Letter of Transmittal

Date: September 26, 2014

To: Professor Heather Sustersic

had132@psu.edu

From: Mary Julia Haverty

mjh5532@psu.edu

Enclosed: AE 481W- Senior Thesis: Structural Technical Report 2

Dear Professor Sustersic,

This report was completed to fulfill the requirement for Technical Report 2 for AE 481W- Senior Thesis. It includes an analysis of all dead and live gravity loads, including snow loads, wind loads, and seismic loads. The report was created through the use of hand calculations as well as an excel spreadsheet. Loading diagrams are provided at the end of each section for your convenience.

I appreciate you taking the time to review this report and I look forward to discussing it with you in the future.

Sincerely,

M. Julia Haverty

``

CORPORATE HEADQUARTERS

Great Lakes Region, U.S.A.

TECHNICAL REPORT 2

M. JULIA HAVERTY
STRUCTURAL OPTION
ADVISOR: H. SUSTERSIC
26 SEPTEMBER 2014

Table of Contents

Executive Summary

The Corporate Headquarters, located in the Great Lakes Region of the United States, is a new 5 story office and retail space designed to serve as new home base for an established and successful US based company. The building's architecture was designed to mirror its surrounding buildings, namely, the newer retail area situated directly to the north of the building. It aims to mirror those buildings through its façade, which changes materials in order to break up the large building. In keeping with that architectural style, the Corporate Headquarters features a façade of glass and face brick, construction crews broke ground on the roughly 660,000 SF building in August 2014 with a projected completion date of Spring 2016.

A challenge in the design of the Corporate Headquarters is the poor existing soil conditions on part of the site. To remedy this problem, aggregate piers will be pushed down below foundation level to support the column spread footings and piers. Grade beams are also utilized in the foundation system.

The floor system in floors 2-5 is a composite floor framing consisting of metal deck on top of steel wide flange members. Average bays are rectangular with typical sizes around 38'-0" x 40'-0". The primary lateral system of the building is HSS braced frames near the building's core.

The primary loading conditions considered in the design of this structure were live loads, dead loads, snow loads, wind loads, seismic loads, and soil loads. To consider these loading conditions, the 2011 Ohio Building Code was set as primary design criteria. 2011 Ohio Building Code adopts IBC 2009, which references ASCE 7-05.

Due to security reasons, detailed location maps are not permitted for this report.

Site Plan and Location

Building Location: Great Lakes Region, U.S.A.

-exact location map not permitted

Site Map

Corporate Headquarters

Great Lakes Region, U.S.A.

IN PROGRESS

Documents Used to Create Report

The following documents were used during the creation of Technical Report 2.

- **Ohio Building Code 2011**

  -incorporates IBC 2009

- **American Society of Civil Engineers**

  -ASCE 7-05: Minimum Design Loads for Buildings

- **Corporate Headquarters**

  -Construction Documents

  -Technical Specifications

- **Vulcraft Deck Catalog**

  -product manual

- **Boise- Cascade**

  -Weight of Building Materials Technical Note

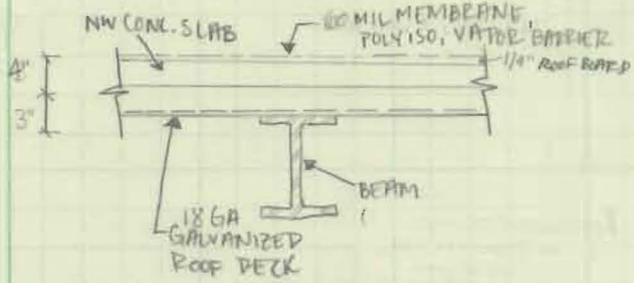
Gravity Load Calculations

GRAVITY LOAD CALCULATIONS

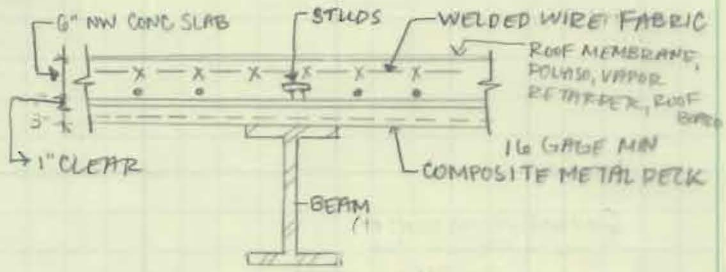
Georgia Pacific Product Spec

Typical Roof Bay Loading

Roof Construction Cross Sections



TYPE R-1 ROOF CONSTRUCTION



TYPE R-2 ROOF CONSTRUCTION

Roof R-1 Dead Load

adhered membrane = 1 PSF
1/4" DENSDECK ROOF BOARD = 1.2 PSF
polyisocyanurate roof insulation = 3 PSF
vapor retarder = 1 psf
4" Concrete Slab = 150 × (4/12) = 50 psf
18 GA galvanized metal deck = 2.9 psf
Ceilings = 5 psf
MEP = 15 psf
Sprinklers = 3 psf
Framing = 10 psf

Roof R-1 Dead Load = 92.1 = 92 psf

Roof R-2 Dead Load

adhered membrane = 1 PSF
1/4" Densdeck Roof Board = 1.2 PSF
Polyisocyanurate roof insulation = 3 PSF
vapor retarder = 1 psf
6" concrete slab = 150 pcf × (6/12) = 75 psf
16 GA composite metal deck = 2.2 psf
ceilings = 5 psf
MEP = 15 psf
sprinklers = 3 psf
Framing = 10 psf

Roof R-2 Dead Load = 116.4 psf

- though the majority of the roof is R-1 construction, enough typical bays are constructed in the R-2 style that I felt it should be included
  - approximately 1/3 of the roof is R-2

Roof Live Loading

ASCE 7-05 - Table 4-1

$L_r = 20$ psf

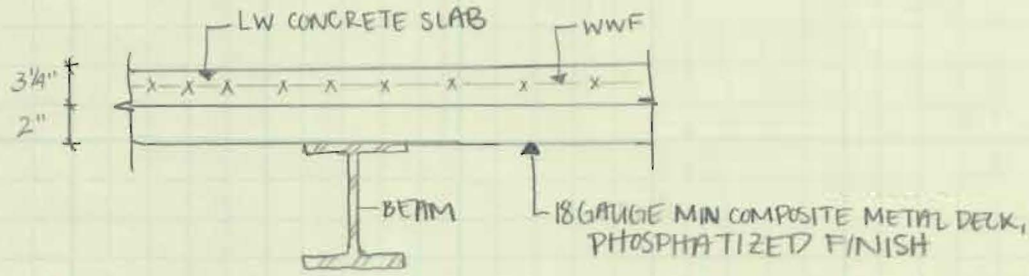
Structural Drawings - Sheet S-001

$L_r = 25$ psf

* I believe that the code value was increased due to the rooftop mechanical units and the frequency with which they may be serviced

Typical Floor Bay Loading

Floor construction Cross Section



- Concrete slab appears to be an architectural feature and has no additional topping.

Floor Dead Load

3 1/4" Concrete Slab = $(115)(\frac{3.25}{12}) = 31.15$ psf
18 Gauge Composite Metal deck = 2.8 ~ 3 psf
Ceiling = 5 psf
MEP = 10 psf
Sprinklers = 3 psf
Framing = 10 psf

Typical Floor Dead Load = 62 psf

Floor Live Load

From ASCE 7-05 Table 4-1 & § 4.2.2

Offices - 50 psf
Partitions - 15 psf
65 psf

Lobby "Public Area" — 100 psf

These live load values match those listed on S-001 of the structural drawings.

Typical Floor Bay Live Load = 65 psf

Courtyard Loading

From S-001,

	Courtyard Grass Area	Courtyard Tree Area
Dead	201 psf	441 psf
Live	100 psf	100 psf

Non-Typical Loading Conditions

Non-Typical Dead Load Values

- All Floor and Roof types have dead load values listed on sheet S-001 that match ASCE 7-05 typical values.

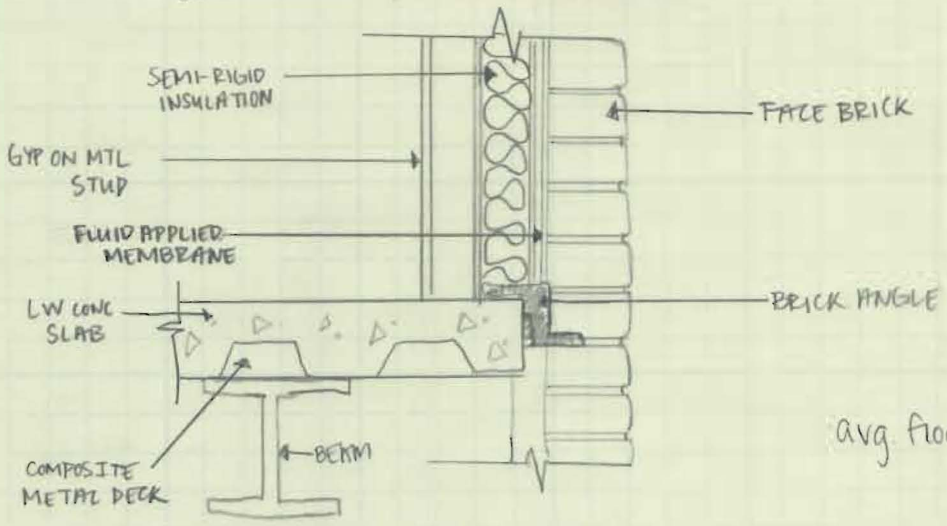
- Courtyard Tree area Soil dead load = 360 psf
  - the courtyard grass area lists a soil load of 120 psf on S-001 while the tree area lists 360 psf dead load
  - this value is large due to the trees taking root in the courtyard

Non-Typical Live Load Values

Load	Design Value	ASCE 7-05 value	Reasoning
Kitchen Area/Fridge	150 psf	125 psf	The higher load is due to heavy foot traffic in this area during lunch and the mobility of the equipment in the area, including future appliances
Typical Roof	25 psf	20 psf	As stated in roof live loading section, I believe this was done due to more frequent access to the RTU's.

Typical Exterior Wall Load

Typical Cavity Wall Section



avg floor height ~ 16' 6"

Gravity Load Path – Wall System

The weight of the cavity wall system is supported by the brick relief angle and the concrete floor slab. The weight of the face brick is taken by the angle, which then sends it to the floor system. The floor system puts the weight of the brick and cavity wall on the edge beams, which send it to the transfer girders. Those girders eventually meet one of the braced frames, which in turn send the load down to the foundation system. and into the soil.

Cavity Wall Dead Load

Brick veneer: 39 psf × (16.5') = 643.5 plf

semi-rigid insulation:
5/8" gypboard on 3 5/8" metal stud } 12 psf × 16.5' = 198 plf

fluid applied membrane: 2 psf × 16.5' = 33 plf

Total Cavity Wall Dead Load = 847.5 = 848 plf

SNOW LOAD CALCULATIONS

Flat Roof Snow Load, p_f

$p_f = 0.7 C_e C_t I p_g$ (ASCE 7-05 Eq. 7-1)

From S-001:

$p_g = 20$ psf
occupancy category II
$I = 1.1$
$C_e = 1.0$
$C_t = 1.1$
exposure B

$p_f = 0.7(1.0)(1.1)(1.1)(20)$ $p_f = 16.94 \sim 17$ psf $\underline{p_f = 17 \text{ psf}}$

Snow Drift

- calculated for drift from mechanical penthouse roof
- windward snow drift

$h_b = \frac{p_s}{\gamma}$ in this case, $p_s = p_f = 17$ psf

$\gamma = 0.13 p_g + 14$ but cannot exceed 30 pcf
$= 0.13(20) + 14$
$\gamma = 16.6$

$h_b = \frac{17 \text{ psf}}{16.6 \text{ pcf}}$ $h_b = 1.02$ ft $h_c / h_b > 0.2 \rightarrow$ drift loads must be calculated

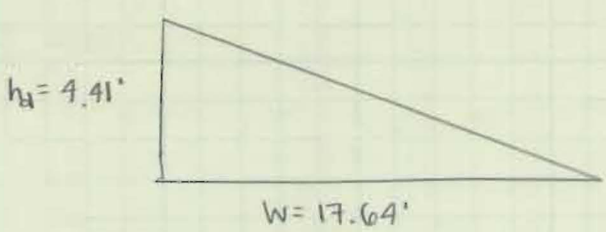
$h_c = 15'-1.5"$

$\ell_u = 394' \rightarrow$ from Figure 7-9, $h_d \sim 5.5$ ft

$h_d = 0.43 \sqrt[3]{\ell_u} \sqrt[4]{p_g + 10} - 1.5 = 0.43 \sqrt[3]{394} \sqrt[4]{20+10} - 1.5$
$h_d = 5.88'$
$3/4\, h_d = 4.41'$

$h_d > h_b$ $h_d < h_c \rightarrow w = 4h_d$ $w = 4(4.41)$ $w = 17.64'$

Drift Density $p_d = h_d \gamma$ $p_d = 4.41(16.6)$ $\underline{p_d = 73.21 \text{ psf}}$



Wind Load Calculations

WIND LOAD CALCULATIONS

· calculated using ASCE 7-05

1) Occupancy Category – II from drawings

confirmed in table 1-1

2) Wind Load Importance Factor

From drawings, V = 90 mph, category II
$I = 1.00$, From drawing, $I = 1.00$ → match

3) Basic Wind Speed, from Figure 6-1, confirmed on dwgs

$V = 90$ mph

4) Wind Load Parameters

a. Wind Directionality Factor, K_d, from Table 6-4, confirmed on dwgs.

$K_d = 0.85$

b. Exposure Category (§6.5.6.3)

Exposure B → confirmed in drawings (Case 2, not low rise building)

c. Topographic Factor, K_{zt} (§6.4.2.1 & §6.5.7, Table 6-4)

$K_{zt} = 1.0$ → confirmed on dwgs → $K_{zt} = 1.0$ (2-D escarpment.) no hill

d. Gust Effect factor (§6.5.8)

From commentary

$n_1 = 100/H$ → average value $= 100/83 = 1.20$

$n_1 = 75/H$ → lower bound $= 75/83 = .90$ ← use this value to be conservative

exposure B factors (from Table 6-2)

$\alpha = 7.0$
$z_g(ft) = 1200$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$\ell(ft) = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min}(ft) = 30$

$\bar{z}$ = equivalent height of structure $= .6h = .6(83) = 49.8'$

For buildings without concrete shear walls, a simplified procedure can be used

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.90(367.05)}{74.80} \qquad N_1 = 4.417$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{49.8}{33}\right)^{1/3} = 367.05 \text{ ft}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{49.8}{33}\right)^{1/4} 90\left(\frac{88}{60}\right) = 65.84 \text{ ft/s}$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

h = mean roof height = 83'
B = horizontal dimension of building normal to wind direction
BNS = 326'
BEW = 394'

$$Q_{NS} = \sqrt{\frac{1}{1+.63\left(\frac{326+83}{367.05}\right)}} = .77$$

$$Q_{EW} = \sqrt{\frac{1}{1+.63\left(\frac{394+83}{367.05}\right)}} = .74$$

$$G = 0.925\left(\frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right)$$

From §6.5.8.1, g_Q & g_v = 3.4

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .30\left(\frac{33}{49.8}\right)^{1/6} = .28$$

$$G_{NS} = 0.925\left(\frac{1+1.7(3.4)(.28)(.77)}{1+1.7(3.4)(.28)}\right) \qquad G_{NS} = .79$$

$$G_{EW} = 0.925\left(\frac{1+1.7(3.4)(.28)(.74)}{1+1.7(3.4)(.28)}\right) \qquad G_{EW} = .78$$

1.6184

e. Enclosure Classification (§6.5.9 & §6.2)

- Building is enclosed as it does not meet "open" and "partially enclosed" conditions

f. Internal Pressure Coefficient Figure 6-5

$GC_{pi} = \pm 0.18$ for enclosed buildings

CP Values (Figure 6-6)

- Wind typically blows Southwest in the area the Corporate Headquarters is located. It tends to hit more vertically than horizontally, so for this reason, the north side of the building will be considered windward and the south side will be considered leeward.

L = horizontal dimension of building parallel to wind direction
B = horizontal dimension of building normal to wind direction

Wall C_p Values

Windward wall: $C_p = 0.8$ use with q_z

leeward wall: $B = 326'$ $L = 394'$ $L/B = 394/326 = 1.21$

- need to interpolate to find C_p

$$x = \frac{(1.21-1)(-0.3+0.5)}{(2-1)} + -0.5$$

L/B	C_p
0-1	-0.5
1.21	x
2	-0.3

$C_p = -0.46$ use with q_h

side wall: $C_p = -0.7$, use with q_h

Roof C_p Values

- Roof has 0° slope
  - horizontal distance from windward edge = 394' $h = 83'$

  394 > 2h

  $C_p = -0.3, -0.18$

Find Wind Pressures

$K_z = 2.01 (z/z_g)^{2/\alpha}$ z = height of floor above ground

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

$p = q G_f C_p$

Floor #	z (ft)	K_z	q_z (psf)
2	20	0.62	11.0
3	37.33	0.75	13.14
4	54	0.83	14.61
5	68.67	0.89	15.64
roof	83.33	0.94	16.53

Excel Equations: $p = qCP_i$ $G_{NS} = .79$ $G_{EW} = .78$ $q_h \sim 16.5$

Building width NS = 326' Building width EW = 394'

* See excel sheet for values

Wind Pressure for Roof

$P = q_h G C_p$

$q_h = 0.00256[2.01(83.33/1200)^{2/7}](1.0)(.85)(90^2)(1.0) = 16.78$ psf

N-S direction

windward: $p = 16.78(.79)(-0.3) = -3.98$ psf

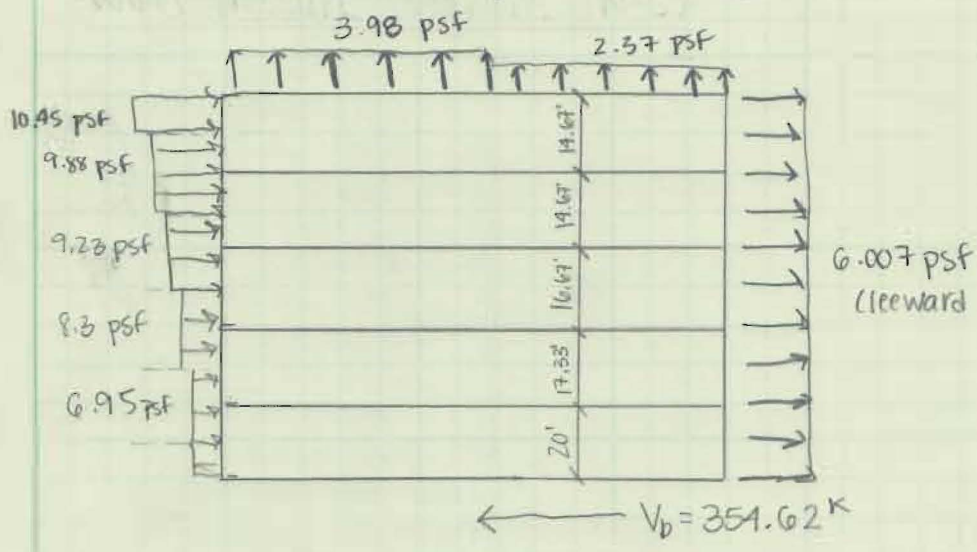
leeward: $p = 16.78(.79)(-0.18) = -2.37$ psf

E-W direction

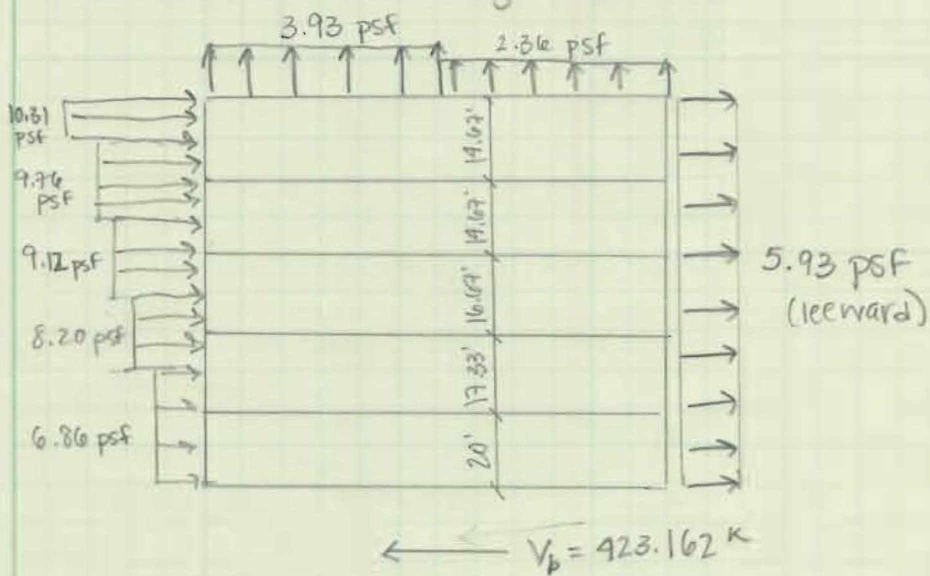
windward: $p = 16.78(.78)(-0.3) = -3.93$ psf

leeward: $p = 16.78(.78)(-0.18) = -2.36$ psf

Wind Pressure Diagram (N-S)



Wind Pressure Diagram (E-W)



Wind Pressure (North-South Direction)						
Floor	z (ft)	qz (PSF)	Windward Pressure (PSF)	Leeward Pressure (PSF)	Tributary Area	Force (K)
2	20	11	6.952	-6.007	6096	78.99808
3	37.33	13.14	8.30448	-6.007	5542	79.31423
4	54	14.61	9.23352	-6.007	5314	80.98813
5	68.67	15.64	9.88448	-6.007	4782	75.99307
roof	83.33	16.53	10.44696	-6.007	2390	39.32497
				Base Shear=		354.6185

Wind Pressure (East-West Direction)						
Floor	z (ft)	qz (PSF)	Windward Pressure (PSF)	Leeward Pressure (PSF)	Tributary Area	Force (K)
2	20	11	6.864	-5.93096	7368	94.27329
3	37.33	13.14	8.19936	-5.93096	6698	94.64491
4	54	14.61	9.11664	-5.93096	6422	96.63571
5	68.67	15.64	9.75936	-5.93096	5780	90.69007
roof	83.33	16.53	10.31472	-5.93096	2888	46.91754
				Base Shear=		423.1615

Seismic Load Calculations

SEISMIC LOAD CALCULATIONS

1) Building not exempt (§ 11.1.2)

2) Design Spectral Response Acceleration (§11.4)

a) Site Classification: C

b) Acceleration Parameters

$S_S = 0.175$ g

$S_1 = 0.051$ g

c) Site Class Effects (§11.4.3)

$F_a = 1.2$

$F_v = 1.7$

$S_{ms} = F_a S_s = 1.2(0.175) = 0.21$ g $\qquad S_{ms} = 0.21$ g

$S_{m_1} = F_v S_1 = 1.7(0.051) = 0.0867$ g $\qquad S_{m_1} = 0.0867$ g

d) Determine Spectral Acceleration Parameters (§11.4.4)

$S_{DS} = 2/3\, S_{ms} = 2/3\,(0.21)$ $\qquad S_{DS} = 0.14$

$S_{D_1} = 2/3\, S_{m_1} = 2/3\,(0.0867)$ $\qquad S_{D1} = 0.0578$

3) Find Seismic Design Category

Occupancy Category: II

Importance factor = 1.0

$S_{DS} < 0.167 \longrightarrow$ Category A

-matches category given in drawings → SDC A

4) Analysis Procedure Selection

§11.7 - "Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 11.7."

§11.7.1- Seismic loads shall be taken as "E" and combined with other load combinations from Sections 2.3 and 2.4

§1.4 - Eqn 1.4-1 $\qquad F_x = 0.01 W_x$

5-9) Skip due to SDC A

10) Calculate effective total seismic weight (W)

Roof: DL + 20% SL

W_{RF} = (Roof area)[DL + 20% SL]

Roof area = (326×394) − (174×152) = 101996 SF ~ 102000 SF

1/3 Roof area (R-2) = 34000 SF

W_{RF} = (102000 − 34000)[92 + .2(17)] + (34000)[117 + .2(17)]

W_{RF} = 10581 K

Floor: DL

Floor area (Floors 1-2) = 128444 SF

Floor area (Floors 3-5) = 102000 SF

∴ W_F = (2)128444(62) + 102000(62)(3) = 34900 K

Courtyard: Assumed to be handled as Roof area

W_C = DL + 20% SL

Courtyard area = 174×152 = 26448 SF

W_C = 26448 (441 + .2(17) = 11754 K

Total Building Load = $W_{RF} + W_F + W_C$ = 57235 K = W

11) Calculate Seismic Base Shear (V)

$V = C_s W$ $V = 0.01(57235)$ $V = 572.35^K$

12) Vertical Distribution of Seismic Forces

SDC A, so $F_x = 0.01\,W_x$

Base shear $= 572.35^k$

$F_{1^{st}\ floor\ (ground\ level)} = 0.01(128444 \times 62) = 79.64^k$

$F_{level\ 2} = 0.01(128444 \times 62) = 79.64^k$

$F_{level\ 3} = 0.01[(102000 \times 62)/1000 + 11754] = 180.78^k$
* includes courtyard

$F_{level\ 4} = 0.01[102000 \times 62] = 63.24^k$

$F_{level\ 5} = 0.01[102000 \times 62] = 63.24^k$

$F_{Roof} = 0.01[10581] = 105.81^k$