Shawn Jones
Structural Option
Advisor: Dr. Hanagan
Hershey Academic
Support Center
Hershey, PA
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Technical Report 1 – Structural Systems

Executive Summary

This report is an analysis of the existing structural systems in The Hershey Academic Support Center found in Hershey, PA. The Hershey Academic Support Center is part of the Hershey Medical Center complex and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is used for auxiliary purposes of the Hershey Medical Center and accommodates 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center section. The building footprint encompasses a total area of 150,000 square feet. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The building consists of a conventional structural steel system with composite beam floor framing and a precast concrete and glass facade. Moment connections placed on the columns as well as braced steel frames provide additional support to resist the wind and lateral loads throughout the building. The basic structural systems are similar to those found in more recent buildings built in the Hershey Medical Center complex.

The code basis for this building is BOCA 1996, but to keep the entire set of calculations standard with the current structural technology, ASCE 7-02 was used to calculate Wind and Seismic forces. When performing my spot check analysis, I found that the calculated members were slightly different than the proposed members in the design. For the gravity loading, the beams and girders I calculated were slightly larger than in the original design which is most likely due to the code difference (ASD to LRFD) and the ¾" camber found on the beams. For the lateral loading, I calculated the braced frames to take the entire load, but in reality there are moment connections in place to help better distribute it. Also, the change in design code can be attributed for part of the difference.

General Building Information

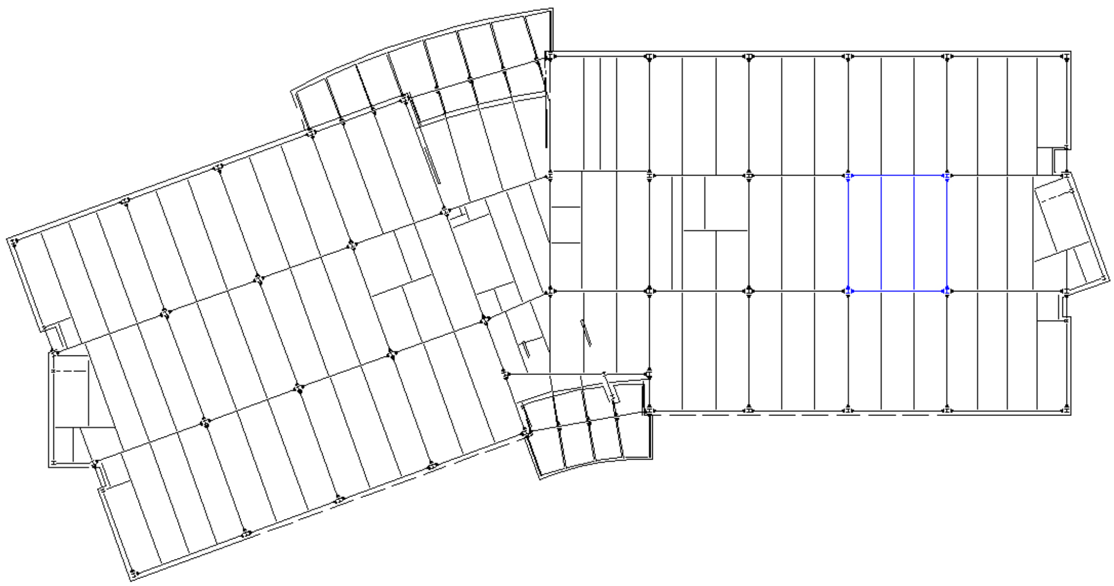
The Hershey Academic Support Center is part of the Hershey Medical Center complex and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is used for auxiliary purposes of the Hershey Medical Center and accommodates 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center section. The building footprint encompasses a total area of 150,000 square feet. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The building consists of a conventional structural steel system with composite beam floor framing and a precast concrete and glass facade. Moment connections placed on the columns as well as braced steel frames provide additional support to resist the wind and lateral loads throughout the building. My building has a unique lateral framing system because it contains a composite floor system, extensive use of moment connections, and a braced frame system as well. This excess of lateral bracing could be due to in part the weight located on the roof from the Mechanical Penthouse. Another factor in the design could be that the architecture of the building would be best maintained by the current use of moment connections instead of other options like shear walls or more braced frames. A cost analysis for each system should be conducted to see if the moment connections ran up a greater overall cost like I predict they did compared to other designs. Two major considerations that I will have to take into account for further technical reports are the affects of wind uplift on the Mechanical Penthouse and the roof as well as snow drifting. These may prove to be significant and should be calculated to check.

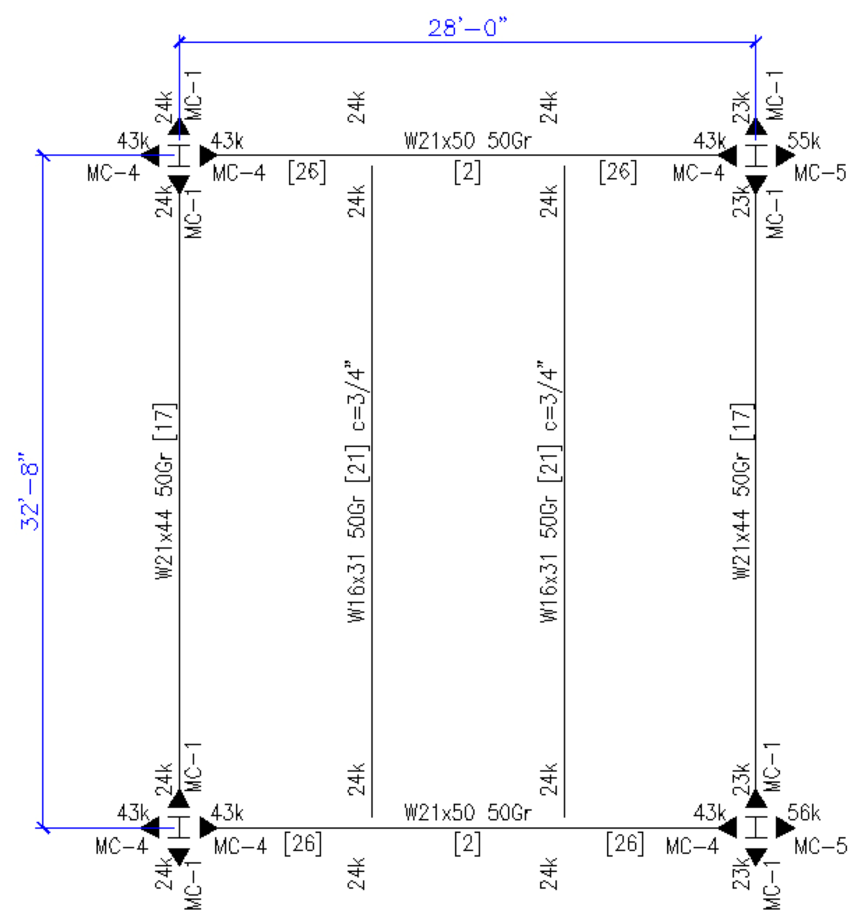
Foundation

The foundation for this structure is a deep foundation system consisting of caissons and grade beams. The concrete slab on grade is 4" thick and reinforced by WWF. Footings are placed under the columns and step footings were used at the corners of the building for extra support. All exterior footings must extend 3'-6" below the finished grade to protect from frost. Footings have been designed for a net soil bearing pressure of 6,500 psf. If spread footings are not desired, geopiles may be used in place as long as they meet the same criteria.

Floor System

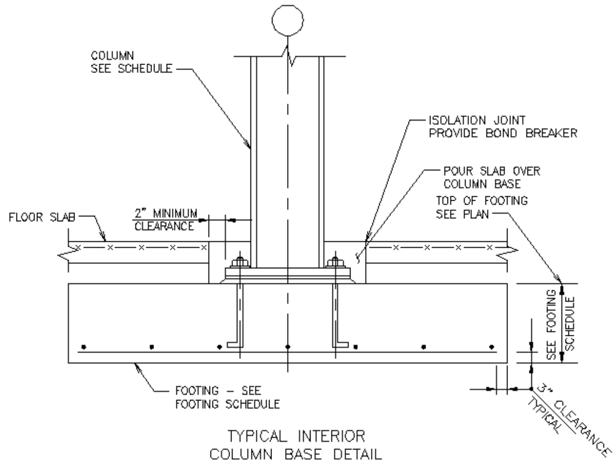
The floor system at the Hershey Academic Support Center utilizes a composite beam floor framing system with 3" 20 gauge Vulcraft galvanized steel metal decking and 6x6 W1.4XW1.4 Welded Wire Fabric between the steel members and the concrete. The 2.5" Lightweight concrete slab along with the decking provide a total slab thickness of 5.5" and a total system depth at the girder of 26.5". To hold together the decking and concrete slab, ¾"ø x 4 ½" longheaded steel studs were used. Moment connections are used throughout the building to prevent wind force. Each typical bay is 28' by 32'-8" and consists of W21x50 and W21x44 girders with W16x31 interior beams that have ¾" camber. Material strength is given as 4000 psi for the concrete and ASTM A-572 Grade 50 steel in the beams and girders. Spray on cementitious fireproofing was used to meet the fire rating required for the building. The floor framing system along with a typical interior bay is shown below.





Columns

Columns for the building vary in size but the typical column size is a W14x120 and a W14x176. The columns are spaced identically in each wing in 28' to 33'-8" bays. Cross bracing is used for extra building support and is shown below (See Picture).



TYPICAL INTERIOR
COLUMN BASE DETAIL

DETAIL A

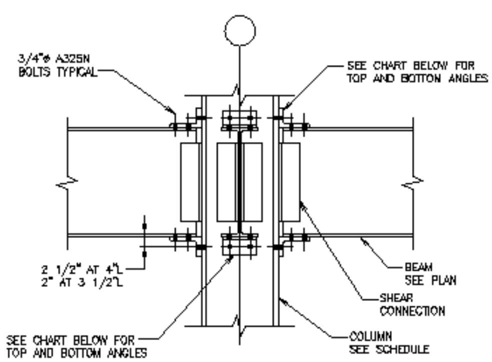
NOT TO SCALE

Roof

This building utilizes an EPDM membrane roofing system with rigid insulation placed on a 3" lightweight concrete slab with 3" deep 20 gauge composite steel metal deck underneath. Girder size is increased slightly to W18x40 and W21x76 and the moment connections at the columns were increased in strength with more bolts. The Mechanical Penthouse is located on the roof and houses all the major mechanical components for the entire building.

Connections

Almost every girder to column connection encountered in the Hershey Academic Support Center is a partially restrained moment connection with connecting angles used between beams and girders. The most common connection used in a typical bay is a L6 x 4 x 7/8 x 0'7" steel angle with 4 bolts to a girder and 2 bolts to a column. Top and bottom plates are also used for some connections for extra support. A diagram of a typical moment connection (Right) and a beam to girder connection (Left) is shown below.

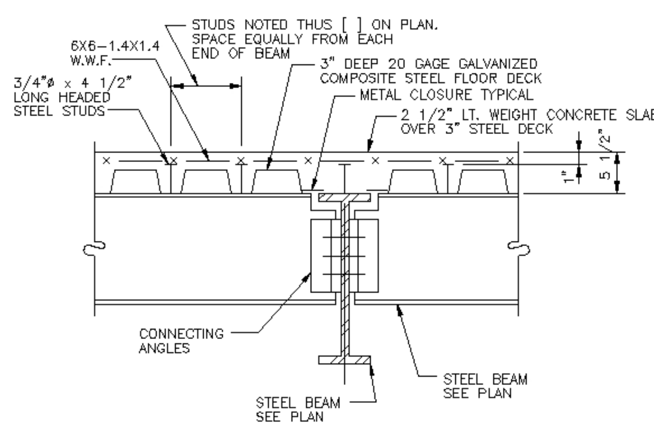


WIND MOMENT CONNECTION–
TOP AND BOTTOM ANGLES
MC–1 THRU MC–10

SEE PLAN FOR LOCATIONS

DETAIL A

NOT TO SCALE



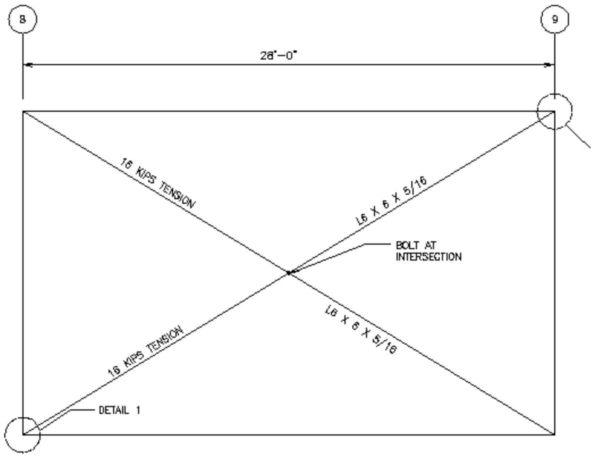
TYPICAL BEAM TO GIRDER

DETAIL 1

NOT TO SCALE

Lateral System

For the lateral system, this building utilizes a composite floor system, moment connections at the columns, and braced framing to handle the lateral loads. There are three different strengths of column moment connections used throughout the building depending on the number of bolts used and every column is outfitted with at least one type. For extra resistance to lateral load forces, braced frame cross bracing is used between some columns for extra stability. Three different bracing sizes were used to help accommodate some of the different widths in the building ranging from 28’ down to 17’ cross member bracing (See Picture).



X–BRACING ELEVATION B
1/4" = 1'–0"

Envelope

The two wings of the building form a slight angle out from the center and are clad with a repeating window pattern laced with precast concrete panels. The center of the building has a concrete slab canopy leading into the main lobby, which is encased by glass and extends across the entire first floor of the building. A sheet of glass is located from the canopy to the top of the building. To break up the repeating window pattern, both sides of the building have fire rated concrete encased stairwells that jut out to the side.

Design Codes

Building Code: 1996 BOCA code was used originally, but ASCE 7-02 was used in this technical report.

Cast-In-Place Concrete: Use code excerpts “Specifications for Structural Concrete for Buildings” (ACI 301) and “Building Code Requirements for Reinforced Concrete” (ACI 318).

Masonry: Materials and construction will follow the ACI 530/ASCE 5: Building Code Requirements for Masonry Structures.

Structural Steel: The 9th edition Allowable Stress Design code from the American Institute of Steel Construction “Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings” as well as AISC “Code of Standard Practice for Steel Buildings and Bridges” was used for the original project. AISC Load and Resistance Factor Design, 3rd Edition was used for my calculations.

Welding: Welds shall conform to The American Welding Society Code for Buildings (AWS D1.1)

Metal Decking: Design and fabrication will follow the latest edition of the “Specification of the Steel Deck Institute”.

Material Strength

Cast-In-Place Concrete:

~All concrete will be stone aggregate concrete with a minimum compressive strength of 4000 psi at 28 days.

~Elevated floor slabs on composite metal deck will use light-weight aggregate concrete conforming to ASTM C330.

~Concrete fill for masonry walls will be pea gravel concrete with a minimum compressive strength of 3000 psi at 28 days.

Masonry:

~All masonry block shall have a minimum compressive strength net area of 1900 psi as provided by ASTM C90 and ASTM C145.

~All grout used shall have a minimum compressive strength of 1800 psi at 28 days.

~Block foundation walls must meet a minimum compressive strength net area of 1900 psi.

Structural Steel:

~All structural steel beams will be Fy = 50,000 psi as given by ASTM A-572.

~All columns, angles, channels, and miscellaneous steel will be Fy = 36,000 psi as given by ASTM A-36

~Steel tubing will be Fy = 46,000 psi as given by ASTM A500 Grade steel.

~All steel pipe will be Fy = 36,000 psi by ASTM A-501 or Fy = 35,000 psi by ASTM A-53 type “E” or “S”, Grade B.

~All welded connections shall be done with E70XX Electrodes with 3/16” minimum material.

~All bolted connections will use ¾” ø ASTM A325N high strength bolts minimum.

Metal Deck:

~All metal floor deck shall be 3” VLI – Galvanized 20 Gage composite decking and will be designed to resist a floor shear load of 2000 plf and a roof shear load of 3000 plf as well as uplift loads.

Metal Studs:

~All metal studs 16 Gage and heavier shall have a minimum steel yield strength of 50 ksi by ASTM A 446 Grade D.

~All metal studs 18 Gage and lighter shall have a minimum steel yield strength of 33 ksi by ASTM A 446 Grade A.

Gravity Loads

All gravity load calculations found in the existing building used BOCA 1996 as their design standard. For simplicity and current accurate standards, I will use ASCE 7-02 to find, factor, and calculate all gravity loads in the building. If a uniform difference in sizes is noticed, it may be do to this change.

Dead Loads:

Roof:

~Decking = 3 psf
~Insulation = 3 psf
~Steel = 6 psf
~Membrane = 2 psf
~MEP = 8 psf
~Ballast = 8 psf
~Total Roof Dead Load = 30 psf

Penthouse Floor:

~Decking = 2 psf
~3” Lightweight Concrete = 45 psf
~4” Top Slab = 50 psf
~Steel = 10 psf
~MEP = 10 psf
~Finishing = 3 psf
~Miscellaneous = 5 psf
~Total Penthouse Dead Load = 125 psf

Office Floor:

~3” Decking = 2 psf
~2.5” Lightweight Concrete = (2.5” + (3”/2))*(115 pcf/12) = 40 psf
~Steel = 5 psf
~MEP = 10 psf
~Plenum Air System = 10 psf
~Finishing = 3 psf
~Total Office Dead Load = 70 psf

Live Loads:

Roof = 30 psf + snow drifting
High Density File Storage = 200 psf, uniformly distributed
Main Floor = 100 psf (with corridors and partitions)
Mechanical Penthouse = 150 psf
Stairs = 100 psf

Snow Loads:

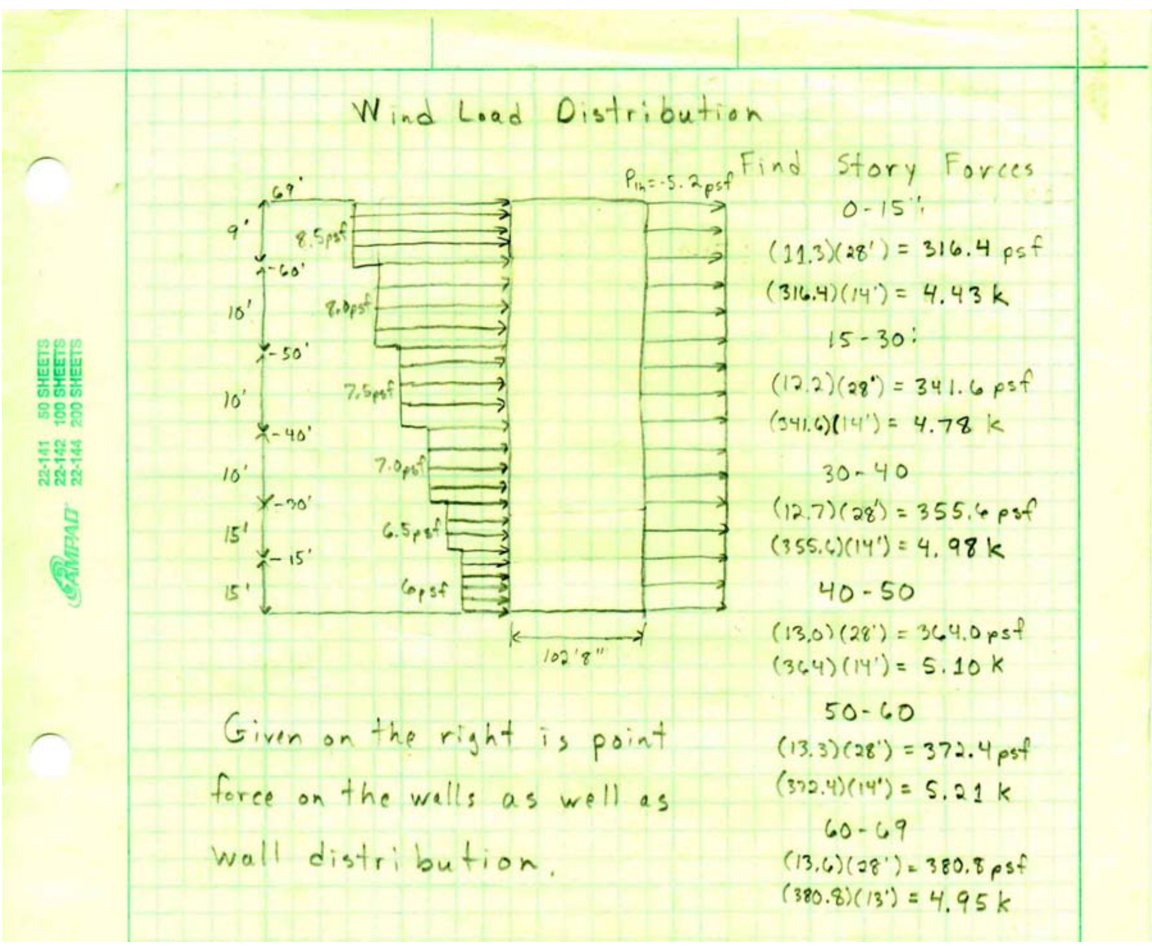
Use the equation $p_f = 0.7*C_e*C_t*I*p_g$ where:
~From Table 7-2, Exposure Factor C_e = 1.0 (Terrain Category B, Partially Exposed)
~From Table 7-3, Thermal Factor C_t = 1.0
~From Table 7-4, Importance Factor I = 1.0 (Category II)
~From Figure 7-1, Ground Snow Load p_g = 30 psf
~Total Snow Load = 21 psf

Wind Loads

z (ft)	K_z	q_z	(P_{wz}) N-S	(P_{lh}) N-S	(P_{tot}) N-S	(P_{wz}) E-W	(P_{lh}) E-W	(P_{tot}) E-W
0-15	0.85	9.06304	6.079937	-5.21265	11.29259	6.257873	-3.21912	9.476997
20	0.9	9.59616	6.437581	-5.21265	11.65023	6.625984	-3.21912	9.845107
25	0.94	10.02266	6.723695	-5.21265	11.93635	6.920472	-3.21912	10.1396
30	0.98	10.44915	7.00981	-5.21265	12.22246	7.21496	-3.21912	10.43408
40	1.04	11.0889	7.438982	-5.21265	12.65163	7.656692	-3.21912	10.87582
50	1.09	11.62202	7.796626	-5.21265	13.00928	8.024802	-3.21912	11.24393
60	1.13	12.04851	8.08274	-5.21265	13.29539	8.319291	-3.21912	11.53841
70	1.17	12.47501	8.368855	-5.21265	13.58151	8.613779	-3.21912	11.8329

	N-S	E-W
Story Shear @ 0	21.21098	6.811023
Story Shear @ 1	43.07454	13.87904
Story Shear @ 2	46.18385	15.10357
Story Shear @ 3	48.39108	15.97283
Story Shear @ 4	50.09928	16.64556
Story Shear @ 5	35.30126	10.81774

The charts shown above summarize the results found from my wind calculation analysis. Shown below is the wind loading for a typical building wall as well as story forces. Specific calculations of wind forces are located in the Appendix.



Seismic Loads

Hershey 5					
Vertical Distribution N-S					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Hershey 5					
Vertical Distribution E-W					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
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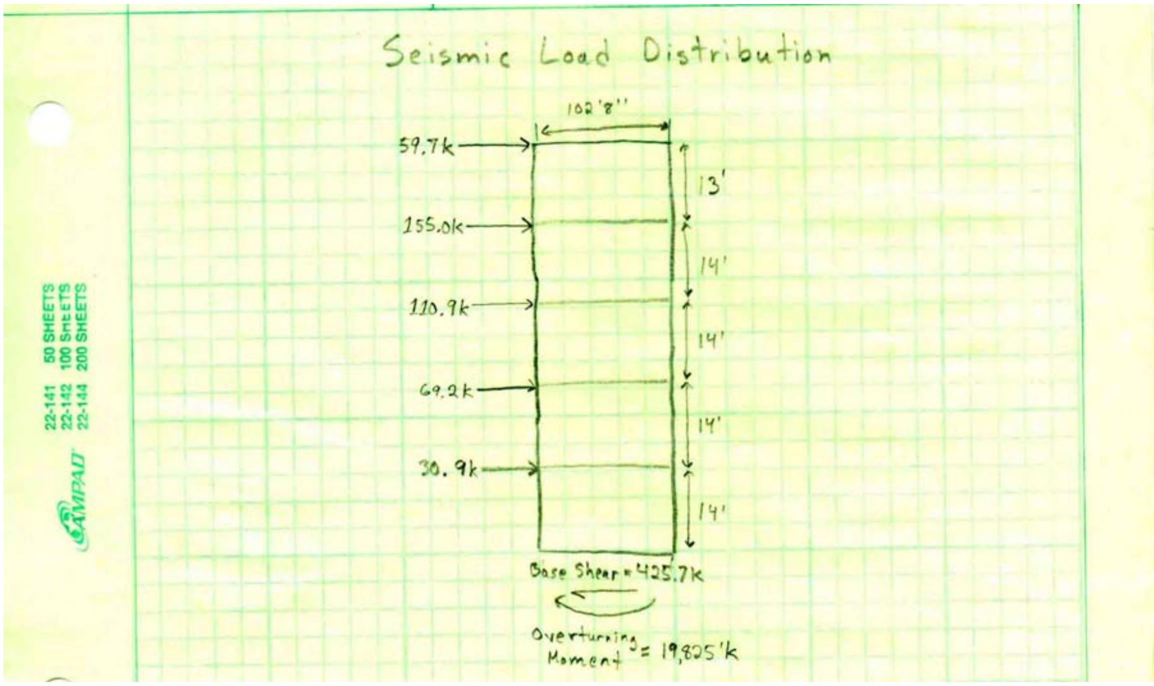
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

The charts shown above summarize the results found from my seismic calculation analysis. Shown below is the seismic loading for a typical building as depicted by story forces. Specific calculations of seismic forces are located in the Appendix.



Further Considerations

This is a list of design parameters that were not covered in this Technical Report, but are still significant to the complete understanding of the building systems:

~Wind Uplift on the Mechanical Penthouse as well as the roof

~Snow Drift in conjunction with the Mechanical Penthouse

~Story Drift from lateral forces

~Deflection checks from gravity and lateral forces

~Specific Lateral Load calculations and values

Appendix

Wind Loading Calculations

	Hershey 5	Assumptions and Information
(K_{zt}) Topographic Factor	1	Code 6.5.7.2, Figure 6-4, $K_{zt} = (1+(k_1)*(k_2)*(k_3))^2$
(K_d) Directional Wind Factor	0.85	Code 6.5.5.4, Table 6-4
(V) Basic Wind Speed	70	Given by Structural Notes
(I) Importance Factor	1	Code 6.5.5, Table 6-1
(C_t) Peroid Parameter	0.02	Code 9.5.3.2, Table 9.5.5.3.2
(h) Building Height in Feet	69	Height to the 5th Story
(f) Frequency in Hz	2.08849378	Code 9.5.3.2, Table 9.5.5.3.2, f = 1/((C_t)*((h)^0.75))
Exposure Category C	α	Given by Structural Notes
(α)	9.5	Code 6.5, Table 6-2
(zg (ft))	900	Code 6.5, Table 6-2
(^a)	2/19	Code 6.5, Table 6-2
(^b)	1	Code 6.5, Table 6-2
(α bar)	1/6	Code 6.5, Table 6-2
(b bar)	0.65	Code 6.5, Table 6-2
(c) alsdrj	0.2	Code 6.5, Table 6-2
(L (ft))	500	Code 6.5, Table 6-2
(Є bar)	1/5	Code 6.5, Table 6-2
(z min)	15	Code 6.5, Table 6-2
Rigid Structures N-S		
*Exposure C, Table 6-2		
(g_q) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(g_v) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(z bar) Wind Coefficient	41.4	Code 6.5.8.2, Table 6-2, z bar = 0.6(h)
(L_z) Turbulence Scale Factor	523.199457	L_z = L*((z bar)/33)^(Є bar)
(I_z) Turbulence Intensity	0.19258196	Code 6.5.8.1, Equation 6-5, Iz = (c)*(33/(z bar))^(1/6)
(B) Perpendicular to Wind	268.33	Code 6.3, Given in Plan
(L) Parallel to Wind	102.67	Code 6.3, Given in Plan
(Q) Background Response	0.82260391	Code 6.5.8.1, Equation 6-6, Q = SQRT(1/(1+(0.63*((B+h)/L_z)^0.63)))
(G) Gust Factor	0.83856209	Code 6.5.8.2, Equation 6-8, G = 0.925*((1+(1.7*(g_q)*(I_z)*(Q)))/(1+(1.7*(g_v)*(I_z))))
Rigid Structures E-W		
(B)	102.67	
(Q)	0.8729702	
(G)	0.86310353	
Flexible Structures N-S		
*Exposure B, Table 6-2		

(g_q)	3.4	
(g_v)	3.4	
(z bar)	41.4	
(L_z)	523.199457	
(I_z)	0.19258196	
(B) Perpendicular to Wind	268.33	
(L) Parallel to Wind	102.67	
(Q)	0.82260391	
(β) Damping Ratio	0.05	Code 6.3, Section 9
(n_1) Natural Frequency	2.08849378	Code 6.5.8.2
(V_z) Mean Hourly Wind Speed	69.3038272	Code 6.5.8.2, Equation 6-14, V_z = ((b bar)*(B40/33)^(α bar)*(V)*(88/60)
(η_h) R_l Coefficient	9.5649541	Code 6.5.8.2, Equation 6-13, η_h = 4.6*(n_1)*(h)/(V_z)
(η_B) R_l Coefficient	37.1965816	Code 6.5.8.2, Equation 6-13, η_B = 4.6*(n_1)*(B)/(V_z)
(η_L) R_l Coefficient	47.6475145	Code 6.5.8.2, Equation 6-13, η_L = 4.6*(n_1)*(L)/(V_z)
(R_h) R_l Coefficient	0.09908316	Code 6.5.8.2, Equation 6-13, R_h = (1/η_h)-(1/(2*(η_h^2)))*(1-(2.718281828^(-2*η_h)))
(R_B) R_l Coefficient	0.02652281	Code 6.5.8.2, Equation 6-13, R_h = (1/η_B)-(1/(2*(η_B^2)))*(1-(2.718281828^(-2*η_B)))
(R_L) R_l Coefficient	0.02076722	Code 6.5.8.2, Equation 6-13, R_h = (1/η_L)-(1/(2*(η_L^2)))*(1-(2.718281828^(-2*η_L)))
(N_1) Reduced Frequency	15.7667889	Code 6.5.8.2, Equation 6-12, N_1 = (n_1*L_z)/V_z
(R_n) Resonance Coefficient	0.0241168	Code 6.5.8.2, Equation 6-11, R_n = (7.47*N_1)/((1+(10.3*N_1))^(5/3))
(R) Resonance Response Factor	0.02615683	Code 6.8.5.2, Equation 6-10, R = (1/β)*R_n*R_h*R_B*(0.53+(0.47*R_L)))
(g_R) Gust Coefficient	4.36152676	Equation 6-9, g_R = (SQRT((2*(LN(3600*n_1))))+(0.577/(SQRT((2*LN(3600*n_1))))))
(G_f) Gust Factor	0.8388954	Equation 6-8, G_f = 0.925*((1+(1.7*I_z*(SQRT(((g_q)^2)*((Q)^2)+((g_R)^2)*((R)^2)))))/(1+(1.7*g_v*I_z
Flexible Structures E-W		
*Exposure B, Table 6-2		
(B) Perpendicular to Wind	102.67	
(L) Parallel to Wind	268.33	
(Q)	0.8729702	
(η_B)	14.2323744	
(η_L)	124.527686	
(R_B)	0.06779395	
(R_L)	0.0079981	
(R)	0.04158559	
(G_f)	0.863897	
(Cp) Windward	0.8	Code 6.5.11.2, Figure 6-6
(Cp) Leeward N-S	-0.5	Code 6.5.11.2, Figure 6-6, L/B
(Cp) Leeward E-W	-0.3	Code 6.5.11.2, Figure 6-6, L/B
(q_z)*K_z Velocity Pressure	10.6624	Code 6.5.10, Equation 6-15, (qz)*Kz = 0.00256*K_{zt}*K_d*(V^2)*I
(q_h) Velocity Pressure at z	12.4323584	Code 6.5.12.2, Table 6-3, q_h = ((h-C131)/(C132-C131))*(A132-A131)*((qz)*Kz)+(((qz)*Kz)*A
(P_{wz})*q_z N-S	0.69048283	(Pwz)*qz = (Cp Windward)*G
(P_{wz})*q_z E-W	0.67111632	(Pwz)*qz = (Cp Windward)*G_f

Leeward Wind Pressure		
(P_{lh}) N-S	-5.2126522	$P_{lh} = q_h$*(Cp Leeward N-S)*G
(P_{lh}) E-W	-3.21912374	$P_{lh} = q_h$*(Cp Leeward E-W)*G_f
Windward Pressure N-S		
(P_{wz}) 0-15	6.07993738	
(P_{wz}) 20	6.43758076	
(P_{wz}) 25	6.72369546	
(P_{wz}) 30	7.00981016	
(P_{wz}) 40	7.43898221	
(P_{wz}) 50	7.79662559	
(P_{wz}) 60	8.08274029	
(P_{wz}) 70	8.36885499	
Windward Pressure E-W		
(P_{wz}) 0-15	6.25787348	
(P_{wz}) 20	6.62598369	
(P_{wz}) 25	6.92047185	
(P_{wz}) 30	7.21496002	
(P_{wz}) 40	7.65669226	
(P_{wz}) 50	8.02480247	
(P_{wz}) 60	8.31929063	
(P_{wz}) 70	8.6137788	

K_z	q_z	z (ft)
0.85	9.06304	0-15
0.9	9.59616	20
0.94	10.022656	25
0.98	10.449152	30
1.04	11.088896	40
1.09	11.622016	50
1.13	12.048512	60
1.17	12.475008	70

Total Pressure N-S		$P = P_{wz} + P_{lh}$
(P_{tot}) 0-15	11.2925896	
(P_{tot}) 20	11.650233	
(P_{tot}) 25	11.9363477	
(P_{tot}) 30	12.2224624	
(P_{tot}) 40	12.6516344	
(P_{tot}) 50	13.0092778	
(P_{tot}) 60	13.2953925	
(P_{tot}) 70	13.5815072	

Total Pressure E-W		$P = P_{wz} + P_{lh}$
(P_{tot}) 0-15	9.47699723	
(P_{tot}) 20	9.84510743	
(P_{tot}) 25	10.1395956	
(P_{tot}) 30	10.4340838	
(P_{tot}) 40	10.875816	
(P_{tot}) 50	11.2439262	
(P_{tot}) 60	11.5384144	
(P_{tot}) 70	11.8329025	

Leeward Shear N-S	
(B) Perpendicular to Wind	268.33
Shear @ Ground	-9790.97675
Shear @ Floors	-19581.9535
Shear @ Roof	-9091.62127

Leeward Shear E-W	
(B) Perpendicular to Wind	102.67
Shear @ Ground	-2313.55204
Shear @ Floors	-4627.10408
Shear @ Roof	-2148.29833

Winward Shear N-S	
(B) Perpendicular to Wind	268.33
Shear @ 0	11420.0072
Shear @ 1	23492.5862
Shear @ 2	26601.8991
Shear @ 3	28809.1274
Shear @ 4	30517.3301
Shear @ 5	15719.304

Windward Shear E-W	
(B)Perpendicular to Wind	102.67
Shear @ 0	4497.47109
Shear @ 1	9251.94054
Shear @ 2	10476.4621
Shear @ 3	11345.7212
Shear @ 4	12018.4522
Shear @ 5	6190.63668

Seismic Loading Calculations

	Hershey 5	
Seismic Use Group I		Table 9.1.3
Site Classification D		Table 9.4.1.2
Seismic Design Category B		Given in Structural Notes
Number of Stories	5	Mechanical Penthouse is considered as 6th floor
(h) Height	69	From Plans
(L) Building Length	268.33	From Plans
(B) Building Width	102.67	From Plans
Roof Dead Load	15	See Calculations
Snow Load	21	See Calculations
Floor Dead Load	70	See Calculations
Exterior Wall Load	30	Assumed
(S_s)	0.23	Figure 9.4.1.1(a)
(S_1)	0.07	Figure 9.4.1.1(b)
(F_a)	1.6	Table 9.4.1.2.4(a) Site Classification D
(F_v)	2.4	Table 9.4.1.2.4(b) Site Classification D
(S_{MS})	0.368	$S_{MS} = S_s*F_a$
(S_{M1})	0.168	$S_{M1} = S_1*F_v$
(S_{DS})	0.2453333	$S_{DS} = (2/3)*S_{MS}$
(S_{D1})	0.112	$S_{D1} = (2/3)*S_{M1}$
(w_{roof})	662.50927	$w_{roof} = ((L*B*DL_{roof})+(6*WL_{ext}*((2*L)+(2*B)))+(0.2*L*B*SL))/1000$
(w_{floors})	2195.5809	$w_{floors} = ((L*B*DL_{floor})+(12*WL_{ext}*((2*L)+(2*B))))/1000$
(W)	9444.8328	$W = w_{roof} +((5 -1)*w_{floors})$
(R)	3	Table 9.5.2.2, Ordinary Composite Moment Frame
(I)	1	Table 9.1.4, Seismic Use Group I
(T_a) N-S	0.8283947	Table 9.5.5.3.2, T_a (N-S) = $C_t*((h)^\wedge(x))$, Steel Moment Frame
(T_a) E-W	0.8283947	Table 9.5.5.3.2, T_a (E-W) = $C_t*((h)^\wedge(x))$, Steel Moment Frame
(C_s)	0.0817778	$C_s = (S_{DS})/(R/I)$
($C_{s\ max}$) N-S	0.0450671	$C_{s\ max}$ (N-S) = $(S_{D1})/(T*(R/I))$
($C_{s\ max}$) E-W	0.0450671	$C_{s\ max}$ (E-W) = $(S_{D1})/(T*(R/I))$
($C_{s\ min}$)	0.0107947	$C_{s\ min} = 0.044*(I)*(S_{DS})$
(V) N-S	425.65108	V (N-S) = C_s*W
(V) E-W	425.65108	V (E-W) = C_s*W
(k) N-S	1.1641973	k (N-S) = 1+((T-0.5)/2)
(k) E-W	1.1641973	k (N-S) = 1+((T-0.5)/2)

Hershey 5					
Vertical Distribution N-S					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Hershey 5					
Vertical Distribution E-W					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Spot Check Analysis

AE Senior Thesis | Spot Check | Technical Assignment 1

First Solve the loading for a typical Office Floor:

Dead Load = 70 psf (See Loading for specifics)

Live Load = 100 psf (Main Floor)

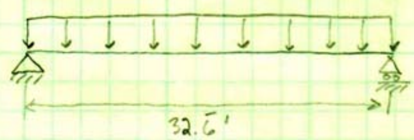
Use Live Load Reduction → $L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

$A_{TOT} = (9.\bar{3}')(32.\bar{6}') = 304.\bar{8}\ ft^2$, $A_I = 2A_T = 2(304.\bar{8}) = 609.\bar{7}\ ft^2$

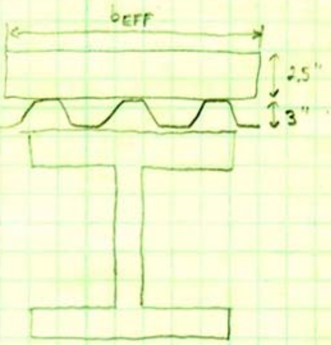
$L = (100)\left(0.25 + \frac{15}{\sqrt{609.\bar{7}}}\right) = 85.74\ psf$

Using Load Factors: $1.2DL + 1.6LL = 1.2(70) + 1.6(85.74) = 221.84_{psf}$

$P_u = 221.84\ psf$, $\omega_u = (9.\bar{3}')(221.84) = 2.06\ klf$



Typical Beam Calculation



Lightweight (115pcf)

Given: $f'_c = 4ksi$ & $f_y = 50ksi$

From Above, $\omega_u = 2.06\ klf$

$M_u = \frac{\omega_u l^2}{8} = \frac{(2.06)(32.\bar{6}')^2}{8} = 274.78'k$

Assume $a = 1"$, $b_{EFF} = \begin{cases} l_n = 112" \\ \frac{(32.\bar{6})}{4}(12) = \boxed{98"} \end{cases}$

$Y_2 = 5.5" - \frac{a}{2} = 5"$

Use LRFD Table 5-14, Try W18×40 where $\phi_b M_p = 294'k$

PNA @7 for $Y_2 = 5"$ → 400 so $\phi_b M_p = 400'k$ & $\Sigma Q_n = 148k$

$\Sigma Q_n = 0.85 f'_c b a \rightarrow a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{148}{0.85(4)(98)} = 0.444$

$Y_2 = 5.5'' - \frac{0.444}{2} = 5.28'' \rightarrow \phi_b M_N = 403'k$ by interpolation

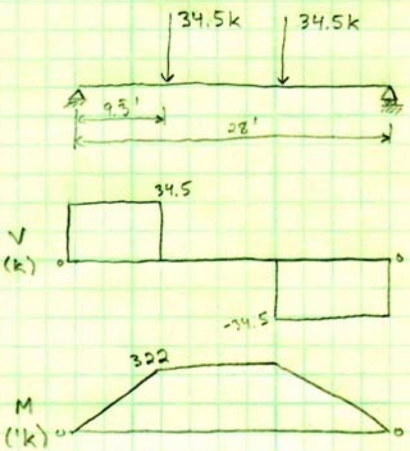
From the Structural notes, Minimum stud capacity is 9.7k

$\frac{\Sigma Q_n}{\text{Stud capacity}} = \frac{148}{9.7} = 15.26 \rightarrow$ Use 32 shear studs

Overall Design W18×40 with 32 shear studs

The beam specified in the drawings is W16×31. I believe my beam is larger because an older code was used for factoring and there is a perscribed camber of $c = \frac{3}{4}''$ for the given design.

Typical Girder Calculation



$P = \frac{\omega_u l}{2} = \frac{(2.06)(32.\bar{6})}{2} = 33.6k$

$M_u = (33.6)(9.\bar{3}) = 313.6'k$

Assume $a = 1''$, $b_{EFF} = \begin{cases} l_n = 112'' \\ \frac{(28)}{4}(12) = \boxed{84''} \end{cases}$

$Y_2 = 5.5'' - \frac{a}{2} = 5''$, LRFD Table 5-14

Try W21×44 where $\phi_b M_p = 358'k$

$A_s = 13.0 in^2$, $\Sigma Q_n = A_s f_y = (50)(13) = 650k$

Find $a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{650}{0.85(4)(84)} = 2.28$, $Y_2 = 5.5'' - \frac{2.28}{2} = 4.36''$

For PNA @ 7 + $Y_2 = 4.36''$ → $\phi_b M_p = 481'k$ by interpolation

For shear studs: $\Sigma Q_n = 163k$ & capacity is given as 33k

$$\frac{\Sigma Q_n}{\text{stud capacity}} = \frac{163}{33} = 4.9 \rightarrow \text{Use 10 shear studs}$$

Overall Design W21x44 with 10 studs

The girder specified in the drawings is given as W21x44. The design also calls for W21x50 Girders and this increase in size can be attributed to the load of the beams framing into the member.

Typical Column Design

Dead Load = $70psf + \overset{\text{Self weight}}{5psf} = 75psf$ (Office Design)

Live Load = 100psf (Main Floor)

Use Live Load Reduction → $L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$ where

Column Tributary Area = $(28')(32.\bar{6}') = 914.\bar{6}\ ft^2$

$A_T = (4\ \text{floors})(914.\bar{6}) = 3658.\bar{6}\ ft^2$, $A_I = 4A_T = 4(3658.\bar{6}) = 14,634.\bar{6}\ ft^2$

Reduction Factor $= \left(0.25 + \frac{15}{\sqrt{14,634.\bar{6}}}\right) = 0.374 \not> 0.4$ for multiple story buildings, $L = (100)(0.4) = 40psf$, Factor the loading

$1.2DL + 1.6LL = 1.2(75) + 1.6(40) = 154psf$, $P_u = 154psf$,

$P_{FLOOR} = (154)(3658.\bar{6}) = 563.43K$, $w_u = 2.06 + \underset{\text{Self Weight}}{(0.05)}(1.2) = 2.12\ klf$

$M_u = \frac{wl^2}{12} = \frac{(2.12)(28)^2}{12} = 138.51'k$, Calculate for the wall

Wall Load = (28')(55') = 1540 ft² where wall height = 69'-14' = 55'

Assume Wall Load of 30 psf, $P_{wall} = (1540)(30) = 46.2k$

$P_u = P_{floors} + P_{wall} = 563.43k + 42.6k = 606.03k$

Using $M_u = 138.51'k$ & $P_u = 606.03k$, Try W14 shape

$P_{eff} = P_u + \alpha M_u$ where $\alpha = \frac{24}{d} = \frac{24}{14} = 1.71$, $P_{EFF} = 606.03 + (1.71)(138.51)$

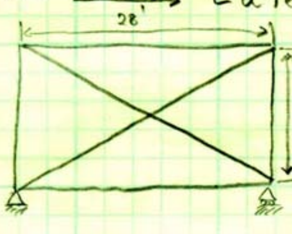
$P_{EFF} = 842.88k$ Assume Pinned connection, K = 1 (conservative)

KL = 14', From LRFD Table 4-2, Try W14×90

969 k > 842.88k ALLOW, Overall Design is W14×90

The column specified in the drawings is listed as a W14×120. The differences can be attributed possibly to not considering a heavy enough exterior wall load or a smaller live load reduction used by the designer.

Base Shear: $\xrightarrow{425k}$ Lateral Force Check



Use the Given member, $L6\times6\times\frac{5}{16}$

$A_g = \frac{425}{50} = 8.5 in^2$, Use LRFD Table 1-7

Choose $L6\times6\times\frac{7}{8} \rightarrow A_g = 9.75 in^2$

The designed member is not adequate due to the large calculated Seismic Force (Critical Force)