof	5	1170	72	89155.05042	0.159023	13.806	2.1954704
Office	4	3176	56	188937.804	0.337002	37.4768	12.629761
Office	3	3176	42	141499.672	0.252388	37.4768	9.4587056
Office	2	3176	28	94142.06444	0.167918	37.4768	6.2930328
Office	1	3176	14	46908.17882	0.083669	37.4768	3.1356303
Garage	abve grnd	3176	0	0	0	37.4768	0
Total		17050		560642.7697	1	201.19	201.19

DESCRIPTION OF THE STRUCTURAL SYSTEM

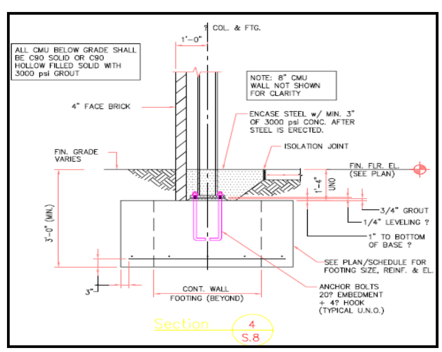
Northbrook Corporate Center is a four story office building. Including a garage, the height of each story is 14 feet, and the total accumulated height of a building is 74 feet. The height of a building varies going around the perimeter because side and rear elevations of the garage are completely above ground. Each floor of a building provides roughly 26,000 square feet of usable space. Structural system of the building resists all gravity and lateral loads applied on a building. Gravity load carried by the system include the live office load, superimposed dead load, self weight of the structure, snow loads, the weight of the brick wall around the perimeter of a building, and the weight of the glass curtain wall at the entrance of a building.

FOUNDATION

Northbrook Corporate Center is relatively light since it is only five stories high above the foundation. The foundation of a building consists of concrete footing with and without the pier. The footing size varies from 3'x3'x14" to 11'x11'x28". Footing around the perimeter of a building is a continuous, at 2 feet wide and 28 inches high (see fig below). An example of typical column footing with the pier and typical exterior wall footing are shown bellow.



TYPICAL COLUMN FOOTING WITH



EXTERIOR WALL FOOTING

Concrete column piers are generally spaced in a 30’x30’ grid; column piers in the area of the center or entrance of the building are spaced as design requires it. The bottom of the footing is at least 3 feet below the ground. Garage does not occupy all of the building’s floor area, and some of the ground below the first floor will not be excavated; thus, portion of the first story is supported directly by the foundation.

FRAME

Frame of the building is composed of steel columns, steel girders, steel beams, steel joists, and concrete slab on metal decking. Typical columns sizes are W12x60, W12x65 and W12x72. In most cases each column extends from the garage floor to middle of third floor where it is connects to and continued by smaller, lighter column. Typical joist (26k7) is supported by a steel girder, typically W24x68. All girder/joist to column connections are designed for a moment of 40 ft-kips to resist wind and seismic loads. All lateral loads are resisted by moment connections. The loads are transferred in this order: A four inch thick concrete slab on metal decking is held in place by steel joists; steel joists are connected to steel girders, girders are supported by steel columns, and columns stand on footings.

STRUCTURAL FLOOR/SLAB

Northbrook Corporate Center’s floor system is composed of 4 inch concrete slab on metal decking held in place by 26K7 steel joists. Joists are spaced 3 feet apart center to center, and are held from both sides by W24X68 steel girders. Concrete is poured on 9/16” – 26 GA. UFS form deck, and is reinforced with 6x6 – W2.9xW2.9 WWF; thus, the total slab thickness is 4 inches. Strength of the concrete and steel is 3,000 psi and 50,000 psi respectively.

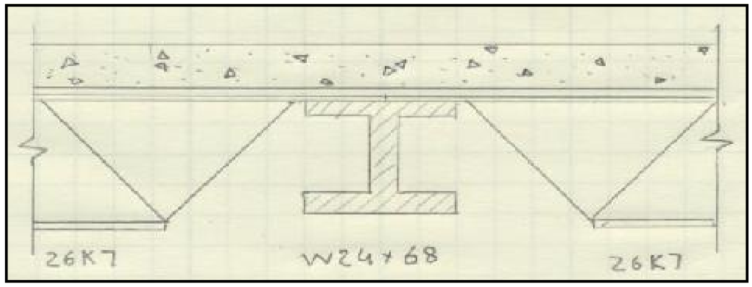
LATERAL LOAD RESISTING SYSTEM

To resist wind and seismic loads the design incorporates moment resisting connections. As indicated above all girder/joist to column connections are design for a moment of 40 ft-kips to resist lateral loads. The adequacy of this system is better demonstrated in structural analysis section of this report.

STRUCTURAL ANALYSIS

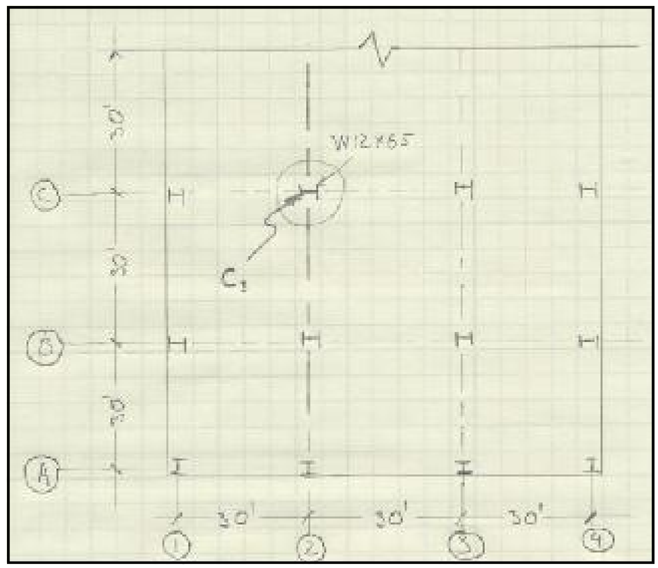
In this section the report will demonstrate the feasibility of the described structural design. Because the scope of this report is limited, only three spot checks were performed to demonstrate the adequacy of the design. The spot checks are demonstrated in the following order: the girder check, the column check and the lateral resisting system check.

GIRDER CHECK



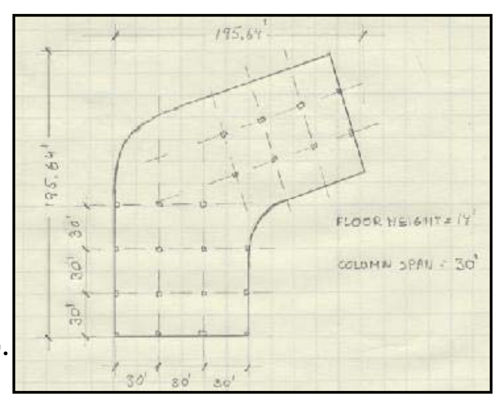
A typical 30'x30' bay is chosen to perform a girder spot check. The girder spans 29 feet from column to column. The dead weight of 60 psf is imposed on a girder by the concrete slab, steel joists, ceiling, and collateral weights. Live loads include office space loads and partition loads, total equaling 80 psf. The total uniformly distributed load on the girder is 180 kips. According to table 5.4 in the LRFD, W24x68 is sufficient to carry the uniformly distributed load of 183 kips, thus the girder design is satisfactory. All assumptions and calculations are provided in the appendix.

COLUMN CHECK

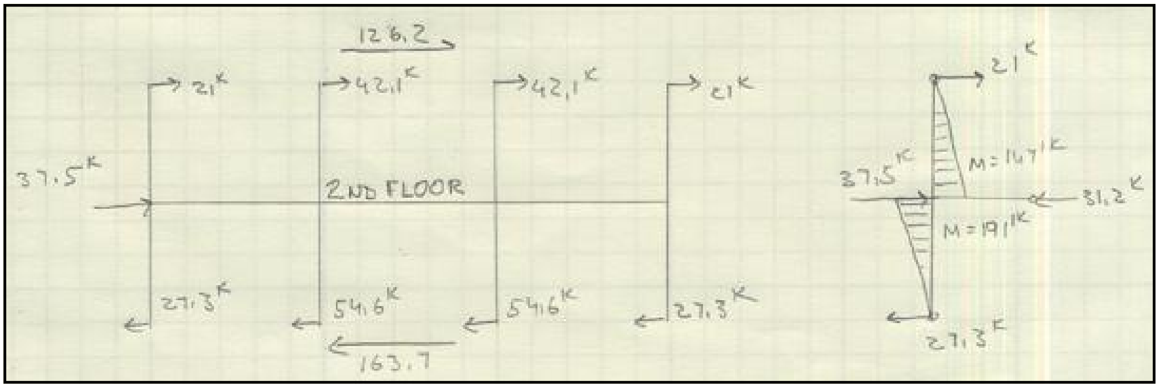


An interior column check was performed on the second floor of a building (see the figure on a right). The worst case scenario was considered as the moment from gravity loads was calculated. The tributary area of a column is 900 SF per floor. The total axial load from the roof down to second floor was calculated to be 430.2 kips. Adding the moment applied by the girders, the total axial effective load equaled 603.2 kips. Assuming KL=12, W12x65 column can withstand a total axial load of 687 kips (LRFD). The spot check of column C1 proves that the design is satisfactory. All assumptions and calculations are provided in the appendix.

MOMENT RESISITING CONNECTION CHECK



Due to the buildings geometry several assumptions are made in order to help analyze the adequacy of moment resisting connections (see fig on a right). It is assumed that 7x4 column grid can be used as an equivalent resisting lateral load system. It is found that the seismic shear is the controlling load case. The shear above the second floor is 126.2 kips, and 163.7 below (see fig below).



The maximum moment applied on each column is calculated to be 27 ft-kips, that is less than 40 ft-kips provided by moment resisting connection. The moment resisting connections of 40 ft-kips are sufficient to resist the lateral loads; hence, the lateral load resisting design is satisfactory.

All in-depth calculations are provided in the appendix at the end of this report.

APPENDIX A

LATERAL LOAD DESIGN

ASCE7-02 WIND LOAD CALCUALATION

Location data	Value	Reference	Ch/Fg/Tb
Occupancy Type	2	1.5.1	T 1-1
Importance Factor	2	6.5.5	T 6-1
Surface Roughness	B	6.5.6.2	NA
Exposure Factor	B	6.5.6.3	NA
Topographic	1	6.5.7.2	Fg 6-4

Building Dimentions (ft)	Value	Reference
Height Above Base	74.33	9.5.5.3
Height Above Ground	74.33	6.3
Horiz. Length Parallel	232.146	6.3
Horizontal Dimension Ratio	1.187	F 6-6
Horiz. Lenth Perpind.	195.64	6.3

Wind Velocity (mph)	Value	Reference	Ch/Fg/Tb
Basic Wind Speed	90	6.5.4	F 6.1
Wind Directinality	0.85	6.5.4.4	T 6-4
3-sec Gust Power Law	7	6.3	T 6-2
Mean Wind Speed Factor	0.25	6.5.8.2	T 6-2
Wind Coefficient (b)	0.45	6.5.8.2	T 6-2
Wind Coefficient (z)	44.6	6.5.8.2	T 6-2
Mean Hourly Wind Speed	64.05	6.5.8.2	Eq 6-14
Height atm Boundary	1200	6.3	T 6-2
Velocity Pressure Exp.	0.90332	6.5.6.6	T 6-3

Integral Length Scale	Value	Reference	Ch/Fg/Tb
Integral Length Scale Factor	320	6.5.8.1	T 6-2
Integral Length Scale Exp.	0.333	6.5.8.1	T 6-2
Integral Length Scale, Turb.	353.796	6.5.8.1	Eq 6-7
Turbulence Intensity Factor	0.3	6.3	T 6-2
Intensity of Turbulence	0.392	6.5.8.1	Eq 6-5

Fundamental Period	Value	Reference	Ch/Fg/Tb
Period Coefficient	0.02	9.5.3.2	T 9.5.5.3.2
Approx. Fund. Period	0.51	9.5.3.2	Eq 9.5.3.3-1
Natural Frequency	1.96	6.5.8.2	1/T

Resonance	Value	Reference	Ch/Fg/Tb
R_l Coeffiecient	0.091	6.5.8.2	Eq 6-13
R_l Coeffiecient	0.912	6.5.8.2	Eq 6-13
R_l Coeffiecient	0.0091	6.5.8.2	Eq 6-13
Reduced Frequence	10.82	6.5.8.2	Eq 6-12
Resonance Coefficient	0.031	6.5.8.2	Eq 6-11
Damping ration	0.05	6.3	Section 9
Resonant Response Factor	0.1658	6.5.8.2	Eq 6-10

Gust Effect Factor	**Value**	**Reference**	**Ch/Fg/Tb**
Gust Coefficient	3.4	6.5.8.2	Eq 6-8
Gust Coefficient	3.4	6.5.8.2	Eq 6-8
Gust Coefficient	4.347	6.5.8.2	Eq 6-9
Background response	0.80821	6.5.8.1	Eq 6-6
Gust Factor	0.819462	6.5.8.2	Eq 6-8

Wind Pressure	**Value**	**Reference**	**Ch/Fg/Tb**
Velocity Pressure	17.6256	6.5.10	Eq 6-15
Volocity Pressure at z	15.99	6.5.12.2	T 6-3

External Pressure Coefficient	**Value**	**Reference**	**Ch/Fg/Tb**
Windward Side	0.8	6.5.11.2	F 6-6
Leeward Side	-0.3	6.5.11.2	F 6-6
Final Pressure (psf)	15.48577	P = qGCp - qGCp	

z (ft)	**Kz (T 6-3)**	**q**	**P**	**P total**
0-15	0.57	10.04	6.58624	10.512878
20	0.62	10.93	7.17008	11.0963349
25	0.66	11.64	7.63584	11.5617894
30	0.7	12.34	8.09504	12.0206881
40	0.76	13.4	8.7904	12.7155919
50	0.81	14.28	9.36768	13.2924931
60	0.85	14.99	9.83344	13.7579475
70	0.89	15.69	10.29264	14.2168462
80	0.93	16.39	10.75184	14.675745

ASCE7-02 SEISMIC LOAD CALCUALATION

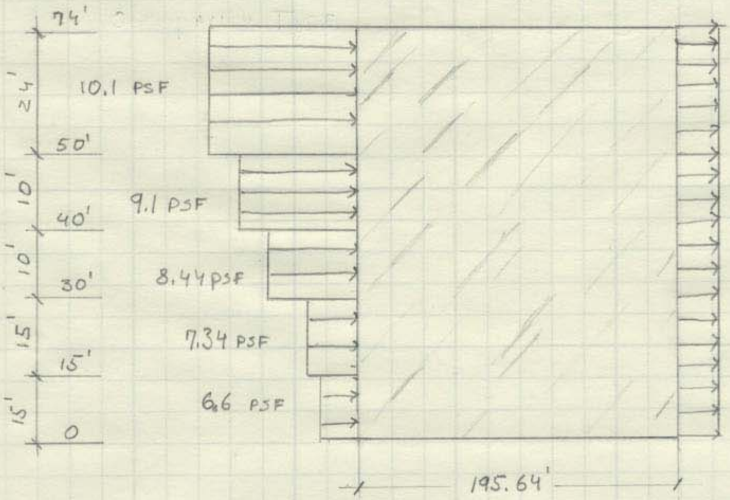
k=1.005	Level	Weight (w_x)	h_x	$w_x h_x^k$	C_{vx}	V	F_x(k)
Roof	5	1170	72	89155.05042	0.159023	13.806	2.1954704
Office	4	3176	56	188937.804	0.337002	37.4768	12.629761
Office	3	3176	42	141499.672	0.252388	37.4768	9.4587056
Office	2	3176	28	94142.06444	0.167918	37.4768	6.2930328
Office	1	3176	14	46908.17882	0.083669	37.4768	3.1356303
Garage	abve grnd	3176	0	0	0	37.4768	0
Total		17050		560642.7697	1	201.19	201.19

WIND LOAD DESIGN

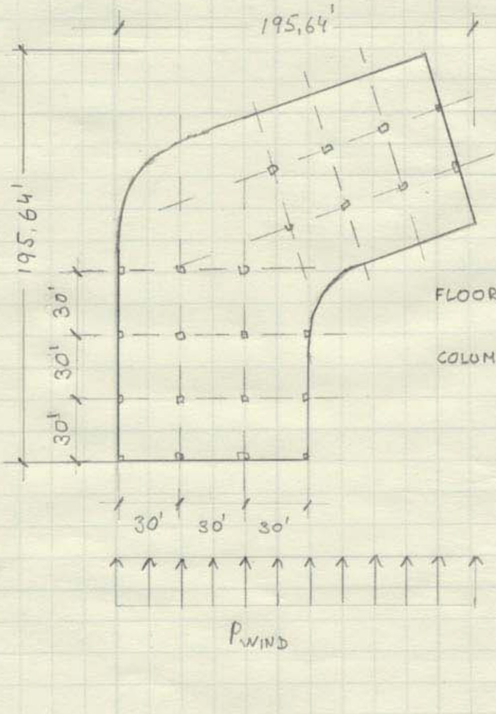
LATERAL LOADS

WIND LOAD CALCULATIONS   ASCE7-02   CH 6.

WIND LOAD DISTRIBUTION



$P_h = 3.93$ PSF

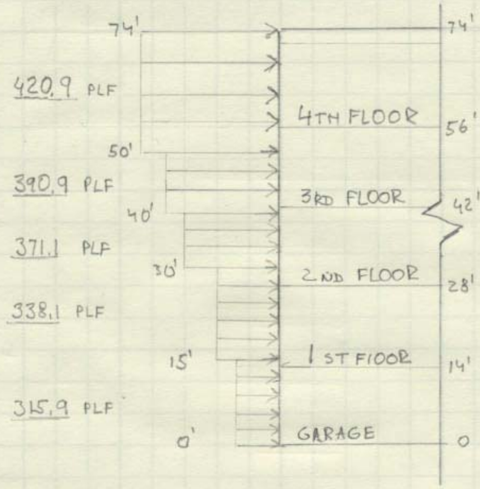


FLOOR HEIGHT = 14'

COLUMN SPAN = 30'

WIND LOAD DESIGN

POINT LOAD AT EACH STORY



74' → $(1.67' + 7')(420.9) = 3.6^K$

56' → $(7' + 6')(420.9) + (7' - 6')(390.9) = 5.9^K$

42' → $(7' - 2')(371.1) + (7' + 2')(390.9) = 5.4^K$

28' → $(7' + 2')(338.1) + (7' - 2')(371.1) = 4.9^K$

14' → $(7' + 1')(315.9) + (7' - 1')(338.1) = 4.6^K$

0

HEIGHT (FT)	P_w (PSF)	W_w (PLF)	POINT LOAD (K)
50-74	14.03	420.9	5.893
40-50	13.03	390.9	5.472
30-40	12.37	371.1	5.195
15-30	11.27	338.1	4.733
0-15	10.53	315.9	4.423

# STORY / HEIGHT	POINT LOAD (K)
ROOF / 74'	3.6
4th / 56'	5.9
3RD / 42'	5.4
2ND / 28'	4.9
1ST / 14'	4.6

LATERAL LOADS

SEISMIC LOAD CALCULATIONS ASCE 7-02 CH 9

OCUPANCY CATEGORY	II	[TABLE 1-1]
SEISMIC GROUP	I	[TABLE 9.1.3]
SITE CLASSIFICATION	C	ASSUME

ACCELERATION

$S_s = 0.335$ (MAP 9.4.1.1a)

$S_1 = 0.085$ (MAP 9.4.1.1b)

SITE CLASS ADJUSTMENT

$F_A = 1.2$ (TABLE 9.4.1.2.4a)

$F_V = 1.7$ (TABLE 9.4.1.2.4b)

$S_{ms} = 1.2(0.335) = 0.402$

$S_{m1} = 1.7(0.085) = 0.145$

DESIGN SPECTURAL RESPONSE

$S_{DS} = 0.667(0.402) = 0.268$

$S_{D1} = 0.667(0.145) = 0.096$

SEISMIC DESIGN CATEGORY

B (TABLE 9.4.2.1a)

B (TABLE 9.4.2.1b)

SEISMIC BASE SHEAR

$V = C_s W$

$C_s = \frac{S_{DS} \cdot I}{R} < C_s = \frac{S_{D1} \cdot I}{T \cdot R}$

$< C_s = 0.044 I\ SAS$

RESPONSE MODIFICATION FACTOR

$R = 8$ (TABLE 9.5.2.2)

I

$I = 1.0$ (TABLE 9.1.4)

T PERIOD

$T_a = 0.51$ (EQ. 9.5.3.3-1)

SEISMIC LOAD DESIGN

BUILDING DEAD LOADS

AVG. FLOOR AREA = 26000 SF

TYP. FLOOR LOAD: FLOOR WEIGHT = (72 PSF)(26000) = $1,872^K$

PARTITION = (20 PSF)(26000) = 520^K

EXTERIOR WALL = (80 PSF)(14')(700') = 784^K

TOTAL STORY WEIGHT = $3,176^K$

TOTAL BUILDING WEIGHT: ROOF = (45 PSF)(26000) = 1,170

4 STORIES + GARAGE = $5(3176)^{\#}$ = 15,880

TOTAL BUILDING WEIGHT = $17,050^K$

$C_S = \frac{0.268}{8(1)} = 0.0335$

$C_{S\,MAX} = \frac{0.096}{0.51(8/1.0)} = 0.0235$

$C_{S\,MIN} = 0.044(1.0)(0.268) = 0.0118$ ← GOVERNS

$\boxed{V = (0.0118)(17050) = 201.1}$ BASE SHEAR

$K = 1 + \frac{(0.51 - 0.5)}{2} = 1.005$ (TABLE 9.5.3.4)

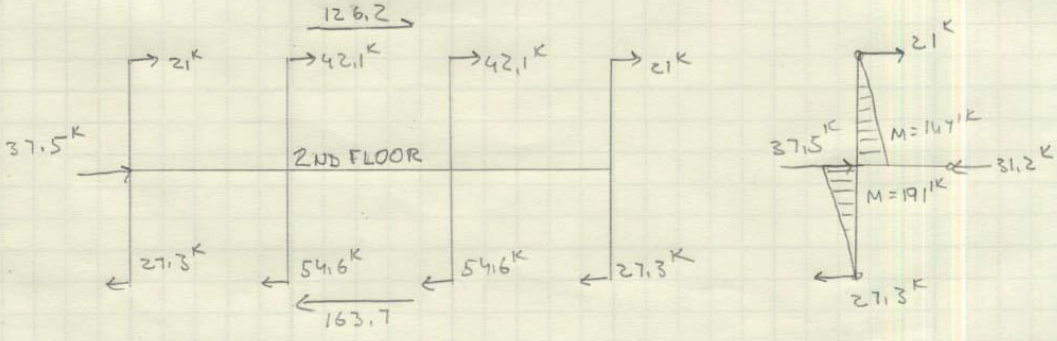
@ 2ND STORY

$V = (0.0118)(17050 - 2(3176)) = 126.236^K$

@ 1ST STORY

$V = (0.0118)(17,050 - 3176) = 163.7^K$

GIVEN:
FRAME MOMENT CONECTION = 40^{1K}



½ MOMENT PER COLUMN = $191^{1K}/7 = 27^{1K} < 40^{1K}$ OK

APPENDIX B

SPOT CHECK

GIRDER CHECK

SPOT CHECK

GIRDER C1-2

DL:
45 PSF CONCRETE
8 PSF ST AND JOISTS
5 PSF COLATERAL
2 PSF CEILING
60 PSF TOTAL

LL:
60 PSF OFFICE OCUP.
20 PSF PARTITION
80 PSF

$\omega = 1.2(60) + 1.6(80) = 200$ PSF

TOTAL UNIFORMLY DISTRIBUTED LOAD:

$$\frac{(200\ \text{PSF})(30\ \text{FT})(30\ \text{FT})}{1000} = \boxed{180^{K}}$$

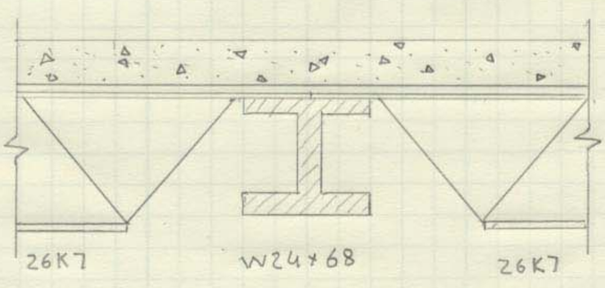
$$L_n = 30\ \text{FT} - \frac{2(12\ \text{IN})(12\ \text{IN/FT})}{2} = 29\ \text{FT}$$

USE TABLE 5.4 LRFD

TRY W24x68

TOTAL CAPACITY = $183^{K} > 180^{K}$ OK

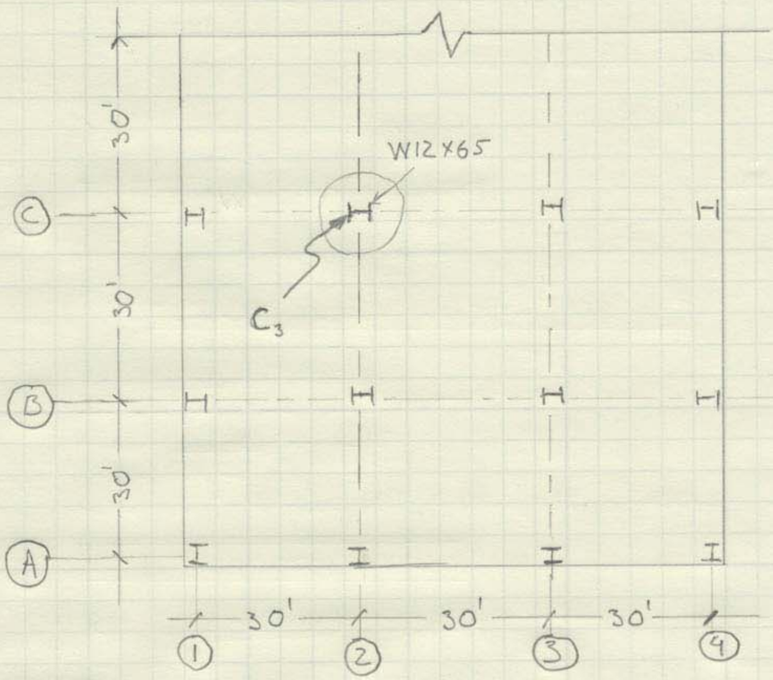
GIRDER W24x68 IS SUFFICIENT TO CARY ALL REQUIRED LOADS



COLUMN CHECK

SPOT CHECK

COLUMN DESIGN



TOTALS:

LL = 80 PSF

DL = 60 PSF

NOTE:

IN THIS DESIGN CHECK MOMENTS PRODUCED BY LATERAL LOADS WILL NOT BE CONSIDERED → EXPEKT SMALLER COLUMN

MOMENT

GIRDER $C1\text{-}2$ SPAN = 30'

TRIB. AREA = (30' FT)(30 FT) = 900 FT^2

$A_T = 900\ FT^2$

$A_I = 2(900) = 1800\ FT^2$

$L_R = 80\ PSF\,(0.25 + 15/\sqrt{1800}) = 48.28\ PSF$

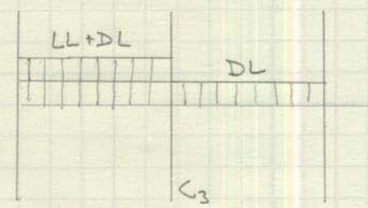
$\omega_{C1\text{-}2} = 1.2(60)(30') + 1.6(48.28)(30') = 4\_.47\ PLF$

COLUMN CHECK

$$FEM_{1-2} = \frac{WL^2}{12} = \frac{(4.47)(30^2)}{12} = 335^{\prime K}$$

GIRDER$_{C2-3}$

WORST CASE:



TRIB AREA = 900 SF

$$w = 1.2(60)(30') = 2,160$$

$$FEM_{C2-3} = \frac{(2.16)(30^2)}{12} = 162^{\prime K}$$

$$\Delta FEM = 335^{\prime K} - 162^{\prime K} = 173^{\prime K}$$

$$M_U = \frac{\Delta FEM}{2} = \frac{173^{\prime K}}{2} = \boxed{86.5^{\prime K}}$$

AXIL LOAD

LL = 80 PSF

DL = 8 STEEL + JOISTS
2 CEILING
5 COLATERAL
45 CONCRETE
60 TOTAL

ROOF
LL = 30 PSF

DL = 25 PSF

$$w = 2[1.2(60) + 1.6(80)] + (1.2(25) + 1.6(30)) = 478 \text{ PSF}$$

$$P = \frac{(30')(30')(478 \text{ PSF})}{1000} = 430.2^K$$

TRY W12 SHAPE COLUMN $d = 12''$

$$\alpha = \frac{24}{d} = \frac{24}{12} = 2$$

ASSUME $KL = (14 \text{ FT} - 2') = 12 \text{ FT}$

$$P_{EFF} = 430.2^K + 2(86.5) = 603.2^K$$

TABLE 4.2 IN LRFD => $\boxed{W\ 12 \times 65}$ $(P_{EFF} = 687^K > 603.2^K)$ OK

$\therefore$ W 12x65 WORKS