Technical Report #1

Due October 5th, 2009

Table of Contents

Executive Summary:

This report is intended to analyze the existing conditions of the structural system of the Butler Health System – New Inpatient Tower Addition in Butler, PA., by making reasonable assumptions and simplifications of the gravity and lateral systems. Wind, seismic and gravity loads have been determined and compared with existing Known values. Wind and seismic forces have not been used to calculate beam or column sizes.

The new addition is approximately 206,000 square feet and 134 feet tall from its lowest elevation to the highest point on the roof. Typical floor levels are 14'-8". Two bottom levels are almost completely underground with the main entrance and lobby area located on the second level on the plan north side and at ground level.

Drilled caissons were used for the foundation system. Grade beams between the caissons on the below grade level areas transfer wall loads to the foundation system and provide interior perimeter walls for the lower levels. The superstructure is composed of steel W-shape members with a steel HSS lateral bracing system. Almost all member connections are shear connections with the exception of a few moment connections at cantilevering beams.

A design load summary was compiled comparing ASCE 7-07/IBC 1607.9 values, with the design architects (HGA), and my determined values. The data on the construction documents for wind and seismic loads were also compared with what were determined using current code sources.

Spot checks were performed on a beam and a non-lateral column element to determine if analysis assumptions and calculations matched relatively closely with the design professional's. A more in depth analysis will be completed in future technical reports.

Introduction:

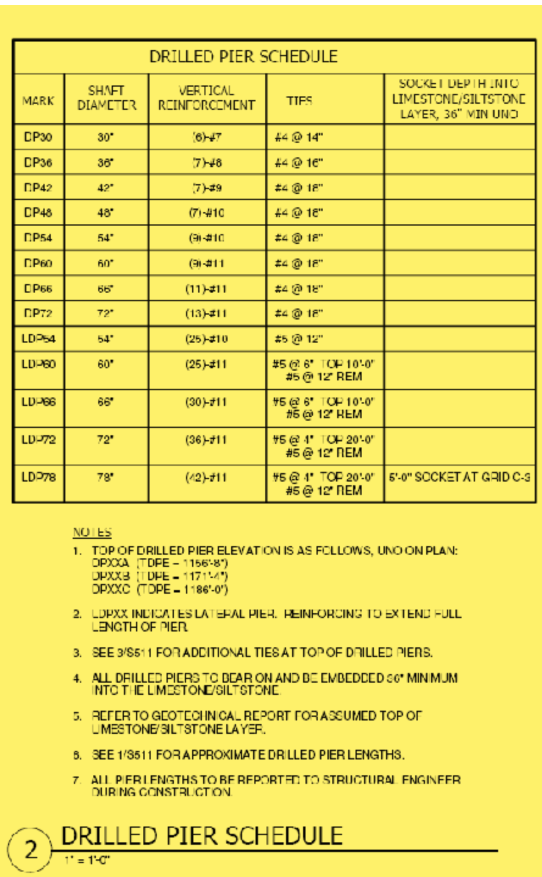
The architectural form of the building on the North side reflects the contour of the topography with the stepped walking path, and the curvature of the roadway with respect to the arcing wall of the North facade. Each level of the new addition has specific functions with the Ground and first floor levels being devoted to emergency generators, elevator pits, mechanical, electrical, boiler, chiller and storage rooms as well as some staff support areas. One quarter of the second floor area is given to training rooms, while approximately another quarter is seating / waiting areas; and the balance is given to an auditorium, chapel, physician lounges, a boardroom and conference rooms. Third floor space is devoted to the Ambulatory Care Unit, operating rooms and outpatient surgery. Fifth floor space is the Critical Care Unit and its support facilities. Floors six and seven are patient recovery rooms. On the top level of the structure is the penthouse level which houses the air handling units and mechanicals.

The design intent for the addition and renovations was to construct the building as economically feasible as possible and complete the renovations with as little disturbance to existing facilities and in the shortest time frame achievable. The need for this design intent was because of the need for deep drilled piers for the foundation system which took more time from the construction timeframe; therefore, construction time needed to be kept at a minimum and a cast in place concrete frame and flat plate system were eliminated as a construction type. A structural steel frame with composite beams and floor slabs with metal lateral bracing was decided upon. This type of structural system is consistent with the original and two existing additions.

For the purpose of this first technical report the focus will be on the first full floor level at/above grade which is the second floor level as this constitutes the bulk of the structure. Wind and seismic loading will be investigated as coming from the plan north/south and east/west directions. The plan north-south direction is the strong axis direction of nearly all columns.

Structural System:

Drilled piers are at the base of the superstructure and provide the majority of the foundation support along with reinforced concrete grade beams. The caissons used for the structural support of the addition ranging in size from 30”-78” are drilled and socketed three feet into sandstone/siltstone at varying depths to provide resistance and support for the differing amounts of loads at each.



DRILLED PIER SCHEDULE

MARK	SHAFT DIAMETER	VERTICAL REINFORCEMENT	TIES	SOCKET DEPTH INTO LIMESTONE/SILTSTONE LAYER, 36" MIN UNO
DP30	30"	(6)-#7	#4 @ 14"	
DP36	36"	(7)-#8	#4 @ 16"	
DP42	42"	(7)-#9	#4 @ 18"	
DP48	48"	(7)-#10	#4 @ 18"	
DP54	54"	(9)-#10	#4 @ 18"	
DP60	60"	(9)-#11	#4 @ 18"	
DP66	66"	(11)-#11	#4 @ 18"	
DP72	72"	(13)-#11	#4 @ 18"	
LDP54	54"	(25)-#10	#5 @ 12"	
LDP60	60"	(25)-#11	#5 @ 6" TOP 10'-0" #5 @ 12" REM	
LDP66	66"	(30)-#11	#5 @ 6" TOP 10'-0" #5 @ 12" REM	
LDP72	72"	(36)-#11	#5 @ 4" TOP 20'-0" #5 @ 12" REM	
LDP78	78"	(42)-#11	#5 @ 4" TOP 20'-0" #5 @ 12" REM	5'-0" SOCKET AT GRID C-3

NOTES
1. TOP OF DRILLED PIER ELEVATION IS AS FOLLOWS, UNO ON PLAN:
DPXXA (TDPE – 1156'-8")
DPXXB (TDPE – 1171'-4")
DPXXC (TDPE – 1186'-0")
2. LDPXX INDICATES LATERAL PIER. REINFORCING TO EXTEND FULL LENGTH OF PIER.
3. SEE 3/S511 FOR ADDITIONAL TIES AT TOP OF DRILLED PIERS.
4. ALL DRILLED PIERS TO BEAR ON AND BE EMBEDDED 36" MINIMUM INTO THE LIMESTONE/SILTSTONE.
5. REFER TO GEOTECHNICAL REPORT FOR ASSUMED TOP OF LIMESTONE/SILTSTONE LAYER.
6. SEE 1/S511 FOR APPROXIMATE DRILLED PIER LENGTHS.
7. ALL PIER LENGTHS TO BE REPORTED TO STRUCTURAL ENGINEER DURING CONSTRUCTION.

2 DRILLED PIER SCHEDULE
1" = 1'-0"

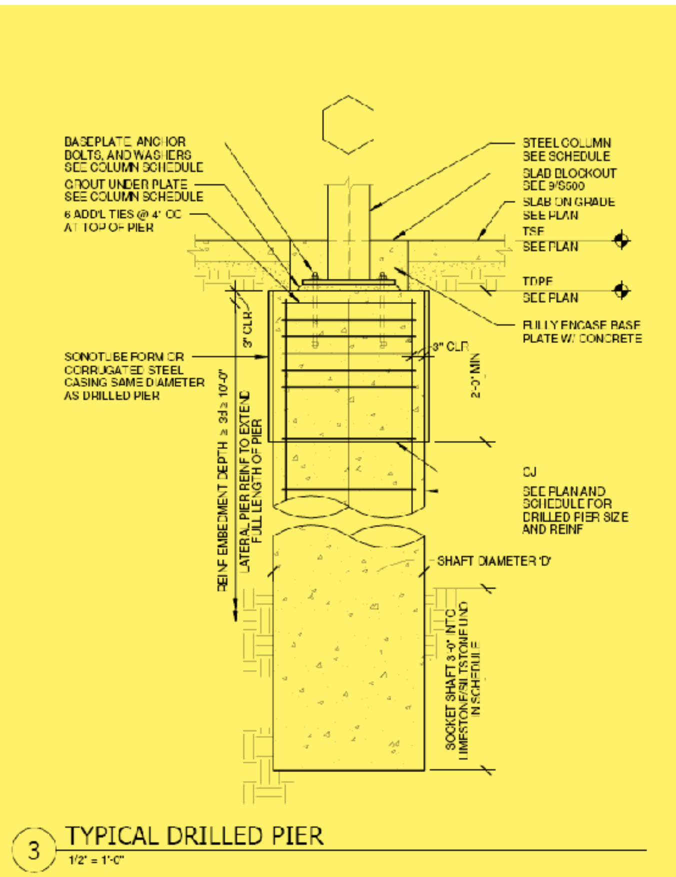


Figure 1.1

Figure 1.2

Piers have been designed for both end bearing and skin friction with an allowable end bearing pressure of 20 TSF and an allowable lateral earth pressure that varies with the depth of the soil strata from a minimum of 3TSf through fill and decomposed rock to a maximum of 12 TSF in the limestone/siltstone layer.

They are comprised of 4000 psi @ 28 days strength concrete, ASTM A615 Grade 60 deformed bars with 12" minimum Class B tension lap splices where required and conform to ACI 318 design code.

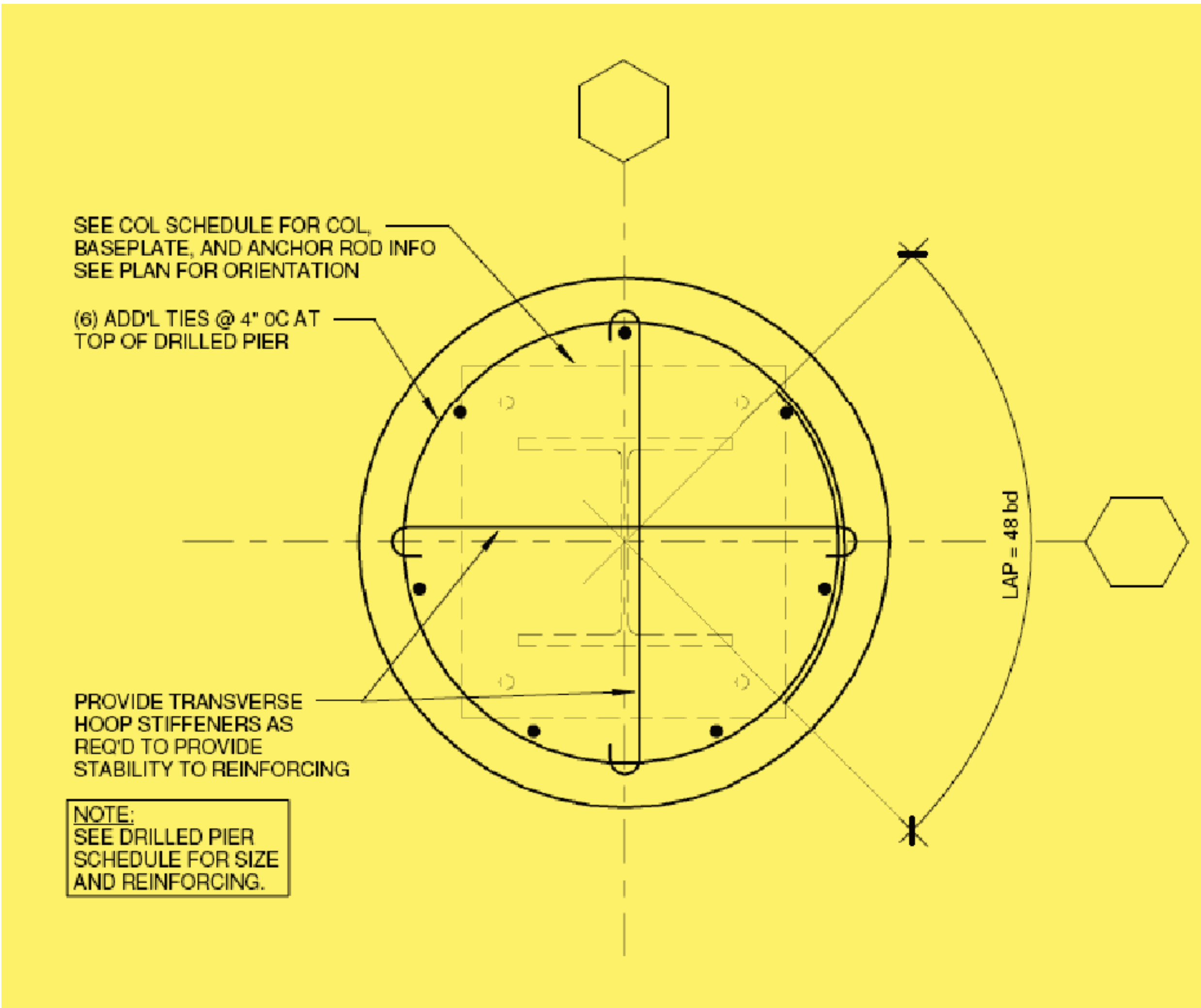


Figure 1.3: Typical drilled pier

The caissons are supporting either reinforced concrete (R.C.) grade beams/walls or the structural steel frame of the superstructure. The (R.C.) grade beams/walls are up to the second level and enclose the area around the stepped down auditorium floor and slab on grade areas. All other building sections are supported, framed and resisting loads using wide flange and HSS shapes.

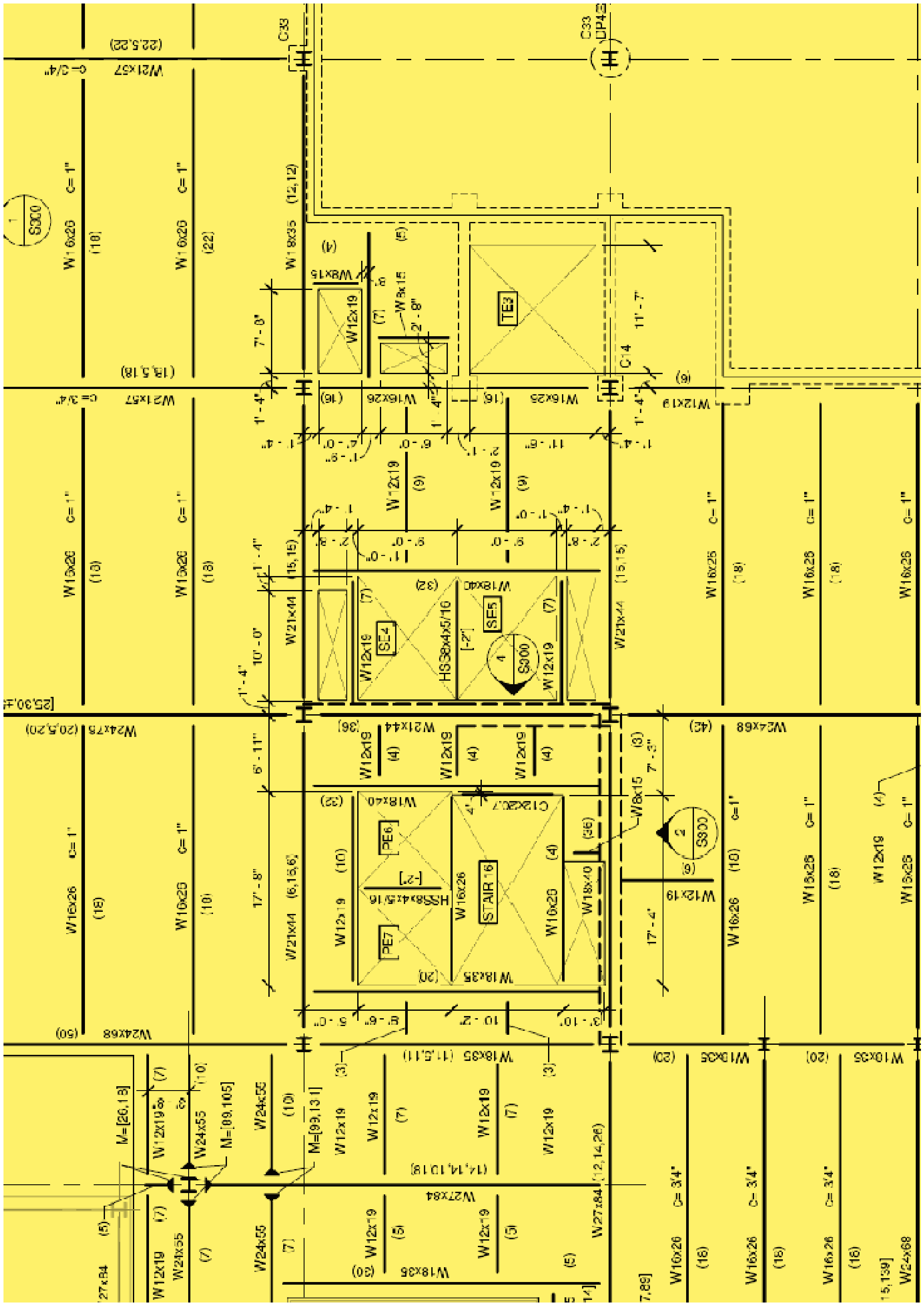
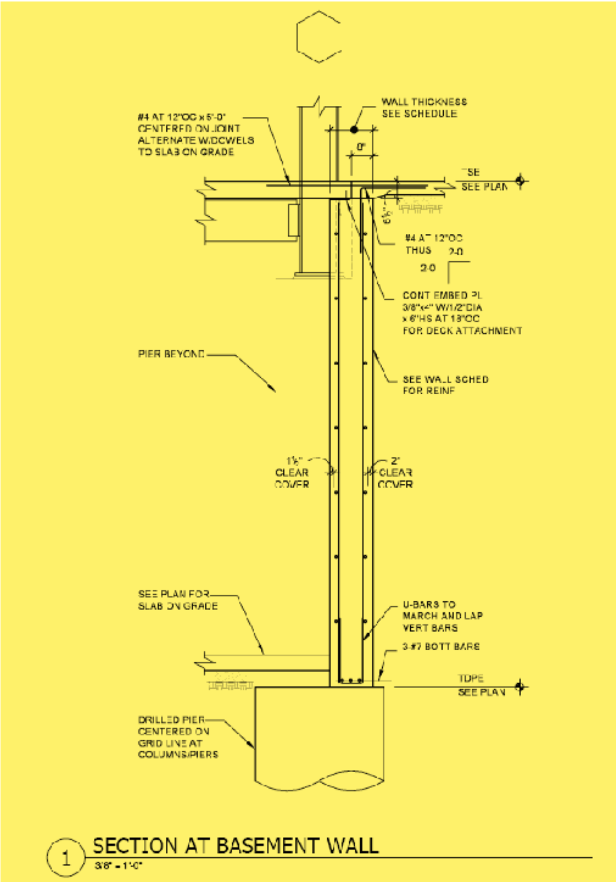


Figure 1.4: Partial 2nd floor framing plan, typical bay and beam sizes.



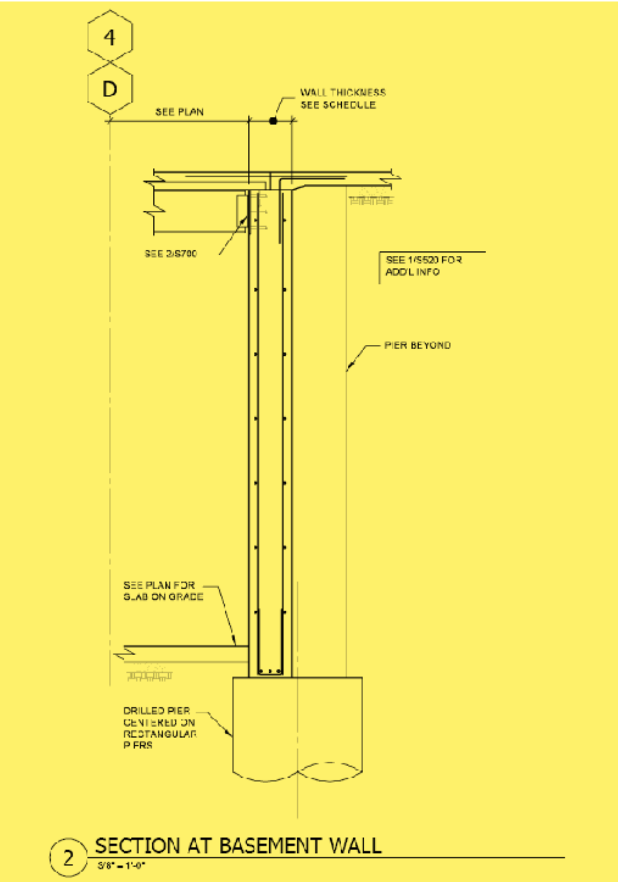


Figure 1.5 Figure 1.6

The structural system for the building addition consists primarily of wide flange members for the beams and columns above ground level. Columns are mostly W14's ranging from 43 - 176 lb/ft. Beams have a wide range of sizes depending their loading and span with the two most common sizes being W18x40 and W16x26. (See Figure 1.4). These elements are ASTM A992 with yield strength of 50 ksi. A typical bay size is either 30' x 30' or 28' x 30' and a floor to floor height of 14'-8".

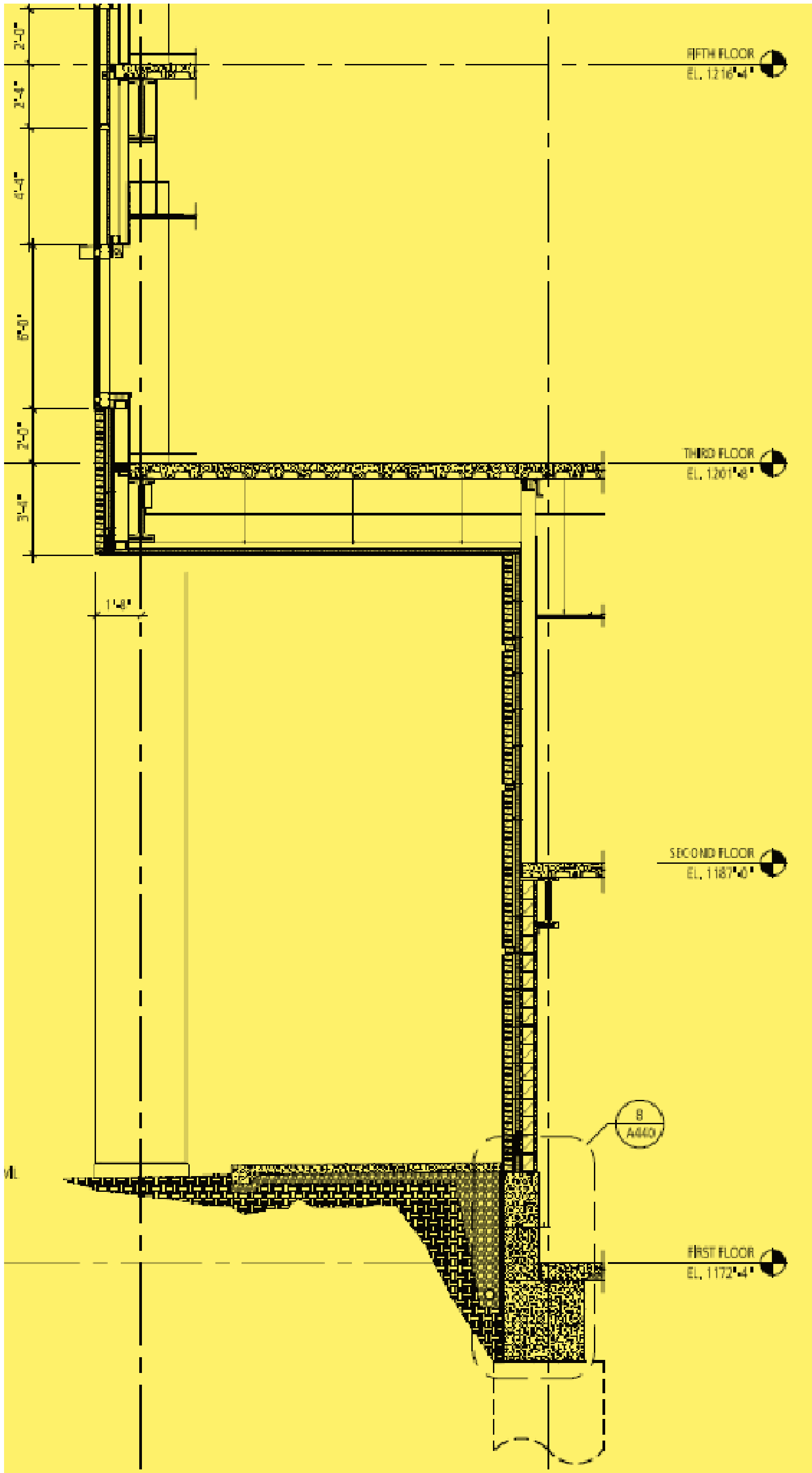


Figure 1.7:

Wall section thru

North Elevation

Floor systems are comprised of wide flange girders and beams supporting composite metal decking and composite concrete floor slabs. Floor thicknesses are 6-1/2" total with 3-1/2", 3500psi @ 28 day strength lightweight concrete and 5" shear stud length and either 6x6 WWF or #4 and #5 deformed bars @12" O.C. generally. All of the composite floor slab thicknesses are the same and all are supported by wide flanges that have been cambered to control deflections both during construction and while in service. All composite beams and composite decks are designed as unshored UNO as per construction document specifications. The size of pours between construction joints for concrete on metal deck is limited to 10000square feet with a maximum dimension of 100 feet.

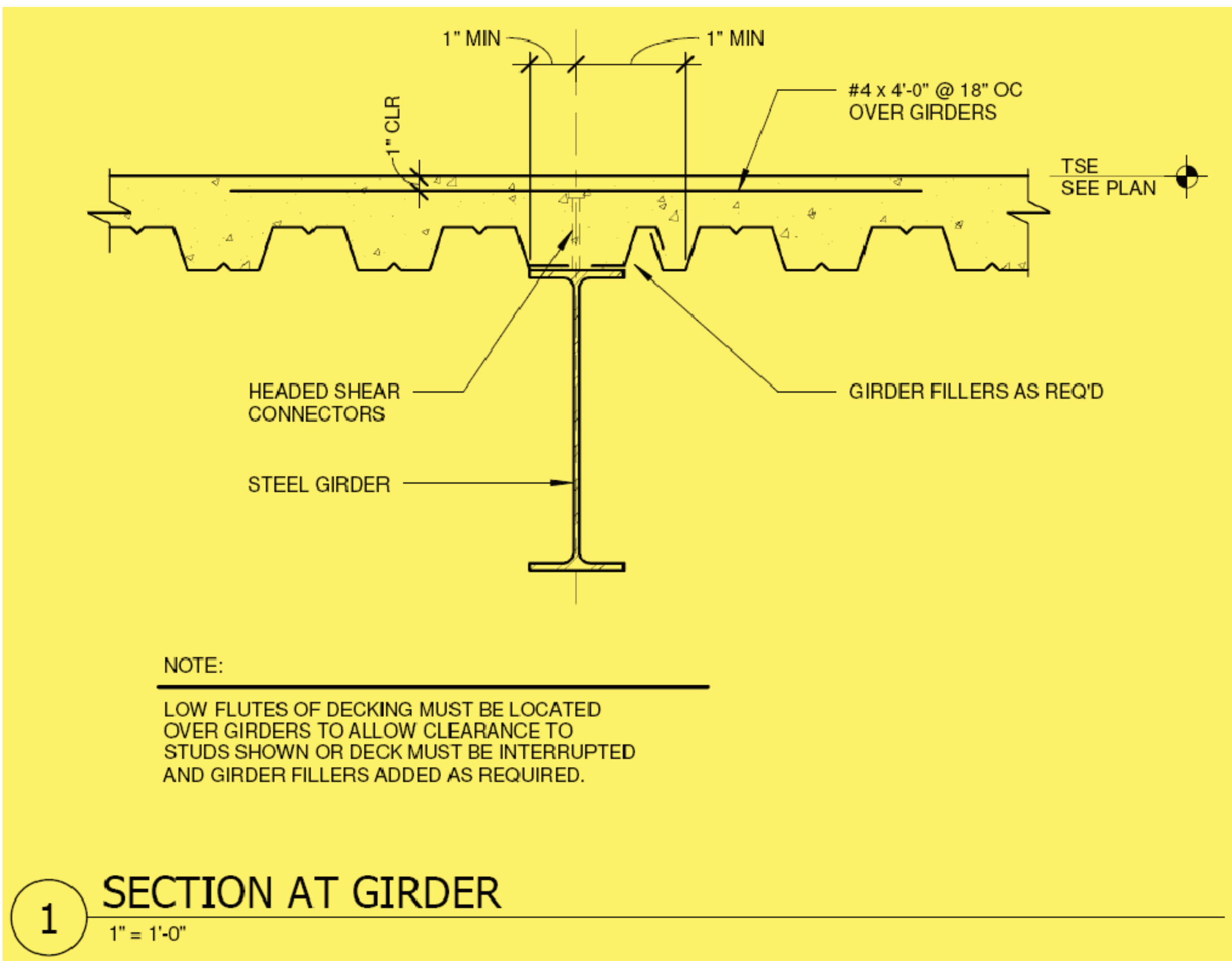


Figure 1.8

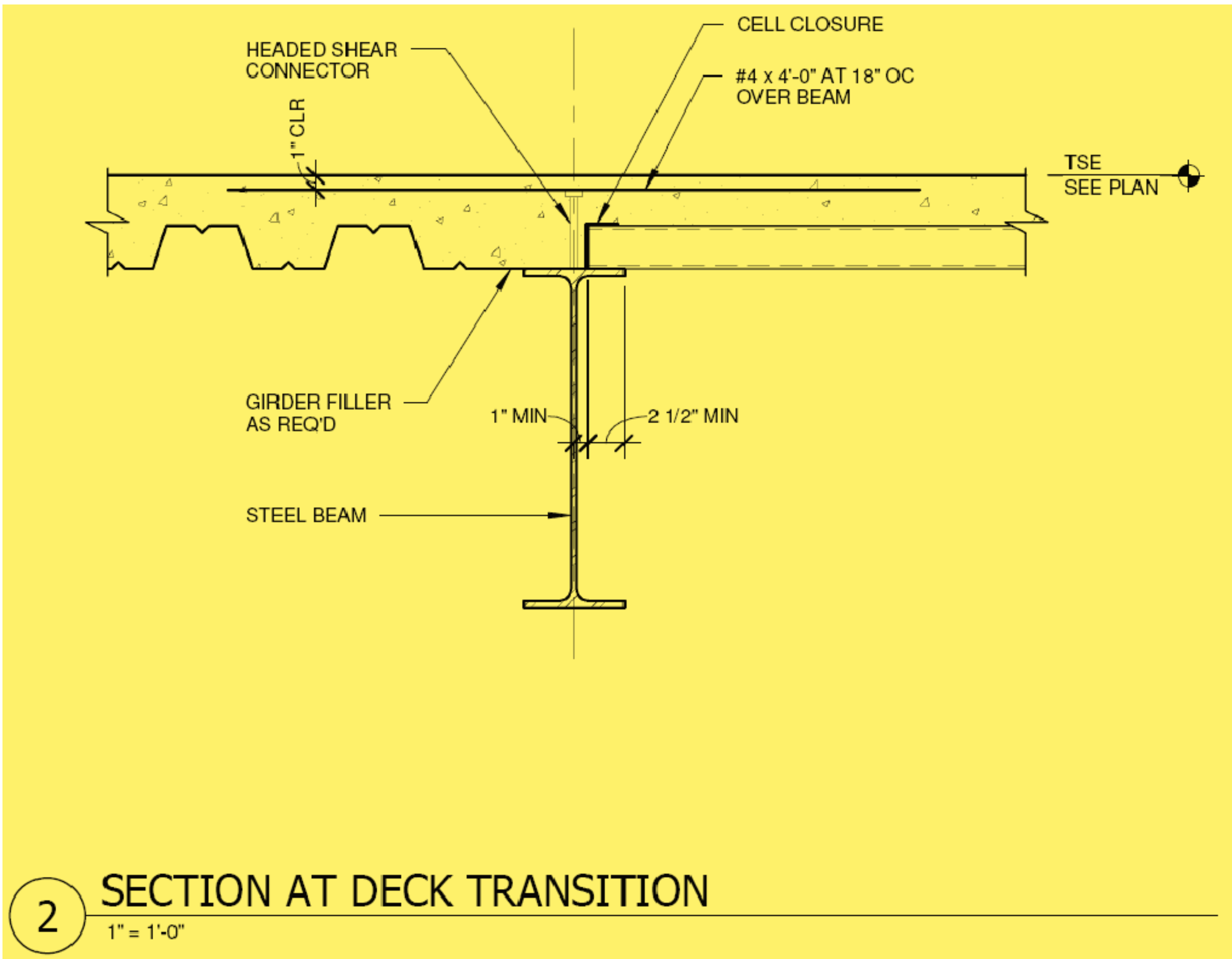


Figure 1.9

The main lateral force resisting system is composed typically of K frame braces made from HSS sections in both directions. HSS sections are mostly 10"x10-1/2" on the first thru the third floor levels and 8"x8-5/16" on the higher levels. The penthouse levels housing the mechanical units have HSS 6"x6-5/16" lateral braces. The HSS sections are ASTM A500 Grade B with a yield strength of 46 ksi.

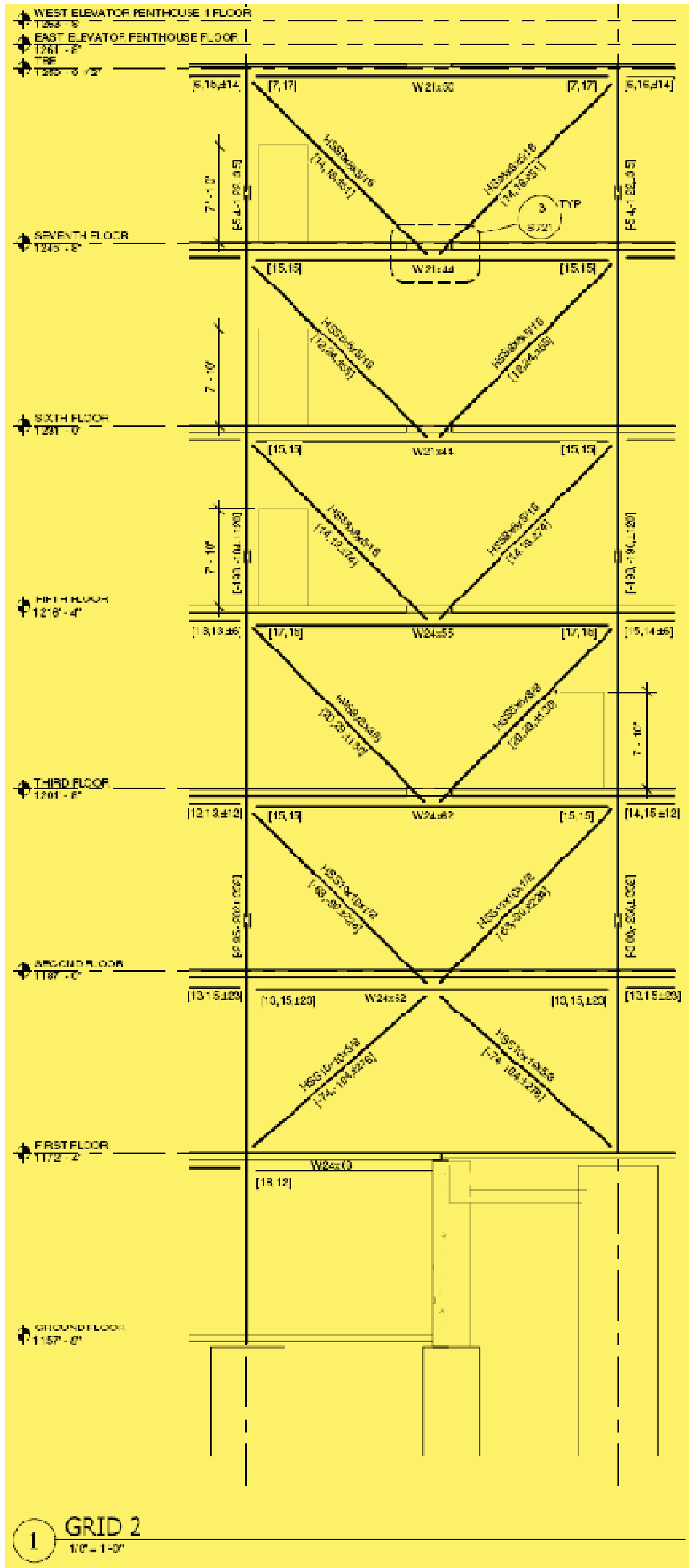


Figure 1.10:

Bracing elevation

Structural system distribution of loads and load paths:

Gravity loads created by dead and live floor, live roof and snow loads are transmitted first through the structural system by the one way composite floor or roof slabs to the supporting beams which transfer the load to the girders through bolted shear connections. The load is taken from the girder into the columns through another bolted shear connection and transferred from the columns directly vertical into the reinforced concrete caissons to the sandstone/siltstone strata layer. The few moment frame connections that are located at each floor level are at the edges of the floor slabs where the addition abuts the existing structure. In these areas moment connections are on both sides of the columns where the girders/beams are cantilevered and not vertically supported on the end next to the existing building.

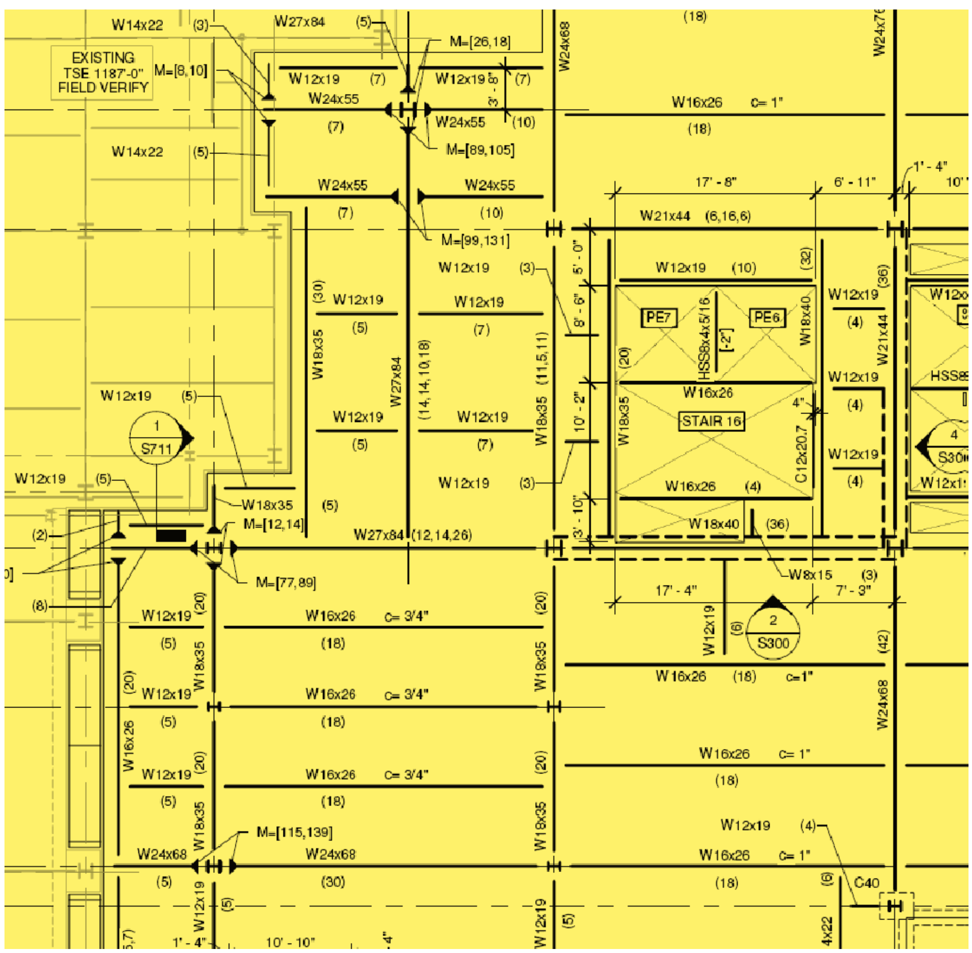


Figure 1.11:

Moment Frame

Locations

Lateral loads caused by wind pressures are calculated using ASCE 7-05 and are resisted by the structure through the use of diagonal Chevron bracing (see Figure 1.10 & 1.12) located at every floor level in both directions.

The differing pressures on the exterior façade are converted to forces per square foot of wall area and are distributed to each floor level by tributary areas. From there the floors are assumed to act as rigid diaphragms and distribute each floor load to the braced frames at each level according to their relative stiffnesses. These loads are then transferred axially through the HSS members and into their corresponding beams. At the beam/girder to HSS connection there is a concentric compressive force from one brace and a concentric tension force from the other brace which cancel each other's vertical components being transferred into the beam/girder; therefore, the force transferred into the member is axial. See figure 1.13 for brace to beam connection and figures 1.14 & 1.15 for how the load is distributed from the initial lateral force to the individual bracing and framing elements. Note how the singe lateral force at the top of the structure creates the same compressive/tensile force from top to bottom in all bracing members, but the load being transferred axially into the columns increases linearly by the force in the top column until the frame reaches its foundation support. From there the load is transferred to the ground.

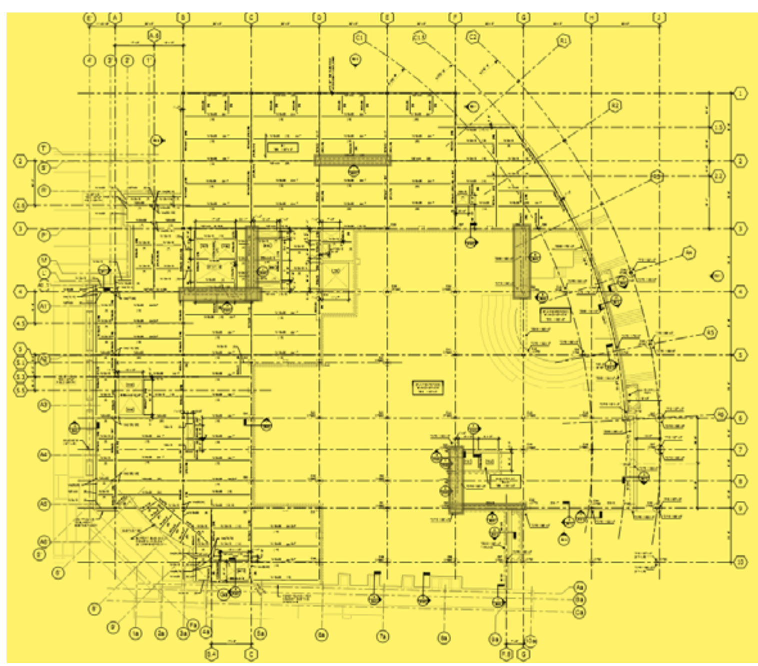


Figure 1.12: Lateral diagonal bracing locations

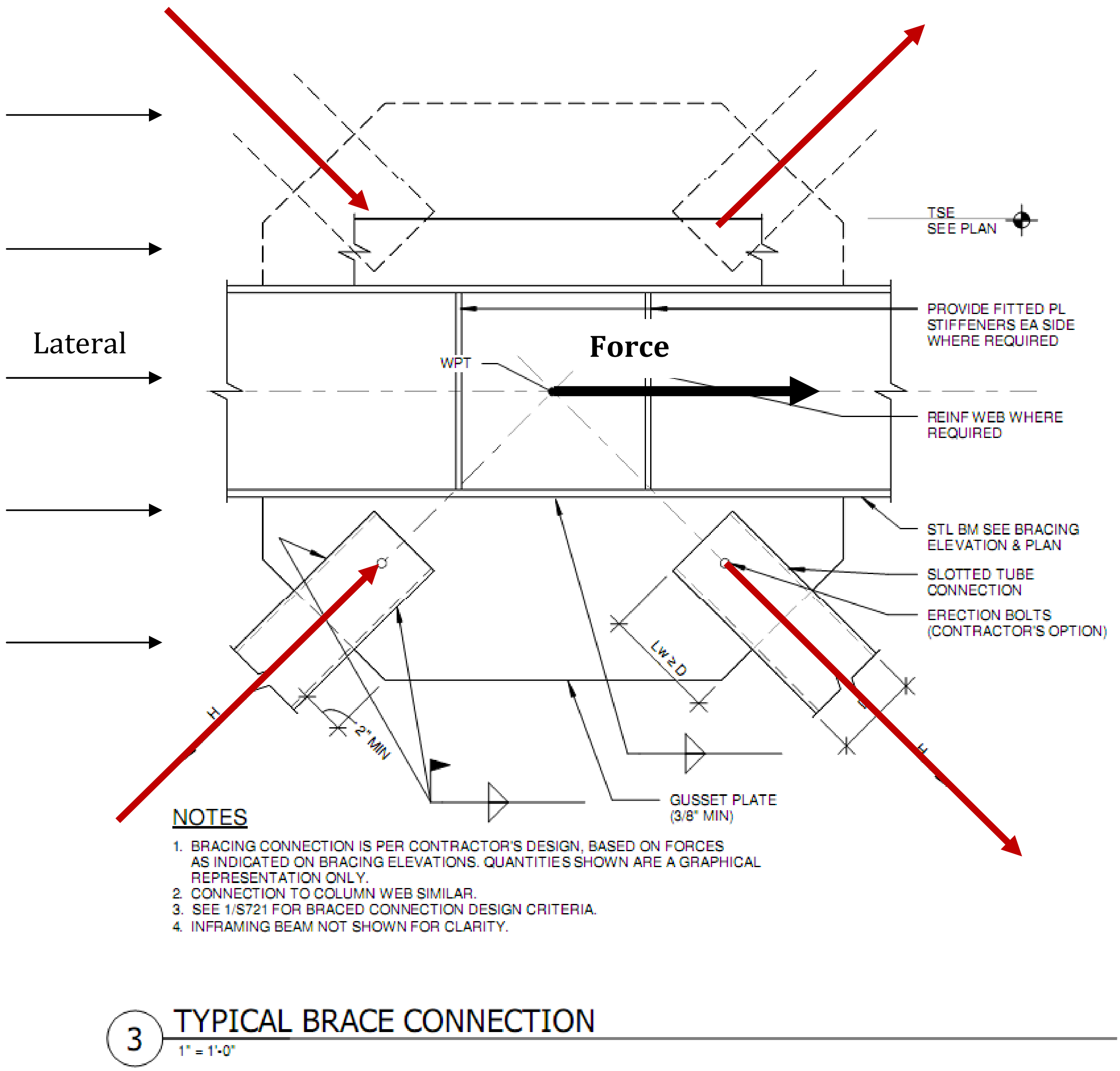


Figure 1.13: Resultant force from wind into beam/girder

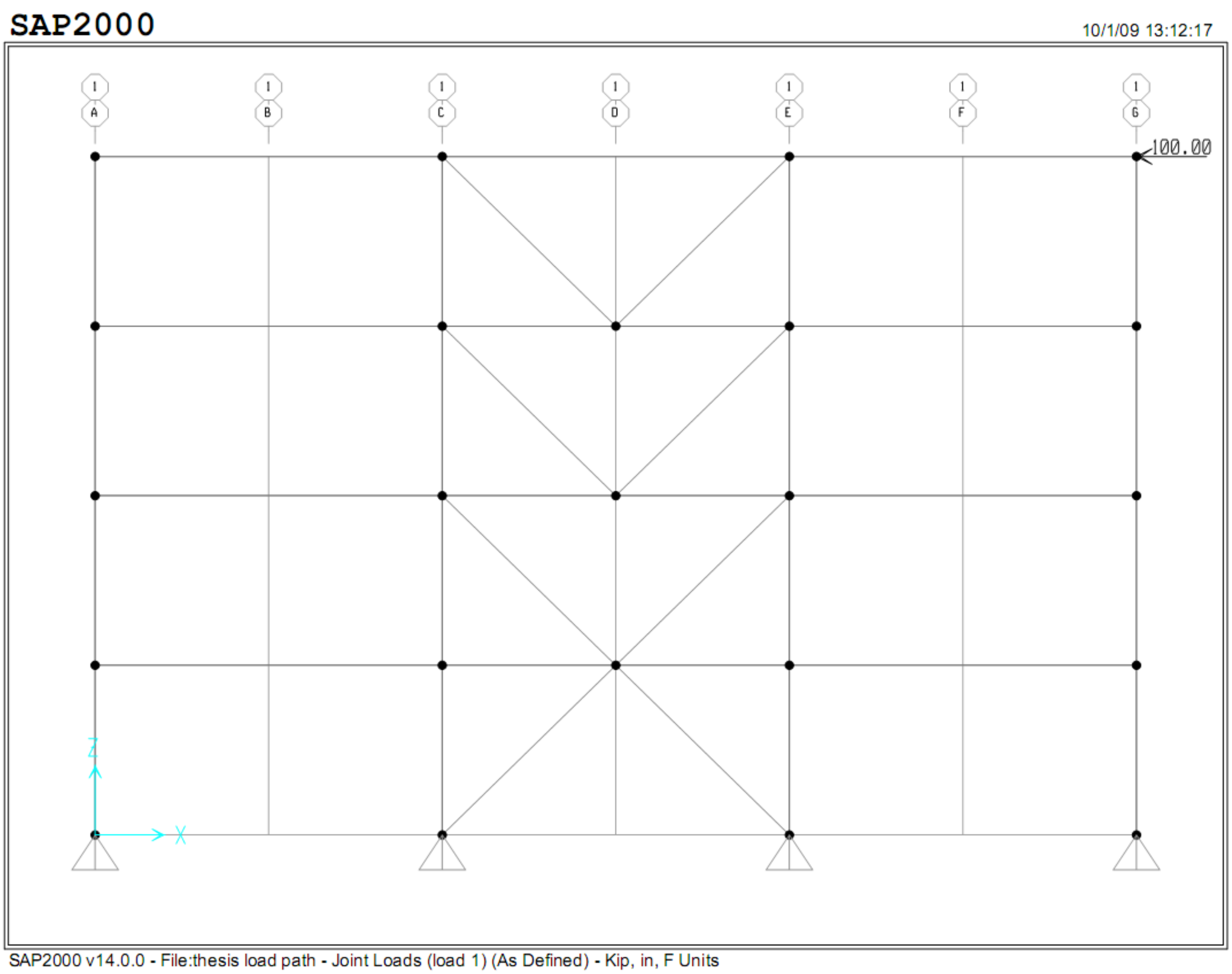


Figure 1.14: A single lateral force to show load path compounding effects for simplicity

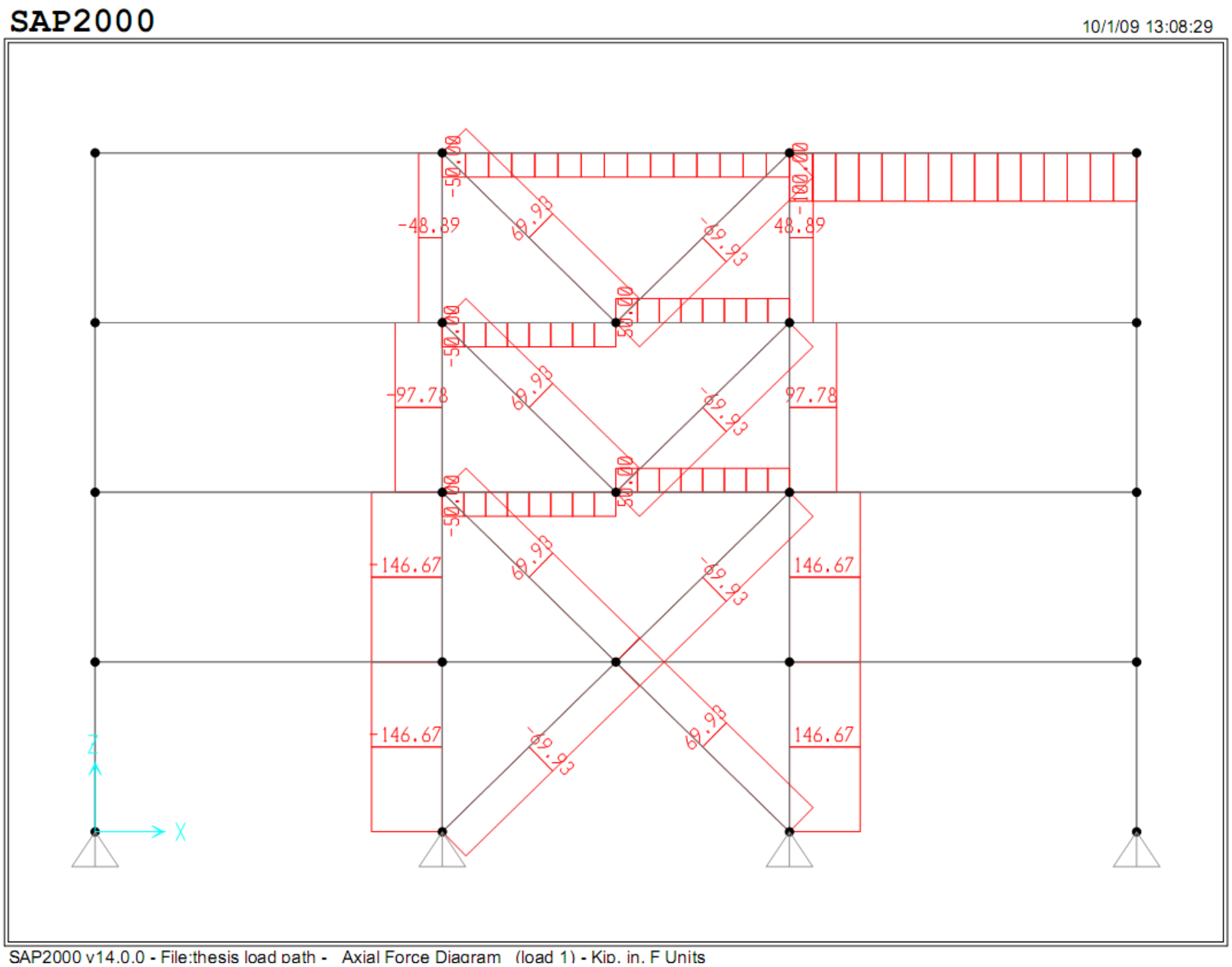


Figure 1.15: lateral force distribution to braced frame and lateral load columns

Design Standards & Codes:

2006 IBC
2000 NFPA 101
2006 Guidelines for Design & Construction of Health Care Facilities
1998 Pennsylvania Department of Health Rules and Regulations for Hospitals
ASCE 7-05: for wind, seismic, snow and gravity loads
ACI 318-05: for concrete construction
AISC Thirteenth Edition: for steel members
Floor deflections: L/240 total, L/360 for live load
Lateral Deflection: for braced frame construction side sway is prohibited, there is however a 6" expansion joint between the new and existing structures for thermal movements.

Design Load Summary:

Gravity Loads					
Description/location	DL/ LL	ASCE 7-05/ IBC 1607.9 values	HGA's values	Reduction available/used	Design value
Concrete floors	DL	90-115pcf	115pcf	NO/NO	115pcf
MEP/partitions/finishes	SDL	20-25psf		NO/NO	35psf
1st floor mechanical	LL		125psf	YES/NO	125psf
2nd floor/ lobby	LL	100psf	100psf	YES/NO	100psf
Hospital floors	LL	40-80psf	80psf	YES/YES	80psf
Stairs & exits	LL	100psf	100psf	NO/NO	100psf
5th floor roof	LL		115psf	NO/NO	115psf
Mech. Penthouse floor	LL		125psf	NO/NO	125psf
Elevator Machine room floor	LL		125psf	YES/NO	
Roof top equipment areas	LL		125psf (or actual equipment wt.)	NO/NO	125psf
Balconies	LL	100psf	100psf	YES/YES	psf
*Snow	LL	24-30psf	24-30psf	NO/NO	24-30psf

* See Appendix C for calculations

Table 1.1

Wind Loads are determined using ASCE 7-05 Section 6.5, which is Main Wind Force Resisting System (MWFRS) method 2- analytical procedure. See Table 1B for design factor values needed in calculations. All values, factors and equations are derived from section 6. To Determine the Gust Effect Factor (G) the structure had to be determined as a rigid structure. To make this assumption 100/h has to be ≤ 1. Making the assumption that h was just under 100 feet based on the fact that the ground and second floors are minimal compared to the rest of the structure and there is only one wall face exposed; therefore the bulk of the structure completely exposed above ground would meet the requirement.
See Appendix A of structure under construction for clarity, lowest level faces west.
See Appendix B for all calculations.

Wind Load Data for Calculations

North-South direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	1.0	6.5.7 (Figure 6-4)
Gust effect factor	G	0.857	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	26.65lb/ft^2	6.5.10
Design wind load	F	determined	

Table 1.2

WIND LOAD

BASIC WIND SPEED (3 SECOND GUST)	90 MPH
WIND IMPORTANCE FACTOR	1.15
WIND EXPOSURE CATEGORY	C
MEAN ROOF HEIGHT	122 FT
INTERNAL PRESSURE COEFFICIENT	±0.18
TOPOGRAPHIC FACTOR, Kzt	1.62 MAX AT BASE 1.09 MIN AT MEAN ROOF HEIGHT

Figure 1.16: Wind load data from construction documents

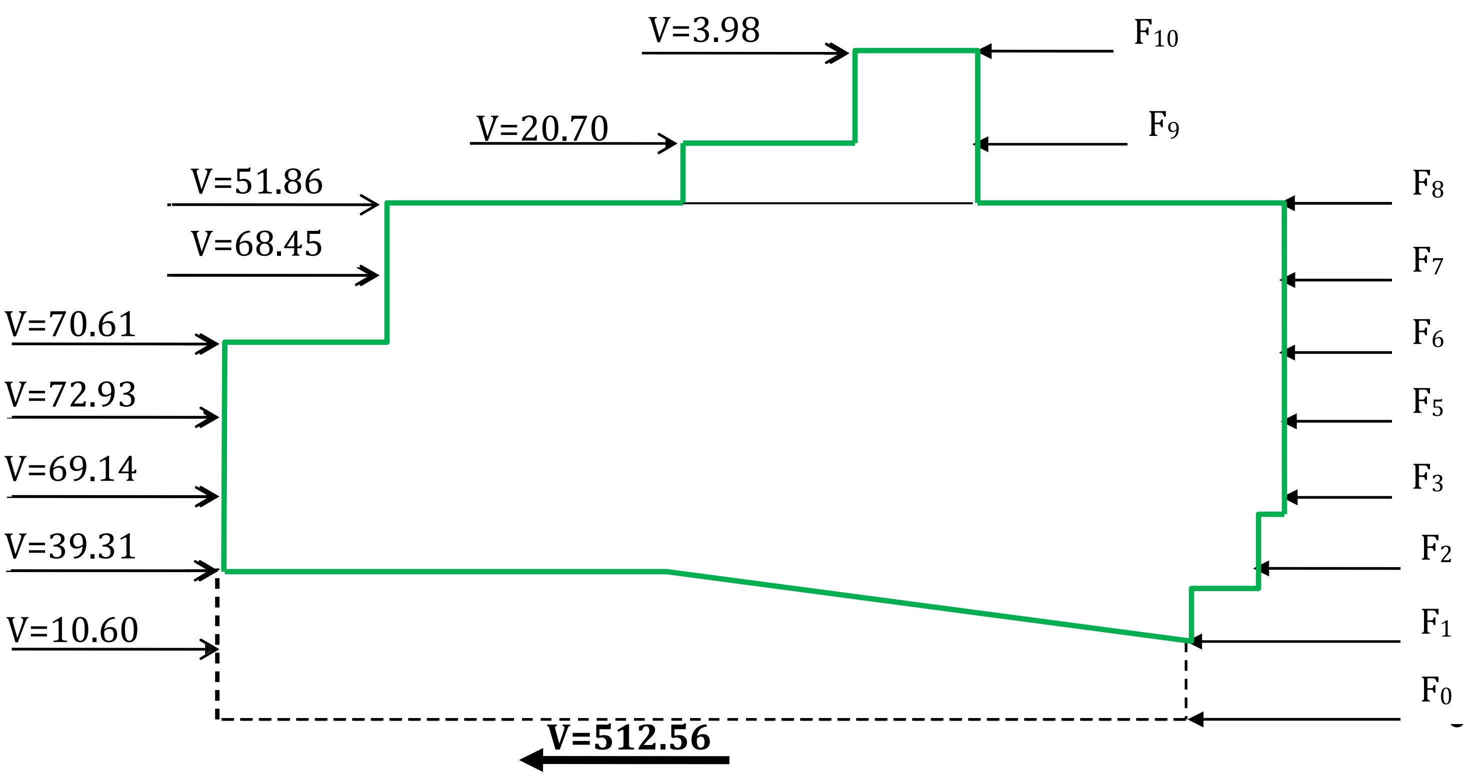


Figure 1.17: Floor level forces and shears for North-South wind

Level	**Height** (ft)	**Force** (lbs/ft ^{2})	**Pressure** (P)kips	**Shear** (V) kips
		Windward + leeward		
0- Ground	0	16.61	0	
1	14'-8"	16.61	10.60	10.60
2	29'-4"	18.35	39.31	39.31
3	44'-0"	19.53	69.14	69.14
5	58'-8"	20.43	72.93	72.93
6	73'-4"	21.24	70.61	70.61
7	88'-0"	21.94	68.45	68.45
8-Roof	102'-8"	22.40	51.86	51.86
9- P.H. 1	122'-0"	23.07	20.70	20.70
10- P.H. 2	135'- 0"	23.53	3.98	3.98
			Base Shear =	**407.58**

Table 1.3: * Note: Assuming leeward pressures are not added to the windward at all levels since some levels are underground or are adjacent to the existing structure. Levels 5 and above are exposed on the east side.

Wind Load Data for Calculations

East-West direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor (Occupancy category IV)	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	1.0	6.5.7.1 (Figure 6-4)
Gust effect factor	G	0.856	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	26.65lb/ft^2	6.5.10
Design wind load	F	determined	

Table 1.4

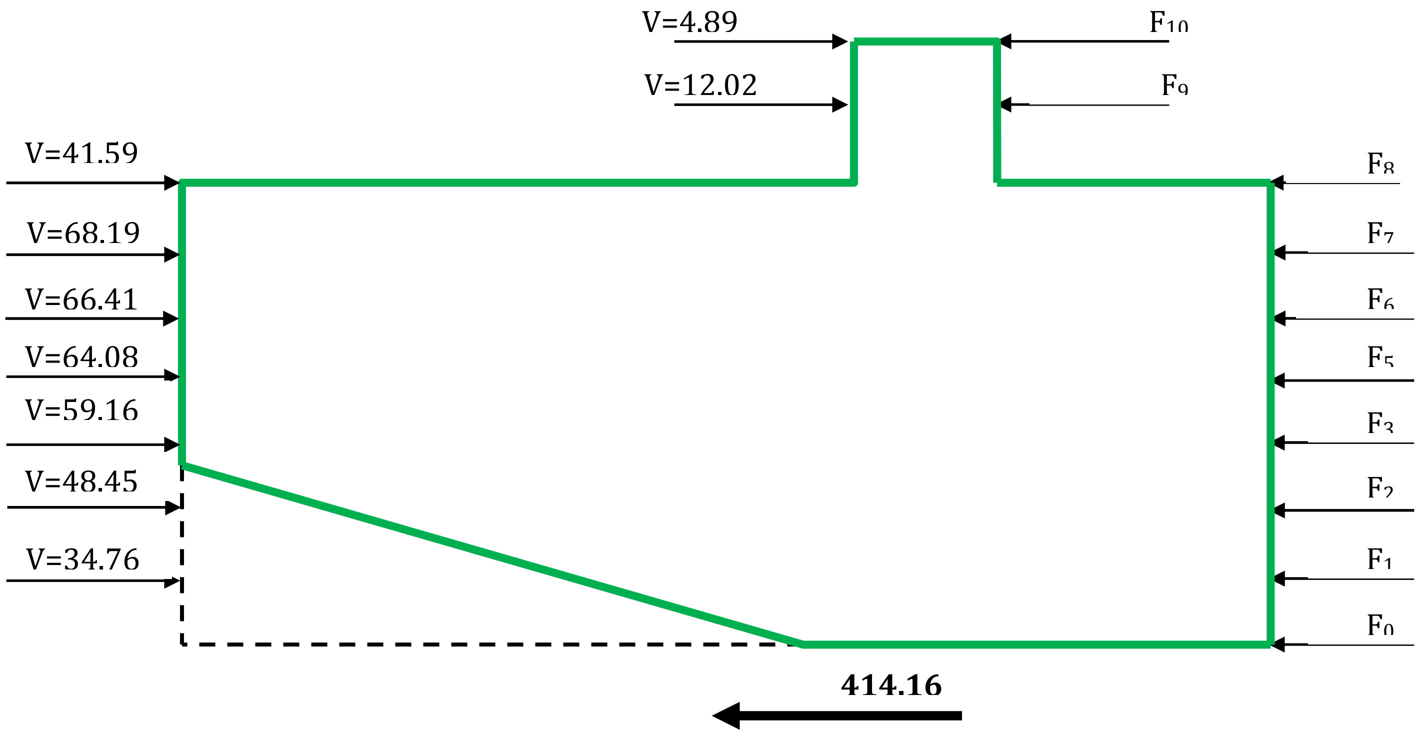


Figure 1.18: Floor level forces and shears for East-West wind

Level	**Height** (ft)	**Force** (lbs/ft ^{2})	**Pressure** (P)kips	**Shear** (V) kips
		Windward + leeward		
0- Ground	0	16.60	14.61	
1	14’-8”	16.60	34.76	34.76
2	29’-4”	18.33	48.45	48.45
3	44’-0”	19.51	59.16	59.16
5	58’-8”	20.41	64.08	64.08
6	73’-4”	21.22	66.41	66.41
7	88’-0”	21.92	68.19	68.19
8-Roof	102’-8”	22.38	41.59	41.59
9- P.H. 1	122’-0”	23.05	12.02	12.02
10- P.H. 2	135’- 0”	23.51	4.89	4.89
			Base Shear =	**414.16**

Table 1.5: * Note: Assuming leeward pressures are not added to the windward at all levels since some levels are underground or are adjacent to the existing structure. Levels 5 and above are exposed on the east side.

Snow loads are determined using ASCE 7-05 Chapter 7. The design values in sections 7.1-7.3 all agree with HGA's values (see Appendix C notes on snow loads.) A minimum roof design load of 30psf will be used for calculations.

SNOW LOAD

GROUND SNOW LOAD, Pg	25 PSF
FLAT ROOF SNOW LOAD, Pf	24 PSF
MINIMUM ROOF DESIGN LOAD	30 PSF
SNOW IMPORTANCE FACTOR	1.2
SNOW EXPOSURE FACTOR, Ce	1.0
THERMAL FACTOR, Ct (BUILDING)	1.0
THERMAL FACTOR, Ct (CANOPIES)	1.2

Figure 1.19: Construction document values

Seismic design criteria are based off of ASCE 7-05 Chapters 11, 12, 14 & 22 for seismic design. Using Table 12.2-1Design Coefficients and Factors for Seismic Force-Resisting Systems, category B: Building Frame Systems #4 (ordinary steel concentrically braced frames), several factors can be obtained. Response modification coefficient (R^a)=3.25, system overstrength factor (Ω_o^g)=2, deflection amplification factor (C_d^b)=3.25, seismic design category C, no limitation to building height.

ASCE 7-05 Section 14.1is where the detailing requirements are specified. The designer's data is listed below. All of the design criteria compare to my calculations except for the discrepancy in the spectral response acceleration (S_1) value. Designed is 0.0055, determined is 0.046 from ASCE 7-05 Figure 22-2.

SEISMIC DESIGN DATA
SPECTRAL RESPONSE ACCELERATION, Ss 0.0127
SPECTRAL RESPONSE ACCELERATION, S1 .... 0.0055
SITE CLASS C
SEISMIC IMPORTANCE FACTOR 1.5
SEISMIC DESIGN CATEGORY (SDC) .... A

Figure 1.20: Construction document data for seismic

The effective seismic weight (W_T) is determined using information from section 12.7.2., and totaled using an excel spreadsheet found in Appendix D.

V= base shear = Cs*W_T
Cs= 0.0456
W_T= 22 635.6 kips
V= 1032 kips
See Appendix D for data and calculations.

Once the base shear has been calculated then this load is distributed to each individual floor level using relative floor stiffnesses which are based off of the proportions of floor areas and ½ column weights of the floor above and below. The square footage of the roof area is broken up and divided out over levels 8, 7, 5 and 3, which are the levels that contain roof areas.
See figures 1.21 &1.22.

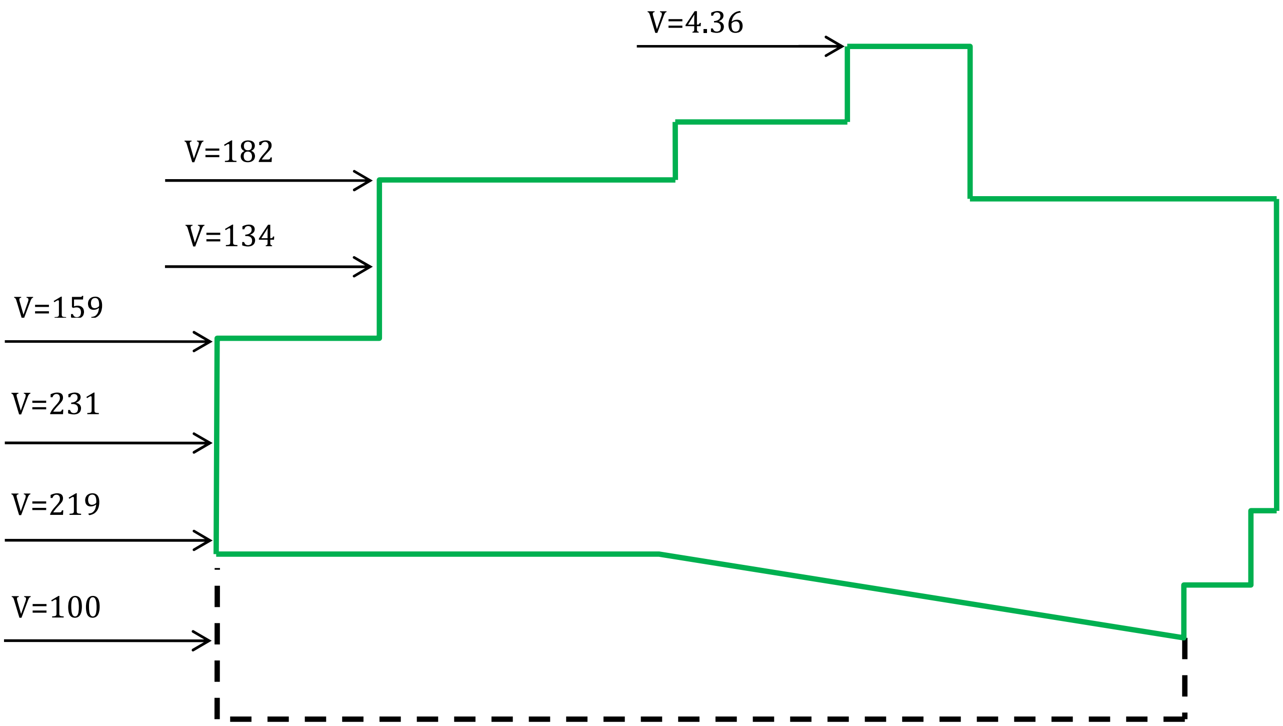


Figure 1.21: North-South Elevation

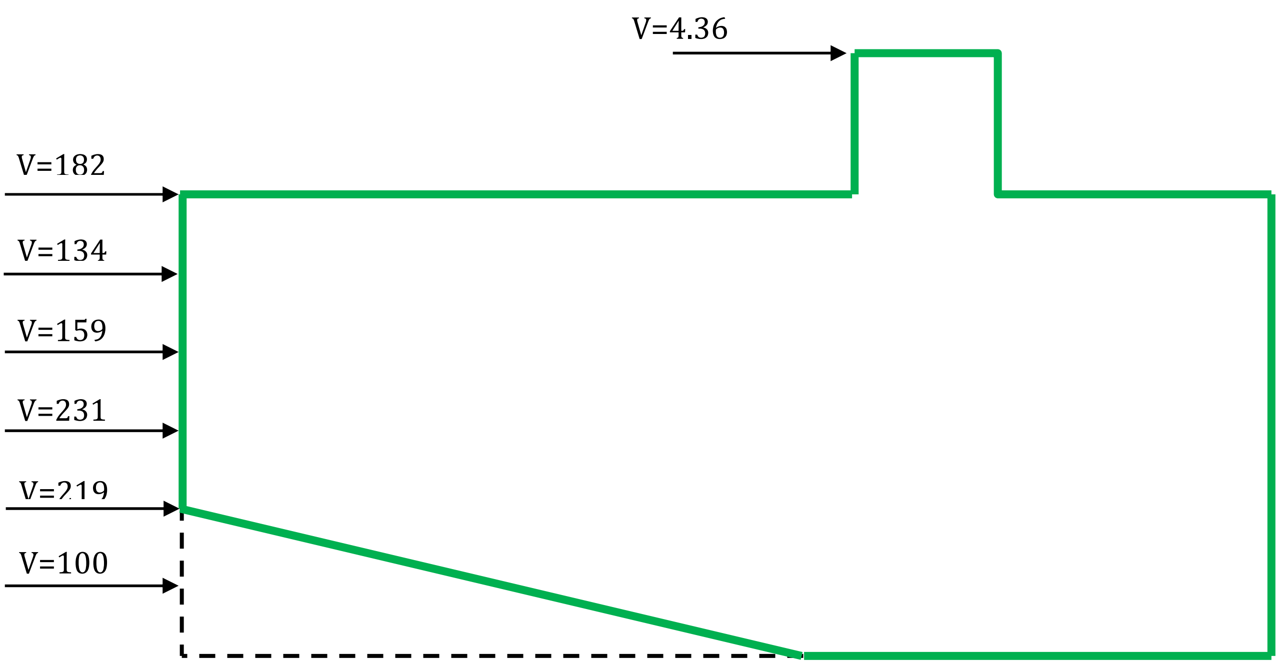


Figure 1.22: East-West Elevation

Total Dead Load for Seismic Calculation

W_T

Load type

Floor Level	square footage	wall square footage	Concrete deck psf 55	Superimposed MEP/Partitions 35	Columns kips	Beams lb/ft² 9.2	equipment psf 5	roof psf 42	exterior walls psf/wall 28.6	Floor weight Totals for relative stiffness
Ground	8240									
Level 1	20405	170	1122.28	714.18	58.39	187.73	102.03	0	4.86	2184.6
Level 2	45545	458	2504.98	1594.08	50.59	419.01	227.73	0	13.10	4796.4
Level 3	42165	458	2319.08	1475.78	68.99	387.92	210.83	0	13.10	4462.6
Level 5	31525	458	1733.88	1103.38	41.83	290.03	157.63	0	13.10	3326.7
Level 6	27720	678	1524.60	970.20	39.50	255.02	138.60	0	19.39	2927.9
Level 7	27760	678	1526.80	971.60	29.86	255.39	138.80	0	19.39	2922.5
Level 8 (roof)	45545							1912.89		1912.9
TOTALS			10731.6	6829.2	289.159	1795.104	975.6	1912.89	82.94	

W_T = 22616.5 kips

C_S = 0.0456

$V = C_S * W_T$ = 1031.31

Table 1.6:

Column Load Summary

W Shapes 14.667	**weight** lbs	**Levels** 1		2		3		5		6		7	
8x40	40	1	0.587	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
12x40	40	1	0.587	2	1.173	2	1.173	2	1.173	1	0.587	0	0.000
12x45	45	0	0.000	2	1.320	2	1.320	0	0.000	0	0.000	0	0.000
12x50	50	1	0.733	2	1.467	2	1.467	1	0.733	1	0.733	0	0.000
12x53	53	3	2.332	3	2.332	3	2.332	0	0.000	0	0.000	0	0.000
12x58	58	0	0.000	0	0.000	0	0.000	2	1.701	2	1.701	0	0.000
12x65	65	2	1.907	1	0.953	1	0.953	0	0.000	0	0.000	0	0.000
12x72	72	1	1.056	1	1.056	1	1.056	0	0.000	0	0.000	0	0.000
12x87	87	0	0.000	1	1.276	1	1.276	0	0.000	0	0.000	0	0.000
12x96	96	2	2.816	1	1.408	1	1.408	0	0.000	0	0.000	0	0.000
14x43	43	0	0.000	0	0.000	0	0.000	1	0.631	0	0.000	9	5.676
14x48	48	0	0.000	0	0.000	0	0.000	3	2.112	4	2.816	11	7.744
14x53	53	0	0.000	0	0.000	0	0.000	3	2.332	4	3.109	5	3.887
14x61	61	2	1.789	0	0.000	3	2.684	17	15.210	17	15.210	8	7.157
14x68	68	2	1.995	0	0.000	2	1.995	10	9.974	9	8.976	3	2.992
14x74	74	4	4.341	0	0.000	4	4.341	1	1.085	1	1.085	0	0.000
14x82	82	7	8.419	0	0.000	9	10.824	0	0.000	0	0.000	2	2.405
14x90	90	7	9.240	18	23.761	18	23.761	4	5.280	4	5.280	0	0.000
14x99	99	0	0.000	1	1.452	1	1.452	0	0.000	0	0.000	0	0.000
14x109	109	4	6.395	9	14.388	7	11.191	1	1.599	0	0.000	0	0.000
14x120	120	3	5.280	0	0.000	1	1.760	0	0.000	0	0.000	0	0.000
14x132	132	2	3.872	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
14x145	145	1	2.127	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
14x159	159	1	2.332	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000
14x176	176	1	2.581	0	0.000	0	0.000	0	0.000	0	0.000		0.000
			58.389 kips		50.586 kips		68.994 kips		41.830 kips		39.498 kips		29.862 kips
		Total column weight=			289.160 kips								

Table 1.7: Total number of columns & weights for seismic calculations

Spot Checks:

A beam spot check was done on the third floor in a corridor area where the typical beam size is a 30' long W16x26 with 18 shear studs and a 1" camber. The thickness of the concrete floor is 6-1/2" with 3" metal deck and 5" shear studs, f'c=3500psi. The beams are spaced at 10' intervals, the metal deck is perpendicular to the beams, and the studs are assumed to be in the strong position since there is 1.67' between studs. The available moment strength is checked against capacity, the live load deflection is checked against Δ lower bound and dead load deflection. The construction load situation was also checked since the beams and girders were designed as unshored. The load combination used was 1.2D+1.6L and in accordance with LRFD provisions.

Mu: 305k*ft
ΦMn: 385k*ft AISC Table 3-19
Vu: 40.65k
ΦVn: 106k AISC Table 3-2
ΔL_b: $985in^4$
ΔDL: 1.2" with 1" camber
ΔLL: 1" with 1" camber

A column check was done on column #45 on the fifth level. This column is a gravity column with no lateral loads and supports level 6, 7 and a roof structure above. It is a W14x61and has an unbraced length of 14'-8". The total factored gravity load from the roof to the top of the fifth floor slab including column self weight was computed and compared to its ΦPn value. The load combination used was 1.2D+1.6L and in accordance with LRFD specifications.

Pu: 394k
ΦPn: 552.7k

Conclusions:

Based on the above spot check of the designed composite beam the required capacity is approximately 80% of the composite beams actual capacity. Since the beams were designed as unshored, this made the construction load situation critical. Deflection caused by the construction load ≈ 1.2" with a 1" built-in camber; therefore, even though a smaller size beam element may have been able to be used more efficiently, ultimately its design was controlled by deflection during concrete placing and the beam size and composite action seem to be appropriate.

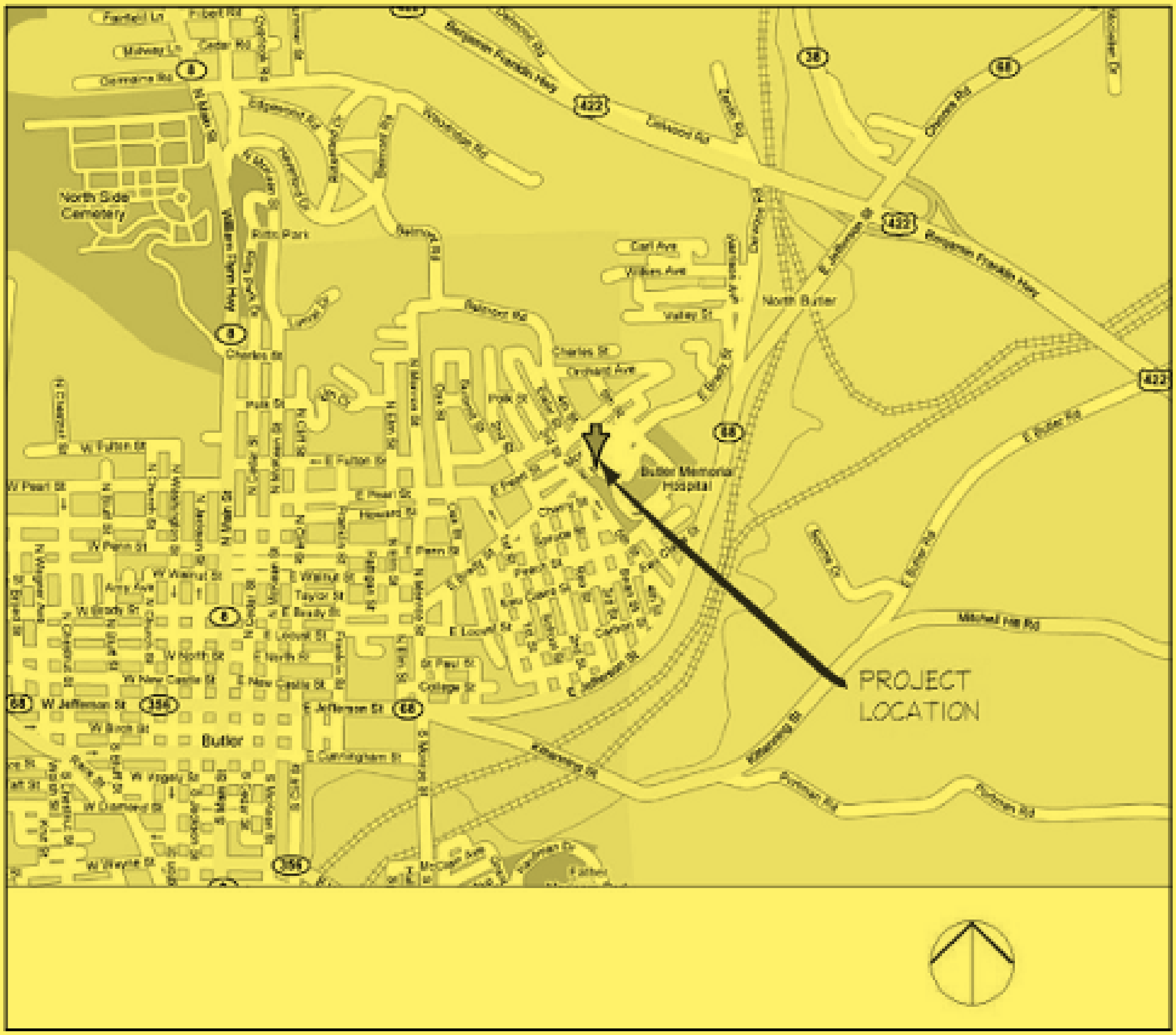
The column check that was done calculates that approximately only 63% of the columns capacity is being utilized. This particular column (#45) was designed as a gravity column with no lateral loads or capacity. The reason for the design load difference could have come from the fact that it is not exactly clear if the design firm took the live load reduction or not into their floor live loads for level 6& 7. If the reduction was not considered and a slightly higher (10psf) roof dead load was considered because of the large AHU located in that area, then the columns capacity would be at approximately 81% of capacity. Splicing into the column below would also be an issue since the next size smaller W14 would not be sufficient to carry the load and a W12 of sufficient capacity would be only 3plf less and require more steel and especially labor to make the connection. This would make it less economically feasible and sacrificing capacity. Therefore it appears that based on all aspects the best column size was chosen.

Overall based the couple of spot checks performed, the design uses the best members available for function and economic considerations.

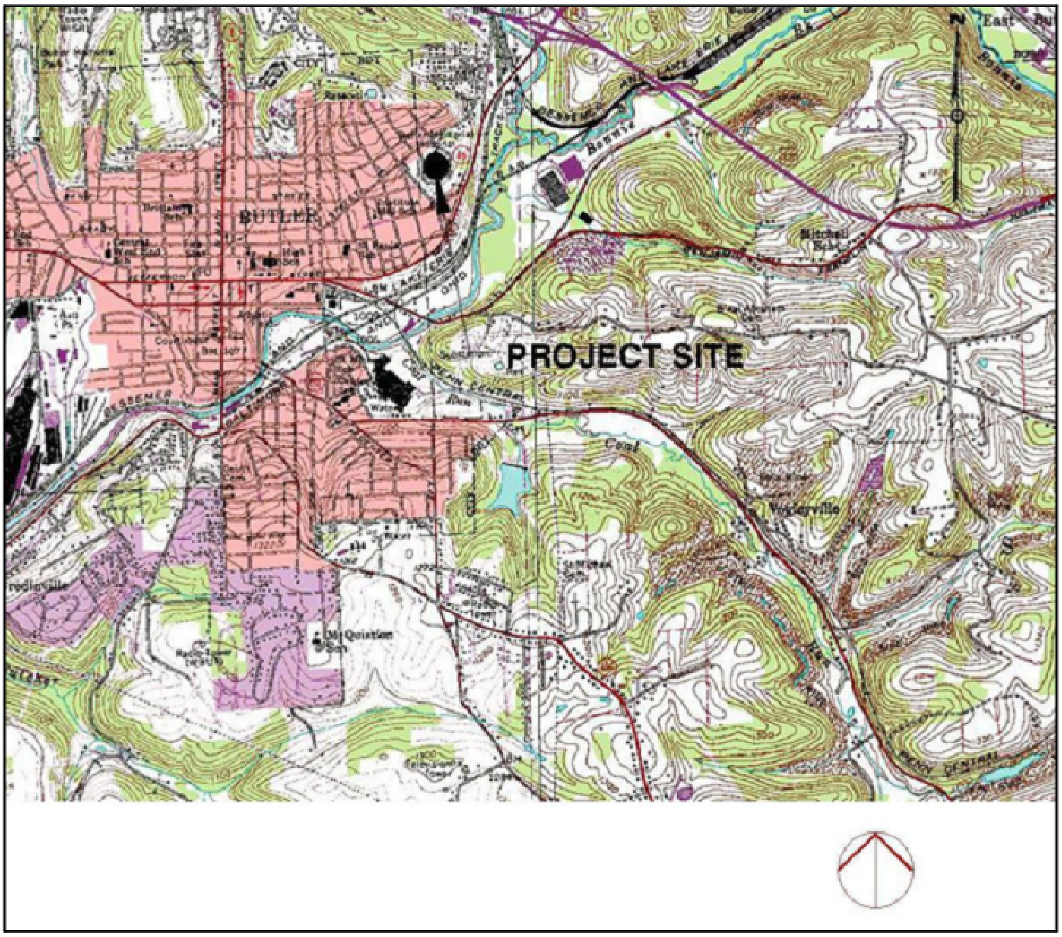
Appendix: A

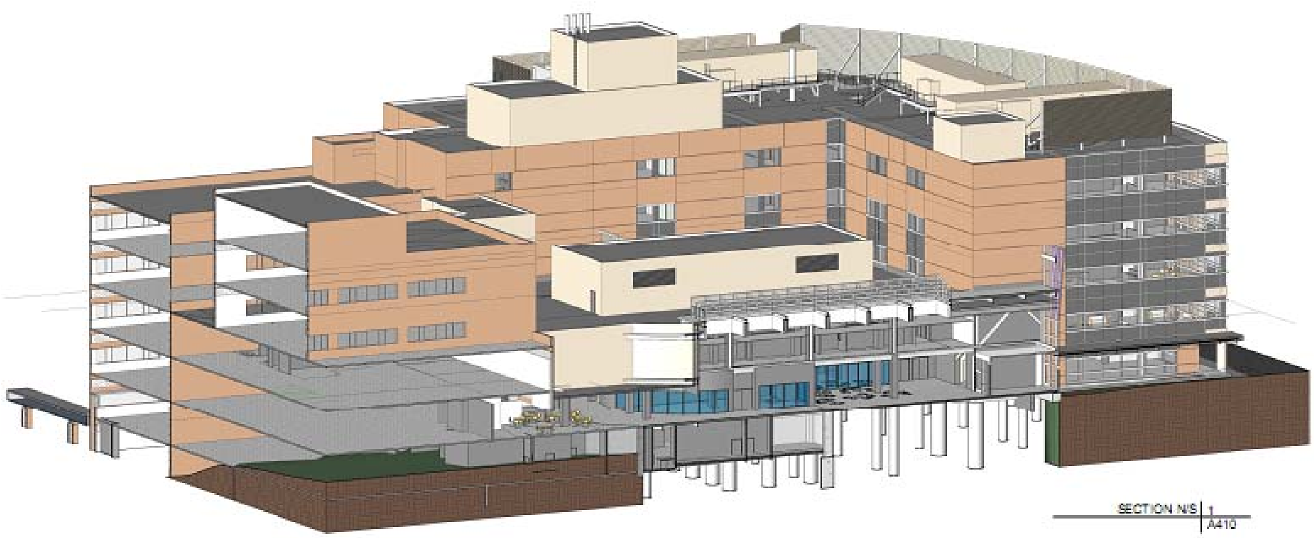
View looking from magnetic north

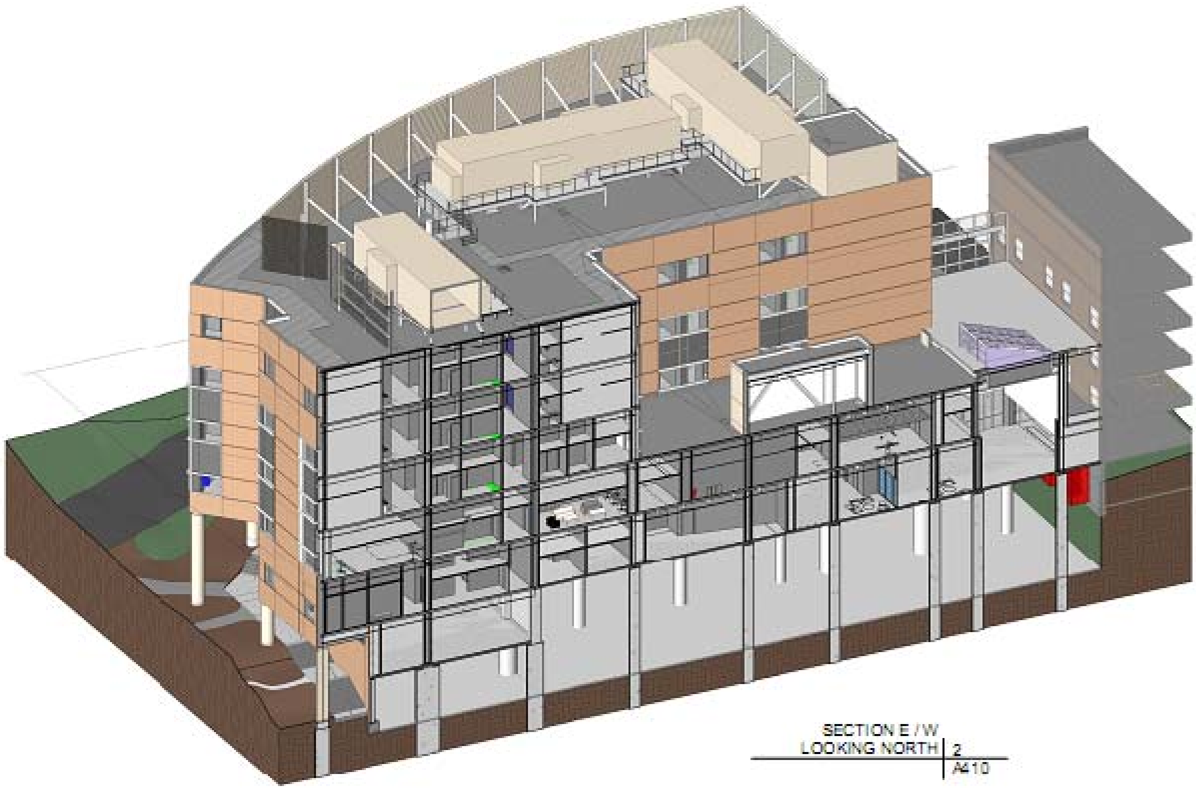
LOCATION MAP



VICINITY MAP







SECTION E / W
LOOKING NORTH 2
A410

MAIN FACADE
NORTHWEST CORNER 3
A400

Appendix B

JIM ROTUNNO TECH 1 WIND CALCULATIONS
9-26-09

Gust effect Factor: Rigid Structure

$$G = 0.925\left(\frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}}\right) \Rightarrow I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

$\bar{z} = 0.6h = 0.6(122') = 73.2' > 30' = \bar{z}_{min}$

$c = 0.20$ (Table 6-2)

$$I_{\bar{z}} = 0.2\left(\frac{33}{73.2}\right)^{1/6}$$

$0.2(0.876) = 0.1752$

$g_Q = 3.4$, $g_v = 3.5$ > 6.5.8.1

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

B = horizontal dimension normal to wind direction
E-W 198'
N-S 210'

$h = 122'$

$L_{\bar{z}} = l\left(\frac{\bar{z}}{30}\right)^{\bar{\epsilon}}$ $\Rightarrow \bar{z} = 73.2$

$= 500\left(\frac{73.2}{30}\right)^{1/5}$ $\bar{\epsilon} = 1/5.0$ > Table 6-2

$= 744.5'$ $l = 500$

(N-S) wind $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{198+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.372}}$$

$= 0.854$

(E-W) wind $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{210+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.3788}}$$

$= 0.851$

(E-W)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.851)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.856$

(N-S)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.854)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.857$

JIM ROTUNNO TECH 1 WIND CALCULATIONS
9-26-09

Velocity Pressure (q_z) evaluated @ height z ⇒ height above ground level

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ (lb/ft^2) ⇒ K_z varies @ levels

$= 0.00256 (K_z)(1.0)(0.85)(90^2)(1.15)$ — $K_{zt} = 1.0$ ie; does not meet all requirements of 6.5.7.1

$= \boxed{20.269\, K_z}$

$q_h = 0.00256\, K_{z@roof} (1.0)(0.85)(90^2)(1.15)$ ⇒ $K_{z_{roof}} = 1.315$ by interpolating From table 6-3

USING Exposure B Case 1 for K_z

$= 0.00256 (1.315)(7917.75)$
$= 26.65$

6.5.12.2.1 MWFRS - rigid buildings All heights

$F = qGC_p - q_i(GC_{pi})$ (lb/ft^2)

$q = q_z$ or q_h

$q_i = q_h$

K_{z1} @	14'-8"	= 0.85
K_{z2}	29'-4"	= 0.975
K_{z3}	44'-0"	= 1.06
K_{z5}	58'-8"	= 1.125
K_{z6}	73'-4"	= 1.183
K_{z7}	88'-0"	= 1.234
K_{z8}	102'-8"	= 1.267
K_{z9}	122-0"	= 1.315 ⇒ K_h
K_{z10}	135-0"	= 1.348

From Table 6-3
EXPOSURE B CASE 1
+ interpolation

TABLE 10: K_z

JIM ROTUNNO TECH 1 WIND CALCULATIONS
9-26-09

PRESSURES (F) (East-West) windward side

$F_i = q_{z_i} G C_p - q_h (GC_{p_i})$ ⇒ $q_z = 20.269(K_{z_i})$ $q_h = 26.65$

$F_1 = 20.269(0.85)(0.856)(0.8) - 26.65(-0.18) = 16.60$ lb/ft^2
$F_2 = 20.269(0.975)(0.856)(0.8) - 26.65(-0.18) = 18.33$
$F_3 = 20.269(1.06)(0.856)(0.8) + 4.797 = 19.51$
$F_5 = 20.269(1.125)(0.856)(0.8) + 4.797 = 20.41$
$F_6 = 20.269(1.183)(0.856)(0.8) + 4.797 = 21.22$
$F_7 = 20.269(1.234)(0.856)(0.8) + 4.797 = 21.92$
$F_8 = 20.269(1.267)(0.856)(0.8) + 4.797 = 22.38$
$F_9 = 20.269(1.315)(0.856)(0.8) + 4.797 = 23.05$
$F_{10} = 20.269(1.348)(0.856)(0.8) + 4.797 = 23.51$

Leeward side

$F = q_h GC_p - (q_h GC_{p_i})$
$= 26.65(0.856)(0.8) - 26.65(0.18) = 13.45$ lb/ft^2 leeward

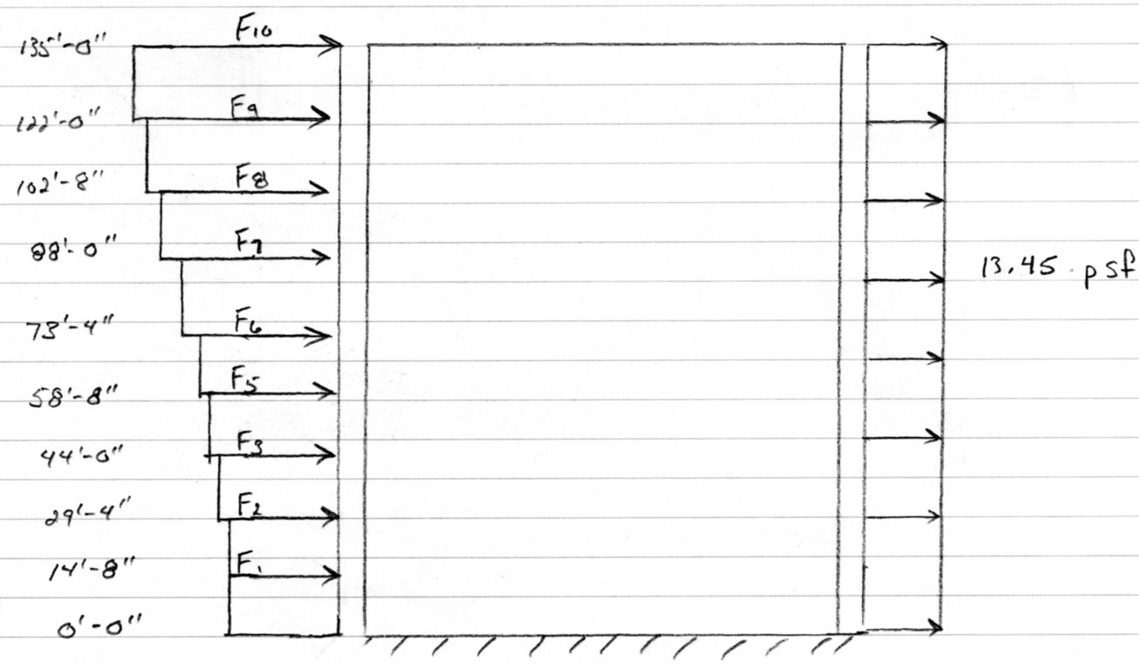
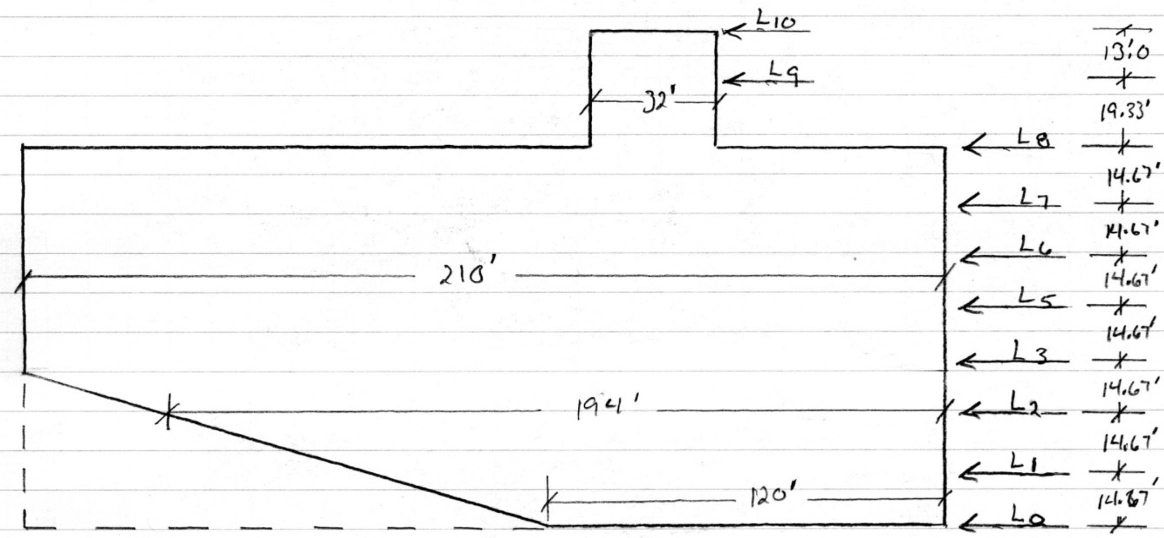


Figure 1M

JIM ROTUNNO TECH 1 WIND CALCULATIONS

East-West Wind



west elevation

Leeward pressure = 13.45 lb/ft² = LW_{E-W}

$P_0 = \frac{1}{2}(14.67')(120')(16.60) = 14.61\ k$

$P_1 = 7.33(120)(16.60) + 7.33(150)(18.33) = 34.76\ k$

$P_2 = 7.33(150)(18.83) + 7.33(194)(19.51) = 48.45\ k$

$P_3 = 7.33(194)(19.51) + 7.33(210)(20.41) = 59.16\ k$

$P_5 = 7.33(210)(20.41) + 7.33(210)(21.22) = 64.08\ k$

$P_6 = 7.33(210)(21.22) + 7.33(210)(21.92) = 66.41\ k$

$P_7 = 7.33(210)(21.92) + 7.33(210)(22.38) = 68.19\ k$

$P_8 = 7.33(210)(22.38) + \frac{19.33}{2}(32)(23.05) = 41.59\ k$

$P_9 = \frac{19.33}{2}(32)(23.05) + \frac{13}{2}(32)(23.51) = 12.02\ k$

$P_{10} = \frac{13}{2}(32)(23.51) = 4.89\ k$

414.16^{k}

JIM ROTUNNO TECH 1 WIND CALCULATIONS
9-26-09

PRESSURES (F) (North-South) windard side

F_1 = 20.269 (k_z)(0.857)(0.8) - 26.65(-0.18) =
= 13.896 (0.85) + 4.797 = 16.61 lb/ft²
F_2 = 13.896 (0.975) + 4.797 = 18.35
F_3 = 13.896 (1.06) + 4.797 = 19.53
F_5 = 13.896 (1.125) + 4.797 = 20.43
F_6 = 13.896 (1.183) + 4.797 = 21.24
F_7 = 13.896 (1.234) + 4.797 = 21.94
F_8 = 13.896 (1.267) + 4.797 = 22.40
F_9 = 13.896 (1.315) + 4.797 = 23.07
F_{10} = 13.896 (1.348) + 4.797 = 23.53

Leeward side

F = 26.65 (0.857)(0.8) - 26.65 (0.18) = 13.47 psf leeward

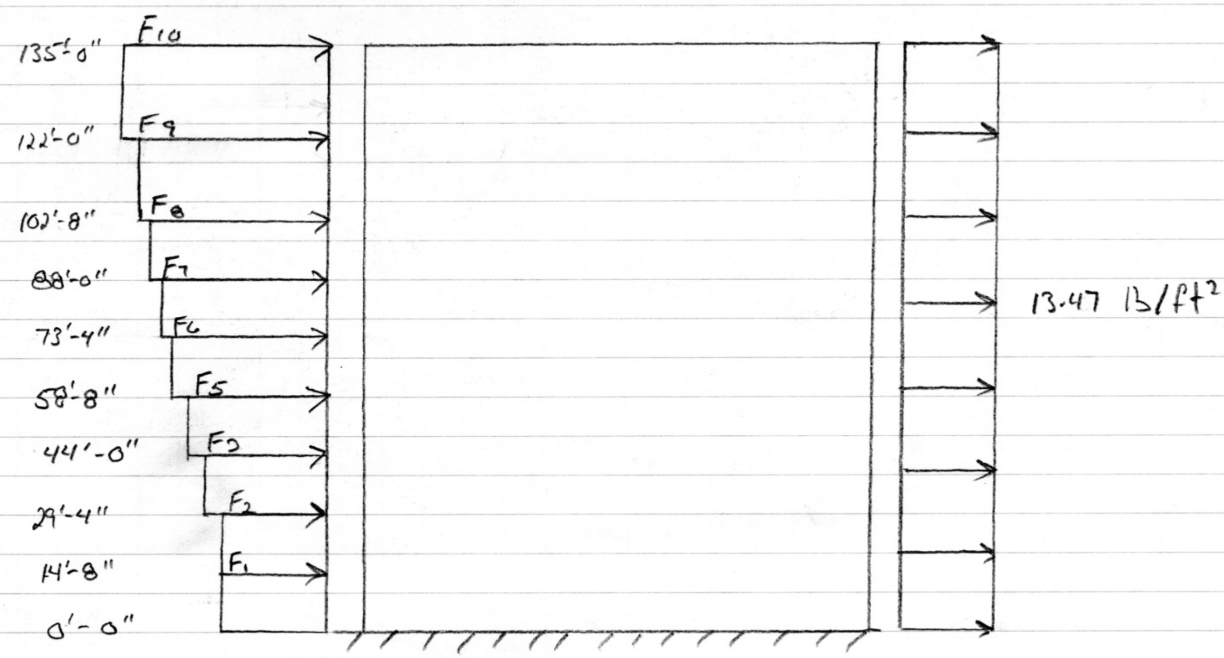
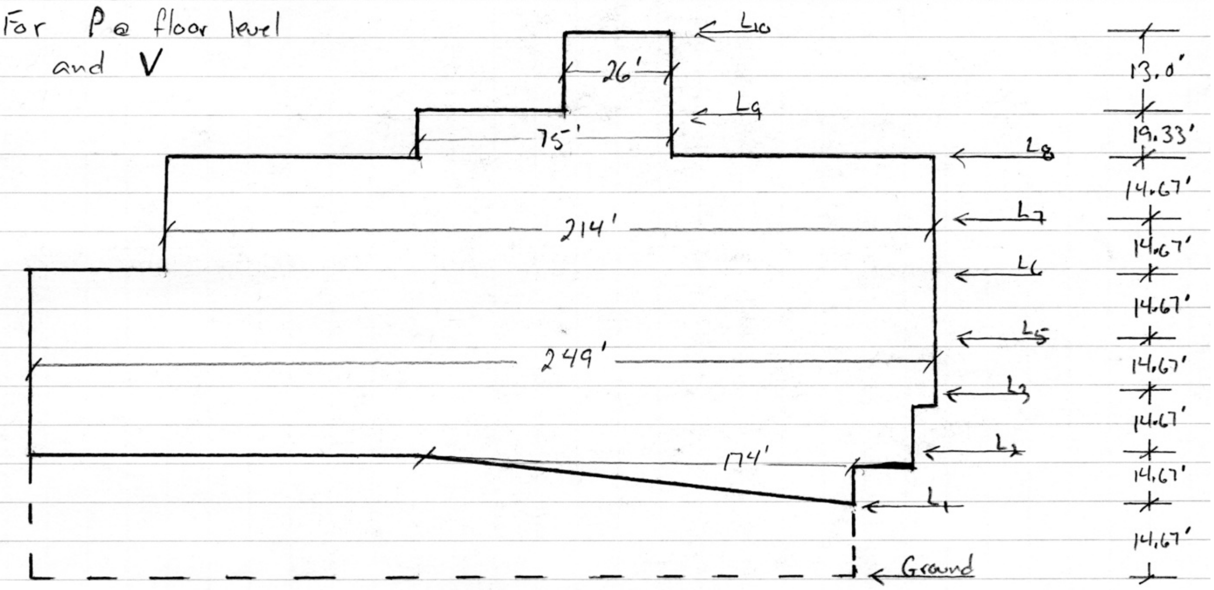


Figure 1N

JIM ROTUNNO TECH 1 WIND CALCULATIONS

North - South Wind

For P @ floor level
and V



Leeward pressure = 13.47 lb/ft^2 = LW_{N-S}

$P_G = 0$

$P_1 = \frac{1}{2}(174')(\frac{1}{2})(14.67')\,16.61 = 10.60\text{ k}$

$P_2 = \frac{1}{2}(14.67')(\frac{3}{4})(174')(16.61) + \frac{1}{2}(14.67)(174)18.35 = 39.310\text{ k}$

$P_3 = 7.33(249')(18.35) + 7.33(249')(19.53) = 69.14\text{k}$

$P_5 = 7.33(249')(19.53) + 7.33(249')(20.43) = 72.93$

$P_6 = 7.33(249)(20.43) + 7.33(214')(21.24) = 70.61$

$P_7 = 7.33(214)(21.24) + 7.33(214')(22.40) = 68.45$

$P_8 = 7.33(214)(22.40) + \frac{19.33}{2}(75')(23.07) = 51.86$

$P_9 = \frac{19.33}{2}(75')(23.07) + \frac{13.0}{2}(26')(23.53) = 20.70$

$P_{10} = \frac{13}{2}(26')(23.53) = 3.98$

Appendix C

JIM ROTUNNO TECH 1 SNOW LOAD CALCULATIONS

USING ASCE 7-05 CHAPTER 7

Flat roof snow load (p_f)

$p_f = 0.7\, C_e\, C_t\, I\, p_g$

$p_g \Rightarrow$ Butler, PA Figure 7-1 = ground snow load
$= 25$ psf

$C_e \Rightarrow$ Table 7-2 terrain category C roof partially exposed
$= 1.0$

$C_t \Rightarrow$ Table 7-3
$= 1.0$

I = Table 7-4 Category IV
$= 1.2$

$p_f = 0.7(1.0)(1.0)(1.2)(25) = 21$ psf

7.3 where p_g exceeds 20 lb/ft²
$p_f = 20I$
$= \boxed{24 \text{ psf}}$ not 21 psf

Appendix D

JIM ROTUNNO TECH 1 SEISMIC ANALYSIS

Occupancy Category IV

Determine the design spectral responce acceleration

$S_{DS} = 2/3\, S_{MS}$

$S_{MS} = F_a S_S \Rightarrow F_a$ = site coefficient Table 11.4.1

Site Class C

$S_S = .12 \quad S_1 = 0.046 \Rightarrow 0.0055$

$F_a = 1.2$

$S_{MS} = 1.2(0.12) = 0.144$

$S_{DS} = 2/3\, S_{MS} = 2/3(0.144) = 0.096$

$S_{D1} = 2/3\, S_{M1} \Rightarrow S_{M1} = F_v S_1$

$= 1.7(0.046) = 0.0782$

$S_{D1} = 2/3(0.0782) = 0.0521$

Importance Factor 1.5

SDC ⇒ Seismic design category = A ⇒ Table 11.6.1 occupancy IV

$S_{DS} < 0.167$

From Table 11.6.2

occupancy IV

$S_{D1} < 0.067 \;\therefore A$

Calculate the seismic base shear

$V = C_S W \quad$ W = Total dead load for seismic load determination

C_S = Seismic Responce coefficient ASCE 7-05 §12.8.1

$= S_{DS} / {}^{R}/_{I}$

$= 0.096 / {}^{3.25}/_{1.5} = 0.0443$ and $\le S_{D1}/T(R/I)$ for $T \le T_L$

$T: T_a = C_t h_n^x$ eq. 12.8.7

T_L = § 11.4.5 Fig 22-15

$= 12$

$T = 0.02(135\,ft)^{0.75} = 0.792\,sec$

$S_{D1}/T(R/I) = 0.0782 / 0.792(3.25/1.5)$

$C_S = 0.0456$

JIM ROTUNNO TECH 1 SEISMIC ANALYSIS

Effective seismic weight W_T as defined 12.7.2
1) 25% LL for for storage areas
2) partitions - minimum of 10 psf
3) total operating weight of permanent equipment use 10psf
4) where the flat roof snow load exceeds 30psf use 20% p_f < 30 ∴ Not required
plus the total Dead load

55psf weight of concrete slab + metal deck concrete 3½" ↕ metal 3" ↕

$$115\,pcf\left(\frac{3.5 + 3/2}{12''}\right) + 3\,psf = 50.92\ lb/ft^2 \quad \text{use } \underline{55psf}$$

Superimposed Dead Loads ⇒ MEP, partitions, finishes
35psf use 35psf for increased needs in hospitals

89.16 kips columns from column schedule (tabulated using excel)

beams (see next page)

5psf operating weight of permanent equipment - 5 psf of total building area

12psf roof - concrete deck, insulation, EDPM - 42 psf
building square footage from construction documents
L_1 = 20 405 sq. ft
L_2 = 45 545 sq. ft
L_3 = 42 165 sq. ft
L_5 = 31 525 sq. ft
L_6 = 27 720 sq. ft
L_7 = 27 760 sq. ft
Roof ⇒ L_8 = 46000

Exterior wall weight ⇒ brick veneer 120 lb/ft³

$$120\frac{lb}{ft^3}\left(\frac{3.25''}{12''}\right) = 32.5\ lb/ft^2$$

Story height = 14'-8"

JIM ROTUNNO TECH I SEISMIC CALCULATIONS

Level by Level weight analysis (Example)

concrete MEP Equip. columns beams brick facade

$W_1 = 55(20405) + 35(20405) + 5(20405) + 58390 + 187730 + 4860$
$= 2184.6$ k

exterior walls are estimated @ 20% glass + 80% brick
average curtain wall weighs 12.5-15 lb/ft² use 15 psf
0.20(15)(sq. ft. of wall per floor) + 0.80(32.5)(sq. ft. of ext. wall per floor) =
lb of exterior wall per level → divide this by the level's square footage
to get a lb/sq. ft per level then add into excel spreadsheet
$0.20(15) + 0.80(32) = 28.6$ lb/ft² of exterior wall

Beams
weight of beams @ each floor level is tabulated by
taking (3) spot checks of average bays and calculating the
% steel per floor area and superimposing it to all floors

spot check 1: @ level 3
Typical bay = 30' × 28' = 840 sq. ft
→ 4 - 18×40-30' 2 21×50-28'
$4(40)(30) + 2(50)(28) = 7600$ lb
7600 lb / 840 sq. ft = 9.04 lb/ft²

2: Typical bay level 2 30' × 30' = 900 sq. ft
4 - 16×26 · 2 - 24×55
$4(26)(30) + 2(55)(30) = 6420$
$6420/900 = 7.13$ lb/ft²

3: Typical bay level 5 30×28 = 840 sq. ft
5 - 18×40-30' 2-24×62-28'
$5(40)(30) + 2(62)(28) = 9472$ lb
9472 lb / 840 = 11.3 lb/ft²

$(11.3 + 9.04 + 7.13)/3 = 9.14$ average
USE 9.2 lb/ft²

JIM ROTUNNO TECH 1 SEISMIC CALCULATIONS

Distributing the total base shear to individual levels
The roof square footage needs to be divide up between levels 3, 5, 7, + 8

3 = 45 545 - 31 525 = 14 020 — 14020/45545 = 31%
5 = 31 525 - 27760 = 3765 — 3765/45545 = 8%
7 = 27760 - 2060 = 25700 — 25700/45545 = 56%
8 = 45545 - 14020 - 3765 - 25700 = 2060 — 2060/45545 = 5%

From excel take the floor weight totals (summed across) and add the % of roof weight to levels 3 5 + 7 to get total floor level weights

$1 = 2184.6^k$
$2 = 4796.4^k$
$3 = 4462.6^k + 0.31(1913) = 5055.6^k$
$5 = 3326.7 + 0.08(1913) = 3479.7^k$
$6 = 2927.9^k$
$7 = 2922.5 + 0.56(1913) = 3993.8^k$
$8 = 0.5(1913) = 95.65^k$

Total = 22616.5 kips

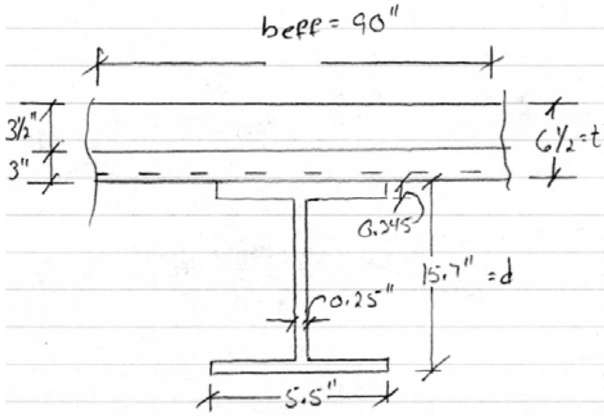
USE THESE LEVEL weights to estimate the % of Total Base Shear that acts at each level.

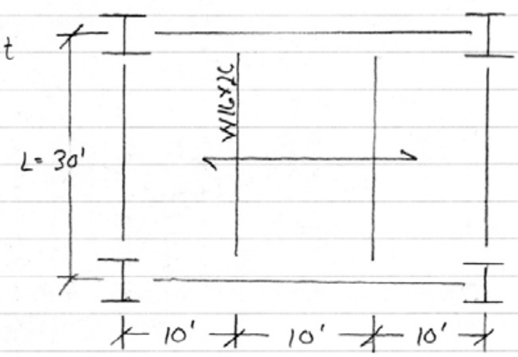
$1 \Rightarrow 2184.6/22616.5 = 9.66\% \Rightarrow 0.0966(1032) = 99.68^k$
$2 \Rightarrow 4796.4/22616.5 = 21.2\% \Rightarrow 0.212(1032) = 218.86^k$
$3 \Rightarrow 5055.6/22616.5 = 22.35\% \Rightarrow 0.2235(1032) = 230.69^k$
$5 \Rightarrow 3479.7/22616.5 = 15.39\% \Rightarrow 0.1539(1032) = 158.78^k$
$6 \Rightarrow 2927.9/22616.5 = 12.95\% \Rightarrow 0.1295(1032) = 133.6^k$
$7 \Rightarrow 3993.8/22616.5 = 17.66\% \Rightarrow 0.1766(1032) = 182.24^k$
$8 \Rightarrow 95.65/22616.5 = 0.42\% \Rightarrow 0.0042(1032) = 4.36^k$

Appendix E

JIM ROTUNNO TECH 1 SPOT CHECKS

BEAM W16x26 30' span





$A_s = 7.68\,in^2$
$I = 301\,in^4$
$f'_c = 3.5$ ksi
$F_y = 50$ ksi
$F_u = 65$ ksi

$b_{eff} = \min \left| \begin{array}{l} S = 10' \\ L/4 = 30/4 = 7.5' = 90" \end{array} \right.$

$C_c = 0.85 f'_c A_c = 0.85(3.5)(3.5)(90") = 937.125^k$
$T_s = A_s F_y = 7.68(50) = 384^k$

$C_c > T_s \therefore$ PNA is in the concrete

$a = \dfrac{T_s}{0.85 f'_c b_{eff}} = \dfrac{384}{0.85(3.5)(90)} = 1.434"$

Assuming full composite action and ΣQ_n is at least 384^k

$Y_1 = TFL \qquad Y_2 = 5.5 = 6.5 - a/2 = 5.78$

$\phi M_n = 385^{k \cdot ft}$ Table 3-19 AISC
$\phi V_n = 106^k$ Table 3-2 AISC

$M_u \Rightarrow$ Dead Load $\Rightarrow$ concrete floor, self weight beam, MEP
Live Load $\Rightarrow$ 100 psf

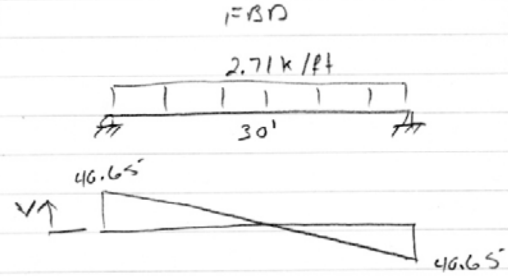
$DL \Rightarrow 1.2[55 + 35)10' + 26]/1000 = 1.1112\ k/ft$
$LL \Rightarrow 1.6(100)(10)/1000 = 1.6\ k/ft$
$w_u = 1.2D + 1.6L = 2.71\ k/ft$

JIM ROTUNNO TECH I SPOT CHECKS

Beam Cont.

$M_u = \frac{w_u L^2}{8} = \frac{2.71(30^2)}{8} = 305.0 \text{ k·ft}$

FBD



$M_u < \phi M_n \therefore OK$

$V_u = 46.65^k < 106^k \therefore OK$

$\Delta_{max} = \frac{5 w l^4}{384 E I}$

$\Delta L_b \Rightarrow I_{LB} = 985 \text{ in}^4$ Table 3-20 AISC

$\Delta LL = \frac{5(W_{LL}) L^4}{384 E I_{LB}} = \frac{5(1.6)(30^4)(1728)}{384(29000)(985)} = 1.02 \text{ in} \approx 1''$ There's is 1" camber already $\therefore$ OK

Construction load situation

assuming bare beam with full lateral bracing from decking

weight of concrete = 55 lb/ft²

W_D (55 lb/ft²)(10') + 26 lb/ft = 576 lb/ft

W_{LL} = 20 lb/ft²(10') = 200 lb/ft

$1.2D + 1.6L = 1.2(0.576) + 1.6(0.200) = 1.0112 \text{ k/ft}$

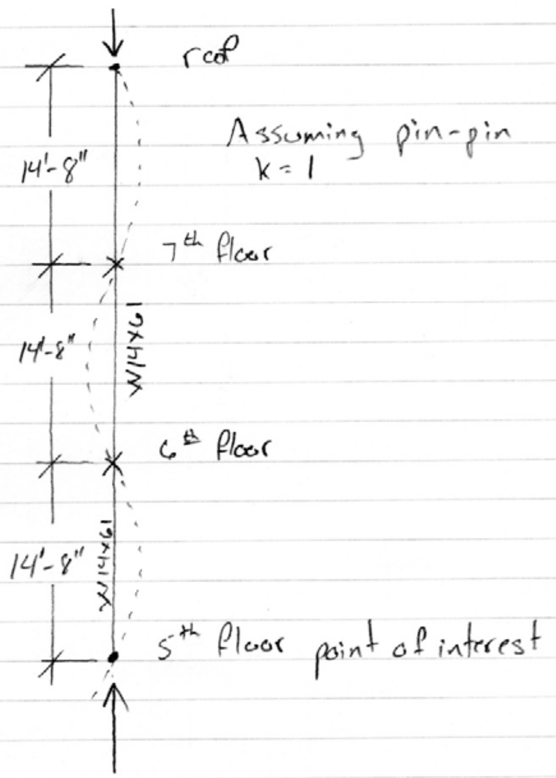
$M_u = 1.0112(30')^2/8 = 113.76 \text{ k·ft}$

ϕM_p of bare beam = 166 k·ft $\therefore$ OK

$\Delta DL = \frac{5(0.576)(30^4)(1728)}{384(29000)(501)} = 1.2''$ with 1" camber $\therefore$ OK

JIM ROTUNNO TECH 1 SPOT CHECK

Column C45



W14X61
$F_y = 50$ ksi
$F_u = 65$ ksi
$\phi P_n = 552.7^k$ @ 14'-8"
AISC Table 4-1
$A_s = 17.9 \text{ in}^2$
$r_y = 2.45''$
$KL/r = 14(12'')/2.45 = 68.57 < 113$
$\therefore$ use $F_{cr} = 0.658^{(F_y/F_e)} F_y$
$F_e = \pi^2 E / 68.57$
$= 60.87$
$F_{cr} = (0.658^{50/60.87})\,50$
$= 35.45$ ksi
$\phi P_n = 0.9(35.45)(17.9) = 571^k$
USE 552.7^k

Calculate Total factored gravity load

Dead Loads

roof framing $= 5.659^k$ column roof-7 $= 0.895^k$ concrete Roof $= 29.61^k$
level 7 framing $= 3.185^k$ column 7-6 $= 0.895^k$ concrete 7 $= 34.65^k$
level 6 framing $= 3.185^k$ column 6-5 $= 0.895^k$ concrete 6 $= 34.65^k$
MEP 6+7 $= 44.1^k$
$1.2D = 1.2(157.724) = 189.3^k$

Live Loads

roof ⇒ no reduction $21'(30')(115 \text{ psf}) = 72.45^k$
level 7 ⇒ reduced ⇒ $80(0.25 + 15/\sqrt{4(21)(30)})(21)(30) = 27.7^k$
level 6 ⇒ reduced ⇒ same as 7
$1.6L = 1.6(72.45 + 27.7 + 27.7) = 204.4$

for LL reduction
$K_{LL} = 4$ for interior column
$A_T = 21'(30')$
ASCE 7-05
SECTION 4.8

$P_u = 1.2D + 1.6L = 189.3 + 204.4 = 393.7^k < 552.7 \therefore$ ok