# William W. Wilkins Professional Building

Columbus, Ohio

## Technical Assignment 1

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AE 481W
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# Executive Summary

This report consists of a detailed description of the structural system and a structural analysis of a typical bay and braced frame in the William W. Wilkins Professional Building.

The Wilkins building is a 6 story medical office building located in Columbus, Ohio. It is essentially an addition to the Grant Riverside hospital across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing.

The structural system for the Wilkins building begins with caissons drilled 25′-2″, on average, to rest on soil with a bearing value of 16,000psf. On each caisson is a pier. Framing into the piers are grade beams. The ground slab is 4″ concrete reinforced with 6x6-W1.4xW1.4 welded wire fabric (WWF) sitting on 6″ porous fill. Floors 2-6 consist of a 3 ½″ concrete slab on 2″ 18 gage composite steel deck welded to the support steel. These slabs are reinforced with 6x6-W2.1xW2.1 WWF. Floor framing generally consists of a W16x31 beam connected compositely to the floor slab. Beams frame into a W24x55 girder. Columns are ASTM 992 Grade 50 rolled W12 steel shapes.

The lateral system consists of V-bracing. Lateral bracing is located on the North and South faces of the building in the West end bay. Additionally there are two braced frames spanning North-South and one spanning East-West in the stairwell, elevator shaft area.

A spot check was performed on a typical floor and roof bay. For the most part the results of these checks showed that the building was somewhat over designed. In addition to floor bays a check was made on a braced frame resisting North-South lateral loads. I found from my lateral analysis that seismic was going to control in the North-South direction. At this time I do not have the actual seismic loads calculated. My lateral analysis resulted in far higher forces in bracing members than the original design accounted for. These differences could be for several reasons which will be discussed in detail later in the report.

# Codes

Building Code:
Ohio Basic Building Code, 1998 (OBBC)

Loads:
ASCE 7-98

Structural Concrete:
The American Concrete Industry (ACI 301-89, 315, 318)

Structural Steel:
Specifications for Design, Fabrication and Erection of Structural Steel for Buildings, latest edition (AISC-ASD)

Welding:
American Welding Society, latest edition (AWS D1.1)

Steel Deck:
Steel Deck Institute Specifications, latest edition (SDI)

Cold Formed Light Gage Steel Framing:
Design of Cold-Formed Steel Structural Members, latest edition (AISI)

# Material Strengths

| | |
|---|---|
| Rolled W shapes | ASTM 992 GR 50 |
| Tubing | ASTM A500 GR B |
| Pipes | ASTM A501 or A53 type E or S |
| Other Structural Steel | ASTM 36 |
| WWF | ASTM A-185 |
| Rebar | ASTM A-615 GR 60 |
| Shear Studs | ASTM A108 GR 101, 1015, 1017 or 1020 |
| Bolts | ASTM A325 or A490 |
| Welds | E70XX or better |
| Caissons | 3000psi |
| All Other Concrete | 3500psi |

# Structural Systems

Foundation:

The foundation for the William W. Wilkins Professional Building consists of piers resting on caissons with grade beams spanning between. See Figure 1 below. Caissons are drilled an average of 25′-2″ to rest on sand/gravel with a bearing pressure of 16,000psi. Concrete with a minimum 28 day strength of 3,000psi was used for the caissons. Ranging in diameter from 48″ to 84″ these caissons are reinforced with #9, 10 or 11 bars with #3 or 4 stirrups at 12 or 18 inches. Piers and grade beams have a minimum 28 day strength of 3,500psi. On average piers are 1′x1′ while grade beams vary from 12″x32″ to 24″x32″. Both are reinforced with #6, 7 or 8 bars with #3 stirrups at 12″.

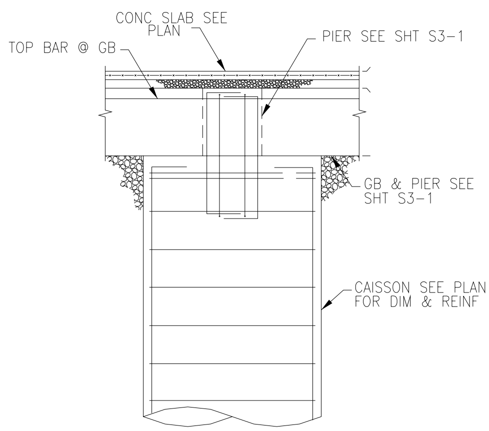



Figure 1: Typical Caisson Detail

Floor system:

The floor system in the Wilkins building is designed compositely. Floor slabs comprise of 3 ½″ normal weight concrete on 2″ 18 gage composite steel deck reinforced with W2.1xW2.1 welded wire fabric (WWF). Decking is welded to support steel. Slab on grade (SOG) varies slightly consisting of 4″ concrete on 6″ porous fill reinforced with W1.4xW1.4 WWF. A typical bay consists of W16x31 beams spanning 32′-4″ in the East-West direction framing into W24x55 girders spanning 30′-9″ in the North-South direction. ¾″ diameter by 4 ½″ long headed studs are spaced evenly across members to transfer loads. Roof framing of a typical bay utilizes the same size members only non-composite. On the East face there is a slight overhang supported by W12x14 beams framing into W16x26 girders. Moment connections are used where beams connect to columns and girders.

## Columns:

Columns are ASTM 992 Grade 50 rolled W12 steel shapes with splices on the third and fifth floors. Splice connections use welds and ¾" diameter A325 bolts, web bolts are slip critical, to connect plates. See Figure 2 below. The largest columns are W12x136 and are part of the lateral system as well. Gravity columns range from W12x40s at the roof level to a max of W12x106 at ground level. Base plates are either 18x18 or 20x20 with thicknesses ranging from 1 ¾" to 2 ½". Connections consist of (4) anchor bolts of varying sizes. See Figure 3 below.

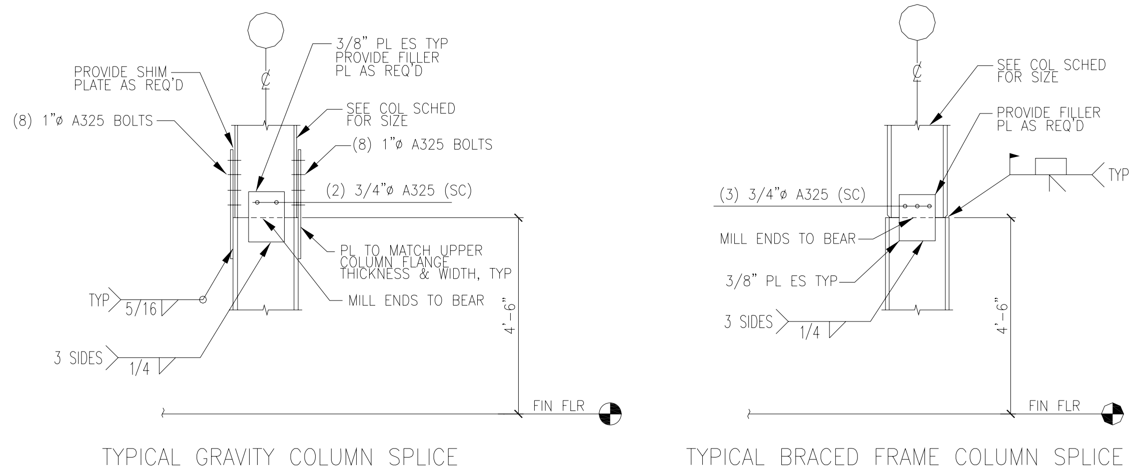



Figure 2 Typical Column Splice Details

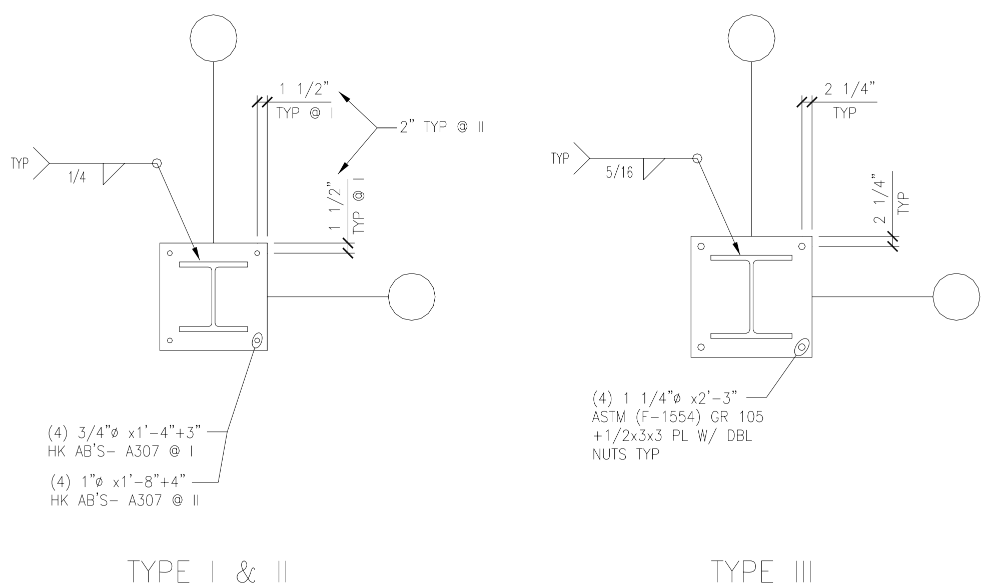



Figure 3: Typical Base Plate Details

Lateral System:

Lateral loads are resisted in the Wilkins building using braced frames. Two frames spanning North-South are located near the elevator shafts. Frames spanning East-West are split up with one by the elevator shafts, one on the exterior South-East bay and one on the exterior North-East bay. Lateral bracing in these frames are ASTM A500 Grade B tubes ranging in size from TS5x5x.1875 to TS8x8x.25. A typical braced frame is shown in Figure 4 below. The tube steel is welded to gusset plates that connect to main framing members.

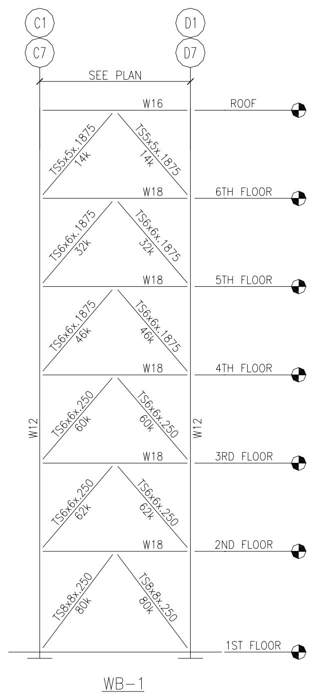



Figure 4: Typical Braced Frame

# *Loads*

| | Existing loads (psf) | My loads (psf) | Source |
|---|---|---|---|
| Live: | | | ASCE 7-05 |
| Office floor | 50 | 50 | |
| Corridors | 80 | 80 | |
| Lobbies & Stairs | 100 | 100 | |
| Library Stacks | 150 | 150 | |
| Roof | | 20 | |
| Snow | 25 + Drift | 20 | |
| | | | |
| Dead: | | | |
| Partitions | 20 | 10 | ASCE 7-05 |
| Metal Deck | | 2.4 | Catalogues |
| Concrete | | 54 | Catalogues |
| Beams | varies | varies | LRFD |
| Ceiling | 5 | 2 | |
| MEP | | 3 | |
| Misc. | 5 | 3.6 | |
| Total: | 30 | 75 | |
| | | | |
| Roof Dead: | 22 + steel | 22 + steel | |

| Wind(MWFRS) | Psi |
|---|---|
| 0-20' | 16 |
| 21''40' | 19 |
| 41'-60' | 21 |
| 61'-80' | 23 |
| 81'-100' | 24 |

My wind:

For complete calculations see Appendix.

| $P=qGCp - q_i(GCp_i)$ | | | | | | |
|---|---|---|---|---|---|---|
| | Windward (psf) | | Leeward (psf) | | Total (psf) | |
| height | N-S | E-W | N-S | E-W | N-S | E-W |
| 0-15' | 9.813934 | 9.813934 | -7.65776 | -10.0237 | 17.47169 | 19.83761 |
| 20' | 10.4132 | 10.4132 | -7.65776 | -10.0237 | 18.07096 | 20.43688 |
| 25' | 10.89262 | 10.89262 | -7.65776 | -10.0237 | 18.55038 | 20.9163 |
| 30' | 11.37204 | 11.37204 | -7.65776 | -10.0237 | 19.0298 | 21.39572 |
| 40' | 12.09116 | 12.09116 | -7.65776 | -10.0237 | 19.74892 | 22.11484 |
| 50' | 12.69043 | 12.69043 | -7.65776 | -10.0237 | 20.34819 | 22.71411 |
| 60' | 13.16985 | 13.16985 | -7.65776 | -10.0237 | 21.30702 | 23.67294 |
| 70' | 13.64926 | 13.64926 | -7.65776 | -10.0237 | 21.30702 | 23.67294 |
| 80' | 14.12868 | 14.12868 | -7.65776 | -10.0237 | 21.78644 | 24.15236 |
| 84.67 | 14.24854 | 14.24854 | -7.65776 | -10.0237 | 21.90629 | 24.27221 |

Seismic:

For complete calculations see Appendix.

| Story | $F_x$(k) |
|---|---|
| 2 | 24.26172 |
| 3 | 47.07634 |
| 4 | 69.43915 |
| 5 | 91.70517 |
| 6 | 114.1482 |
| R | 47.09417 |

# Spot Checks

Through spot checks I have found that the building is slightly over designed in areas. W16x31 beams connecting to W24x55 girders are used almost exclusively. For example, I found for a typical floor bay a W14x22 beam would suffice with a W21x50 girder. On the roof I found a W14x22 beam with a W24x55 girder would work. My analysis of a typical column yielded mixed results. I found some of the members to be over designed and some to be under designed.

There are several possible reasons for these differences. In terms of design procedure ASD was utilized in the initial design, however, I performed my checks using LRFD. Also, framing plans indicate that the same beam size was used almost exclusively. This was probably done for ease of construction. My check was done on a typical office bay. Other bays consisting of corridors will have higher live loads causing greater reactions. On the roof bays I did not take into account the loads due to roof top units. It is for these reasons I feel my designs are slightly different.

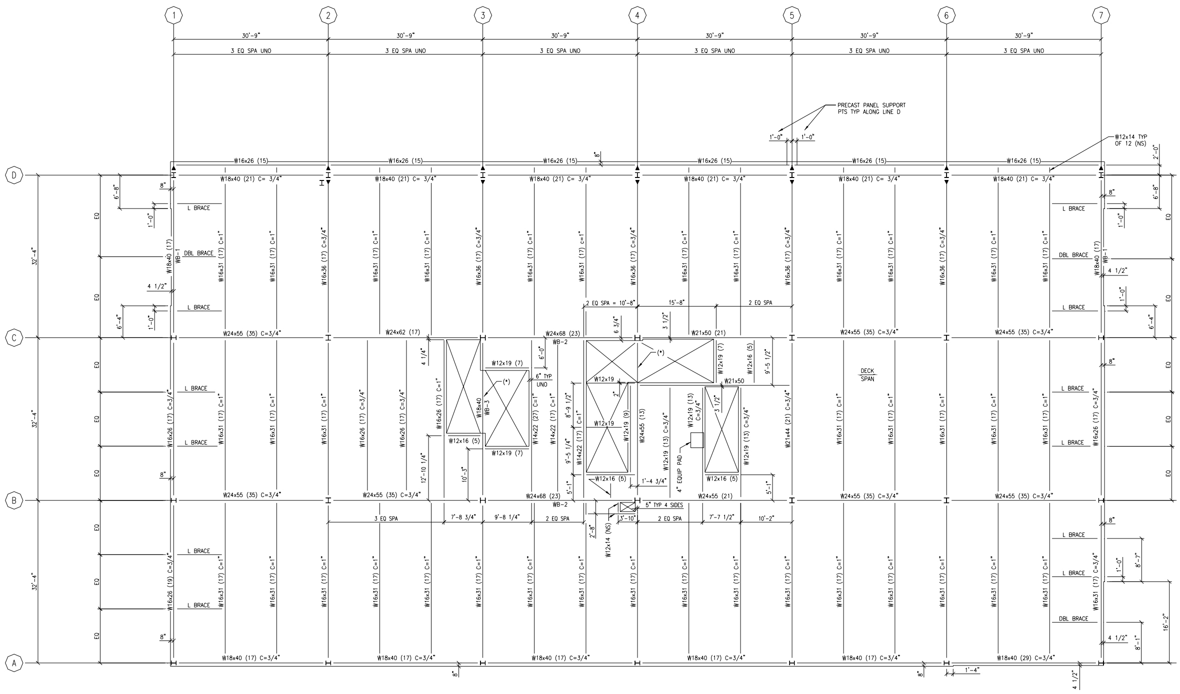
In checking a braced frame I found the member forces to be significantly higher than the design engineer found. I do not have the seismic loads calculated at the time of design to compare my loads to. I did find the wind loads to be the same, however, I found seismic to control. This may be due to differences in seismic loads due to changes in the code. The design engineer used ASCE 7-98 to calculate loads whereas I used ASCE 7-05. The Ohio Basic Building Code may not have mandated that the building be designed for seismic loads. If this is the case wind would control, which would explain the lower forces. This may account for some of the differences. Further investigation will be performed to verify loads.

Future items to be checked include cladding and roof design, foundation design and drift and deflection of the building. I will need to consider roof uplift. Design of Windows need to be checked for pressure differentials, transfer of loads, resistance to seismic and high wind loadings to maintain a fully enclosed building and projectile resistance. The foundation will need to be checked for stability, bearing pressure and possible uplift from the water table level.

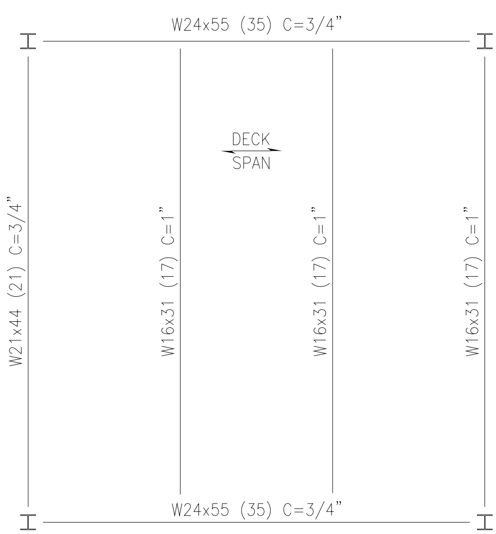
For complete calculations see Appendix.

# *Appendix*

## A1. Typical Floor Plan



## A2. Enlarged Typical Bay



## A3. Wind Loads

| Height | $K_z$ | $q_z$ |
|---|---|---|
| 0-15' | 0.57 | 10.04659 |
| 20' | 0.62 | 10.92787 |
| 25' | 0.66 | 11.6329 |
| 30' | 0.7 | 12.33792 |
| 40' | 0.76 | 13.39546 |
| 50' | 0.81 | 14.27674 |
| 60' | 0.85 | 14.10048 |
| 70' | 0.89 | 15.68678 |
| 80' | 0.93 | 16.39181 |
| 84.67 | 0.94 | 16.56806 |

| | |
|---|---|
| G | 0.85 |
| Gcpi | 0.18 |

| Cp | | |
|---|---|---|
| | Leeward | Windward |
| N-S | 0.332 | 0.8 |
| E-W | 0.5 | 0.8 |

| $P=qGCp - q_i(GCp_i)$ | | | | | | |
|---|---|---|---|---|---|---|
| | Windward(psf) | | Leeward(psf) | | Total(psf) | |
| height | N-S | E-W | N-S | E-W | N-S | E-W |
| 0-15' | 9.813934 | 9.813934 | -7.65776 | -10.0237 | 17.47169 | 19.83761 |
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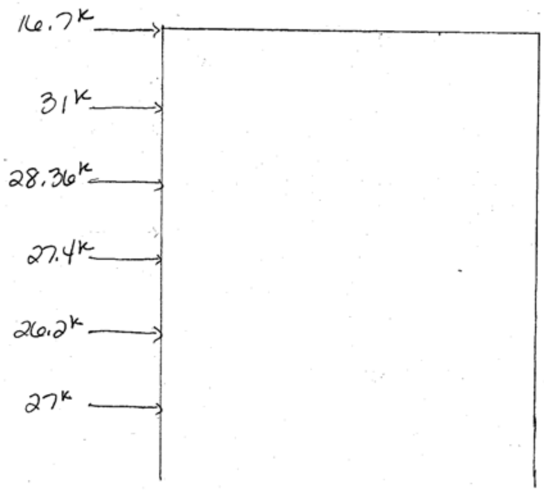
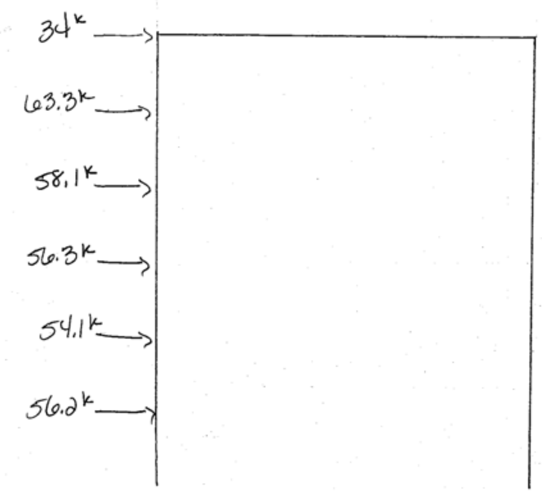
| 60' | 13.16985 | 13.16985 | -7.65776 | -10.0237 | 20.82761 | 23.19353 |
| 70' | 13.64926 | 13.64926 | -7.65776 | -10.0237 | 21.30702 | 23.67294 |
| 80' | 14.12868 | 14.12868 | -7.65776 | -10.0237 | 21.78644 | 24.15236 |
| 84.67 | 14.24854 | 14.24854 | -7.65776 | -10.0237 | 21.90629 | 24.27221 |

| | | | Story Force (kip) | | Cumulative Shear (kip) | | OM (ft-kip) | |
|---|---|---|---|---|---|---|---|---|
| Floor | Height | Trib. Ht | N-S | E-W | N-S | E-W | N-S | E-W |
| 1 | 0 | 0 | 0 | 0 | 156.66 | 322.3 | 7567.7418 | 15526.36 |
| 2 | 16.33 | 14.835 | 27 | 56.2 | 156.66 | 322.3 | 440.91 | 917.746 |
| 3 | 29.67 | 13.335 | 26.2 | 54.1 | 129.66 | 266.1 | 777.354 | 1605.147 |
| 4 | 43 | 13.33 | 27.4 | 56.3 | 103.46 | 212 | 1178.2 | 2420.9 |
| 5 | 56.33 | 13.335 | 28.36 | 58.1 | 76.06 | 155.7 | 1597.5188 | 3272.773 |
| 6 | 69.67 | 14.17 | 31 | 63.6 | 47.7 | 97.6 | 2159.77 | 4431.012 |
| Roof | 84.67 | 7.5 | 16.7 | 34 | 16.7 | 34 | 1413.989 | 2878.78 |




N-S




E-W

## Wind Loads

Category II ⇒ $I = 1.0$

$V = 90\text{mph}$

Bldg: $h = 84'-8'' = 84.67'$
$101'-6''$ x $187'-2''$
↑ E-W ↑ N-S

$K_d = 0.85$
$K_{zt} = 1.0$

Exposure = B
$K_h$ = Case 2 → 0.94

$K_z$ - varies

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

$= 0.00256(1.0)(0.85)(90)^2(1.0) K_z$
$= 17.6256 K_z$

flexible if $f \le 1.0\text{Hz}$ $\quad f = \frac{1}{T}$

$T_a = C_t h_n^x \quad = 0.02(84.67)^{0.75} = 0.56$

Table 12.8-2 $\quad C_T = 0.02 \quad x = 0.75$

$f = \frac{1}{0.56} = 1.8 > 1.0$ ∴ Rigid

$G = 0.85$

$GC_{pi} = \pm 0.18$ fig. 6-5

$C_p$: windward : 0.8
fig. 6-6 leeward : N-S = $187'-2'' / 101'-6'' = 1.84$ ⇒ 0.332
E-W = $101'-6'' / 187'-2''$ ⇒ 0.5

$P = qGC_p - q_i(GC_{pi})$

for windward $\quad q = q_z$
$q_i = q_h$

for leeward $\quad q_i = q_h$
$q = q_h$

2nd floor

N-S = [9.81(6.835) + 10.4(5) + 10.9(3) + 7.66(14.835)] (101'-8")
= [151.75 + 113.64](101'-8") = 27^k
E-W = [151.75 + 14.835(10.02)](187'-2")
= 56.2^k

3rd floor

N-S = 101'-8" [6.335(12.1) + 11.4(5) + 10.9(2) + 7.66(13.335)]

= 26.2^k

E-W = 187'-2" [13.335(10.02) + 155.45]

= 54.1^k

4th floor

N-S = 101'-8" [7.66(13.33) + 12.1(3.665) + 12.7(9.665)]
= 27.4^k

E-W = 187'-2" [10.02(13.33) + 167.1]
= 56.3^k

5th floor

N-S = 101'-8" [7.66(13.335) + 12.7(0.335) + 13.17(10) + 13.6(3)]
= 28.36^k

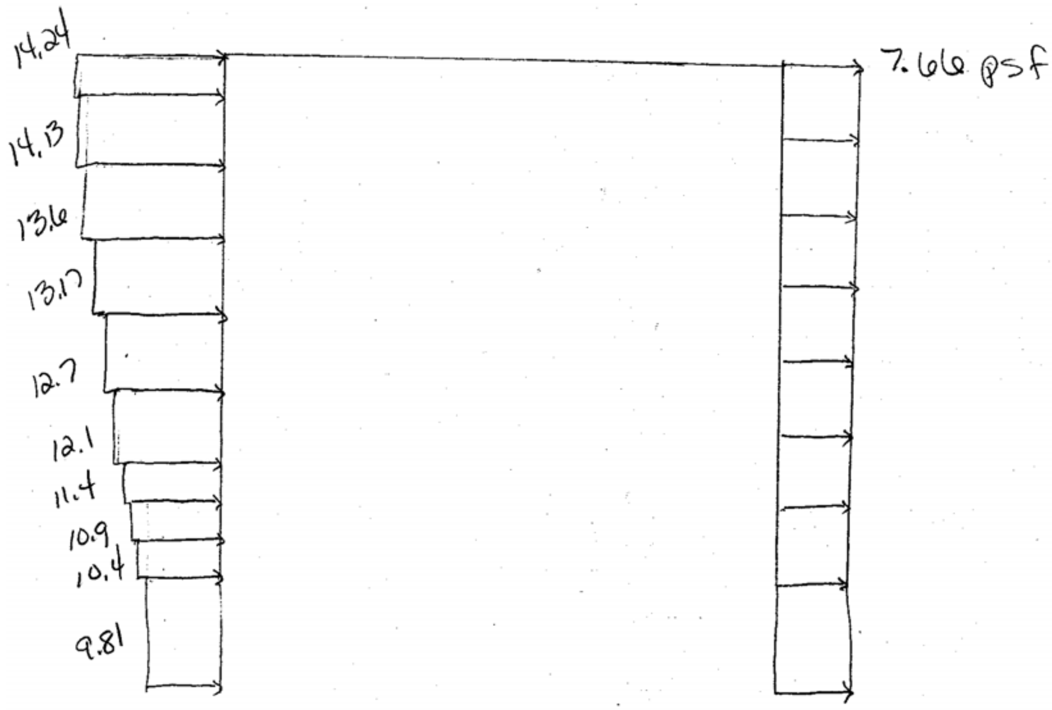
E-W = 187'-2" [10.02(13.335) + 176.75]
= 58.1^k

6th floor

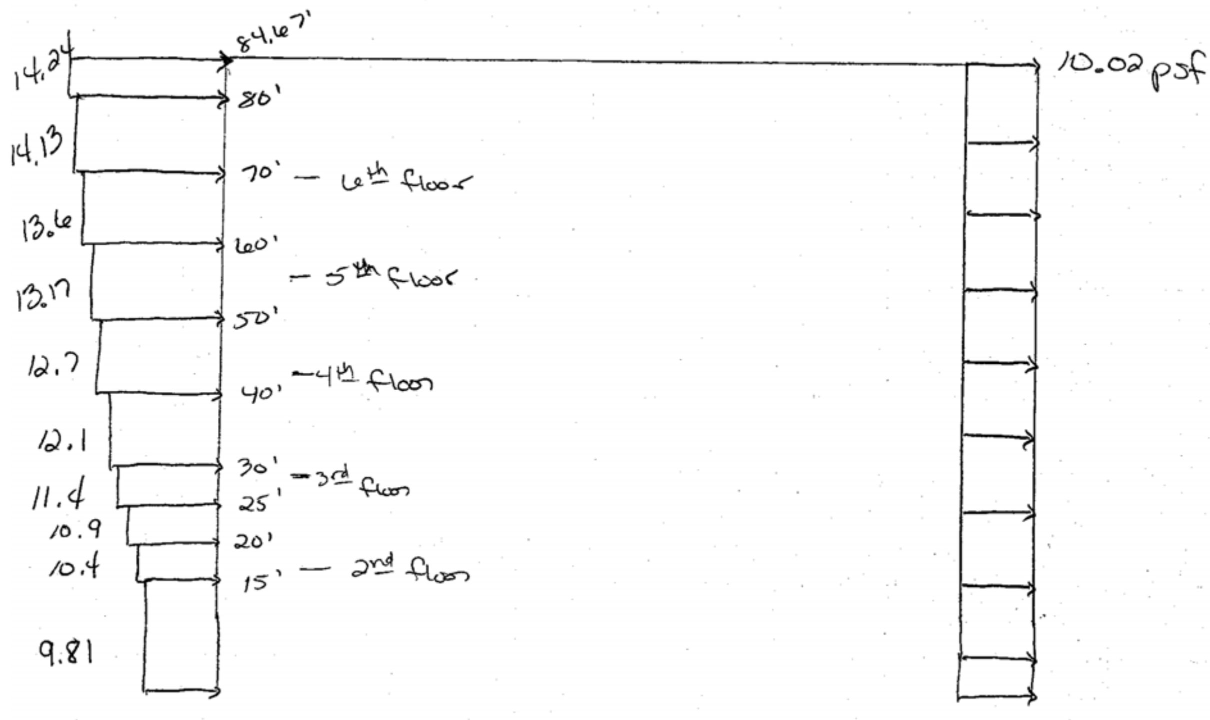
N-S = 101'-8" [7.66(14.17) + 13.6(7) + 14.13(7.17)]
= 31^k
E-W = 187'-2" (10.02(14.17) + 196.5]
= 63.3^k

Roof

N-S = 101'-8" [7.66(7.5) + 14.13(2.83) + 14.24(4.67)]
= 16.7^k
E-W = 187'-2" [10.02(7.5) + 106.5]
= 34^k


14.24
14.13
13.6
13.17
12.7
12.1
11.4
10.9
10.4
9.81
7.66 psf

N-S

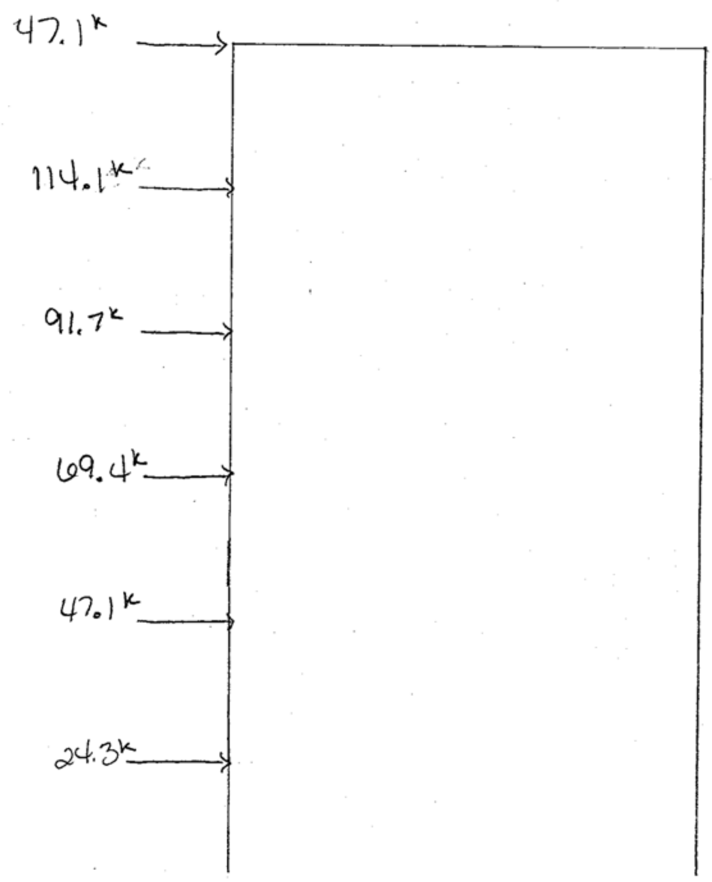

84.67'
80'
70' — 6th floor
60'
— 5th floor
50'
40' —4th floor
30' —3rd floor
25'
20'
15' — 2nd floor
14.24
14.13
13.6
13.17
12.7
12.1
11.4
10.9
10.4
9.81
10.02 psf

E-W

**A4. Seismic**

| Story | V | k | $h_x$ | Area (sq. ft.) | $W_x$ | $h_x^k W_x$ | $C_{vx}$ | $F_x(k)$ |
|---|---|---|---|---|---|---|---|---|
| 2 | 393.7248 | 1.03 | 16.33 | 18023.4 | 1441.872 | 25603.69 | 0.061621 | 24.26172 |
| 3 | 393.7248 | 1.03 | 29.67 | 18906.3 | 1512.504 | 49680.23 | 0.119567 | 47.07634 |
| 4 | 393.7248 | 1.03 | 43 | 19029.3 | 1522.344 | 73279.97 | 0.176365 | 69.43915 |
| 5 | 393.7248 | 1.03 | 56.33 | 19029.3 | 1522.344 | 96777.58 | 0.232917 | 91.70517 |
| 6 | 393.7248 | 1.03 | 69.67 | 19029.3 | 1522.344 | 120462 | 0.289919 | 114.1482 |
| R | 393.7248 | 1.03 | 84.67 | 19029.3 | 513.7911 | 49699.04 | 0.119612 | 47.09417 |

| | |
|---|---|
| $\sum W_x =$ | 8035.199 |
| $\sum h_x^k W_x =$ | 415502.5 |



## Earthquake Loads

Occupancy Cat. II $\Rightarrow$ $I = 1.0$

$S_S = 0.15$
$S_1 = 0.06$

Site Class D

$F_v = 2.4$
$F_a = 1.6$

$S_{MS} = F_a S_S$
$= 1.6(0.15) = 0.24$

$S_{M1} = F_v S_1$
$= 2.4(0.06) = 0.144$

$S_{DS} = \frac{2}{3} S_{MS}$
$= \frac{2}{3}(0.24) = 0.16$

$S_{D1} = \frac{2}{3} S_{M1}$
$= \frac{2}{3}(0.144) = 0.096$

Design Category

$S_{DS} \Rightarrow A$
$S_{D1} \Rightarrow \boxed{B}$

$R = 3.25$

$$C_S = \frac{S_{DS}}{\frac{R}{I}}$$

$$C_S = \frac{0.16}{3.25} = 0.049$$ use this

$T_a = 0.56$
$T_L = 12$

$T_a < T_L$

$$C_S \leq \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.096}{0.56(3.25)} = 0.054$$

**A5. Snow Loads**

Snow Loads

$P_f = 0.7 C_e C_t I P_g$

terrain Category B
partially exposed
$C_e = 1.0$
$C_t = 1.0$
$I = 1.0$
$P_g = 25psf$

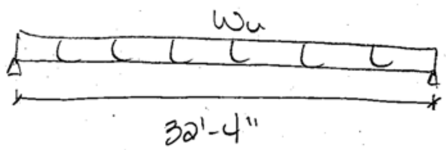
$P_f = 0.7(25) = 17.5psf$

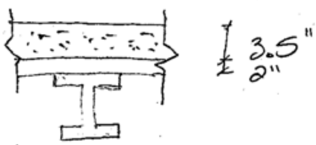
$P_{fmin} = 20I = 20psf$

**A6. Spot Checks**

Floor bay- beams and girders
Roof bay- beams and girders
Typical column
Lateral bracing along column lines B&C

## Check Beams




DL = 80psf
LL = 50psf ← designing for typical office area

30'-9" bay ⇒ 3 equal spaces

Trib. width = $\frac{30'\text{-}9"}{3}$ = 10'-3"

$w_u = [1.2(80) + 1.6(50)](10'\text{-}3") = 1.8$ klf

$b_{eff} \le \frac{1}{4}$ span = 0.25(32'-4") = 97" ← controls

$\le$ spacing = 10'-3" = 123"

Assume 1 weak stud per rib ~ $Q_n = 17.2^k$

for 3/4" $\phi$ normal weight concrete
$f'_c$ = 3500psi

$V_u = \frac{w_u l}{2} = 29^k$

$M_u = \frac{w_u l^2}{8} = 235.2^{1k}$

assume $a = 1"$, $Y_2 = 3.5 + 2 - \frac{1}{2} = 5$

| Shape | $\phi M_n$ | $\Sigma Q_n$ | # Studs | Equiv. wt | a | $Y_2$ |
|---|---|---|---|---|---|---|
| W16x31 | 296 | 114 | 14 | 1142plf | .04 | 5.3 |
| W16x26 | 245 | 96 | 12 | 961 | 0.33 | 5.34 |
| W14x30 | 259 | 111 | 14 | 1110 | 0.38 | 5.31 |
| W14x22 | 241 | 199 | 24 | 951 | 0.69 | 5.16 |

$a = \frac{\Sigma Q_n}{0.85 f'_c b}$

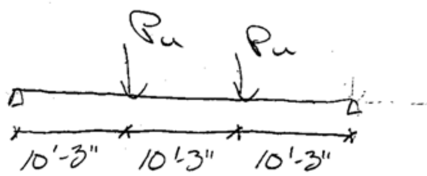
$= \frac{\Sigma Q_n}{288.6}$

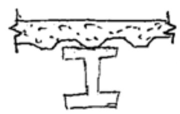
all $Y_2$'s are larger than assumed. As $Y_2$ increases so does moment capacity so all choices are ok.
An extra 5psf was added to the dead load as self weight allowance. This equates to 51.25plf which is greater than any of the above beam self weights.

W14x22 is most economical, however it is more likely to have deflection problem under construction loads. W16x26 is the next economical choice.

The engineer used a W16x31 with 17 shear studs. Based on my calculations this is overdesigned. Part of the difference may be due to my use of LRFD compared to their design in ASD.
My feeling, however, is that a W16x31 was used to keep the structural system simple and uniform. In other areas such as corridors a larger live load is applied. For construction simplicity they may have chosen a slightly larger member to maintain uniformity.

## Check Girders



$P_u = 2(29) = 58^k$

$V_u = 58^k$
$M_u = 58(10.25) = 594.5^{'k}$

Deck parallel, don't know what brand used. Assume $\frac{w_r}{h_r} < 1.5$

and use $Q_n = 18.3^k$ to be conservative.

$b_{eff} \leq 32'-4"$
$\leq \frac{1}{4}(30'-9") = 92.25" \leftarrow$ controls

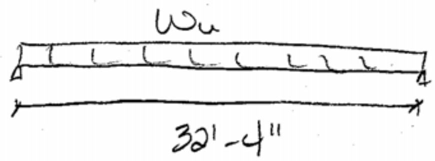
assume $a = 1$
$Y_2 = 5.5$

| Shape | $\phi M_n$ | $\Sigma Q_n$ | # Studs | Equiv Wt | a | $Y_2$ |
|---|---|---|---|---|---|---|
| W18x50 | 610 | 414 | 46 | 1998 | 1.5 | 4.75 |
| W18x46 | 605 | 492 | 54 | 1955 | 1.79 | 4.6 |
| W21x48 | 635 | 355 | 40 | 1876 | 1.3 | 4.85 |
| W21x50 | 624 | 285 | 32 | 1858 | 1.04 | 4.98 |
| W24x55 | 685 | 204 | 24 | 1931 | 0.74 | 5.13 |
| [illegible] | [illegible] | [illegible] | | | | |

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{\Sigma Q_n}{274.4}$

W21x50 is the most economical choice. $Y_2$ is close enough to 5' to still work. The design engineer used a W24x55 with 35 studs. These differences could be from different design methods (ASD vs. LRFD). Or again, possibly from beams with higher live loads, this will place a larger reaction on the girder.

## Check Roof Beam




DL = 25 psf
LL = 20 psf

Trib width = 10'-3"

$w_u = [1.2(25) + 1.6(20)](10'\text{-}3")$

$= 636\ plf$

$V_u = 10.3^k$

$M_u = 84.9^{'k}$

$\Delta = \frac{l}{360} = 1.089"$

$$\Delta_L = \frac{5(20)(10.25)(32.67)^4(12)^3}{384(29{,}000{,}000)I}$$

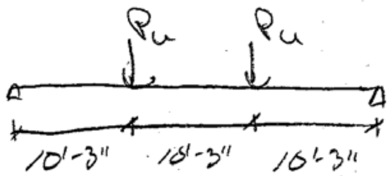
$I = 166.4$

W14x22 $I = 199\ in^4$, $\phi M_p = 123^{'k}$, $\phi V_n = 85.1^k$ OK

If unbraced W12x53 $\phi M_n = 155^{'k}$, $I = 425$ $\phi V_n = 113^k$

Design engineer used W16x31. This may be to keep uniform with all other beams in building. I would imagine that the added load from roof top units will require an increase in member size.

Check Roof Girder




$P_u = 2(10.3) = 20.6^k$

$V_u = 20.6^k$
$M_u = 20.6(10.25) = 211.15^{'k}$

$\Delta_L = \frac{l}{360} = 1.025''$

$$\Delta = \frac{Pa}{24EI}(3l^2 - 4a^2) = \frac{20.6(10.25)}{24(29,000)I}\left(3(30.75)^2 - 4(10.25)^2\right)(12)^3$$

$I = 1236$

W24x55 $I = 1360\ in^4$
$\phi M_p = 506^{'k}$
$\phi V_n = 252^k$ OK

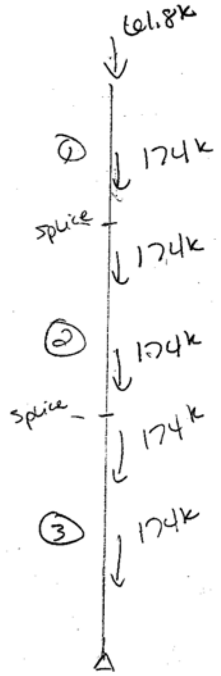
Design Engineer also has a W24x55.

## Check Column

Typical Column will have girders on 2 sides and beams on 2 sides.

At each floor $P_u = 2(29) + 2(58) = 174^k$

Check Column B6 typ. gravity column




Longest unbraced length for roof - 5th $L_b = 15'$

$L_b$ 5th - 3rd $= 13.33'$

$L_b$ 3rd - ground $= 16.33'$

Roof Load $= 2(10.3) + 2(20.6) = 61.8^k$

Gravity loads only - not taking into account RTUs.

Section 1 : $P_u = 235.8^k$
$L_b = 15'$

W12x40 $P_u = 265^k$

Section 2 : $P_u = 583.8^k$
$L_b = 13.33'$

W12x65 $P_u = 661^k$

Section 3 : $P_u = 931.8^k$
$L_b = 16.33'$

W12x106 $P_u = 989^k$

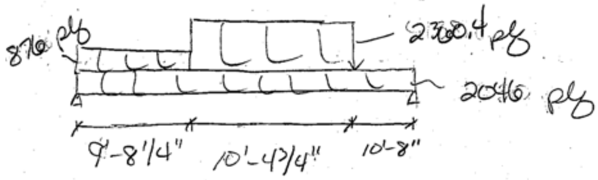
Design Engineer used W12x45, W12x65 and W12x96 for sections 1-3 respectively. Differences may be due to ASD vs LRFD, load differences or due to not including roof top units.

Lateral Check:

$$\left.\begin{array}{ll} \text{Corridor} \Rightarrow (1.2(80) + 0.5(80))(6) = 816 \\ \text{Office} \Rightarrow (1.2(80) + 0.5(50))(10.167) = 1230.1 \end{array}\right\} 2046$$

$$\text{Lobby} \Rightarrow (1.2(80) + 0.5(100))(6) = 876$$

$$\text{stairs} \Rightarrow (1.2(80) + 0.5(100))(16.167) = 2360.4$$




$$30.75 V_{uL} = 2046\frac{(30.75)^2}{2} + 876(9.6875)(25.91) + 2360.4(10.4)(15.87)$$

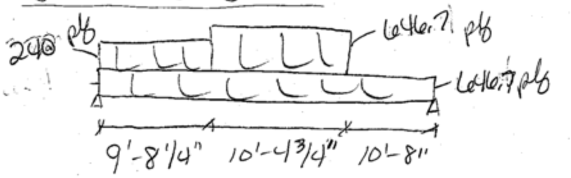
$V_{uL} = 51.3^k$ $V_{uR} = 44.65^k$

typical floor girder loading

$1.2(25) + 0.5(20) = 40$ psf

$40(16.167) = 646.7$ plf
$40(6) = 240$ plf

typical roof girders:




$V_{uL} = 15.37^k$

$V_{uR} = 13.57^k$

Left Column roof

$$\frac{646.7(30'\text{-}9")}{2} = 9.9^k$$

$$7.96(16.167)\underbrace{(1.2(25) + 0.5(20))}_{40} = 5.15^k$$

$P_u = 15^k$

Right Column roof

$9.9^k$

$$13.3(40)(16.167) + 40(8.35)(2.08) = 9.3^k$$

$P_u = 19.2^k$

Left column floors

$$\frac{2046 (30'\text{-}9'')}{2} = 31.5^k$$

$$(\underbrace{1.2(80) + 0.5(50)}_{121})(8'\text{-}6'')(7'\text{-}8.75'') + 121(16.167)(7'\text{-}2\tfrac{3}{4}'') = 22.9^k$$

$P_u = 54.4^k$

right column floors

$31.5^k$

$121(12.96)(7.6) = 11.9^k$

$P_u = 43.4^k$

$1.2D + 1.6W + 0.5L$
$1.2D + 1.0E + 0.5L$

Seismic base shear = $393.7^k$
wind base shear N-S direction = $156.7^k$

$1.6(156.7) = 250.72^k$

Seismic controls

2 braced frames so each takes half

$F_2 = 12.13^k$
$F_3 = 23.5^k$
$F_4 = 34.7^k$
$F_5 = 45.85^k$
$F_6 = 57^k$
$F_R = 23.55^k$

Results from RISA 3D analysis