# Crossroads at Westfields

**Building II**

**Chantilly, Va**

**STEPHEN LUMPP**

**Structural option**

**Faculty Consultant: Dr. Andres Lepage**

**Technical Report 1**

## EXECUTIVE SUMMARY

This report contains analysis regarding the original design of Building II of the Crossroads at Westfields. It covers an overview of the structural systems, applicable codes and design loads including gravity and lateral loads. Even though two different types of code were used the designs came out relatively similar in many cases. Several spot checks were made and some discrepancies were revealed but overall the designs are close.

Again, even though two different codes were used, IBC 2003 for the original design and ASCE 7-05 for this report, most loads came out very similar. The dead load for the typical floor and roof were the only loads that had any differences. All live loads and snow loads were identical. A detailed analysis was conducted for the lateral forces of wind and seismic, resulting in seismic loads controlling. The calculated dead loads for this report were slightly higher resulting in a conservative design of self weight of the building and spot checks. This may be the reason why seismic loads controlled in a non-seismic region.

The spot checks revealed that the dead load calculation was fairly accurate, about 14% higher for the floors and about 7% higher for the roof. The composite beam and girder check were very accurate with respect to the actual design. The overdesign of the framing members in this report are due to the slight increase of dead load. The column spot check was also very accurate with the exception of how the analysis was completed. The analysis in this report assumed a new design for each floor while the actual design had 2 columns spliced only once. This resulted in the original design having a more conservative approach. The extra framing at the west end of the building was verified when solving for snow loads. The 9.5' tall screen wall coupled with the roof extending 275' in length allowed for snow drift. The surcharge load was then solved for and verified the original design.

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## OVERALL INTRODUCTION

The Crossroads at Westfields are two identical office buildings mirroring each other on site. Although the project is currently on hold, these two buildings will offer over 300,000 GSF of office space to future tenants. Located in the Westfields Corporate Center in Chantilly, Virginia, the site is located at the crossing of the Stonecroft Blvd. and Lee Rd., hence the name.

**Site Plan**

Building II, identical to Building I, is a 5- story office building with floor plans that offer column-free spans of over 41 feet. The large open floor plan creates long spans that require the beams to be cambered to pass deflection criteria. The structure consists composite steel beam framing with moment frames to resist lateral loading. The roof is supported by joists and steel decking, and the future mechanical units will have composite slab pads similar to each floor.

**Typical Floor Plan**

## FOUNDATION SYSTEMS

The Foundation system consists of reinforced cast-in-place concrete spread footings. According to the Geotechnical report recommendations prepared by ECS, Ltd the allowable soil bearing values vary throughout the site. Foundations bearing on the natural 'weathered rock' soil classification will be designed with an allowable soil bearing of 6000 psf while foundations bearing on engineered fill will be designed for soil bearing of 3000 psf. The concrete strength shall be 3000 psi.

According to recommendations in the Geotechnical Report, the Slab on Grade will bear on the natural soil. The slab is a 4" thick cast-in-place concrete with 6x6–10/10 welded wire mesh (WWM), laid on a 6-mil fiberglass reinforced polyethylene vapor barrier and 4" of washed gravel. Interior SOG will have a compressive strength of 3000 psi, while exterior SOG will have a strength of 4500 psi.

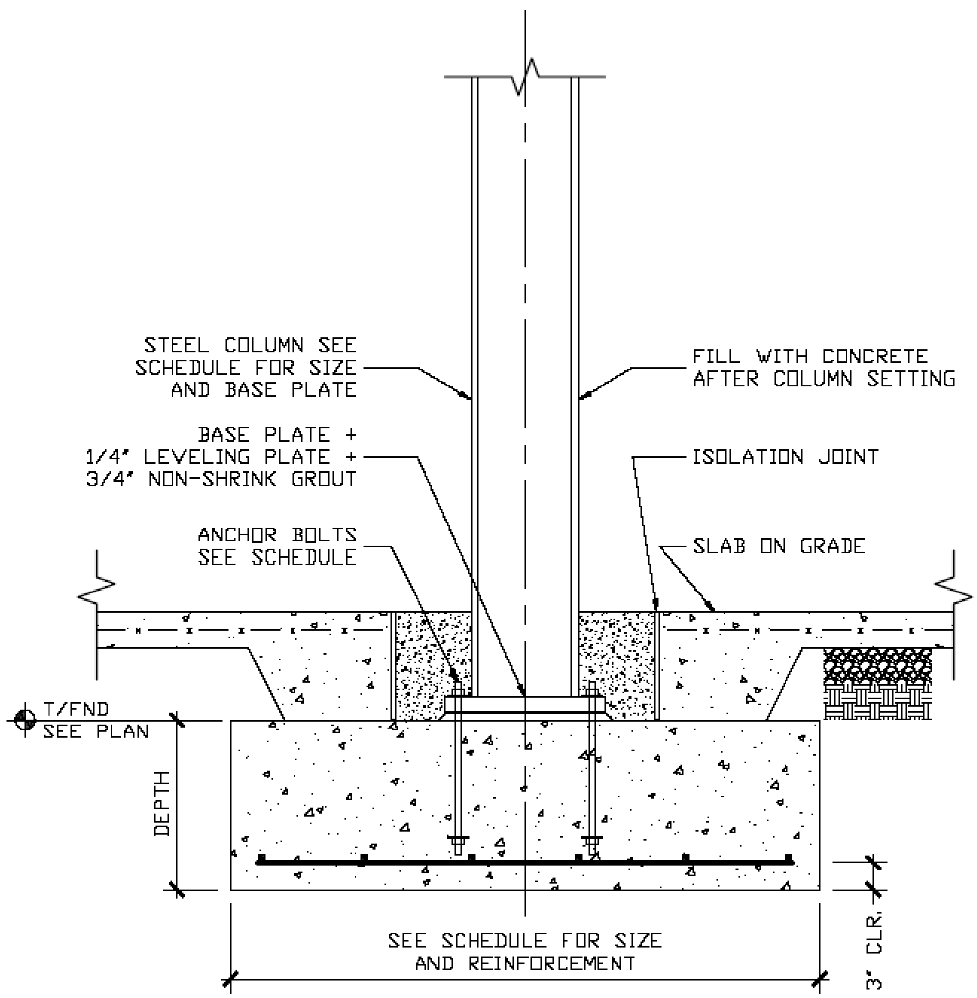



**Figure 1**- Typical Foundation section

## FLOOR SYSTEMS

A typical floor in the Building II consists of 3” 20 gauge composite steel deck with 3-1/4” lightweight concrete slab totaling a total slab thickness of 6-1/4”. The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi. The floor is supported by A992 wide flange beams with studs dimensioned at ¾” in diameter and 5 ¼” in length. The beams are spaced at 10’ o/c and span 41’-8” in a typical exterior bay and 30’-0” in a typical interior bay, as you can see in Figure 2 below. Depending on the floor, the beams will be cambered from an 1” to 1½” and will vary in size and weight. Typical interior girders are W24-62 spanning 30’-0”, while typical exterior girders vary in size and also span 30’-0”.

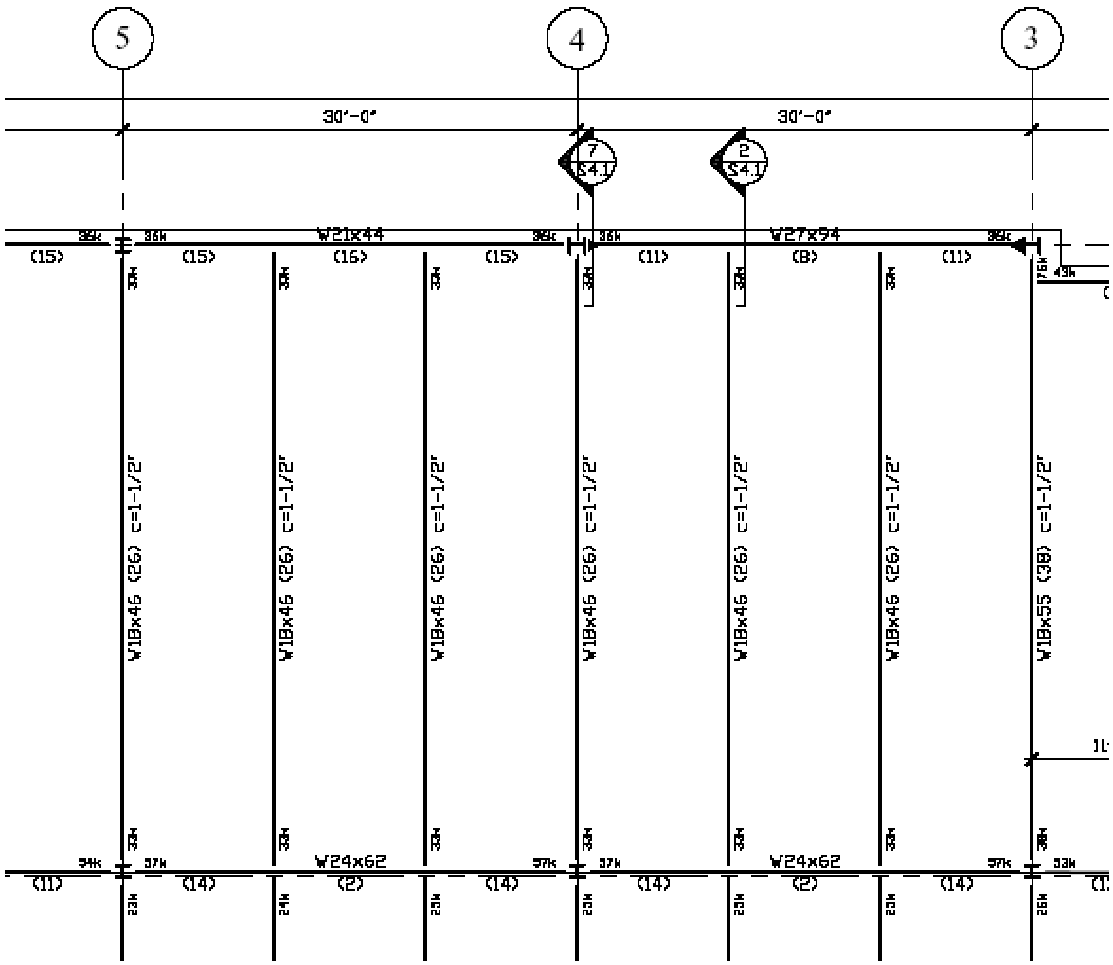



**FIGURE 2** – Typical exterior floor bay

## ROOF SYSTEM

As seen in Figure 3, the roof system is comprised of 1-1/2"  22 gauge Type B wide rib galvanized roof deck, on K series bar joists and steel girders.  Light-gage framing makes up the  4' parapet and the screen wall encompassing the roof. Precast panels frame into each floor including the roof.

Rooftop Mechanical pads for future tenant equipment shall be constructed similar to the typical floor system consisting of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4".  The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi.

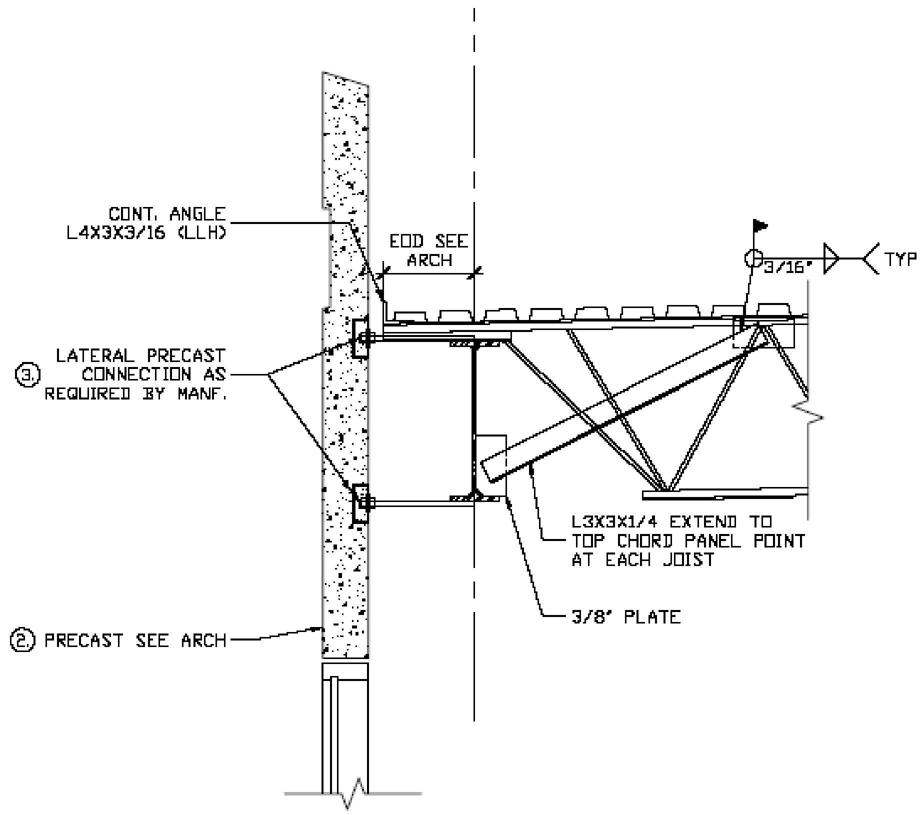



**FIGURE 3** – Typical exterior roof section

## LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consists of a number of structural steel moment frames running in both directions. Lateral loading is transferred from precast panels (connected at each floor) to each individual floor. Once transferred into the floor system, the load is transferred into composite beams which make up the framing and then into the columns. The columns and beams are connected by a moment connection seen in Figure 4. the columns transfer the rest of the load into the foundation.

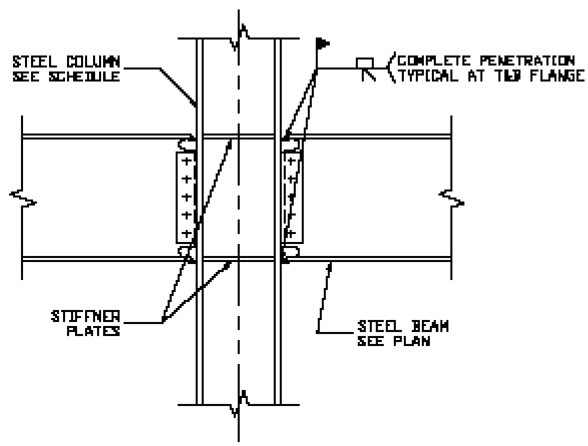



**FIGURE 4** – Typical Beam to Column Moment connection

Figure 5 clearly shows the four moment frames positioned in each direction, North-South and East-West, supporting the building laterally. In both directions the moment frames are positioned symmetrically about the center axis. The North-South lateral system is 2 sets of parallel moment frames anchoring each end bay. The East-West lateral system is a set of 2 moment frames on each exterior side of the building. The beam sizes vary.

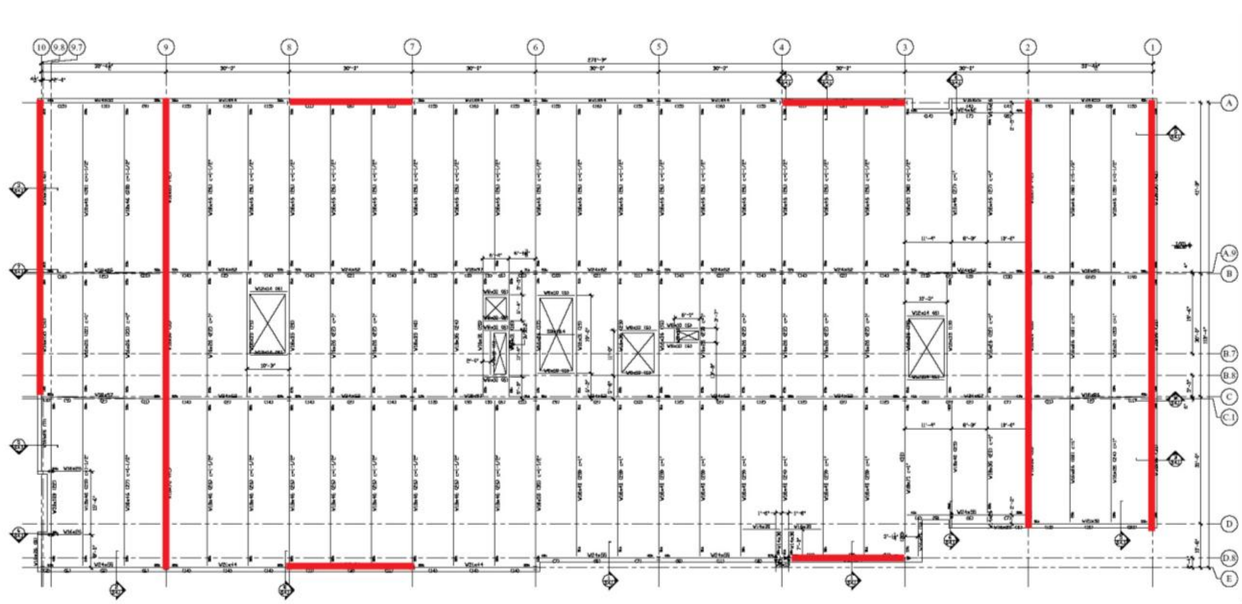


**FIGURE 5** – Typical Floor plan with moment frames

### COLUMN SYSTEM

Having a very uniform design layout the column system consists of typical exterior bays of 30’-0” x 41’-8” and interior bays of 30’-0” x 30’-0”. All of the columns consist of either a gravity resisting member or a combined lateral and gravity resisting member. Each columns is spliced at 4 feet past the third floor, regardless of its resisting system. All columns vary in size depending on location and load resistance capabilities.

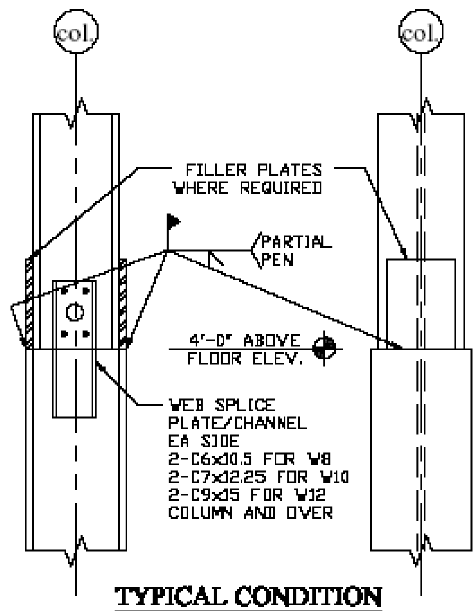



**FIGURE 6** – Typical splice connection

## APPLICABLE CODE

### Design Codes used for Original Design:

- International Building Code, 2003 Edition
- Virginia Uniform State Building Code, 2003
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 02, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Ninth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-02

### Code Substitutions/ Additional References used for Thesis Design:

- International Building Code, 2006 Edition
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-08

## MATERIALS AND PROPERTIES

| | |
|---|---|
| **Steel:** | |
| Wide flange shapes | 50 ksi (A992) |
| Square or Rectangular Tubes | 46 ksi (A500 Grade B) |
| Round Pipes | 42 ksi (A500 Grade B) |
| Miscellaneous Steel | 36 ksi (A36) |
| Bolts | 36/45 ksi (A325N/A490N) |
| Steel Studs | 60 ksi (A108) |
| Weld Strength | 70 ksi (E70XX) |
| **Concrete:** | |
| Foundations, Int. Wall & Int. SOG | f'c = 3000 psi |
| Ext. SOG and Pads | f'c = 4000 psi |
| Deck supported slabs (lightweight) | f'c = 3000 psi |
| **Reinforcement:** | |
| Stirrups and Ties | 40 ksi (A615) |
| All other | 60 ksi (A615) |
| Welded Wire Fabric: | (A185) |
| **Cold-Formed Steel Framing:** | |
| 20 Gage | 33 ksi (A653) |
| 18 Gage | 33 ksi (A653) |
| 16 Gage | 50 ksi (A653) |

Note: Material strengths are based on American Society for Testing and Materials (ASTM) Standard ratings.

## DESIGN LOADS

All of the Design loads for this technical report were all calculated referencing ASCE 7-05: *Minimum Design Loads for Buildings and other structures.* The actual design loads referenced IBC 2003 and but there wasn't much discrepancy other than calculating the dead load per floor, as seen in Table 1 below. All of the same Live loads and Snow loads were calculated the same resulting in very similar results in design. The dead load calculations can be seen in the Appendix under Table A-1a and Table A-1b.

| Design Loads | | | |
|---|---|---|---|
| **Live Loads** | | | |
| Area | Actual Design | Thesis Design | Code/Table |
| Lobby | 100 psf | 100 psf | 100 (ACSE Min.) |
| Office | 100 psf | 100 psf | 50 (ASCE Min.) |
| Corridors | 100 psf | 100 psf | 80 (ASCE Min.) |
| Roof | 20 psf | 20 psf | 20 (ASCE Min.) |

| Dead Loads | | | |
|---|---|---|---|
| Area | Actual Design | Thesis Design | Code/Table |
| Floor | 79.3 psf | 90.0 psf | Table A-1a |
| Roof | 28.5 psf | 30.0 psf | Table A-1b |

| Snow Loads | | | |
|---|---|---|---|
| Value | Actual Design | Thesis Design | Code/Table |
| Pg | 25.0 psf | 25.0 psf | ASCE 7-05 Chapter 7 |
| Ce | 1.0 | 1.0 | |
| Ct | 1.0 | 1.0 | |
| Cs | 1.0 | 1.0 | |
| I | 1.0 | 1.0 | |
| Pf calculated | 17.5 psf | 17.5 psf | |
| Pf | 20.0 psf | 20.0 psf | |

**TABLE 1 – Design Loads**

Lateral loads were calculated almost all by hand and inserted into the tables on the following pages. The hand calculations can be found on pages 20-27 in the Appendix. After calculating the lateral loads it was concluded that Seismic actually controlled over Wind, even in this non-seismic region as Table 6. This is partially due to the wind Exposure Factor and the lighter weight with respect to its height.

## WIND LOADS

| Design Wind Pressures, p in the E-W Direction | | | | | | |
|---|---|---|---|---|---|---|
| Location | Height above ground z(ft) | q (psf) | External pressure qGCp (psf) | Internal pressure qGCp (psf) | Net Pressure p (psf) | |
| | | | | | (+GCpi) | (-GCpi) |
| Windward | 0-15 | 10.05 | 6.68 | ± 2.80 | 3.88 | 9.48 |
| | 20 | 10.93 | 7.27 | ± 2.80 | 4.47 | 10.07 |
| | 25 | 11.99 | 7.97 | ± 2.80 | 5.17 | 10.77 |
| | 30 | 12.34 | 8.20 | ± 2.80 | 5.40 | 11.00 |
| | 40 | 13.40 | 8.91 | ± 2.80 | 6.11 | 11.71 |
| | 50 | 14.28 | 9.49 | ± 2.80 | 6.69 | 12.29 |
| | 60 | 14.98 | 9.96 | ± 2.80 | 7.16 | 12.76 |
| | 68 | 15.55 | 10.34 | ± 2.80 | 7.54 | 13.14 |
| Leeward | ALL | 15.55 | -3.62 | ± 2.80 | -6.42 | -0.82 |
| Side | ALL | 15.55 | -9.05 | ± 2.80 | -11.85 | -6.25 |
| Roof | 68 | 15.55 | -11.63 ° | ± 2.80 | -14.43 | -8.83 |
| | 68 | 15.55 | -6.46 † | ± 2.80 | -9.26 | -3.66 |
| | 68 | 15.55 | -3.88 ‡ | ± 2.80 | -6.68 | -1.08 |

° from windward edge to 68 ft
† from 68 to 136 ft
‡ from 136 to 275 ft

**TABLE 2 – Design Wind Pressures**

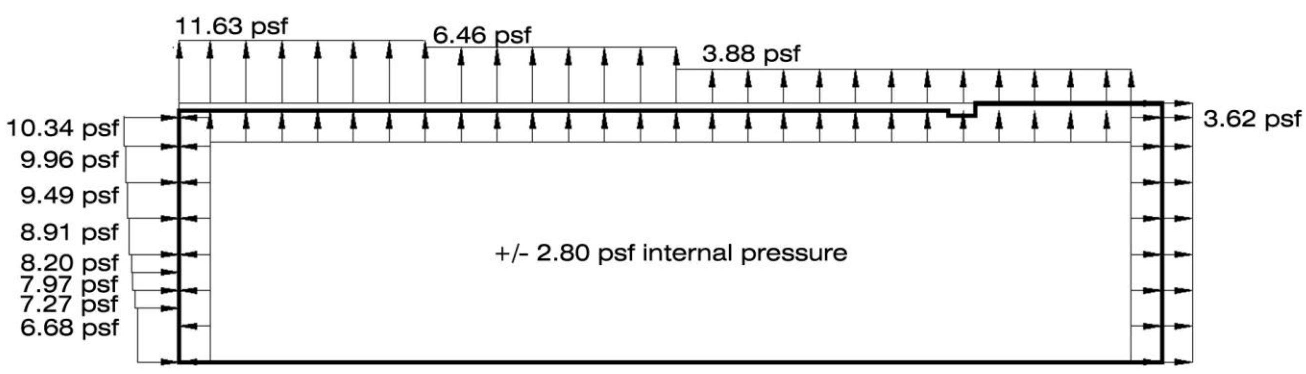



**FIGURE 7 - Design Wind Pressures in E-W Direction**

| Design Wind Pressures, p in the N-S Direction | | | | | | |
|---|---|---|---|---|---|---|
| Location | Height above ground z(ft) | q (psf) | External pressure qGCp (psf) | Internal pressure qGCp (psf) | Net Pressure p (psf) | |
| | | | | | (+GCpi) | (-GCpi) |
| Windward | 0-15 | 10.05 | 6.43 | ± 2.80 | 3.63 | 9.23 |
| | 20 | 10.93 | 6.99 | ± 2.80 | 4.19 | 9.79 |
| | 25 | 11.99 | 7.67 | ± 2.80 | 4.87 | 10.47 |
| | 30 | 12.34 | 7.89 | ± 2.80 | 5.09 | 10.69 |
| | 40 | 13.40 | 8.57 | ± 2.80 | 5.77 | 11.37 |
| | 50 | 14.28 | 9.13 | ± 2.80 | 6.33 | 11.93 |
| | 60 | 14.98 | 9.58 | ± 2.80 | 6.78 | 12.38 |
| | 68 | 15.55 | 9.94 | ± 2.80 | 7.14 | 12.74 |
| Leeward | ALL | 15.55 | -6.21 | ± 2.80 | -9.01 | -3.41 |
| Side | ALL | 15.55 | -8.70 | ± 2.80 | -11.50 | -5.90 |
| Roof | 68 | 15.55 | -11.56 ° | ± 2.80 | -14.36 | -8.76 |
| | 68 | 15.55 | -10.69 † | ± 2.80 | -13.49 | -7.89 |
| | 68 | 15.55 | -6.71 ‡ | ± 2.80 | -9.51 | -3.91 |

° from windward edge to 34 ft
† from 34 to 68 ft
‡ from 68 to 115 ft

**TABLE 3 – Design Wind Pressures**

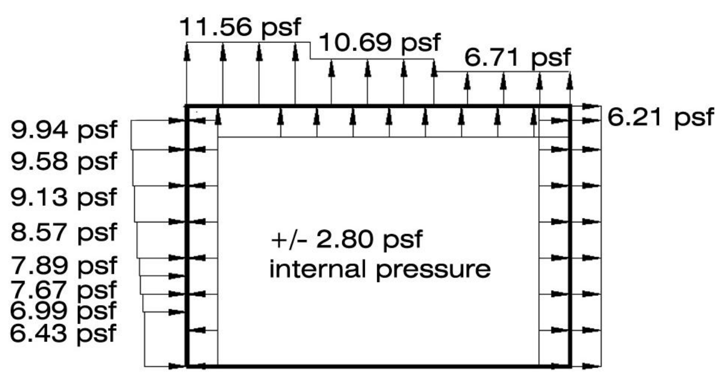



**FIGURE 8 – Design Wind Pressures in the N-S Direction**

## SEISMIC LOADS

| Seismic Force Story Distribution | | | | | | |
|---|---|---|---|---|---|---|
| Floor | $w_x$ | $h_x$ | k | $w_x h_x^k$ | $\Sigma\ w_i h_i^k$ | $C_{vx}$ |
| Base | -- | -- | -- | -- | -- | -- |
| 2 | 2760.10 | 14.25 | 2.00 | 560472.81 | 19622948.41 | 0.029 |
| 3 | 2771.10 | 27.50 | 2.00 | 2095644.38 | 19622948.41 | 0.107 |
| 4 | 2739.70 | 40.25 | 2.00 | 4438485.23 | 19622948.41 | 0.226 |
| 5 | 2739.70 | 54.00 | 2.00 | 7988965.20 | 19622948.41 | 0.407 |
| Roof | 981.70 | 68.00 | 2.00 | 4539380.80 | 19622948.41 | 0.231 |

**TABLE 4 – Seismic Force Distribution**

| Floor | $F_x$ (kips) | Story Shear Vx (kips) | Moment (k-ft) |
|---|---|---|---|
| Roof | 81.92 | - | - |
| 5 | 144.17 | 81.92 | 5570.51 |
| 4 | 80.10 | 226.09 | 7785.27 |
| 3 | 37.82 | 306.19 | 3223.96 |
| 2 | 10.11 | 344.01 | 1040.02 |
| Base | 354.12 | 354.12 | 144.13 |
| | | Overturning Moment (k-ft) | 17763.89 |

**TABLE 5 – Seismic Story Shear**

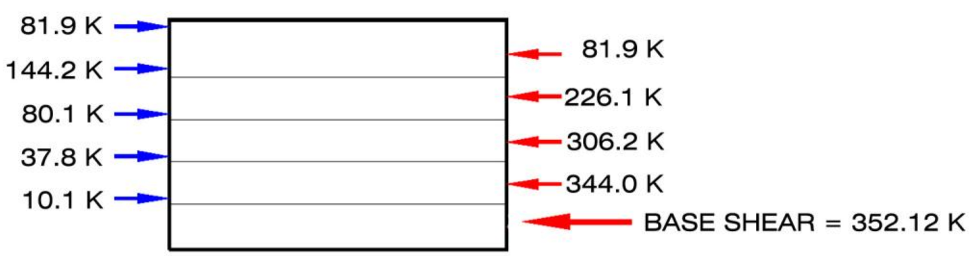



**FIGURE 8 – Story Shear Diagram**

| LATERAL LOADS – Worst Case (Base Shear) | |
|---|---|
| | |
| **Seismic** | **352.12 K (Controls)** |
| **Wind N-S** | **252.0 K** |
| **Wind E-W** | **89.3 K** |

**TABLE 6 – Worst Case Base Shear**

### Spot Check Summary

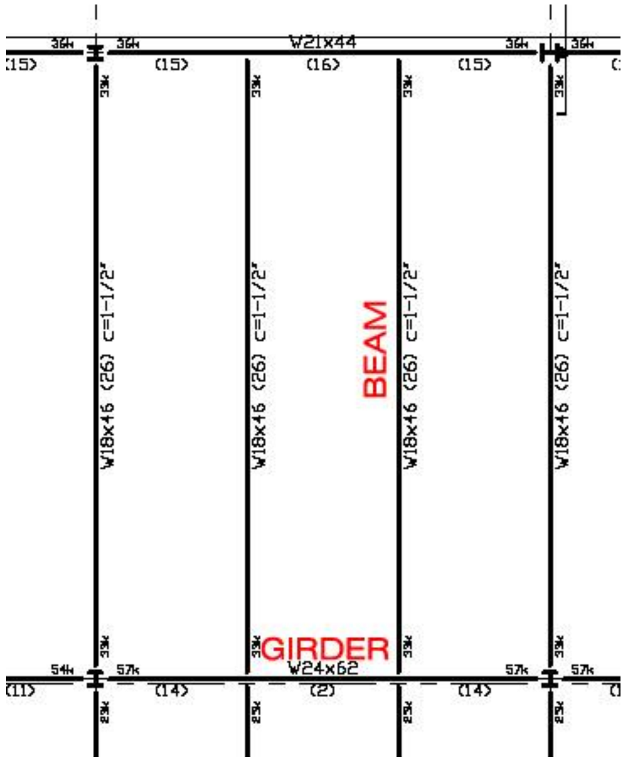



**FIGURE 9 – Typical exterior bay**

### COMPOSITE BEAM

Actual Design: W18x46 (26 studs)
Thesis Design: W18x40 (50 studs)

Conclusion: The W18x40 (50) was slightly more economical but did fail in deflection without the camber. If the W18X46 would have been used, 28 studs would have been required which is very close to the actual design. One obvious difference is the *actual design* dead load is about 11 psf lower than the *thesis design* dead load.

*See hand calculations pg 30 and 31 in Appendix*

### COMPOSITE GIRDER

Actual Design: W24X62 (30 studs)
Thesis Design: W24x62 (28 studs)

Conclusion: With almost identical designs, the thesis design has actually less strength than the actual design. Placement of the studs is probably the reason behind this, as you can see from Figure 9 that the studs are concentrated toward the both reactions.

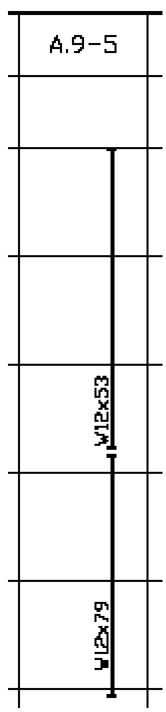
*See hand calculations pg 32 and 33 in Appendix*

**COLUMN (A.9-5)**

| | Actual Design: | Thesis Design: |
|---|---|---|
| Roof: | W12X53 | W12X40 |
| 5th Floor: | W12X53 | W12X40 |
| 4th Floor: | W12X53 | W12X53 |
| 3rd Floor: | W12X79 | W12X65 |
| 2nd Floor: | W12X79 | W12X96 |

Conclusion: There are many differences in the design of the columns. First, the dead loads used in the thesis design were slightly higher resulting in a greater design at the bottom floor. Secondly, the column is spliced once between the third and fourth floors, therefore the design of the 4th floor column requires it to be carried all the way to the roof. Same goes for the 3rd floor, it is governed by the design of the 2nd floor. In the thesis design, the columns were designed separately per their respective floor, hence the less conservative design at the 3rd floor, 5th floor and roof. When comparing the "governing" floors (2nd and 4th) the thesis design is the same if not conservative.

*See hand calculations pg 34and 35 in Appendix*



# APPENDIX

## Dead Load Calculations:

| TABLE A-1a - Building Weight<br>Dead Load Calculations | | | | | |
|---|---|---|---|---|---|
| **Roof** | **Load** | | **Floor** | **Load** | |
| Roofing | 12.0 | psf | Flooring | 1 | psf |
| Deck | 1.7 | psf | Topping | 53.1 | psf |
| Framing | 3.0 | psf | Deck/Sub-floor | 2 | psf |
| Insulation | 6.0 | psf | Framing | 8 | psf |
| Ceiling | 1.8 | psf | Other | 9.4 | psf |
| Sprinklers | 2.0 | psf | Ceiling | 1.8 | psf |
| Mech & Elec | 2.0 | psf | Sprinklers | 2 | psf |
| Misc. | 0.0 | psf | Mech & Elec | 2 | psf |
| | | | Misc. | 0 | psf |
| Total Dead | 28.5 | psf | Total Dead | 79.3 | psf |

| TABLE A- 1b - Building Weight<br>Dead Load Calculations | | | | | |
|---|---|---|---|---|---|
| **Roof** | **Load** | | **Floor** | **Load** | |
| Roofing | 15.0 | psf | Flooring | 2 | psf |
| Deck | 2.0 | psf | Topping | 60 | psf |
| Framing | 3.0 | psf | Deck/Sub-floor | 2 | psf |
| Insulation | 4.0 | psf | Framing | 10 | psf |
| Ceiling | 2.0 | psf | Other | 10 | psf |
| Sprinklers | 2.0 | psf | Ceiling | 2 | psf |
| Mech & Elec | 2.0 | psf | Sprinklers | 2 | psf |
| Misc. | 0.0 | psf | Mech & Elec | 2 | psf |
| | | | Misc. | 0 | psf |
| Total Dead | 30.0 | psf | Total Dead | 90.0 | psf |

**Wind Calculations:**

WIND CALCULATIONS – ASCE 7-05, METHOD 2

- BASIC WIND, $V = 90$ MPH (FIGURE 1) (6.5.4)
- EXPOSURE B (6.5.6.3)
- OCCUPANCY CATEGORY II – NON-HURRICANE
- IMPORTANCE FACTOR, $I = 1.0$ (6.5.5)
- DIRECTIONALITY FACTOR, $K_D = .85$ (6.5.4)
- TOPOGRAPHIC FACTOR, $K_{ZT} = 1.0$ (6.5.7)
  → NO HILLS, RIDGES, ESCARPMENTS

→ DESIGN WIND PRESSURE ON PARAPET

PARAPET $h = 73'$-$0''$

↳ BY LINEAR INTERPOLATION $K_H = .90$

- VELOCITY PRESSURE $q_P$ ON PARAPET

$$q_P = .00256(K_h)(K_{zt})(K_d)(V^2)I$$
$$= .00256(.9)(1.0)(.85)(90^2)(1.0)$$
$$\boxed{q_P = 15.90 \text{ PSF}}$$

- COMBINED NET PRESSURE COEFFICIENT, $GC_{pn}$ (6.5.12.2.4)

$\boxed{GC_{pn} = 1.5}$ WINDWARD PARAPET

$\boxed{GC_{pn} = -1.0}$ LEEWARD PARAPET

- COMBINED NET DESIGN PRESSURE $P_P$

$$P_P = q_P\, GC_{pn}$$
$= 15.90(1.5) = +23.85$ PSF ON WINDWARD PARAPET

$= 15.90(-1.0) = -15.90$ PSF ON LEEWARD PARAPET

FORCES: $23.85(5') = 119.25$ PLF (WW)

$-15.90(5') = -79.5$ PLF (LW)

## GUST EFFECT FACTOR

- ENCLOSED
- RIGID

$$G = .925\left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_v I_z}\right)$$

$I_z = c\left(\frac{33}{z}\right)^{1/6}$

$z = .6h$

$g_v, g_Q = 3.4$

$\epsilon = 1/3$
$l = 320$
$z_{min} = 30$ ft
$c = .3$
(TABLE C-2)

$L_z = l\left(\frac{\bar{z}}{33}\right)^{\epsilon}$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_z}\right)^{.63}}}$$

$z = .6h = .6(68') = \underline{\underline{40.8}}$

$40.8 > 30 = z_{min} \therefore$ OK

$I_z = .3\left(\frac{33}{40.8}\right)^{1/6} = \underline{\underline{.29}}$

$L_z = 320\left(\frac{40.8}{33}\right)^{1/3} = \underline{\underline{343}}$

$$Q_{E\text{-}W} = \sqrt{\frac{1}{1 + .63\left(\frac{115+68}{343}\right)^{.63}}}$$

$Q_{E\text{-}W} = \underline{\underline{.837}}$

$$Q_{N\text{-}S} = \sqrt{\frac{1}{1 + .63\left(\frac{275+68}{343}\right)^{.63}}}$$

$Q_{N\text{-}S} = \underline{\underline{.783}}$

$$G_{E\text{-}W} = .925\left(\frac{1 + 1.7(3.4)(.29)(.837)}{1 + 1.7(3.4)(.29)}\right) = \underline{\underline{.83}}$$

$$G_{N\text{-}S} = .925\left(\frac{1 + 1.7(3.4)(.29)(.783)}{1 + 1.7(3.4)(.29)}\right) = \underline{\underline{.80}}$$

VELOCITY PRESSURES $q_z$ & $q_h$

$q_z = .00256 K_z K_{zt} K_d V^2 I$

$= .00256 K_z (1.0)(.85) 90^2 (1.0)$

$q_z = 12.63 K_z$

VARIES SEE TABLE

PRESSURE COEFFIENTS, $C_p$ (CASE 2 APPLIES EXPOSURE B)

• WALL PRESSURE COEFFIENTS, $C_p$

EAST-WEST WIND

• $L/B = \frac{275}{115} = 2.39$

| | |
|---|---|
| WINDWALL WALL: | $C_p = .8$ |
| LEEWARD WALL: | $C_p = -.28$ |
| SIDE WALL: | $C_p = -.7$ |

NORTH-SOUTH WIND

• $L/B = 115/275 = .42$

$C_p = .8$
$C_p = -.5$
$C_p = -.7$

• ROOF PRESSURE COEFFIENTS, $C_p$

EAST-WEST

$h/L = 68/275 = .25 \leq .5$

$0 \rightarrow h/2$ : $C_p = -.9$,
$h/2 \rightarrow h$ : $C_p = -.9$,
$h \rightarrow 2h$ : $C_p = -.5$,
$> 2h$ : $C_p = -.3$

NORTH-SOUTH

$h/L = 68/115 = .591$

$C_p = -.93$
$C_p = -.86$
$C_p = -.54$
INTERPOLATED

$q_i = q_h = 15.55$ PSF (6.5.12.2.1)

• INTERNAL PRESSURE COEFFICIENTS ($GC_{pi}$)

FIGURE 6.5 → ENCLOSED BLDG'S → $\boxed{GC_{pi} = \pm .18}$

DESIGN WIND PRESSURES, $p_z$ & $p_h$

EAST-WEST

WINDWARD WALL: $p_z = q_z GC_p - q_h(GC_{pi})$

$= q_z(.83)(.8) - 15.55(\pm.18)$

$p_z = .66 q_z \pm 2.80$ (EXT ± INT. PRES)

LEEWARD WALL, SIDE WALLS, ROOF: $p_h = q_h GC_p - q_h GC_{pi}$

$= 15.55(.83)C_p - 15.55(\pm.18)$

$p_h = 12.90 C_p \pm 2.80$ (EXT ± INT. PRES)

SEE TABLE

NORTH-SOUTH

WINDWARD WALL: $p_z = q_z G C_p - q_h(GC_{pi})$

$= q_z(.80)(.8) - 15.55(\pm.18)$

$p_z = .64 q_z \pm 2.80$

LEEWARD WALL, SIDE WALLS, ROOF: $p_h = q_h GC_p - q_h GC_{pi}$

$= 15.55(.80)C_p - 15.55(\pm.18)$

$p_h = 12.44 C_p \pm 2.80$

SEE TABLE

## Wind Force Calculations

| Lateral Forces E-W Direction – Table A-2a | | | | |
|---|---|---|---|---|
| | Force | Story Shear | Moment | |
| Floor | Fx, (kips) | V, (kips) | M (ft-k) | |
| Base/1 | - | 89.3 | 0.0 | |
| 2.0 | 17.3 | 72.0 | 246.5 | |
| 3.0 | 19.0 | 53.0 | 251.8 | |
| 4.0 | 20.3 | 32.7 | 268.8 | |
| 5.0 | 21.5 | 11.2 | 284.9 | |
| Roof | 11.2 | - | 156.8 | |
| Width | 115.0 | | 1208.8 | Overturning Moment |

| Lateral Forces N-S Direction – Table A-2b | | | | |
|---|---|---|---|---|
| | Force | Story Shear | Moment | |
| Floor | Fx, (kips) | V, (kips) | M (ft-k) | |
| Base/1 | - | 252.0 | 0.0 | |
| 2.0 | 50.5 | 201.5 | 719.6 | |
| 3.0 | 53.6 | 147.9 | 710.2 | |
| 4.0 | 56.7 | 91.2 | 751.3 | |
| 5.0 | 59.9 | 31.3 | 793.7 | |
| Roof | 31.3 | - | 438.2 | |
| Width | 275 | | 3413.0 | Overturning Moment |

## Seismic Calculations:

SEISMIC CALCULATIONS (ASCE 7-05)

- SEISMIC GROUP I
- SITE CLASS C
- IMPORTANCE FACTOR = 1.0

* MAPPED ACCELERATIONS, $S_S$ & $S_1$

→ $S_S$ & $S_1$ FROM INPUTING LATITUDE & LONGITUDE INTO USGS GROUND MOTION PARAMETER CALCULATOR

$S_S$ (.2 sec) = .183

$S_1$ (1.0 sec) = .064

* SOIL MODIFIED ACCELERATIONS, $S_{MS}$ & $S_{M1}$

SITE COEFFICIENTS: (TABLE 11.4-1) $F_A = 1.2$, (TABLE 11.4-2) $F_V = 1.7$ } SITE CLASS C

$S_{MS} = F_A S_S = 1.2(.183) = .22$

$S_{M1} = F_V S_1 = 1.7(.064) = .109$

* DESIGN ACCELERATIONS, $S_{DS}$ & $S_{D1}$

$S_{DS} = 2/3\, S_{MS} = 2/3\,(.22) = \underline{\underline{.147}}$ (EQN. 11.4-3)

$S_{D1} = 2/3\, S_{M1} = 2/3\,(.109) = \underline{\underline{.073}}$ (EQN. 11.4-4

DETERMINE SEISMIC DESIGN CATEGORY (SDC)

1. $S_S = .183 > .15$, $S_1 = .064 > .04$ } ∴ NOT AUTOMATICALLY ASSIGNED TO SDC A
2. OCCUPANCY CATEGORY II
3. $S_1 < .75$ ∴ NOT SDC E OR SDC F
4. CHECK APPROX PERIOD, $T_A < .8\,T_S$

$T_a = C_t h_n^x$  $C_t = .028$, $x = .8$ } TABLE 12.8-2 (STEEL MOMENT RESISTING FRAMES)

$T_a = C_t h_n^x = .028 (68')^{.8}$

$T_a = .819$ SEC

$T_S = S_{D1}/S_{DS} = \frac{.073}{.147} = .497 \Rightarrow .8T_S = .8(.497) = \underline{.398}$

$\Rightarrow$ CHECK $T_a < .8T_S$

$.819 > .398$ ∴ USE TABLES 11.6-1, 11.6-2

$\rightarrow \quad .067 < S_{D1} < .133 \rightarrow$ SDC B

$S_{DS} \longrightarrow$ SDC A

∴ USE SEISMIC DESIGN CATEGORY B

* NO PLAN OR VERTICAL IRREGULARITIES (12.3.2)

* DETERMINE THE RESPONSE MODIFICATION COEF., R

$\rightarrow$ TABLE 12.2-1

↳ STEEL SYSTEM NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE

- RESPONSE MOD. COEFF., $R = 3.0$
- SYSTEM OVERSTRENGH FACTOR, $\Omega_0 = 3.0$
- DEFLECTION AMPLIFICATION, $C_d = 3.0$

DETERMINE SEISMIC RESPONSE COEFFICIENTS, $C_s$

$\rightarrow$ BOTH DIRECTIONS (SAME R)

$C_s = \frac{S_{D1}}{T(R/I)} = \frac{.073}{.819(3/1)} = .0297 \approx .030$

$C_s = \frac{S_{DS}}{R/I} = \frac{.147}{3/1} = .049 > .0297$

CONTROLS ✓

$.0297 > .01$

∴ $\boxed{C_s = .030}$

## Seismic Load Calculations:

| TABLE A-3 - Building Weight | | | | | | |
|---|---|---|---|---|---|---|
| Floor self-weight | | | | | | |
| **Floor** | **Area** | | **Dead Load** | | **Weight** | |
| Base | - | - | - | - | - | |
| 2 | 31032 | SF | 79.3 | PSF | 2460.8 | K |
| 3 | 31170 | SF | 79.3 | PSF | 2471.8 | K |
| 4 | 31170 | SF | 79.3 | PSF | 2471.8 | K |
| 5 | 31170 | SF | 79.3 | PSF | 2471.8 | K |
| Roof | 31150 | SF | 28.5 | PSF | 887.8 | K |
| | | | | | 10764 | K |

| TABLE A-4 - Building Weight | | | | | | | |
|---|---|---|---|---|---|---|---|
| Column self-weight | | | | | | | |
| **Column** | **Quantity** | **Linear Weight** | | **Height** | | **Total Weight** | |
| W8x40 | 2 | 40 | PLF | 14.3 | ft | 1.1 | K |
| W12X50 | 2 | 50 | PLF | 36.5 | ft | 3.7 | K |
| W12X53 | 9 | 53 | PLF | 36.5 | ft | 17.4 | K |
| W12X58 | 2 | 58 | PLF | 36.5 | ft | 4.2 | K |
| W12X65 | 3 | 65 | PLF | 31.5 | ft | 6.1 | K |
| W12X65 | 4 | 65 | PLF | 36.5 | ft | 9.5 | K |
| W12X72 | 2 | 72 | PLF | 31.5 | ft | 4.5 | K |
| W12X79 | 4 | 79 | PLF | 31.5 | ft | 10.0 | K |
| W12X87 | 4 | 87 | PLF | 31.5 | ft | 11.0 | K |
| W12X96 | 2 | 96 | PLF | 31.5 | ft | 6.0 | K |
| W12X106 | 2 | 106 | PLF | 31.5 | ft | 6.7 | K |
| W14X53 | 9 | 53 | PLF | 36.5 | ft | 17.4 | K |
| W14X61 | 2 | 61 | PLF | 36.5 | ft | 4.5 | K |
| W14X120 | 4 | 120 | PLF | 36.5 | ft | 17.5 | K |
| W14X132 | 8 | 132 | PLF | 31.5 | ft | 33.3 | K |
| W18X86 | 2 | 86 | PLF | 36.5 | ft | 6.3 | K |
| W18X106 | 4 | 106 | PLF | 36.5 | ft | 15.5 | K |
| W18X119 | 5 | 119 | PLF | 31.5 | ft | 18.7 | K |
| W18X119 | 8 | 119 | PLF | 36.5 | ft | 34.7 | K |
| W18X143 | 8 | 143 | PLF | 31.5 | ft | 36.0 | K |
| W18X175 | 3 | 175 | PLF | 31.5 | ft | 16.5 | K |
| | | | | TOTAL WEIGHT= | | 281 | K |

| TABLE A-5 - Building Weight | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Precast panels | | | | | | | | |
| **Floor** | **Perimeter** | | **Height** | | **Self-weight** | | | |
| Base | - | - | - | - | - | - | - | - |
| 2 | 780 | LF | 5 | ft | 57.5 | PSF | 224.3 | K |
| 3 | 780 | LF | 5 | ft | 57.5 | PSF | 224.3 | K |
| 4 | 780 | LF | 5 | ft | 57.5 | PSF | 224.3 | K |
| 5 | 780 | LF | 5 | ft | 57.5 | PSF | 224.3 | K |
| Roof | 175 | LF | 5 | ft | 57.5 | PSF | 50.3 | K |
| | | | | | | | 947 | K |

**Total weight of Building = 11,992 K**

**Snow Calculations:**

<u>SNOW LOADS</u> (ASCE 7-05)

- DETERMINE GROUND SNOW LOAD, $p_g$
  - → FIGURE 7-1 CHANTILLY, VA
  - ⇒ $\boxed{P_g = 25 \text{ PSF}}$
- DETERMINE SNOW DENSITY, $\gamma$

$$\gamma = .13 p_g + 14 \le 30 \text{ PCF}$$

$$\gamma = .13(25) + 14 = 17.3 \le 30 \checkmark$$

$$\boxed{\gamma = 17.3 \text{ PCF}}$$

- IMPORTANCE CATEGORY = II
- IMPORTANCE FACTOR = 1.0
- THERMAL FACTOR $(C_t)$ = 1.0, EXPOSURE FACTOR $(C_e)$ = 1.0
- SLOPED ROOF FACTOR $(C_s)$ = 1.0
- FLAT ROOF SNOW LOAD

$$P_f = .7 C_e \cdot C_t \cdot I \, P_g = .7(1.0)(1.0)(1.0)(25) = 17.5 \text{ PSF}$$

$$P_{f_{min}} = 20 \text{ PSF} > 17.5 \quad \therefore \boxed{P_f = 20 \text{ PSF}}$$

- DESIGN ROOF SNOW LOAD

$$P_s = P_f \cdot C_s = 20(1.0) = \boxed{20 \text{ PSF} = P_s}$$

- BALANCED SNOW HEIGHT, $h_b$

$$h_b = \frac{P_s}{\gamma} = \frac{20}{17.3} = 1.16 \text{ ft}$$

- DETERMINE $h_c$ (CLEAR HEIGHTS)

$h = 9.5'$ SCREENWALL $\qquad 9.5 - 1.16 = 8.34 = h_c$

$h = 4'$ PARAPET

$$\frac{h_c}{h_b} = \frac{8.34}{1.16} = 7.2 > .2$$

∴ DESIGN FOR DRIFT

## SCREEN WALL:

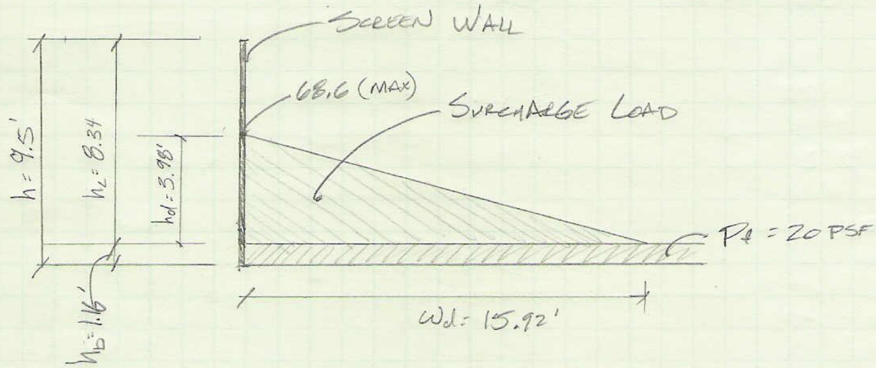
→ WINDWARD DRIFT → FIGURE 7.7 → $h_d = 3/4(5)$

$\boxed{h_d = 3.98}$

$W_d = 4h_d = 4(3.98') = \boxed{15.92\text{ FT}}$

SURCHARGE LOAD

$P_d = \gamma \cdot h_d = (17.3\text{ PCF})(3.98') = \underline{68.6\text{ PSF}}$




## PARAPET

$h = 4'$
$h_b = 1.16'$
$h_c = 2.84$

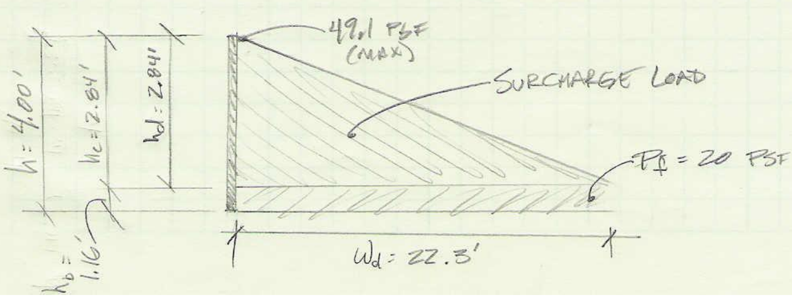
$\frac{h_c}{h_b} = \frac{2.84}{1.16} = 2.45 > 2$ ∴ DESIGN FOR DRIFT

$h_d = 3.98 > 2.84'$ ∴ USE $\underline{2.84'}$ FOR $h_d$

$W_d = \frac{4h_d^2}{h_c} = \frac{4(3.98)^2}{2.84} = \underline{22.3'}$

SURCHARGE LOAD

$P_d = (17.3)(2.84') = \underline{49.1\text{ PSF}}$

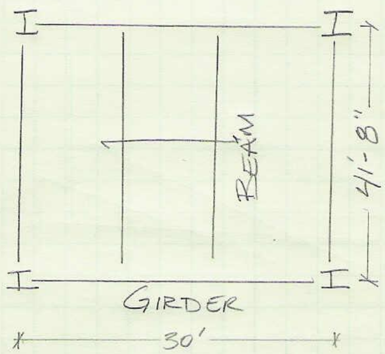


## Spot Checks: (Composite Beam)

SPOT CHECK

TYPICAL BAY 41'-8" x 30'-0"

- COMPOSITE BEAM
  - DESIGNED AS W18x46 (26)
- GIRDER
  - DESIGNED AS W24x62 (30)




- SLAB : 3¼" LIGHTWEIGHT
  $f'_c = 3000$ psi
- DECK : 3" - 20 GA
- STUDS : ¾"$\phi$, 5¼"

LOADS:

DEAD = 90.0 PSF
LIVE = 100 PSF

COMPOSITE BEAM

TRIB AREA = 10'(30') = 300 SF

LIVE LOAD REDUCTION $K_L = 2$ (INTERIOR BEAM)

$$L = 100\left(.25 + \frac{15}{\sqrt{2(300)}}\right) = 86.2 \text{ PSF}$$

$$w_D = 90.0\,(10') = .900 \text{ KLF}$$
$$w_L = 86.2\,(10') = .862 \text{ KLF}$$

$$w_u = 1.2D + 1.6L = 1.2(.90) + 1.6(.862) = 2.46 \text{ KLF}$$

$$M_u = \frac{w\ell^2}{8} = \frac{2.46\,(41.67')^2}{8} = 533 \text{ K}\cdot\text{FT}$$

- ASSUME $a = 2"$, $Y_2 = 6\tfrac{1}{4}" - \frac{2"}{2} = 5.25"$ (USE $Y_2 = 5$ TO BE CONSERVATIVE)

- $b_{eff}$ $\begin{cases} 10' = 120" \checkmark \\ \tfrac{1}{4}(500") = 125" \end{cases}$

TABLE 3-21 $\Longrightarrow$ LIGHTWEIGHT, ¾"$\phi$, $\perp$ DECK, WEAK, $f'_c = 3$ ksi

$Q_N = 17.2^K$

TABLE 3-19

| MEMBER | PNA | $\phi M_u \geq 533'^k$ | $\Sigma Q_n$ | # STUDS | EQ. WT STUDS | EQ. WT BEAM | TOTAL WT |
|---|---|---|---|---|---|---|---|
| W16x45 | 4 | 533 | 365 | 44 | 440 | 1875 | 2315 |
| W18x40 | 3 | 555 | 430 | 50 | 500 | 1667 | 2167 |
| W18x46 | 6 | 533 | 240 | 28 | 280 | 1917 | 2197 |

CHECK W18x40

$$a = \frac{\Sigma Q_n}{.85 f'_c B_{eff}} = \frac{430}{.85(3)(120)} = 1.41 < 2 \quad \therefore \text{CONSERVATIVE}$$

CONSTRUCTION LOAD:

DEAD: CONC. WT = $\frac{12(3.25 + 3/2)}{144}$ (115 PSF) = 45.5 PSF

LIVE: CONSTR. LL = 20 PSF

TOTAL $w_{u\,CONST}$ = 1.2(45.5) + 1.6(20) = 86.6 PSF (10') = .866 KLF

$$M_u = \frac{.866(41.67)^2}{8} = 188'^k < \phi M_p \; W18\times40 = 294'^k$$

CONSTRUCTION DL DEFLECTION:

$$l/360 = \frac{500}{360} = 1.39''$$

$$\Delta = \frac{5}{384}\,\frac{.455(41.67)^4\,1728}{29000(612)} = 1.74'' - 1\tfrac{1}{2}''^{*} = .24'' < 1.39'' \quad \therefore \underline{OK}$$

* NOTE CAMBER = $1\tfrac{1}{2}''$ ON PLANS

LIVE LOAD DEFLECTION:

$$\Delta = \frac{5}{384}\,\frac{.862(41.67)^4\,1728}{29000\,(1470)} = 1.37'' < 1.39'' \quad \therefore \underline{OK}$$

$I_{LB\text{-}W18x40}$

$\therefore$ USE COMPOSITE W18x40 w/50 STUDS

FOR TYPICAL BEAM

## Composite Girder:

COMPOSITE GIRDER (EXT. BAY + INT. BAY)

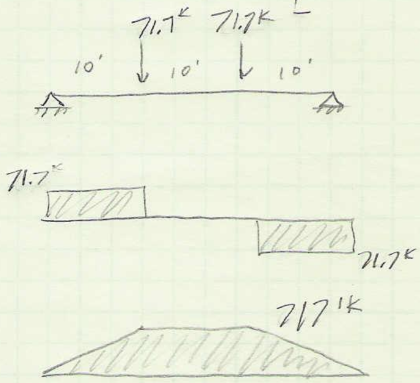
TRIB AREA $= 30' \times \left(\frac{41.67}{2} + \frac{30}{2}\right) = 1075$ SF

LL REDUCTION $K_{LL} = 2$ (INT. BEAM)

$$L = 100\left(.25 + \frac{15}{\sqrt{2(1075)}}\right) = 57.3 \text{ PSF}$$

$$\omega_u = 1.2(.900) + 1.6(.573) = 2.00 \text{ KLF}$$

$$\text{POINT LOADS} = \left[\frac{2.00\left(\frac{41.67}{2} + \frac{30}{2}\right)}{2}\right] \cdot 2 = 71.7^K \text{ @ } 10' \text{ \& } 20'$$




$$B_{EFF} \begin{cases} 41.67(12) = 500 \\ 41.67(12)/4 = 125 \checkmark \end{cases}$$

ASSUME $a = 1 \Rightarrow Y_2 = 5.75$

(USE 5.5 TO BE CONSERV.)

TABLE 3-21 ⇒ LWC, ‖ DECK, $f'_c = 3$ KSi

$Q_n = 17.1^K$

TABLE 3-19

| MEMBER | PNA | $\phi M_n > 717^K$ | $\Sigma Q_n$ | # STUDS | EQ. WT STUDS | EQ WT BEAM | TOTAL WT |
|---|---|---|---|---|---|---|---|
| W21×62 | 7 | 767 | 228 | 28 | 280 | 1860 | 2140 |
| W24×62 | 7 | 827 | 228 | 28 | 280 | 1860 | 2140 |
| W24×68 | 7 | 932 | 251 | 30 | 300 | 2040 | 2340 |

CHECK: W24×62

$$a = \frac{\Sigma Q_n}{.85 f'_c B_{eff}} = \frac{228}{.85(3)(125)} = .715 < 1 \therefore \text{OK}$$

## CONSTRUCTION DEAD LOAD DEFLECTION

$$P = .900\ \text{KLF}\left(\frac{41.67}{2} + \frac{30}{2}\right) = 32.2^K$$

$$\Delta = \frac{PL^3}{28EI} = \frac{32.2(30')^3(144)}{28(29000)(\underset{I_{W24\times62}}{1550})} = .100"$$

$$\Delta = L/360 = \frac{30(12)}{360} = 1" > .100" \therefore \underline{OK}$$

## LIVE LOAD DEFLECTION

$$P = .573\left(\frac{41.67}{2} + \frac{30}{2}\right) = 20.53^K$$

$$\Delta = \frac{PL^3}{28EI} = \frac{20.53(30)^3\ 144}{28(29000)(\underset{I_{CB\ W24\times62}}{2650})} = .037" < 1" \therefore \underline{OK}$$

$\therefore$ USE COMPOSITE <u>W24 X 62 W/ 28 STUDS</u> FOR

TYPICAL GIRDER

## Column (Gravity Only):

COLUMN SPOT CHECK: A.9-5

- INTERIOR GRAVITY COLUMN (NO LATERAL LOAD)
- COLUMN SPLICED 4'-0" ABOVE 3RD FLOOR

LOADS:

DEAD: FLOOR = 90.0 PSF
ROOF = 30.0 PSF

LIVE: FLOOR = 100 PSF
ROOF = 20 PSF
SNOW = 20 PSF

TRIB AREA: $\left(\frac{41.8}{2} + \frac{30}{2}\right) * 30 = 1077 \times 4 = 4308 = A_I$

ROOF COLUMN (KL = 14.0')

→ NO LIVE LOAD REDUCTION

LOAD: = 1.2 (30.0) + 1.6 (20) + .5(20)

$P_u$ = 78.0 PSF (1077 SF) = 84$^k$ ⇒ $\boxed{\begin{array}{l} W12 \times 40 \\ \phi P_n = 304^k \end{array}}$

5TH FLOOR (KL = 13.25')

LL REDUCTION

$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right) = L_o\left(.25 + \frac{15}{\sqrt{4308}}\right) = .478L_o > .4L_o$ ✓ OK

L = 100 (.478) = 47.8

LOAD = 1.2 (90.0) + 1.6(47.8) = 184.5 PSF (1077 SF)

$P_u = 198.7^k + 84^k = 282.7^k$ ⇒ $\boxed{\begin{array}{l} W12 \times 40 \\ \phi P_n = 322^k \end{array}}$

## 4TH FLOOR (KL = 13.25')

LL REDUCTION

$$L = 100\left(.25 + \frac{15}{\sqrt{4308}}\right) = 47.8 \text{ PSF}$$ (SAME AS 5)

LOAD → SAME AS 5 = 184.5 PSF (1077 SF)

$P_u = 198.7^K + 282.7^K = 481.4^K \Rightarrow \boxed{W12\times53 \quad \phi P_n = 519^K}$

## 3RD FLOOR

LL REDUCTION (KL = 13.25')

↳ SAME AS 4 & 5

LOAD → SAME = 184.5 (1077) = $198.7^K$

$P_u = 198.7^K + 481.4^K = 680.1^K \Rightarrow \boxed{W12\times65 \quad \phi P_n = 701^K}$

## 2ND FLOOR (14.25')

NO LIVE LOAD REDUCTION

LOADS:

$1.2(90) + 1.6(100) = 268$ PSF (1077) =

$P_u = 288.6^K + 680.1 = 968.7^K \Rightarrow \boxed{W12\times96 \quad \phi P_n = 1012^K}$