TECHNICAL ASSIGNMENT I

STRUCTURAL CONCEPTS/STRUCTURAL EXISTING CONDITIONS REPORT

329 INNOVATION BOULEVARD
STATE COLLEGE, PA

JEREMY R. POWIS
STRUCTURAL OPTION
ADVISOR: PROFESSOR M. KEVIN PARFITT
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TECHNICAL ASSIGNMENT I

EXECUTIVE SUMMARY

329 Innovation Boulevard is a completed design for multiple commercial tenants. It is located in the Innovation Park at Penn State, State College, PA. It will face Innovation Blvd. directly across from 328 Innovation Boulevard, which hosts the buildings designers, L. Robert Kimball & Associates. Due to the fact that tenants have not currently leased the provided space, the building utilizes an open floor plan to help facilitate any possible tenants.

The building is four stories tall, with a mechanical penthouse located on the roof. The total height is 58', and the footprint is 21,000 SF. It is a steel framed structure with a concrete composite flooring system. The veneer includes brick, aluminum panels, and glass curtain walls. It typically follows the style of the current buildings of Innovation Park.

The following technical report will cover the existing building conditions and structural concepts. It will include a description of the structural system, as well as numerous calculations. These calculations will include a wind and seismic analysis, and multiple spot checks of beams, girders, and columns. The calculations involved the usage of the following codes:

- International Building Code (IBC) 2006
- American Institute of Civil Engineers (ASCE) 7 – 05

The report also includes structural plans, sections, and tables to help aid in procedures used throughout the report. The findings of the report are located in the conclusion.

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The structural system for 329 Innovation Blvd. is designed so that it maximizes space that the open floor plan provides. The building doesn't currently have occupants and therefore uses an open floor plan to easily accommodate the future inhabitants. The structural system catalyzes this by consisting of wide flange beams and columns, with a composite floor system. The lateral resistances is provided by moment connections. The first floor consists of a concrete slab on grade, and the building is supported by strip footings and piers.

FOUNDATION

The foundation consists of a spread footings, pile caps, and piers. The tops of all exterior footings are 3'-4" below grade (unless noted otherwise), and the tops of all interior footings are 0'-8" below grade (unless noted otherwise). The typical footing size is 5'-0"x5'-0"x1'-9". They range from the size to the largest, which is 9'-0"X9'-0"x2'-9". The typical footing does not require reinforcement in the top; however the larger footings receive reinforcement in the top and bottom. There are three pier sizes; they include a 22"x22", 36"x36", and 32"x40". Each pier frames into a pile. Each of these components are designed with a minimum compressive strength, f'_c=4,000 psi, and the reinforcement required ranges from none to #6's through #9's. See Appendix A.2 for typical foundation sections.

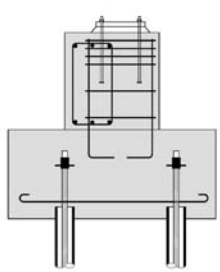


Figure 1.1
Typical Pier and Cap Section

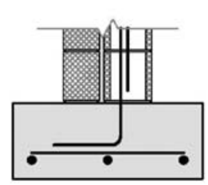


Figure 1.2
Typical Footing Section

COLUMNS

There are four different column designations in the building. The columns start on the ground floor and span from the ground floor to the third floor, which is the splice level. Typically a smaller column is used to span the remainder (third floor to roof.) Wide flange steel shapes ranging from W12x40 to W12x96 are utilized. To induce an open floor plan, the column grid is very regular and remains so throughout the four floors. Three different plate sizes are used, and each column requires four anchor bolts ranging from ¾" to 1½".

FLOOR FRAMING

The floor framing system consists of a composite slab and metal deck on wide flange beams and girders. The concrete used is 3½" lightweight concrete with one layer of 6x6xW1.4xW1.4 WWF. The metal decking used is 3" galvanized wide rib type composite deck. The decking is to be continuous over a minimum of three spans. The total thickness of the flooring system comes to 6½" and therefore, the top of steel (beams and girders) is located at -6½" from the finished floor. The typical size of the beams is W18x35 and they span 33'-3" and the girders range from W18x35 to W21x44 and typically span 30'0". There are minimal interferences on each floor, making each of the three floor systems practically identical.

LATERAL ELEMENTS

Lateral resistance is provided by several full moment connections of beams, girders, and columns. These connections can be found in the middle bay of the building on each end of the building. There are two columns on each end where the two beams and two girders are all connected by full moment connections. Majority of the moment connections occur in the interior of the building, and there are total of twelve moment connections on the exterior frame. The mechanical penthouse located on the roof utilizes flat strap bracing in plane with the stud wall.

Building Code:	Pennsylvania Uniform Construction Code
Concrete Design Code:	American Concrete Institute (ACI) 318-02, Building Code & Commentary
Concrete Design Method:	Equivalent Rectangular Stress Block
Masonry Design Code:	ACI 530 & 530.1
Masonry Design Method:	Allowable Stress Design
Steel Design Code:	American Institute of Steel Construction (AISC), LRFD, Second Edition
Steel Design Method:	Elastic Analysis, Plastic Design

LIVE LOADS

Corridors	100 PSF
Stairs	100 PSF
Public Areas	100 PSF
Mechanical/Electrical Rooms	175 PSF
Open Plan Office (80 PSF + 20 PSF Partitions)	100 PSF
Slabs-On-Grade (U.N.O.)	100 PSF
Slabs-On-Grade (Dock/Receiving)	200 PSF

ROOF LIVE LOADS

Minimum Roof Live Load	20 PSF

DEAD LOADS

Partition Allowance	20 PSF
Lightweight Concrete Slab	115 PCF
MEP	5 PSF
Metal Decking	2-3 PSF (Deck Catalog)
Beam Weight	Specific To Each Member

SNOW LOADS

Terrain Category	C
Ground Snow Load (P_g)	40 PSF
Snow Exposure Factor (C_e)	0.9
Thermal Factor (C_t)	1.0
Snow Importance Factor (I_s)	1.0

WIND LOADS

Minimum Wind Load	10 PSF
Uplift On Roof	20 PSF
Basic Wind Velocity	90 MPH
Wind Importance Factor	1.0
Wind Exposure Category	C
Internal Pressure Coefficient	±0.18
Components And CLadding	By Supplier

SEISMIC LOADS

Seismic Importance Factor (I_E)	1.0
Seismic Response Acceleration (S_s)	16.8%
Spectral Response Acceleration (S_1)	5.9%
Spectral Response Coefficient (S_{DS})	13.4%
Spectral Response Coefficient (S_{D1})	6.7%
Seismic Design Category	A
Site Class	C
Long-Period Transition Period (T_L)	6 Sec.
Seismic Force Resisting System	Undetailed
Response Modification Factor (R)	3.0
Seismic Response Coefficient (C_S)	0.045
Deflection Amplification Factor (C_d)	3.0
Design Base Shear	60 Kips
Analysis Procedure	Eq. Lat. F.

Through the numerous exercises involved with this first examination of the building, I feel that I am able to grasp what is going on in and outside of the building more clearly. The values I obtained through the wind analysis and multiple spot checks, lead me to believe that, for the most part, my assumptions and loadings were correct. I was, however, unable to match the design base shear value for seismic used by the engineers. There could be numerous variables that may have caused this to occur, but by re-calculating the base shear using the constants provided by the plans, I feel that the discrepancy lies within the building weight. I found that their building weight was much lower than the weight I obtained. The weight I obtained even excluded the exterior walls of the building. The location of building and the fact that wind normally controls in that location, may allow for a lower base shear value. The spots checks involving the lateral forces obtained did result in similar member sizes, which leads me to believe a wind controlled loading combination was used.

The results of my wind and seismic analysis are as follows:

SEISMIC (TOTAL BASE SHEAR OF 125^{K})

Floor	Weights			Total	Height (ft)	k	wh^k	Cvx	V (K)	Fx
	Fl/Rf Deck.	Beams	Cols.							
2	1001.4	175.3	40.2	1216.8	14.0	1.11	22773.8	0.0876	125	10.95
3	1001.4	175.3	40.2	1216.8	28.0	1.11	49156.3	0.1892	125	23.65
4	1001.4	175.3	40.2	1216.8	42.0	1.11	77097.5	0.2967	125	37.09
Roof	1001.4	175.3	45.9	1222.6	58.0	1.11	110836.4	0.4265	125	53.31
Totals				**4873.1**			**259863.92**	**1.0000**		**125.00**

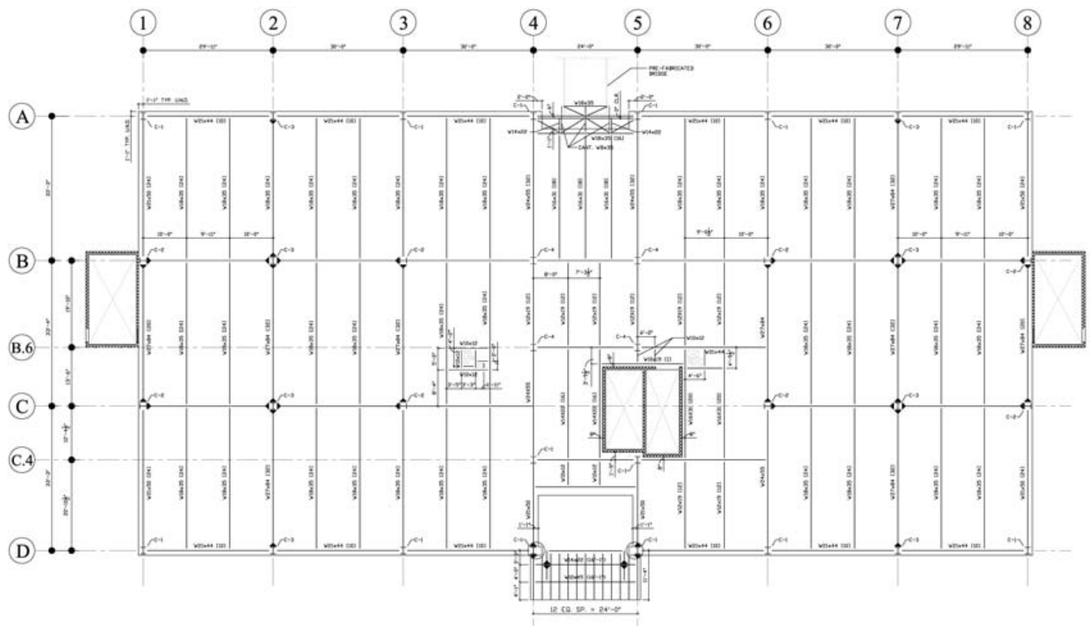
WIND (TOTAL WIND LOADS)

	Surface Area (ft^2)	Total WW Load	Total LW Load	Total Force (K)	
North-South	11950	178.35	100.74	**279.1**	**Transverse To Long Dir.**
East-West	5920	116.00	25.52	**141.5**	**Transverse To Short Dir.**

APPENDICES

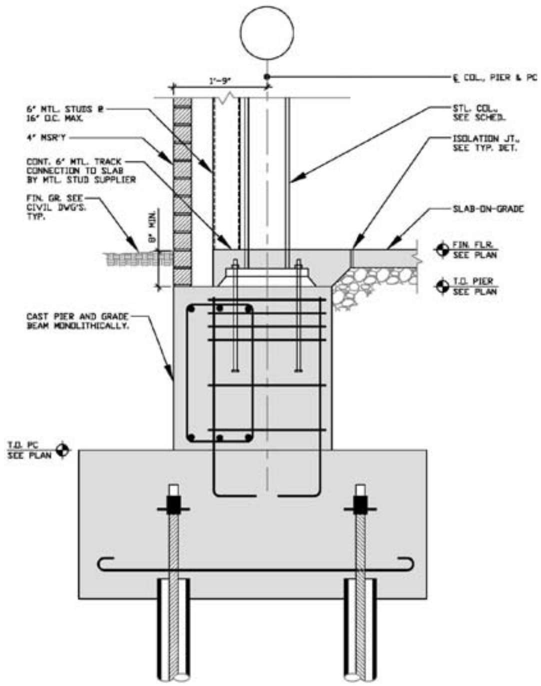
A.1 STRUCTURAL PLANS

Second Floor Framing Plan

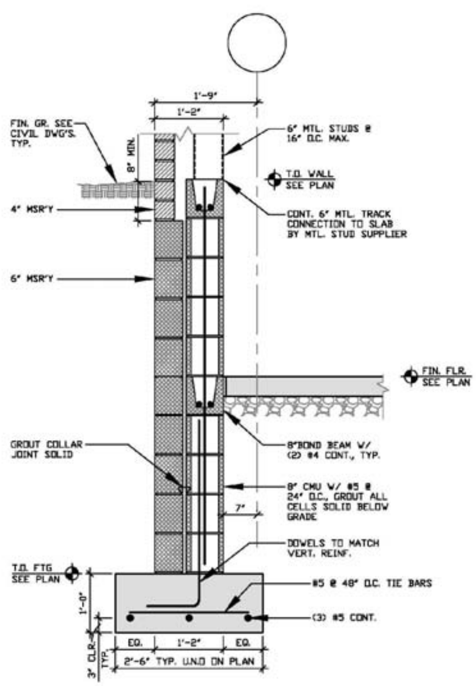


Typical Framing Plan

TYPICAL PIER SECTION



TYPICAL FOOTING SECTION



SEISMIC LOAD ANALYSIS

OCCUPANCY CATEGORY: II (STANDARD OCCUPANCY STRUCTURE)
IMPORTANCE FACTOR (I_E): 1.0
SITE CLASS: C

FROM USGS WEB-PAGE: $S_S = 0.147$
$S_1 = 0.049$

TABLE 11.4-1: $S_S \leq 0.25$, SITE CLASS C → THEREFORE: $F_a = 1.2$

TABLE 11.4-2: $S_1 \leq 0.1$, SITE CLASS C → THEREFORE: $F_v = 1.7$

$S_{MS} = F_a \cdot S_S = 1.2(0.147) = 0.1764$

$S_{M1} = F_v \cdot S_1 = 1.7(0.049) = 0.0833$

$S_{DS} = (2/3) S_{MS} = 0.1176$

$S_{D1} = (2/3) S_{M1} = 0.05553$

R: RESPONSE MODIFICATION COEFFICIENT, TABLE 12.2-1
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE EXCLUDING CANTILEVER COLUMN SYSTEMS:

$R = 3.0$, $\Omega_0 = 3.0$, $C_d = 3.0$ SEISMIC DESIGN CATEGORY A: NOT LIMITED

FUNDAMENTAL PERIOD OF THE STRUCTURE: $T = C_t h_n^x$ $h_n = 58'$
TABLE 12.8-2:
STEEL MOMENT RESISTING FRAMES: $C_t = 0.028$, $x = 0.8$

$T = 0.028(58)^{0.8} = 0.72094$ $T_L = 6$ SEC. FIGURE 22-15

$$C_s = \text{MIN} \begin{cases} S_{DS}/(R/I) = 0.1176/(3.0/1.0) = 0.0392 \\ S_{D1}/[T(R/I)] = 0.05553/[0.72094(3.0/1.0)] = 0.0257 \leftarrow \text{CONTROLS} \\ S_{D1} \cdot T_L/[T^2(R/I)] = 0.05553(6)/[0.72094^2(3.0/1.0)] = 0.2142 \end{cases}$$

SEE SPREADSHEETS FOR CALCULATION OF W

DESIGN SEISMIC BASE SHEAR: $V = C_s \cdot W$

$V = 0.0257(4873^k) = 125^k$

SEISMIC LOAD ANALYSIS (CONT'D)

VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$ WHERE, $C_{vx} = w_x h_x^k / \sum_{i=1}^{n} w_i h_i^k$

$T = 0.5,\ k = 1.0$
$T = 0.72094,\ k = ?$ $\quad k = \frac{(0.72094 - 0.5)(2.0 - 1.0)}{(2.5 - 0.5)} + 1.0$
$T = 2.5,\ k = 2.0$ $\quad k = 1.11$

$\sum w_i h_i^k = (4871)(58)^{1.11} =$

SEE SPREADSHEET FOR C_{vx} VALUES
SEE SPREADSHEET FOR F_x VALUES (VERTICAL DIST. OF SEIS. FORCES)

Due to the fact that the design base shear, V = 125^k, I found is much greater than the given design base shear from the designer (V = 125^k), I re-investigated the seismic analysis with the following calculations. I feel the biggest discrepancy is the weight of the building.

SEISMIC ANALYSIS

USING SPECIFIED DESIGN VALUES:
DESIGN VALUES: $S_S = 0.168 < 0.25$; THEREFORE, $F_a = 1.2$
$S_1 = 0.059 < 0.1$; THEREFORE, $F_V = 1.7$
CALCULATED VALUES:
$S_{MS} = F_a \cdot S_S = 1.2(0.168) = 0.2016$ $\quad S_{DS} = (2/3) S_{MS} = (2/3)(0.2016) = 0.1344$

$S_{M1} = F_V \cdot S_1 = 1.7(0.059) = 0.1003$ $\quad S_{D1} = (2/3) S_{M1} = (2/3)(0.1003) = 0.0669$

DESIGN VALUES: $S_{DS} = 13.4\%$ COMPARED TO 0.1344 OK
$S_{D1} = 6.7\%$ COMPARED TO 0.0669 OK

GIVEN EXPOSURE MODIFICATION COEFFICIENT, $R = 3.0$

FUNDAMENTAL PERIOD = $T = C_t h_n^x$ TABLE 12.8-2: $C_t = 0.028,\ x = 0.8$

$T = 0.028(58)^{0.8} = 0.72094$, $T_L = 6.0$ sec.

$$C_S = \text{MIN} \begin{cases} S_{DS}/(R/I) = 0.1344/(3.0/1.0) = 0.0448 \\ S_{D1}/T(R/I) = 0.0669/(0.72094)(3.0/1.0) = 0.0309 \leftarrow \text{CONTROLS} \\ S_{D1} \cdot T_L / T^2 (R/I) = 0.0669(6)/(0.72094)^2(3.0/1.0) = 0.257 \end{cases}$$

GIVEN $C_S = 0.045$, WHICH IS OBTAINED BY $S_{DS}/(R/I) = 0.0448$

DESIGN BASE SHEAR: $V = C_S W = 60^k$ (GIVEN)
$60 = 0.045 W$
$W = 1333.3^k << 5875^k$

WEIGHT OF COMPOSITE DECKING

Weight of Composite Decking			
Floors		Thickness	DL (PSF)
2^{nd} - 4^{th}	Decking		
Area (ft^2)	20 Gauge	3"	2.14
21,012	Concrete		
	LWC	3.25"	45.52
Total DL			47.7
Total Weight (K/FLR)			1001
Total Weight (K)			**3004**
Roof	Decking		
Area (ft^2)	20 Gauge	1.5"	2.14
21,012	Concrete		
	LWC	3.25"	45.52
Total DL			47.7
Total Weight (K)			**1001**

WEIGHT OF STEEL BEAMS

Approx. Wt. of Steel Beams in Building			
# of Member	**Member Size (#/ft.)**	**Length of Mem. (')**	**Weight (#)**
4	40	33.25	5320
2	50	30	3000
Total			**8320**
Area of Typ. Bay (SF)	**#/SF (Bay)**	**Area of Bldg. (SF)**	**Total Wt./Floor (K)**
997.5	8.34	21012	175.26
Total Weight of Steel (K)			**701**

WEIGHT OF STEEL COLUMNS

Approx. Wt. of Steel Columns in Building				
Column Desig.	# of Columns	Column Size (#/ft)	Length (ft.)	Wt./Col. (#)
C1	12	53	44	2332
		40	14	560
Total/Column				**2892**
C2	8	96	44	4224
		65	14	910
Total/Column				**5134**
C3	8	190	44	8360
		87	14	1218
Total/Column				9578
C4	4	65	44	2860
		45	14	630
Total/Column				**3490**

Column Desig.	# of Columns	Wt./Col. (#)	Total Wt/Col. Group (#)
C1	12	2892	34704
C2	8	5134	41072
C3	8	9578	76624
C4	4	3490	13960
Total Wt. Of Columns in Bldg. (K)			**166.36**

TOTAL WEIGHT OF BUILDING

Summation Of Total Weights	
	Weight (K)
Floor Decking	3004.3
Roof Decking	1001.4
Steel Beams	701.0
Steel Columns	166.4
Total Weight (W)	**4873.1**

VERTICAL DISTRIBUTION FACTORS (C_{VX}) AND VERTICAL DISTRIBUTION OF SEISMIC FORCES (F_X)

Floor	Weights			Total	Height (ft)	k	wh^k	C_{vx}	V (K)	F_x
	Fl/Rf Deck.	Beams	Cols.							
2	1001.4	175.3	40.2	1216.8	14.0	1.11	22773.8	0.0876	125	10.95
3	1001.4	175.3	40.2	1216.8	28.0	1.11	49156.3	0.1892	125	23.65
4	1001.4	175.3	40.2	1216.8	42.0	1.11	77097.5	0.2967	125	37.09
Roof	1001.4	175.3	45.9	1222.6	58.0	1.11	110836.4	0.4265	125	53.31
Totals				**4873.1**			**259863.92**	**1.0000**		**125.00**

WIND LOAD ANALYSIS

BUILDING NAME: 329 INNOVATION BOULEVARD
BUILDING LOCATION: STATE COLLEGE, PA

DESIGN WIND SPEED, V = 90 mph — IBC 2006 FIG. 1609
WIND IMPORTANCE FACTOR: I = 1.0 — ASCE7-05 TABLE 6-1
WIND EXPOSURE CATEGORY: C — IBC 2006 1609.4

VELOCITY PRESSURE EXPOSURE COEFFICIENTS (K_h + K_z) — ASCE7-05 TABLE 6-3

HEIGHT ABOVE GROUND LEVEL (ft)	EXPOSURE C	MWFRS $K_z = 2.01(z/z_g)^{2/\alpha}$
0-15	0.85	0.85
20	0.90	0.90
25	0.94	0.95
30	0.98	0.98
40	1.04	1.04
50	1.09	1.09
60	1.13	1.14

TOPOGRAPHIC FACTOR, K_{zt} — ASCE7-05, SECTION 6.5.7.2

$$K_{zt} = (1 + K_1 K_2 K_3)^2 = 1.0$$

WIND DIRECTIONALITY FACTOR, $K_d = 0.85$ — ASCE7-05, TABLE 6-4

$T = C_t h_n^x = 0.72094$ — SEE SEISMIC CALCS.

ASSUME RIGID FRAME:

$G = 0.925\left[(1 + 1.7 g_Q I_{\bar{z}} Q)/(1 + 1.7 g_v I_{\bar{z}})\right]$ — ASCE7-05 SECTION 6.5.8.1, EQUATION 6-4

$I_{\bar{z}} = c(33/\bar{z})^{1/6} = 0.20(33/34.8)^{1/6} = 0.198$ — EQUATION 6-5

TURBULENCE INTENSITY FACTOR, $c = 0.20$ — TABLE 6-2

EQUIVALENT HEIGHT OF STRUCTURE, $\bar{z} = 0.6h = 0.6(58') = 34.8'$ ← CONTROLS

$\bar{z} = 15'$

$g_Q = g_v = 3.4$

$Q = \left[1/1 + 0.63(B+h/L_{\bar{z}})^{0.63}\right]^{1/2}$ — EQUATION 6-6

$L_{\bar{z}} = l(\bar{z}/33)^{\bar{\epsilon}} = 500(34.8/33)^{1/5} = 505.34$ — EQUATION 6-7

$l = 500'$, $\bar{\epsilon} = 1/5.0$ — TABLE 6-2

B = 102' SHORT DIRECTION, 206' LONG DIRECTION, h = 58' — BUILDING STATISTICS

$Q = \left[1/1 + 0.63(102 + 58/505.34)^{0.68}\right]^{1/2} = 0.587$ ⊥ TO SHORT DIRECTION

$Q = \left[1/1 + 0.63(206 + 58/505.34)^{0.63}\right]^{1/2} = 0.840$ ⊥ TO LONG DIRECTION

WIND LOAD ANALYSIS (CONT'D)

SHORT DIRECTION:

$G = 0.925[1+1.7(3.4)(0.198)(0.587)/(1+1.7(3.4)(0.198))] = 0.721$

LONG DIRECTION:

$G = 0.925[1+1.7(3.4)(0.198)(0.84)/(1+1.7(3.4)(0.198))] = 0.846$

WALL PRESSURE COEFFICIENTS, C_p — FIGURE 6-6

SURFACE	L/B	C_p	USE WITH
WINDWARD WALL	ALL VALUES	0.8	q_z
LEEWARD WALL	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
SIDE WALL	ALL VALUES	-0.7	q_h

$p = q(GC_p) - q_i(GC_{pi})$ — EQUATION 6-23

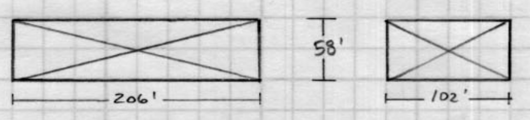
$q_h = 0.00256 K_z K_{zt} K_d V^2 I$ — EQUATION 6-15

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

$q_h = 17.6256(1.13) = 19.92$ psf $= q_i$

SURFACE AREAS: — BUILDING STATISTICS

NORTH-SOUTH	EAST-WEST
$(58')(206') = 11{,}950\ ft^2$	$(58')(102') = 5{,}920\ ft^2$



L/B: $102/206 = 0.495$ $\quad$ $206/102 = 2.02$

C_p: -0.5 $\quad$ -0.3

$G_{pi} = \pm 0.18$ — FIGURE 6-5

WINDWALL: $p = q_z G C_p - q_h(GC_{pi})$

LEEWARD: $p = q_h G C_p$

SIDEWALL: $p = q_h G C_p - q_i(GC_{pi})$

FROM SPREADSHEET:

TRANSVERSE TO LONG DIRECTION: $V_w = 279.1^k$

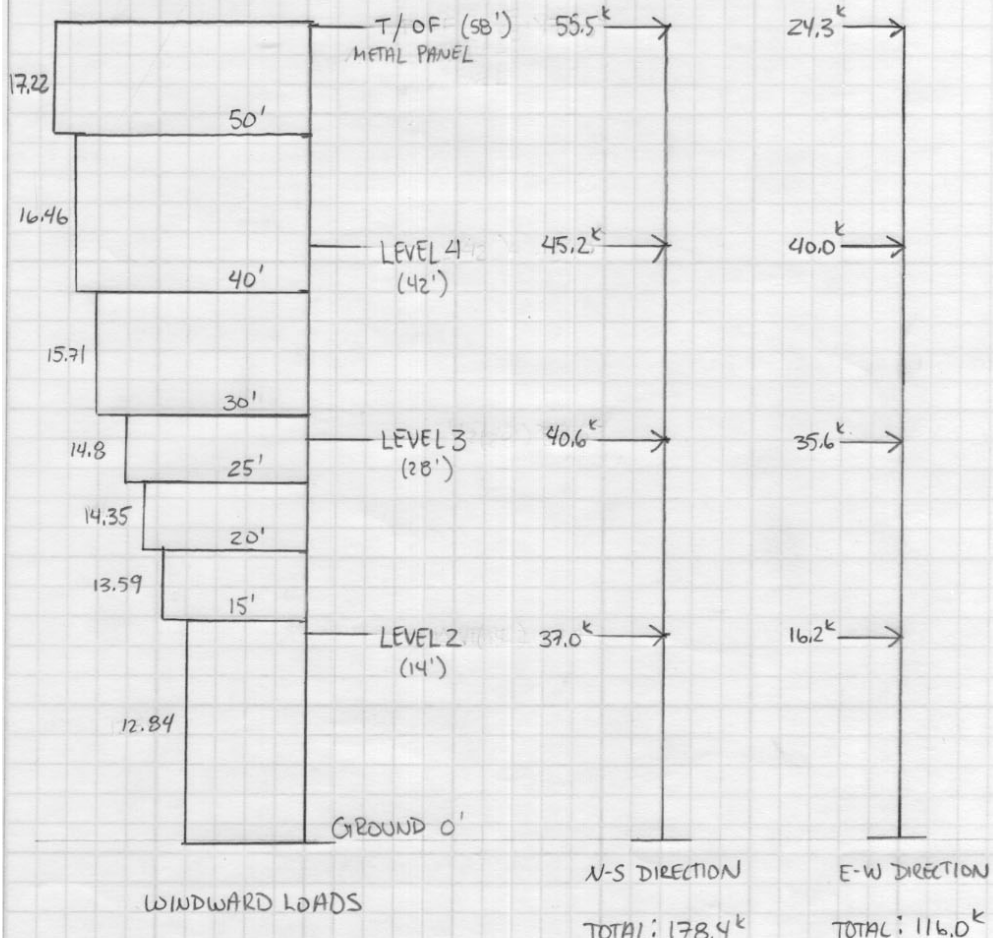
TRANSVERSE TO SHORT DIRECTION: $V_w = 141.5^k$

WIND LOAD ANALYSIS (CONT'D)

MAXIMUM WINDWARD PRESSURES
TAKEN FROM SPREADSHEETS (LONG. DIRECTION)

LOADS @ EACH ALSO TAKEN FROM SPREADSHEETS

LOADING DIAGRAMS:



VELOCITY PRESSURES

Velocity Pressure					
K_z	K_{zt}	K_d	V^2	I	**q**
0.85	1.00	0.85	8100.00	1.00	**14.98**
0.90	1.00	0.85	8100.00	1.00	**15.86**
0.95	1.00	0.85	8100.00	1.00	**16.74**
0.98	1.00	0.85	8100.00	1.00	**17.27**
1.04	1.00	0.85	8100.00	1.00	**18.33**
1.09	1.00	0.85	8100.00	1.00	**19.21**
1.14	1.00	0.85	8100.00	1.00	**20.09**

WINDWARD/LEEWARD PRESSURES

		Long Direction		Short Direction		
Ht. Above Grade	q	WW Pressure	LW Pressure	WW Pressure	LW Pressure	Side Wall
0-15	14.98	12.84	-8.43	11.34	-4.31	-14.83
20	15.86	13.59	-8.43	12.01	-4.31	-14.83
25	16.74	14.35	-8.43	12.67	-4.31	-14.83
30	17.27	14.80	-8.43	13.07	-4.31	-14.83
40	18.33	15.71	-8.43	13.87	-4.31	-14.83
50	19.21	16.46	-8.43	14.54	-4.31	-14.83
60	20.09	17.22	-8.43	15.21	-4.31	-14.83

TOTAL WW LOAD TRANSVERSE TO SHORT DIRECTION

	Elevation (ft)	Pressure	Length (ft)	Load (K)
2	14.00	11.34	102	16.19
	15.00	11.34	102	
	20.00	12.01	102	
	25.00	12.67	102	
3	28.00	12.67	102	35.59
	30.00	13.07	102	
	40.00	13.87	102	
4	42.00	14.54	102	39.95
	50.00	14.54	102	
Roof	58.00	15.21	102	24.28
Total				**116.00**

TOTAL WW LOAD TRANSVERSE TO LONG DIRECTION

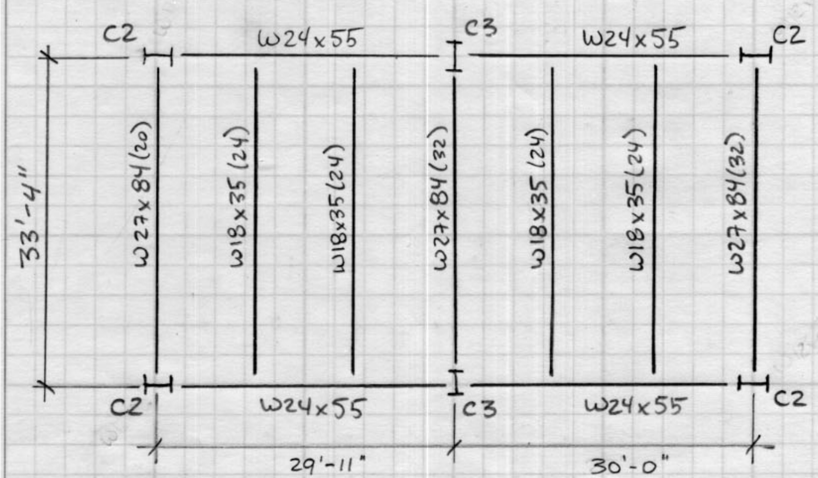
Level	Elevation (ft)	Pressure	Length (ft)	Load (K)
2	14.00	12.84	206	37.03
	15.00	12.84	206	
	20.00	13.59	206	
	25.00	14.35	206	
3	28.00	14.80	206	40.57
	30.00	14.80	206	
	40.00	15.71	206	
4	42.00	16.46	206	45.24
	50.00	16.46	206	
Roof	58.00	17.22	206	55.50
Total				**178.35**

TOTAL WIND LOADS

	Surface Area (ft^2)	Total WW Load	Total LW Load	Total Force (K)	
North-South	11950	178.35	100.74	**279.1**	**Transverse To Long Dir.**
East-West	5920	116.00	25.52	**141.5**	**Transverse To Short Dir.**

SPOT CHECK CALCULATIONS

TYPICAL INTERIOR BAY



C2: W12x96
C3: W12x190

LOADING:

LIVE LOADS

PUBLIC AREAS	100 PSF

DEAD LOADS

PARTITION ALLOWANCE	20	PSF
LWC (115 PCF · 3¼")	50	PSF
MEP	5	PSF
METAL DECKING (3")	2.14	PSF

FACTORED LOAD: $1.2D + 1.6L$

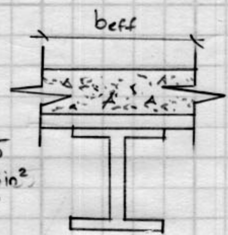
$$1.2(77.14) + 1.6(100) = 252.6 \text{ PSF}$$

BEAM SPOT CHECK

SPOT CHECK CALCS. (CONT'D)

BEAM SPOT CHECK:

FACTORED LOAD = 252.6 PSF (10') = 2526 PLF



W18x35
A_s = 10.3 in²
d = 17.7"

W_u = 2526 PLF + 1.2(40 PSF)(10') = 3.0 KLF
F_y = 50 KSI, F'_c = 4 ksi

$b_{eff} = \begin{cases} l/4 = 33.\bar{3}'/4 = 8.\bar{3}' \leftarrow \text{CONTROLS} \\ \text{SPACING} = 10' \end{cases}$

$M_u = wl^2/8 = 3.0(33.\bar{3}')^2/8 = 416.7'^k$

$T = A_s f_y$
$= 10.3(50)$
$= 515^k = \Sigma Q_n$

$\phi M_n = 0.9\left[515(5.5") + 515\left(\frac{17.7}{2}\right)\right]$

$\phi M_n = 554'^k$

$T = C$
$C = 515^k = 0.85 f'_c\, b_{eff}\, a$
$515 = 0.85(4)(8.\bar{3})(12)a$
$a = 1.51"$

$\boxed{\phi M_n = 554'^k \geq M_u = 416.7'^k \quad \underline{OK}}$

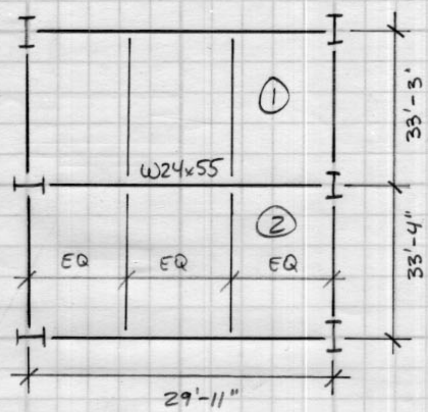
$6.25" - \frac{1.51"}{2} = 5.5"$ OK

THIS DESIGN ADEQUATELY SUPPORTS THE GIVEN LOADS.

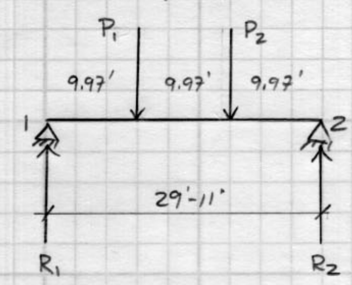
GIRDER SPOT CHECK

SPOT CHECK CALCS. (CONT'D)

SPOT CHECK TYPICAL GIRDER



LOADING DIAGRAM:



LOADS: $W_u = 252.6$ PSF $(9.97') = 2.52$ KLF

1: $\dfrac{2.52 \text{ KLF } (33'\text{-}3")}{2} = 41.9^k = P_1$

2: $\dfrac{2.52 \text{ KLF } (33'\text{-}4")}{2} = 42.0^k = P_2$

$P_{TOTAL} = P_1 + P_2 = 41.9^k + 42.0^k$

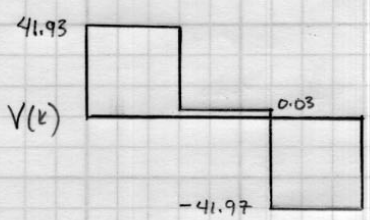
$= 83.9^k$

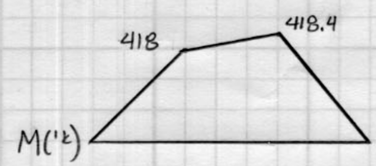
$\circlearrowleft \Sigma M_1 = 0 = -41.9^k(9.97') - 42.0^k(19.94') + R_2(29.91')$

$1255.223'^k = R_2(29.91')$

$41.97^k = R_2$

$41.93^k = R_1$





$M_{MAX} = 418.4'^k$

$\phi M_p = 503'^k$ FOR W24x55

$503'^k > 418.4'^k \therefore$ GIRDER IS ADEQUATE

COLUMN SPOT CHECK

SPOT CHECK CALCS. (CONT'D)

COLUMN CHECK:

COLUMN B-4 DESIGNATED C-4
CARRYING ROOF LOADS

ASSUMPTIONS:
1. INTERIOR GRAVITY COLUMN
2. NO LATERAL LOADS CONTRIBUTE TO LOADING.

COLUMN SCHEDULE

SPAN	C-1	C-2	C-3	C-4
1ST-3RD	W12x53	W12x96	W12x190	$\boxed{\text{W12x65}}$
3RD-ROOF	W12x40	W12x65	W12x87	$\boxed{\text{W12x45}}$

TRIBUTARY AREA: $\left[\tfrac{1}{2}(30') + \tfrac{1}{2}(24')\right]\left[\tfrac{1}{2}(33'\text{-}3") + \tfrac{1}{2}(33'\text{-}4")\right]$

$A_T = 898.9$ SF

ROOF LOADING

LOADS:

SNOW:	$0.7P_s = 0.7(40\text{ PSF})$	=	28 PSF
ROOF LIVE:	$1.6(20\text{ PSF})$	=	32 PSF
SLAB & DECK:	$1.2(30\text{ PSF})$	=	36 PSF
JOISTS:	$1.2(30\text{ PSF})$	=	36 PSF
			132 PSF

TOTAL LOAD:
$132\text{ PSF}(898.9\text{ SF})/1000^{\#} = 118.7^k$

STEEL MANUAL:
EFFECTIVE LENGTH: 16'
W12x45
$\phi_c P_n = 291^k > 118.7^k \therefore$ OK

3RD FLOOR LOADING $(+118.7^k)$

LIVE LOAD REDUCTION:
$L = L_o(0.25 + 15/\sqrt{A_I})$
$= L_o(0.25 + 15/\sqrt{21012})$
$= 0.353L_o$

LOADS:

LIVE:	$100\text{ PSF}(1.6)(0.353)$	=	56.6 PSF
SLAB:	$1.2(30\text{ PSF})$	=	36 PSF
BEAMS:	$1.2(15\text{ PSF})$	=	18 PSF
			110.6 PSF

$A_T = 898.9$ SF

TOTAL LOAD:
$110.6\text{ PSF}(898.9\text{ SF})/1000^{\#} + 118.7^k = 218.1^k$

STEEL MANUAL:
EFFECTIVE LENGTH: 14'
W12x45
$\phi_c P_n = 343^k > 218.1^k \therefore$ OK

COLUMN SPOT CHECK (CONT'D)

2ND FLOOR LOADING (+218.1^k)

LIVE LOAD REDUCTION
$L = 0.353 L_o$

$A_T = 898.9$ SF

LOADS:

LIVE:	100 PSF(1.6)(0.353)	= 56.6 PSF
SLAB:	1.2 (30 PSF)	= 36 PSF
BEAMS:	1.2 (15 PSF)	= 18 PSF
		110.6 PSF

TOTAL LOAD:
110.6 PSF (898.9SF)/1000$^{\#}$ + 218.1^k = 317.5^k

STEEL MANUAL:
EFFECTIVE LENGTH: 14'
W12x65
$\phi_c P_n = 685^k > 317.5^k$
∴ OK

1ST FLOOR LOADING (+317.5^k)

LIVE LOAD REDUCTION
$L = 0.353 L_o$

$A_T = 898.9$ SF

LOADS:

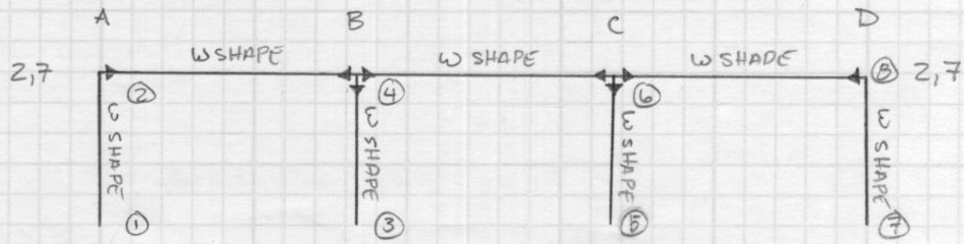
LIVE:	100 PSF(1.6)(0.353)	= 56.6 PSF
SLAB:	1.2 (30 PSF)	= 36 PSF
BEAMS:	1.2 (15 PSF)	= 18 PSF
		110.6 PSF

TOTAL LOAD
110.6 PSF (898.9 SF)/1000$^{\#}$ + 317.5^k = 416.9^k

STEEL MANUAL:
EFFECTIVE LENGTH: 14'
W12x65
$\phi_c P_n = 685^k > 416.9^k$
∴ OK

LATERAL SPOT CHECK CALCS.

FRAME BETWEEN A-7 AND D-7, AS WELL AS
FRAME BETWEEN A-2 AND D-2



▶ DENOTES MOMENT CONNECTION ① DENOTES NODE NUMBER

LOADING: WIND: 55.5^k
LIVE: 20 PSF MIN ROOF LIVE LOAD
DEAD: 115 PCF (LWC) · 3.5" = 33.5 PSF
ROOF DECKING. = 2 PSF
WT. OF JOISTS = 30 PSF
65.5 PSF

REFER TO STAAD/PRO OUTPUT DATA TABLE FOR REACTIONS

COLUMNS ASD ANALYSIS L=16'

MAXIMUM REACTIONS: 53.1^k MAX. USED FOR SYMMETRY
STEEL MANUAL: W8x31 : $P_n/\Omega = 141^k > 53.1^k$ ∴ OK
* DESIGNER USED W12x45 BECAUSE THE COLUMNS CONTINUE TO THE NEXT FLOOR AND MUST SUPPORT THOSE LOADS.

BEAMS ASD ANALYSIS SPAN = 33'-4", 33'-3"

MAXIMUM MOMENT: 3.36×10^3 "k = 280'k USED FOR SYMMETRY
STEEL MANUAL: MOST ECONOMICAL > 280'k = W21x55 $M_{px}/\Omega = 314'^k > 280'^k$
DESIGNER USED W24x55, $M_{px}/\Omega = 334'^k$ ∴ OK
- ASSUMED BEAM IS FULLY BRACED BY STEEL DECK
- CONNECTIONS DESIGNED TO W/STAND MOMENTS AND SHEAR

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:DEAD + LIVE	-0.542	38.280	0.000	0.000	0.000	805.400
3	1:DEAD + LIVE	-27.055	53.091	0.000	0.000	0.000	2.37E 3
5	1:DEAD + LIVE	-14.536	-2.298	0.000	0.000	0.000	1.58E 3
7	1:DEAD + LIVE	-13.367	7.269	0.000	0.000	0.000	1.49E 3

Beam Maximum Moments

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip-in)	d (ft)	Max Mz (kip-in)
1	2	33.250	1:DEAD + LIVE	Max -ve	0.000	0.000	33.250	3.36E 3
				Max +ve	0.000	0.000	13.854	-2.41E 3
2	4	33.333	1:DEAD + LIVE	Max -ve	0.000	0.000	0.000	533.165
				Max +ve	0.000	0.000		
3	6	33.250	1:DEAD + LIVE	Max -ve	0.000	0.000	33.250	1.08E 3
				Max +ve	0.000	0.000	0.000	-904.920
4	1	16.000	1:DEAD + LIVE	Max -ve	0.000	0.000	0.000	805.400
				Max +ve	0.000	0.000		
5	3	16.000	1:DEAD + LIVE	Max -ve	0.000	0.000	0.000	2.37E 3
				Max +ve	0.000	0.000	16.000	-2.83E 3
6	5	16.000	1:DEAD + LIVE	Max -ve	0.000	0.000	0.000	1.58E 3
				Max +ve	0.000	0.000	16.000	-1.22E 3
7	7	16.000	1:DEAD + LIVE	Max -ve	0.000	0.000	0.000	1.49E 3
				Max +ve	0.000	0.000	16.000	-1.08E 3