Technical Report I

2011-2012 AE Senior Thesis

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Table of Contents

Executive Summary

The following technical report summarizes the existing conditions and design concepts present at the UPMC Hamot Women's Hospital. Structural plans were provided by Atlantic Engineering Services. All other plans were provided by Rectenwald Architects Inc. The existing conditions were closely examined and then analyzed using the IBC 2006 building code, which is the design code enforced on the building at its time of construction.

Wind and Seismic loads were calculated using ASCE 7-05. Wind from the North was considered over wind from the South due to a 60' tall 2-D Escarpment present at the base of the North wall. The base shears and overturning moments were determined based on the pressure values attained. A base shear and over turning moment of 1040.3 k and 40230.8 ft-k was determined for the wind from the North, while a base shear and overturning moment of 435.9 k and 18927.2 ft-k was determined for the East or West wind. The seismic load was determined and is the same for both directions due to the building utilizing the same lateral system in both directions. The seismic forces produced a base shear and overturning moment of 278.5 k and 16163.6 ft-k; thus leading to wind controlling the design of the lateral system for both directions. This is likely the case due to the Exposure D classification required by ASCE 7-05, which is due to the buildings close proximity to Lake Erie.

Spot calculations of the gravity structure were also done in order to determine the accuracy of the gravity loads and how conservative the Engineer of Record was in the building design. The calculations proved the members to be adequate for all strength and serviceability concerns, thus meaning that the loads calculated by the other were close to the loads used by the Engineer of Record. The largest concern arose when the calculation of the maximum column load was calculated and compared to the load the column should see. The column appeared to be over designed by 275%, after discussing this with the Engineer of Record and thought on how the hand calculations performed differ from those of the computer it was found that the Engineer of Record imposes a self-limit of 80% stress on all columns, as well as the hand calculations omitted live loading combinations for simplicity.

The intent of this process was to determine how the various structural components behaved as a structural system.

Introduction

Located on the bay the shoreline of Lake Erie, 201 State Street, which will be referred to as UPMC Hamot Women's Hospital, is a 5 story, steel framed healthcare and hospital facility. This site is centrally located on the UPMC Hamot campus, directly between the UPMC Hamot Main Hospital and the UPMC Hamot Heart Institute.

The 163,616 sq. ft. Women's Hospital was completed in early January of 2011. This structure has a very unique history, originally the hospital wanted a four story building, but only had the financing for two levels. Thus the structure was designed for four stories, but only the first two were constructed. Then the hospital decided that a five story structure more suited their needs, so the building was stripped down to the shell (structural steel and floor slabs), the current roof slab was then removed with the columns being truncated 4'-0" above the second story slab. The decision was made to reinforce the columns and beams below this point, as needed, and to build to the desired five stories above.

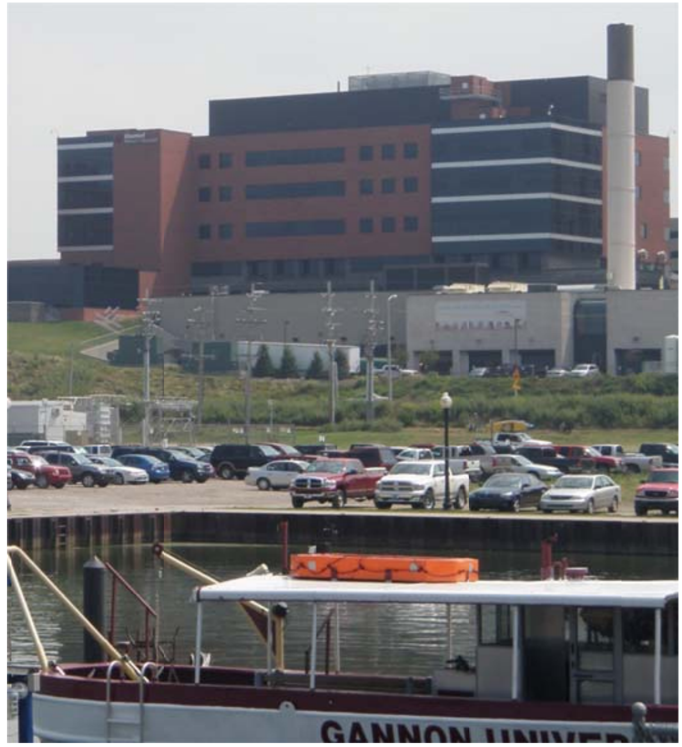


Figure 1: North Façade, Showing 2-D Escarpment

The city of Erie zoned the UPMC Hamot campus as Waterfront Commercial 2 (W-C2), which permits residential, commercial, recreational, and historical uses. This zoning is similar to Waterfront Commercial (W-C), except that this area permits Group Care Facilities. The maximum building height in this zoning district is 100 ft, with a building footprint not greater than 65% of the lot. The exterior lighting of the building must prevent glare to adjoining properties and the lot is required to have 1 parking space per 4 beds.

The five stories of the UPMC Hamot Women's Hospital are topped with a mechanical penthouse that does not cover the entire building footprint. This penthouse houses three air handling units that supply conditioned air to all areas of the building. This is achieved via a large mechanical opening in each floor; this opening is located on the west side of the building and measures approximately 27'-0"± by 30'-0"±.

The UPMC Hamot Women's Hospital was designed to match the Architectural style of the other buildings on the Hamot Medical Center campus. This includes a brick and glass façade that

Figure 2: Interior Water Wall

attempts to allow plenty of natural light into the building without being uncomfortable to the patients. The interior of the building is definitely what most would consider being upscale. The owner of the building was not primarily concerned about cost, but rather wanted the building to put the patients at ease by making them feel as if they were at home. This is primarily achieved through earth tone colors throughout the interior and is driven home with the water wall located in the lobby and the cabinets in every room to hide the hoses and cables that are typical of a hospital; as well as each room being equipped with a Jacuzzi and a very luxurious bathroom, again to achieve a relaxing environment for the patients.

UPMC Hamot Women's Hospital has an exterior façade of 4" nominal face brick, a 3" air space, 1" of rigid insulation, on 6" nominal metal studs w/ R-19 batt insulation filing the wall core. The wall is then closed with 5/8" gypsum wall board. Where applicable the wall system is double pane insulated glass windows. The roof system is EPDM roofing on protection board on polyisocyanurate insulation.

Figure 3: Exterior Building Façade

Structural System

• Foundation

The foundation is unique in that many of the existing foundations also had to increase in size when the building increased in height. The foundation system utilizes both strip and spread footings. The strip footings are typically 2'-0" wide and 1'-0" deep; reinforcement consists of 3-#5 longitudinally and #5 x 1'-6" @ 12" O.C. transverse. The spread footings are the most unique because many of the existing spread footings had to be increased a length, width, and depth. The minimum height of the footings below grade is 3'-6". The typical foundation overbuild details can be found on sheet S403.

Figure 4: Foundation Excavion during Construction

• Floor Construction

The beams are typically W shapes that tend to be framed with the girders spanning the short direction and the beams framing the long direction of the bay. The beams are typically W14x22 composite beams, where concrete slab on deck exists. In the shorter spans (12'-4") the beams become W8x10, and when the tributary spacing is decreased they tend to become W12x19 composite beams. Elsewhere the beams are non-composite. The girders are also composite where applicable.

The elevated floor slabs have a total thickness of 6", consisting of 4" of lightweight 4000 psi concrete on a 2" – 20 GA composite metal deck. These slabs are reinforced with 6x6 – W1.4xW1.4 welded wire fabric.

• Lateral System

The lateral system in the N-S direction consists of a 5 story (6 with penthouse), 49' long braced frame along column line N. This is the alone full height braced frame in the building. The N-S direction also has a full height 42'-8" long moment frame along column line B. The E-W direction utilizes full height moment frames along column line 1 and 17, which are 161' and 173'-4" long, respectively. The columns are spliced 4'-0" above the second floor, where the existing shell remained and was reinforced below. The columns are also spliced at above the 4th floor, at the same 4'-0" elevation. The unique construction sequence has led to the need to reinforce the base of these columns dramatically, especially in the moment frames. The details of these reinforcements can be seen on sheet S400. The column sizes vary from W8 sizes to W14 sizes. The lateral system of the mechanical penthouse is entirely braced frames.

Design Codes & Standards

2006 International Building Code (IBC 2006) with Local Amendments

2006 International Mechanical Code (IMC 2006) with Local Amendments

2006 International Electrical Code (IEC 2006) with Local Amendments

2006 International Fire Code (IFC 2006) with Local Amendments

Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)

Building Code Requirements for Structural Concrete (ACI 318-08)

Building Code Requirements for Masonry Structures (ACI 530)

AISC Manual of Steel Construction, Allowable Stress Design (ASD)

Structural Materials

Structural Steel		
Type	**Standard**	**Grade**
W-Shape Structural Steel	**ASTM A572**	**50**
Hollow Structural Sections (HSS)	**ASTM A500**	**C**
Bars, Plates and Angles	**ASTM A36**	**N/A**
Bolts, Washers, and Nuts	**ASTM A325**	**N/A**

Concrete		
Usage	**Weight**	**Strength**
Footings	**Normal**	**3000 psi**
Slab-on-Grade	**Normal**	**4000 psi**
Concrete on Steel Deck	**Lightweight**	**4000 psi**

Building Loads

Part of this technical report will incorporate the calculation of both gravity and lateral loads. The gravity loads will consist of dead, live, and snow loads. The lateral loads will be analyzed through wind and seismic loading. The intent of this aspect of the report is to lay the groundwork for remainder of this thesis project, as well as begin to determine how conservative the primary designer may or may not have been.

- ## Dead Load

Dead loads were calculated using the most recent data available through the Vulcraft Corporation. Typical floor weight was found to be 59 psf, although allowing for some unknowns a superimposed dead load was decided to be used, which is conservative; thus leaving a typical floor dead load of 69 psf. The roof dead load was also calculated using the Vulcraft Corporation manuals, and the roof dead load was determined to be 15 psf. To be conservative a roof dead load of 20 psf will be used, allowing for future roof coverings to be laid on the initial roof. Appendix A includes the appropriate figures from the Vulcraft Manuals used, as well as detailed calculations for the typical floor and roof dead load.

- ## Live Load

Live Loads were calculated in accordance with IBC 2006 using ASCE 7-05 (Minimum Design Loads for Buildings and Other Structures). The relevant loads derived are tabulated in Table 1 and in Appendix A.

ASCE 7-05 Live Loads	
Space	**Load (psf)**
Lobbies	**100**
First Floor Corridors	**100**
Offices	**50**
Stairs	**100**
Mechanical	**150**
Roof	**20**
Hospitals	
Operating Rooms/Labs	**60**
Patient Rooms	**40**
Corridors, above First Floor	**80**

Table 1: ASCE 7-05 Live Loads

- ## Snow Load

Snow loads were calculated using the procedure outlined in ASCE 7-05 Chapter 7. The city of Erie, PA falls into an area requiring a Case Study (CS) of the ground snow load. A call to the Erie Building Code Official yielded a local requirement for designers to use a ground snow load of 40 psf. The Snow Load Calculations are summarized in Table 2 and detailed calculations are available in Appendix B. Several locations were determined to be potential drift locations, located around the Mechanical Penthouse and the Stair Pop-out. The Mechanical Penthouse yielded a peak drift load of 106.2 psf with a width of 17'-0". The Stair Pop-Out yielded a peak drift load of 58.2 psf with a width of 7'-0". A roof floor plan with mark-ups of the applicable snow drift areas is available in Appendix B.

ASCE 7-05 Snow Loads	
Variable	**Value**
Ground Snow Load, p_g (psf)	**40**
Temperature Factor, C_t	**1.0**
Exposure Factor, C_e	**0.8**
Importance Factor, I_s	**1.1**
Flat Roof Snow Load, p_f (psf)	**24.64**

Table 2: ASCE 7-05 Snow Loads

- ## Wind Load

Wind loads were calculated in accordance with Chapter 6 of ASCE 7-05, Method 2 Main Wind Force Resisting System (MWFRS). In order to use this procedure a few minor simplifications had to be made, such as reducing the five different building heights to three. This was done by taking two of the minor pop-outs (< 5 ft) and simplifying them into the main roof.

The wind loading for this building is very unique and interesting. The building sits on the peak of a 60 ft tall 2-D escarpment, as described in ASCE 7-05. This produces an atypical wind loading pattern in the North-South Direction. This problem is compounded by the building being located on the bay of Lake Erie, this flat open body of water allows for wind velocities to increase rapidly. This leads to a very large wind load at the base of the North wall of the building.

Wind loads on the building are collected by the exterior façade and distributed to the slab, at which point the slab will distribute the forces to the MWFRS, based on the stiffness and location of the various structural elements.

The user should note that the internal pressures are not added to the external windward and leeward pressures. This is due to the fact that the internal pressures effectively cancel themselves out. This has been done in this report as is standard practice in structural engineering.

The wind pressures that engage the North-South lateral system was analyzed as a wind coming from the North. This is due to the large 2-D escarpment located on that side of the building. The wind pressures engage the East-West lateral system was analyzed as a wind coming from the East, although the wind coming from the West would be identical.

Details pertaining to the wind calculations can be found in Appendix C, while a summary of the final wind pressures can be found in Table 3 and Table 4, for a pictorial view of how these pressures are applied to the building see Figure 5 and Figure 6.

ASCE 7-05 Wind Pressures – N-S Direction		
Type	**Height**	**Wind Pressure (psf)**
Windward Walls	0’-15’	59.51
	15’-20’	39.39
	20’-25’	36.35
	25’-30’	34.03
	30’-40’	32.76
	40’-50’	29.87
	50’-60’	28.13
	60’-70’	26.98
	70’-80’	26.40
	80’-90’	26.03
	90’-92’	25.71
Leeward Walls	Full Height	-15.55

Table 3: ASCE 7-05 Wind Pressures in N-S Direction

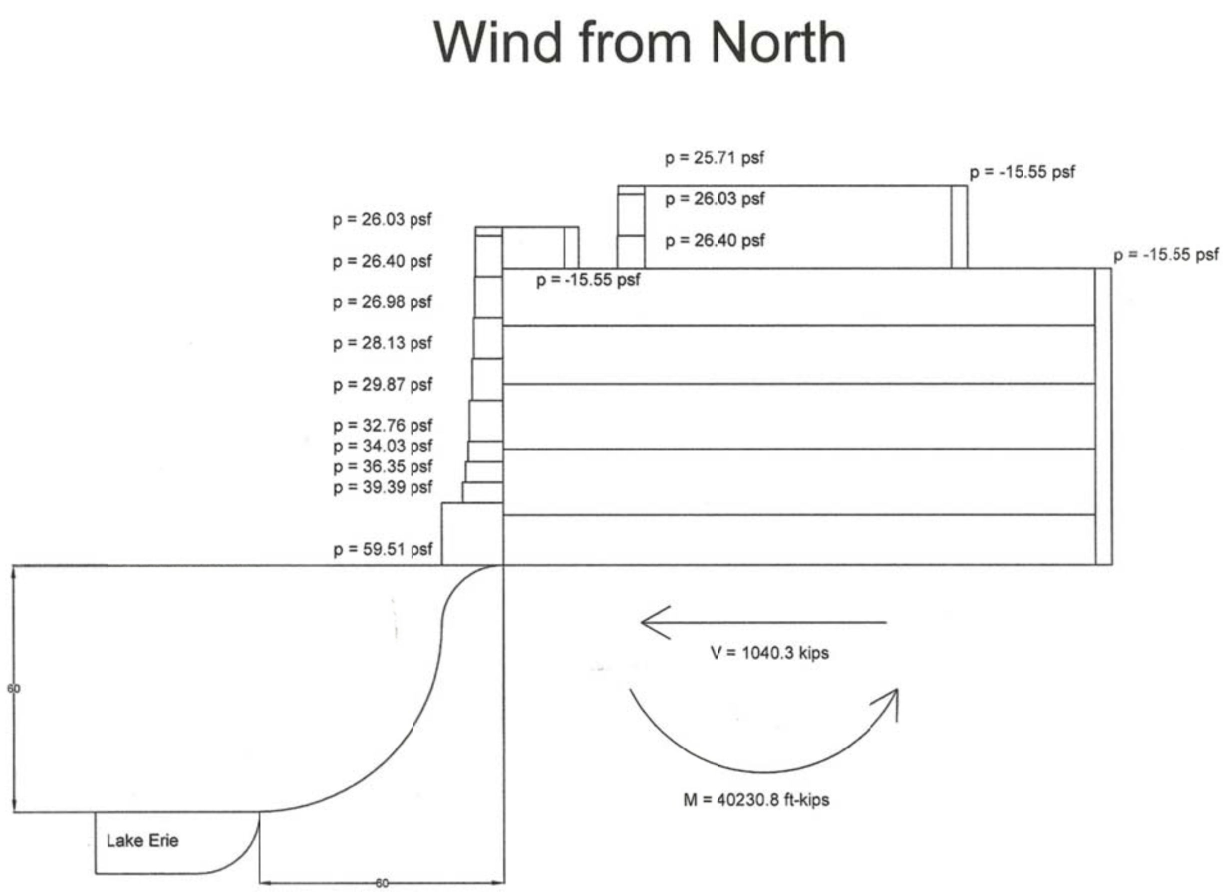


Figure 5: Wind Pressures in N-S Direction, showing 2-D Escarpment

ASCE 7-05 Wind Pressures –E-W Direction		
Type	**Height**	**Wind Pressure (psf)**
Windward Walls	0'-15'	19.20
	15'-20'	19.88
	20'-25'	20.43
	25'-30'	20.99
	30'-40'	21.82
	40'-50'	22.50
	50'-60'	23.05
	60'-70'	23.47
	70'-80'	24.16
	80'-90'	24.44
	90'-92'	24.58
Leeward Walls	Full Height	-14.13

Table 4: ASCE 7-05 Wind Pressures in E-W Direction

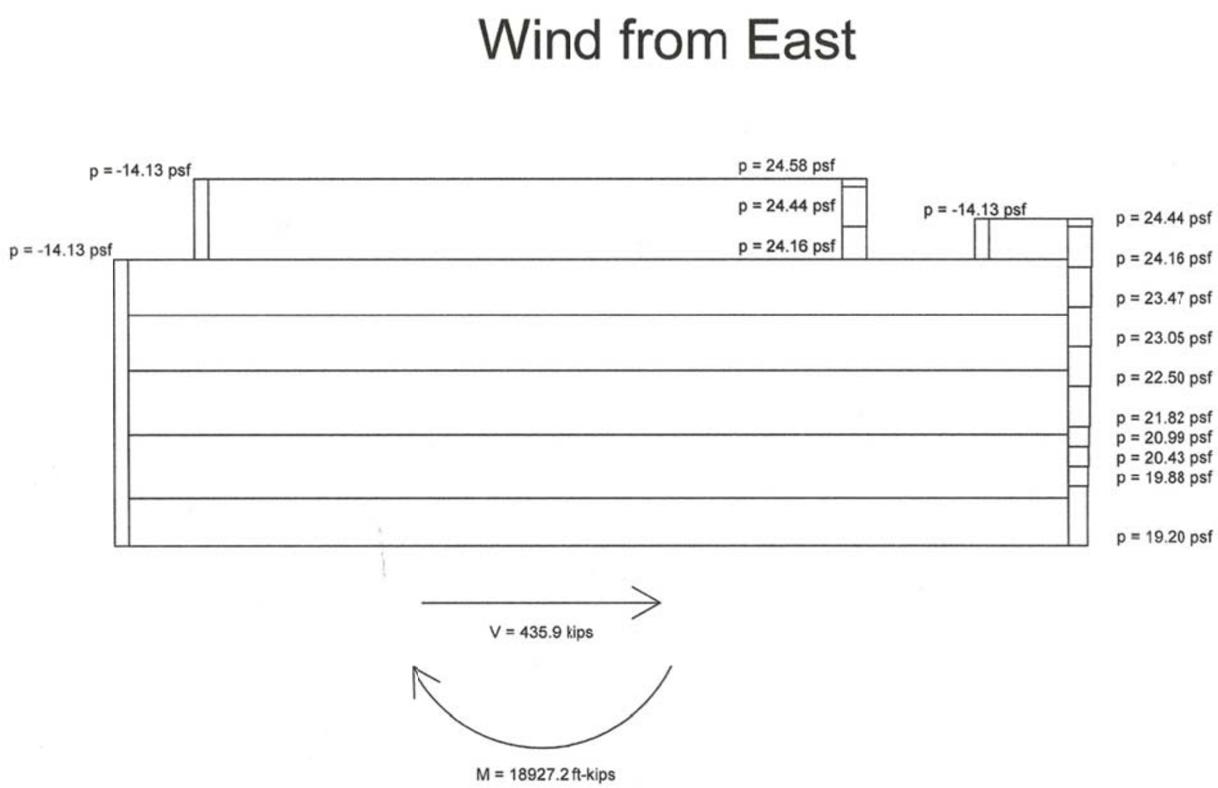


Figure 6: Wind Pressures in E-W Direction

- ## Seismic Load

Seismic loads were calculated as required by ASCE 7-05, Chapter 11 and 12. This section requires the use of the Equivalent Lateral Force Procedure. For this analysis an R-Factor of 3 was chosen, meaning the building is "not specifically detailed for seismic loads".

Seismic loads tend to be very complicated in nature, due to the fact that no two earthquakes are ever the same. This leads to many engineering simplifications within the code to allow us to analyze the structure quickly and efficiently. Wind loads are easier to quantify because it acts as a pressure on the building. Earthquake loads are more difficult to quantify because the loading comes through the motion of the ground. ASCE 7-05 assists the structural engineer by providing a procedure that allows for the complicated loading to be turned into forces applied at the various levels. The overall base shear of the building is controlled by many factors, although the inertial mass of the building can be singled out as one of the most important factors. The mass and height of each level leads to how much of the overall base shear we can apply to that respective level.

Several assumptions had to be made in order to use the Equivalent Force Method in ASCE 7-05. The first assumption is that the mass of each story is lumped at that story level. This is not an outrageous assumption because the majority of a stories mass is located in the slab and beams attributed to that story. The mass associated with columns spanning between levels were divided to the stories above and below based on tributary height between the levels, giving half of the columns mass to the level above and half to the level below. The other major assumption is that the building utilizes a rigid diaphragm. This is a reasonable assumption due to the relative rigidity of the slab compared to that of the lateral system. This is also reasonable due to the absence of shear walls, if shear walls were present as a lateral system in this structure the interaction between the slab and the walls would have to be carefully analyzed and detailed to transfer the large loads that the shear walls would take.

Details pertaining to the seismic calculations can be found in Appendix D, while a summary of the final seismic forces can be found in Table 5, for a pictorial view of the forces being applied at the various story levels see Figure 7.

ASCE 7-05 Seismic Calculations			
Level	**Level Weight (kips)**	**Level Height**	**EQ Force (kips)**
Penthouse	315.4	92’-0”	17.24
Stair Roof	74.3	82’-0”	3.41
Roof	1616.0	72’-0”	60.77
5th Floor	2282.7	58’-0”	61.71
4th Floor	2348.6	44’-0”	41.64
3rd Floor	2401.9	28’-0”	21.36
2nd Floor	2567.1	12’-0”	6.26
Ground Floor	N/A	0’-0”	0

Table 5: ASCE 7-05 Seismic Calculations

Earthquake Forces

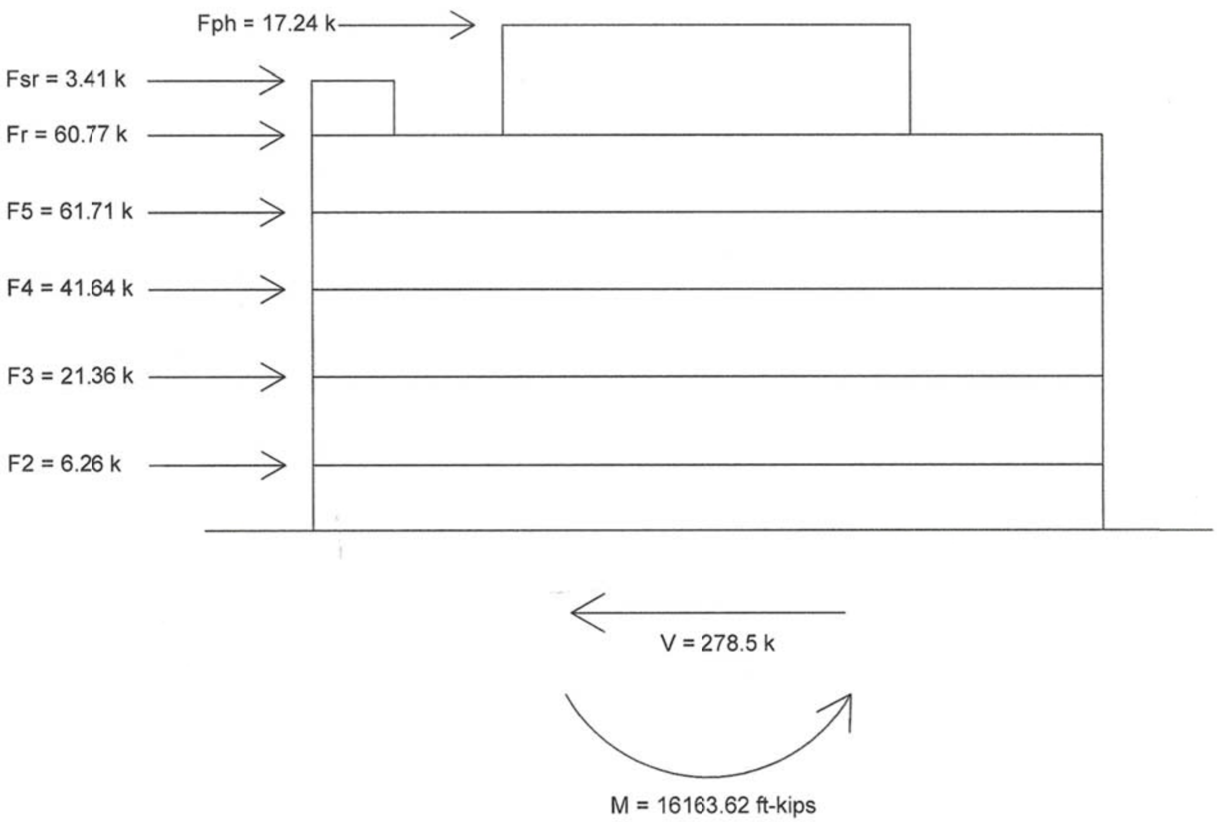


Figure 7: Earthquake Forces at Various Levels

Gravity Load Spot Checks

As part of this technical report spot checks of the existing structure were performed. The purpose of these spot checks is two-fold, to determine that the gravity loads calculated by the author of this report are similar to those used by the Engineer of Record, and to try and allow the author of this report to quantify how conservative the Engineer of Record was in their calculations. Columns M-7, M-5, L-7, and L-5 were chosen to represent the typical bay. This bay was chosen because of the uniformity of the bay mentioned and the adjacent bays in all directions. Complete hand calculations for the members being analyzed below are available in Appendix E; please refer to Appendix A for the calculations of the loading these members were subjected to.

• Decking

The typical floor slab at UPMC Hamot Women's Hospital consists of a2" 20 gage steel deck with 4" of lightweight concrete. Using the Vulcraft Steel deck manual, enclosed in Appendix E, the determination of maximum capacity of the slab was determined. Then the maximum unshored clear span was checked versus the allowable. It was determined that the slab had 275% more capacity then was needed to carry the applied loads, and that the spacing was well within the maximum. Upon further investigation it was determined that the slab was chosen to be a 4" concrete on 2" deck due to the minimum fire rating as specified by code.

• Beam and Girder

The typical composite beam was chosen to be a W14x22[10], this loading on the beam was determined based on a tributary width analysis and the maximum factored shear and moment were determined. The capacity available was then determined based on AISC Steel Manual procedure. Finally the beam was checked to ensure that it passed live load deflection criteria, and unshored wet concrete deflections under construction loading. As is typical the controlling limits of the beam were the live load deflection and the unshored wet concrete deflection.

The typical composite girder was chosen to be a W16x26[18], this loading on the beam was determined based on a tributary width analysis and the maximum factored shear and moment were determined. The capacity available was then determined based on AISC Steel Manual procedure. Finally the beam was checked to ensure that it passed live load deflection criteria, and unshored wet concrete deflections under construction loading. As is typical the controlling limits of the beam were the live load deflection and the unshored wet concrete deflection.

The hand calculations for both the beam and the girder are available in Appendix E, both the beam and the girder were determined to work fine under the applied loading. It appears that the Engineer of Record was not overly conservative with this aspect of the design.

• Column

The typical column was chosen as the column along column L-5, the loading was determined based on a tributary area analysis, with live load reduction being done on all levels except the roof. The K_{LL} value for the live load reduction was determined to be 4, by the methods described in the AISC Steel Manual. The column selected is spliced above the 2nd floor and the 4th floor; thus leading to the need to check the column for 3 different sizes, one of which was an existing 8WF67. The properties for this older beam were determined based on the AISC shape database CD (supplied with AISC Manual 13 ed.).

The column loading vs. column capacity was found to be drastically conservative. This can be explained through various sources. Primarily the Engineer of Record does not allow his columns to be stressed over 80%; thus leading to an already conservative column. Then due to simplifying for the use of hand calculations various live load patterns were not explored. Exploring these combinations would add some inherent moment into the column, thus taking away more of the available capacity. Exploring all of these combinations just is not possible without the use of a sophisticated structural analysis program.

Conclusion

Through this report and the exploration of the various structural systems employed by the Engineer of Record a greater understanding of the UPMC Women's Hospital and the ASCE 7-05 code was developed. Through the various spot checks a greater understanding of the existing structural conditions was determined and a greater understanding of the various design guidelines the Engineer of Record used in design. These checks were determined to be adequate for both strength and serviceability.

The redesign phase of this thesis could include a comparison of the ASCE 7-05 building code and the ASCE 7-10 building code. The UPMC Hamot Women's Hospital was subjected to the ASCE 7-05 code, but the newer ASCE 7-10 code changed the wind loading dramatically. Due to its unique location I feel this would be a great chance to examine the changes of the building code.

Appendix A: Gravity Load Calculations

A.1 – Dead Load Calculations

Dead Loads

Second Floor (Existing) Slab is 3¼" on 2" – 20 GA Composite Deck; Normal Weight or Lightweight Concrete ⇒ Unknown

∴ Use Self-Weight for all slabs as 4" LW Conc. on 2" – 20 GA Composite Deck

Total Slab Thickness = 6"

Theoretical Concrete Volume $= 0.417\ ft^3/ft^2 \times 110\ lb/ft^3 = 46\ lb/ft^2$

Deck Weight = 2 psf

Total Slab Weight	= 48 psf
MEP	= 5 psf
Ceiling/Lights/Floor	= 6 psf
	59 psf
Superimposed DL	10 psf
	69 psf = Total Floor DL

Roof Weight

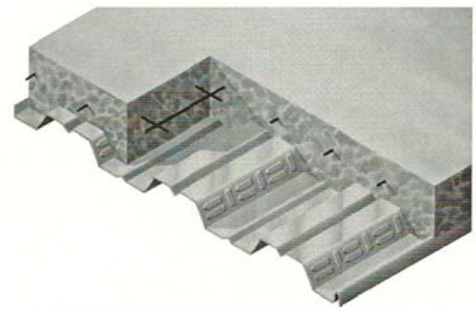
1½" Galvanized Steel Roof Deck – 20 GA = 2 psf
↳ Wide Rib Deck

Roofing	3 psf
Insulation	5 psf
Ceiling/MEP	5 psf
	15 psf

∴ Use 20 psf total
↳ Includes 5 psf Superimposed DL

A.2 – Vulcraft Manual Page for 2VLI Decks

VULCRAFT



SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd³ / 100 ft²	Theo. Concrete Volume ft³ / ft²	Recommended Welded Wire Fabric
4	0.93	0.250	6x6 - W1.4xW1.4
4 1/2	1.08	0.292	6x6 - W1.4xW1.4
5	1.23	0.333	6x6 - W1.4xW1.4
5 1/4	1.31	0.354	6x6 - W1.4xW1.4
5 1/2	1.39	0.375	6x6 - W2.1xW2.1
6	1.54	0.417	6x6 - W2.1xW2.1
6 1/4	1.62	0.438	6x6 - W2.1xW2.1
6 1/2	1.70	0.458	6x6 - W2.1xW2.1

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0
	2VLI22	8'-1	10'-3	10'-7	238	209	186	167	152	120	108	98	90	82	75	69	64	59	55
4.00	2VLI20	9'-6	11'-8	12'-1	268	235	209	187	169	153	140	129	101	92	84	78	72	66	61
(t=2.00)	2VLI19	10'-10	13'-0	13'-2	297	260	230	206	185	168	153	141	130	121	93	86	79	73	68
30 PSF	2VLI18	11'-7	13'-7	13'-7	324	285	253	227	205	187	171	158	146	136	127	119	92	86	80
	2VLI16	12'-3	14'-3	14'-4	377	330	292	261	235	214	195	179	165	153	143	133	118	98	91
	2VLI22	7'-8	9'-10	10'-2	276	243	216	194	155	139	126	114	104	96	88	81	75	69	64
4.50	2VLI20	9'-0	11'-3	11'-7	312	273	243	217	196	178	163	128	117	107	98	90	83	77	72
(t=2.50)	2VLI19	10'-3	12'-5	12'-9	346	302	268	239	215	195	178	164	151	118	108	100	92	85	79
35 PSF	2VLI18	11'-2	13'-1	13'-1	376	331	294	264	238	217	199	183	170	158	147	116	107	100	93
	2VLI16	11'-7	13'-8	13'-10	400	384	340	303	273	248	227	208	192	178	166	155	123	114	106
	2VLI22	7'-4	9'-5	9'-9	315	277	247	197	176	159	143	130	119	109	100	92	85	79	73
5.00	2VLI20	8'-7	10'-9	11'-2	355	312	276	248	224	203	161	146	133	122	112	103	95	88	82
(t=3.00)	2VLI19	9'-9	11'-11	12'-4	394	345	305	272	245	223	203	187	147	135	124	114	105	97	90
39 PSF	2VLI18	10'-9	12'-9	12'-9	400	377	335	300	272	247	227	209	193	180	143	132	122	114	106
	2VLI16	11'-0	13'-1	13'-5	400	400	387	346	311	283	258	237	219	203	189	151	140	130	121
	2VLI22	7'-2	9'-3	9'-7	334	294	262	209	187	168	152	138	126	116	106	98	90	84	78
5.25	2VLI20	8'-5	10'-7	10'-11	377	331	293	263	237	190	171	155	142	130	119	110	101	94	87
(t=3.25)	2VLI19	9'-6	11'-8	12'-1	400	366	324	289	260	236	216	198	156	143	131	121	111	103	95
42 PSF	2VLI18	10'-6	12'-7	12'-7	400	400	355	319	288	263	241	222	205	191	151	140	130	121	113
	2VLI16	10'-9	12'-10	13'-3	400	400	400	367	330	300	274	252	232	215	173	160	148	138	128
	2VLI22	7'-0	9'-1	9'-5	353	311	277	222	198	178	161	147	134	122	113	104	96	89	82
5.50	2VLI20	8'-3	10'-4	10'-9	399	350	310	278	251	201	181	165	150	137	126	116	107	99	92
(t=3.50)	2VLI19	9'-4	11'-6	11'-10	400	387	342	306	275	250	228	182	165	151	139	128	118	109	101
44 PSF	2VLI18	10'-3	12'-5	12'-5	400	400	376	337	305	278	254	234	217	174	160	148	138	128	119
	2VLI16	10'-6	12'-7	13'-0	400	400	400	388	350	317	290	266	246	228	184	170	157	146	136
	2VLI22	6'-8	8'-7	8'-11	400	362	291	258	231	208	188	171	156	143	131	121	112	103	96
6.25	2VLI20	7'-9	9'-10	10'-2	400	400	361	323	260	234	211	192	175	160	147	135	125	115	107
(t=4.25)	2VLI19	8'-9	10'-11	11'-3	400	400	398	356	320	291	233	212	193	176	162	149	137	127	118
51 PSF	2VLI18	9'-8	11'-10	11'-11	400	400	400	392	355	323	296	273	220	202	187	173	160	149	139
	2VLI16	9'-11	12'-0	12'-5	400	400	400	400	400	369	337	310	253	232	214	198	183	170	158

Notes:
1. Minimum exterior bearing length required is 2.00 inches. Minimum interior bearing length required is 4.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.

COMPOSITE

NUCOR VULCRAFT GROUP

53

A.3 – Vulcraft Manual Page for 1.5B Roof Deck

VULCRAFT

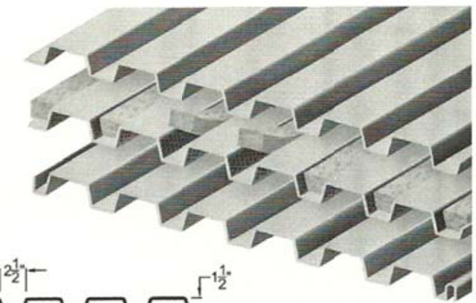
ROOF

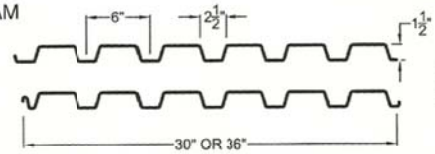
1.5 B, BI, BA, BIA

Maximum Sheet Length 42'-0
Extra charge for lengths under 6'-0
ICC ER-3415
Factory Mutual Approved*
Deck type & gauge — Max. deck span
1.5B22, 1.5BI22.................. 6'-0"
1.5B20, 1.5BI20.................. 6'-6"
1.5B18, 1.5BI18.................. 7'-5"
FM Approvals No. 0C8A7.AM & 0G1A4.AM

1.5B16, 1.5BI16.................. 9'-4"
FM Approvals No. 3029260

* Acoustical Deck is not approved by Factory Mutual





Interlocking side lap is not drawn to show actual detail.

SECTION PROPERTIES

Deck type	Design thickness in.	W psf	Section Properties I_p in⁴/ft	S_p in³/ft	I_n in⁴/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
B24	0.0239	1.46	0.107	0.120	0.135	0.131	2634	60
B22	0.0295	1.78	0.155	0.186	0.183	0.192	1818	33
B20	0.0358	2.14	0.201	0.234	0.222	0.247	2193	33
B19	0.0418	2.49	0.246	0.277	0.260	0.289	2546	33
B18	0.0474	2.82	0.289	0.318	0.295	0.327	2870	33
B16	0.0598	3.54	0.373	0.408	0.373	0.411	3578	33

ACOUSTICAL INFORMATION

Deck Type	Absorption Coefficient 125	250	500	1000	2000	4000	Noise Reduction Coefficient[1]
1.5BA, 1.5BIA	.11	.18	.66	1.02	0.61	0.33	0.60

[1] Source: Riverbank Acoustical Laboratories.
Test was conducted with 1.50 pcf fiberglass batts and 2 inch polyisocyanurate foam insulation for the SDI.

Type B (wide rib) deck provides excellent structural load carrying capacity per pound of steel utilized, and its nestable design eliminates the need for die-set ends.

1" or more rigid insulation is required for Type B deck.

Acoustical deck (Type BA, BIA) is particulary suitable in structures such as auditoriums, schools, and theatres where sound control is desirable. Acoustic perforations are located in the vertical webs where the load carrying properties are negligibly affected (less than 5%).

Inert, non-organic glass fiber sound absorbing batts are placed in the rib openings to absorb up to 60% of the sound striking the deck.

Batts are field installed and may require separation.

VERTICAL LOADS FOR TYPE 1.5B

No. of Spans	Deck Type	Max. SDI Const. Span	Allowable Total (PSF) / Load Causing Deflection of L/240 or 1 inch (PSF) — Span (ft.-in.) ctr to ctr of supports: 5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0	9-6	10-0
1	B24	4'-8	115 / 56	95 / 42	80 / 32	68 / 26	59 / 20	51 / 17	45 / 14	40 / 11	35 / 10	32 / 8	29 / 7
	B22	5'-7	98 / 81	81 / 61	68 / 47	58 / 37	50 / 30	44 / 24	38 / 20	34 / 17	30 / 14	27 / 12	25 / 10
	B20	6'-5	123 / 105	102 / 79	86 / 61	73 / 48	63 / 38	55 / 31	48 / 26	43 / 21	38 / 18	34 / 15	31 / 13
	B19	7'-1	146 / 129	121 / 97	101 / 75	86 / 59	74 / 47	65 / 38	57 / 31	51 / 26	45 / 22	40 / 19	36 / 16
	B18	7'-8	168 / 152	138 / 114	116 / 88	99 / 69	85 / 55	74 / 45	65 / 37	58 / 31	52 / 26	46 / 22	42 / 19
	B16	8'-8	215 / 196	178 / 147	149 / 113	127 / 89	110 / 71	96 / 58	84 / 48	74 / 40	66 / 34	60 / 29	54 / 24
2	B24	5'-10	124 / 153	103 / 115	86 / 88	74 / 70	64 / 56	56 / 45	49 / 37	43 / 31	39 / 26	35 / 22	31 / 19
	B22	6'-11	100 / 213	83 / 160	70 / 124	59 / 97	51 / 78	45 / 63	39 / 52	35 / 43	31 / 37	28 / 31	25 / 27
	B20	7'-9	128 / 267	106 / 201	89 / 155	76 / 122	66 / 97	57 / 79	51 / 65	45 / 54	40 / 46	36 / 39	32 / 33
	B19	8'-5	150 / 320	124 / 240	104 / 185	89 / 145	77 / 116	67 / 95	59 / 78	52 / 65	47 / 55	42 / 47	38 / 40
	B18	9'-1	169 / 369	140 / 277	118 / 213	101 / 168	87 / 134	76 / 109	67 / 90	59 / 75	53 / 63	48 / 54	43 / 46
	B16	10'-3	213 / 471	176 / 354	149 / 273	127 / 214	110 / 172	95 / 140	84 / 115	74 / 96	66 / 81	60 / 69	54 / 59
3	B24	5'-10	154 / 120	128 / 90	108 / 69	92 / 55	79 / 44	69 / 35	61 / 29	54 / 24	48 / 21	43 / 17	39 / 15
	B22	6'-11	124 / 167	103 / 126	87 / 97	74 / 76	64 / 61	56 / 50	49 / 41	43 / 34	39 / 29	35 / 24	31 / 21
	B20	7'-9	159 / 209	132 / 157	111 / 121	95 / 95	82 / 76	72 / 62	63 / 51	56 / 43	50 / 36	45 / 31	40 / 26
	B19	8'-5	186 / 250	154 / 188	130 / 145	111 / 114	96 / 91	84 / 74	74 / 61	65 / 51	58 / 43	52 / 37	47 / 31
	B18	9'-1	210 / 289	174 / 217	147 / 167	126 / 132	108 / 105	95 / 86	83 / 71	74 / 59	66 / 50	59 / 42	54 / 36
	B16	10'-3	264 / 369	219 / 277	185 / 214	158 / 168	136 / 135	119 / 109	105 / 90	93 / 75	83 / 63	74 / 54	67 / 46

Notes: 1. Minimum exterior bearing length required is 1.50 inches. Minimum interior bearing length required is 3.00 inches. If these mininum lengths are not provided, web crippling must be checked.

NUCOR VULCRAFT GROUP

7

A.4 – Live Loads from ASCE 7-05

Live Loads (psf)	ASCE 7-05
Lobbies	100
Hospitals	
Operating Rooms/Labs	60
Patient Rooms	40
Corridors, above First Floor	80
First Floor Corridors	100
Offices	50
Stairs	100
Mechanical	150
Roofs	20

Appendix B: Snow Load & Drift Calculations

B.1 - Snow Load and Drift Calculations

Snow Loads

The city of Erie, PA requires the use of 40 psf for the ground snow load, p_g ⇒ Phone Call 8/31/2011 Scott Heitzenrater

ASCE 7-05

Flat Roof Snow Load

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

$p_g = 40$ psf, see note above

$I = 1.1$ ⇒ Table 7-4 (ASCE 7-05)
→ Occupancy Category II → No Emergency Facilities
↳ Table 1-1 (ASCE 7-05)

$C_t = 1.0$ ⇒ Table 7-3 (ASCE 7-05)

$C_e = 0.8$ ⇒ Table 7-2 (ASCE 7-05)
↳ Fully Exposed
↳ Terrain Category D, on the lake

$p_f = 0.7(0.8)(1.0)(1.1)(40\text{ psf})$

$\boxed{p_f = 24.64\text{ psf}}$

B.2 - Snow Load and Drift Calculations (con't)

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Penthouse Roof)

$\gamma = 0.13\, p_g + 14 = 0.13(40) + 14 = 19.2$ psf

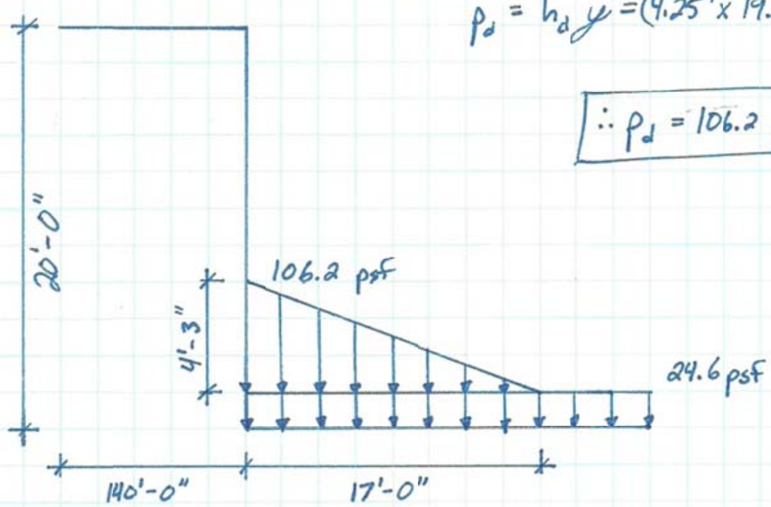
N-S Drift	E-W Drift
$l_u = 60'-0''$	$l_u = 140'-0''$
$h_c = 20'-0''$	$h_c = 20'-0''$
$\therefore h_d = 2.8'$	$\therefore h_d = 4.25'$

$\therefore$ Use $h_d = 4.25'$

$w = 4h_d = 17'-0''$

$p_d = h_d \gamma = (4.25' \times 19.2 \text{ pcf}) + 24.6 = 106.2$ psf

$\boxed{\therefore p_d = 106.2 \text{ psf}}$



B.3 - Snow Load and Drift Calculations (con't)

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Stair Pop-Out)

$\gamma = 0.13 p_g + 14 = 0.13(40) + 14 = 19.2$ pcf

N-S Drift

$l_u = 10'-10''$
$h_c = 10'-0''$

$\therefore h_d = 1.75'$

E-W Drift

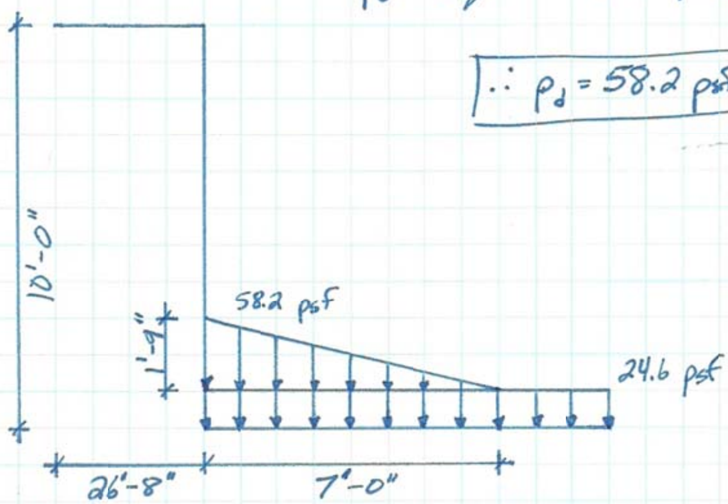
$l_u = 26'-8''$
$h_c = 10'-0''$

$\therefore h_d = 1.75'$

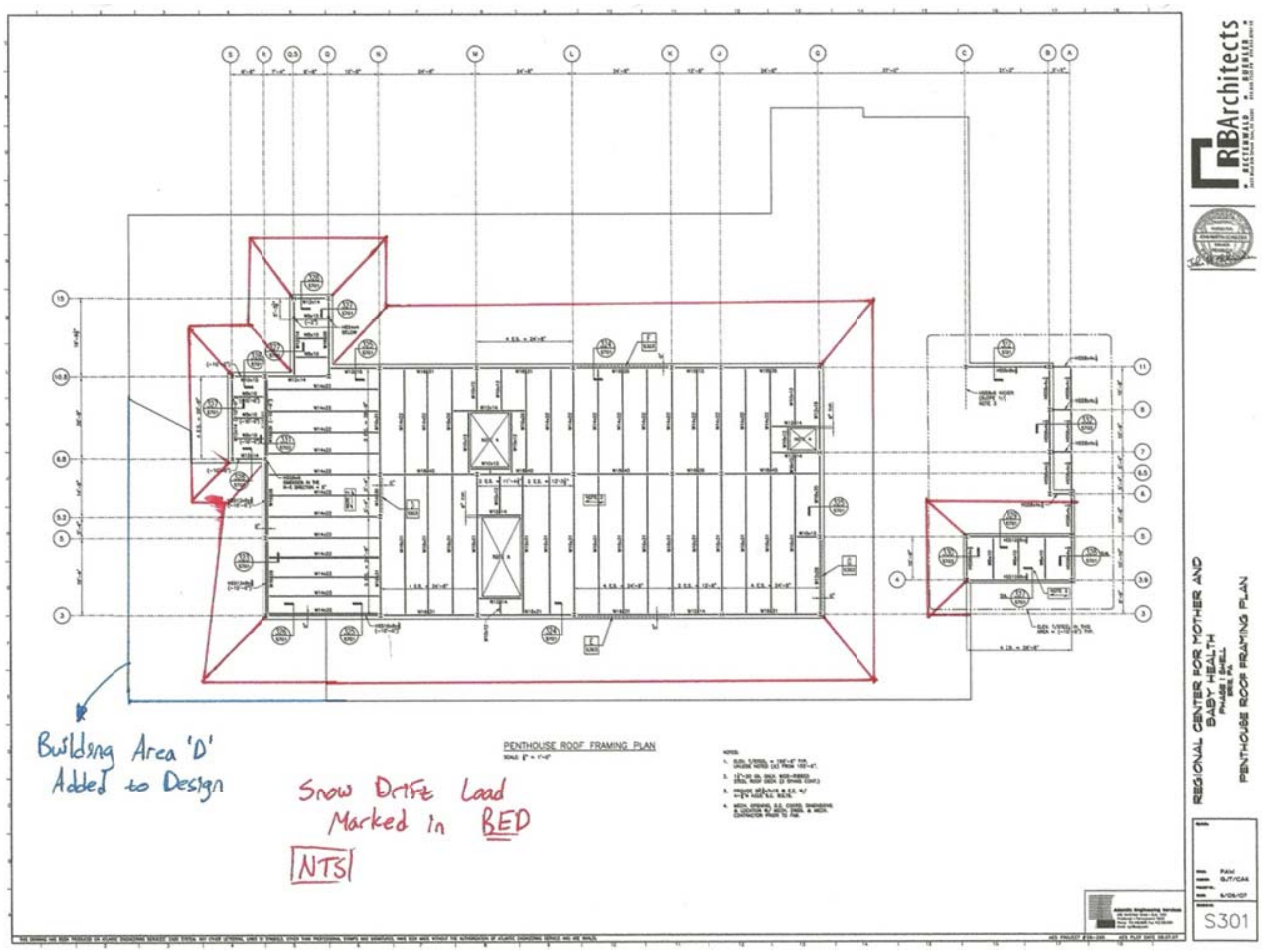
$w = 4 h_d = 4(1.75') = 7'-0''$

$p_d = h_d \gamma = 1.75'(19.2 \text{ pcf}) + 24.6 = 58.2$ psf

$\boxed{\therefore p_d = 58.2 \text{ psf}}$



B.4 - Drift Plan



Appendix C: Wind Load Calculations

C.1 – Wind Calculations

Wind Loads

ASCE 7-05

Method 2 – Analytical Procedure

Assume: Enclosed Building
Rigid Building

Wind From North

$V = 90$ mph → Figure 6-1

$K_d = 0.85$ → Table 6-4

$I = 1.15$ → Table 6-1

Occupancy Category = III → Table 1-1

K_h & K_z → Table 6-3 → Case 2

Surface Roughness D → Exposure D

70'-80' = 1.38
60'-70' = 1.34
50'-60' = 1.31
40'-50' = 1.27
30'-40' = 1.22
25'-30' = 1.16
20'-25' = 1.12
15'-20' = 1.08
0'-15' = 1.03

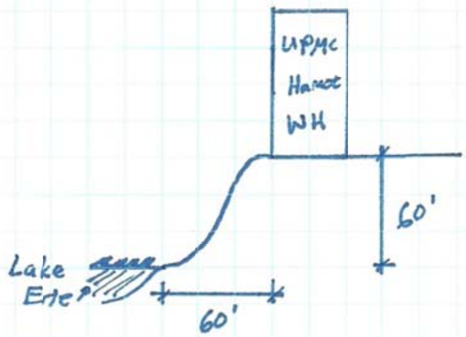
80'-90' = 1.40
90'-92' = 1.41 → Interpolated Value

C.2 – Wind Calculations (con't)

Wind Loads (cont)

$K_{zt} \Rightarrow$ Fig 6-4

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



$$K_1 = 0.95(1.0)$$

$$K_2 = \left(1 - \frac{|x|}{\mu L_H}\right)$$

$$= \left(1 - \frac{0}{4(60)}\right)$$

$$= 1$$

- 2D Escarpment
- Exposure D
- $H/L_H = 60/60 = 1.0$

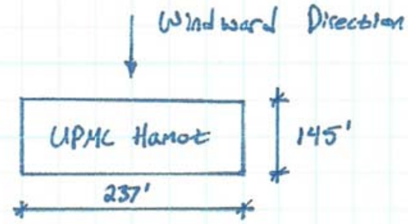
$$K_3 = e^{-\gamma z/L_H}$$

$\gamma = 2.5$

$z = 80$	$= 0.036$	$z = 90$	$= 0.024$
$z = 70$	$= 0.054$		
$z = 60$	$= 0.082$		
$z = 50$	$= 0.125$		
$z = 40$	$= 0.189$		
$z = 30$	$= 0.287$		
$z = 25$	$= 0.353$		
$z = 20$	$= 0.435$		
$z = 15$	$= 0.535$		
$z = 0$	$= 1.0$		

C.3 – Wind Calculations (con't)

Wind Loads (cont)



Plan View

$K_{zt\,70} = 1.105$
$K_{zt\,60} = 1.162$
$K_{zt\,50} = 1.252$
$K_{zt\,40} = 1.391$
$K_{zt\,30} = 1.620$
$K_{zt\,25} = 1.783$
$K_{zt\,20} = 1.997$
$K_{zt\,15} = 2.275$
$K_{zt\,0} = 3.803$

$K_{zt\,80} = 1.070$
$K_{zt\,90} = 1.046$

$L/B = 145/237 = 0.612$

Gust Factor ⇒ Sec 6.5.8

$G = 0.85$

Enclosed Building ⇒ Figure 6-5

$GC_{pi} = +/- 0.18$

C_p Values → Figure 6-6

$C_p = 0.8$ → Windward Wall
$C_p = -0.5$ → Leeward Wall
$C_p = -0.9$ ⇒ Roof ⇒ 0' to 39'
$C_p = -0.9$ ⇒ Roof ⇒ 39' to 78'
$C_p = -0.5$ ⇒ Roof ⇒ 78' to 145'

C.4 – Wind Calculations (con't)

Wind Loads (cont)

q_z Values ⇒ Section 6.5.10

$q_{z80} = 30.91$
$q_{z70} = 31.56$
$q_{z60} = 33.24$
$q_{z50} = 35.81$
$q_{z40} = 40.06$
$q_{z30} = 41.92$
$q_{z25} = 45.33$
$q_{z20} = 49.80$
$q_{z15} = 79.40$

$q_{z90} = 30.36$
$q_{z92} = 29.89 = q_h$

Windward Wall Pressures ⇒ Sec 6.5.12.4.2

h = 80'	$p_{80} = 26.40$
h = 70'	$p_{70} = 26.98$
h = 60'	$p_{60} = 28.13$
h = 50'	$p_{50} = 29.87$
h = 40'	$p_{40} = 32.76$
h = 30'	$p_{30} = 34.03$
h = 25'	$p_{25} = 36.35$
h = 20'	$p_{20} = 39.39$
h = 15'	$p_{15} = 59.51$
h = 90	$p_{90} = 26.03$
h = 92	$p_{92} = 25.71$

Leeward Wall Pressures ⇒ Sec 6.5.12.4.2

$p = -15.55$

C.5 – Wind Calculations (con't)

Wind Loads (cont)

Wind From East or West

$V = 90$ mph ⇒ Figure 6-1

$K_d = 0.85$ ⇒ Table 6-4

$I = 1.15$ ⇒ Table 6-1

Occupancy Category = III → Table 1-1

$K_h + K_z$ → Table 6-3 → Case 2

Surface Roughness D → Exposure D

70-80 = 1.38
60-70 = 1.34
50-60 = 1.31
40-50 = 1.27
30-40 = 1.22
25-30 = 1.16
20-25 = 1.12
15-20 = 1.08
0-15 = 1.03

80-90 = 1.40
90-92 = 1.41 ⇒ interpolated Value

$K_{zt} = 1.0$ ⇒ No Ridge in this direction ⇒ Sec 6.5.7.2

Gust Factor → Sec 6.5.8

$G = 0.85$

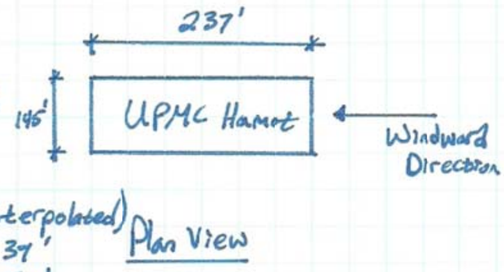
Enclosed Building ⇒ Figure 6-5

$GC_{pi} = \pm 0.18$

C.6 – Wind Calculations (con't)

Wind Loads (cont)

C_p Values ⟹ Figure 6-6



$C_p = 0.8$ ⟹ Windward Wall
$C_p = -0.37$ ⟹ Leeward Wall (Interpolated)
$C_p = -0.9$ ⟹ Roof ⟹ 0' to 39'
$C_p = -0.9$ ⟹ Roof ⟹ 39' to 78'
$C_p = -0.5$ ⟹ Roof ⟹ 78' to 156'
$C_p = -0.3$ ⟹ Roof ⟹ > 156'

$L/B = 237/145 = 1.63$

q_z Values ⟹ Section 6.5.10

$q_{z80} = 27.97$
$q_{z70} = 27.16$
$q_{z60} = 26.55$
$q_{z50} = 25.74$
$q_{z40} = 24.73$
$q_{z30} = 23.51$
$q_{z25} = 22.70$
$q_{z20} = 21.89$
$q_{z15} = 20.88$

$q_{z90} = 28.38$
$q_{z92} = 28.58 = q_h$

C.7 – Wind Calculations (con't)

Wind Loads (cont)

Windward Wall Pressures ⇒ Sec 6.5.12.4.2

$p_{80} = 24.16$
$p_{70} = 23.47$
$p_{60} = 23.05$
$p_{50} = 22.50$
$p_{40} = 21.82$
$p_{30} = 20.99$
$p_{25} = 20.43$
$p_{20} = 19.88$
$p_{15} = 19.20$

$p_{90} = 24.44$
$p_{92} = 24.58$

Leeward Wall Pressures ⇒ Sec 6.5.12.4.2

$p = -14.13$

C.8 – Wind Calculations (con't)

Justin Kovach
AE Senior Thesis 2011-2011

UPMC Hamot Womens Hospital
Erie, PA

Base Shear and Overturning Moment Calculator

Description: Wind from North

Length of Main Wall Perpendicular to Wind	237	ft
Length of Stair Wall Perpendicular to Wind	20	ft
Length of Penthouse Wall Perpendicular to Wind	160	ft

Main Building

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	70 ft	26.40 psf	12.5 kips	888.5 ft-kips
70 ft	60 ft	26.98 psf	63.9 kips	4156.3 ft-kips
60 ft	50 ft	28.13 psf	66.7 kips	3666.7 ft-kips
50 ft	40 ft	29.87 psf	70.8 kips	3185.6 ft-kips
40 ft	30 ft	32.76 psf	77.6 kips	2717.4 ft-kips
30 ft	25 ft	34.03 psf	40.3 kips	1109.0 ft-kips
25 ft	20 ft	36.35 psf	43.1 kips	969.2 ft-kips
20 ft	15 ft	39.39 psf	46.7 kips	816.9 ft-kips
15 ft	0 ft	59.51 psf	211.6 kips	1586.7 ft-kips

Stair Pop-Out

h_{top} =	h_{bot} =	p =	V =	M =
82 ft	80 ft	26.03 psf	1.0 kips	84.3 ft-kips
80 ft	72 ft	26.40 psf	4.2 kips	321.0 ft-kips

Mechanical Penthouse

h_{top} =	h_{bot} =	p =	V =	M =
92 ft	90 ft	25.71 psf	8.2 kips	748.7 ft-kips
90 ft	80 ft	26.03 psf	41.6 kips	3540.1 ft-kips
80 ft	72 ft	26.40 psf	33.8 kips	2568.2 ft-kips

Suction

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	0 ft	15.55 psf	265.3 kips	9552.4 ft-kips
82 ft	72 ft	15.55 psf	3.1 kips	239.5 ft-kips
92 ft	72 ft	15.55 psf	49.8 kips	4080.3 ft-kips

Total

V_{tot} = 1040.3 kips
M_{tot} = 41230.8 ft-kips

Justin Kovach
AE Senior Thesis 2011-2011

UPMC Hamot Womens Hospital
Erie, PA

Base Shear and Overturning Moment Calculator

Description: Wind from East

Length of Main Wall Perpendicular to Wind	145	ft
Length of Stair Wall Perpendicular to Wind	15	ft
Length of Penthouse Wall Perpendicular to Wnd	75	ft

Main Building

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	70 ft	24.16 psf	7.0 kips	497.5 ft-kips
70 ft	60 ft	23.47 psf	34.0 kips	2212.0 ft-kips
60 ft	50 ft	23.05 psf	33.4 kips	1838.2 ft-kips
50 ft	40 ft	22.50 psf	32.6 kips	1468.1 ft-kips
40 ft	30 ft	21.82 psf	31.6 kips	1107.4 ft-kips
30 ft	25 ft	20.99 psf	15.2 kips	418.5 ft-kips
25 ft	20 ft	20.43 psf	14.8 kips	333.3 ft-kips
20 ft	15 ft	19.88 psf	14.4 kips	252.2 ft-kips
15 ft	0 ft	19.20 psf	41.8 kips	313.2 ft-kips

Stair Pop-Out

h_{top} =	h_{bot} =	p =	V =	M =
82 ft	80 ft	24.44 psf	0.7 kips	59.4 ft-kips
80 ft	72 ft	24.16 psf	2.9 kips	220.3 ft-kips

Mechanical Penthouse

h_{top} =	h_{bot} =	p =	V =	M =
92 ft	90 ft	24.58 psf	3.7 kips	335.5 ft-kips
90 ft	80 ft	24.44 psf	18.3 kips	1558.1 ft-kips
80 ft	72 ft	24.16 psf	14.5 kips	1101.7 ft-kips

Suction

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	0 ft	14.13 psf	147.5 kips	5310.6 ft-kips
82 ft	72 ft	14.13 psf	2.1 kips	163.2 ft-kips
92 ft	72 ft	14.13 psf	21.2 kips	1738.0 ft-kips

Total

V_{tot} = 435.9 kips
M_{tot} = 18927.2 ft-kips

Appendix D: Seismic Calculations

D.1 – Seismic Calculations

EQ Loads

ASCE 7-05

$R = 3$ – Not Specifically Detailed for Seismic ⇒ Table 12.2-1

$I = 1.25$ ⇒ Table 11.5-1

$T = C_u T_a$ $\qquad$ $T_L = 12$ ⇒ Fig 22-15

$C_u = 1.7$ ⇒ Table 12.8-1

$T_a = C_t h_n^{x} = 0.028\,(92'')^{0.8} = 1.043$

$\therefore T = 1.7(1.043) = 1.773$

$S_{DS} = 0.175$, $S_{D1} = 0.078$ } From USGS

$$C_s = \min \begin{cases} S_{DS}/(R/I) = 0.175/(3/1.25) = 0.0729 \\ S_{D1}/(T \cdot R/I) = 0.078/(1.773 \cdot 3/1.25) = 0.0183 \\ S_{D1} \cdot T_L/(T^2 \cdot R/I) = 0.078\,(12)/(1.773^2 \cdot 3/1.25) = 0.1241 \end{cases}$$

$\therefore C_s = 0.0183$

$V = C_s W = 0.0183\,(11{,}606)$

$$\boxed{\therefore V = 212.39^{k}}$$

D.2 – Seismic Calculations (con't)

EQ Loads (cont)

$W_{PH} = 315.4^k$ | $h_{PH} = 92'$
$W_{SR} = 74.3^k$ | $h_{SR} = 82'$
$W_R = 1616.0^k$ | $h_R = 72'$
$W_5 = 2282.7^k$ | $h_5 = 58'$
$W_4 = 2348.6^k$ | $h_4 = 44'$
$W_3 = 2401.9^k$ | $h_3 = 28'$
$W_2 = 2567.1^k$ | $h_2 = 12'$

$k = 1.5265 \Rightarrow$ Interpolation

PH	$w_{PH} h_{PH}^k$ =	313,750
SR	$w_{SR} h_{SR}^k$ =	62,005
R	$w_R h_R^k$ =	1,105,756
5	$w_5 h_5^k$ =	1,122,849
4	$w_4 h_4^k$ =	757,774
3	$w_3 h_3^k$ =	388,724
2	$w_2 h_2^k$ =	113,976
		3,864,834

$C_{vPH} = 0.08118$
$C_{vSR} = 0.01604$
$C_{vR} = 0.28611$
$C_{v5} = 0.29053$
$C_{v4} = 0.19607$
$C_{v3} = 0.10058$
$C_{v2} = 0.02949$

D.3 – Seismic Calculations (con't)

<u>EQ Loads</u> (con't)

$F_{PH} = C_{vPH}\ V = 17.24^k$
$F_{SR} = C_{vSR}\ V = 3.41^k$
$F_R = C_{vR}\ V = 60.77^k$
$F_5 = C_{v5}\ V = 61.71^k$
$F_4 = C_{v4}\ V = 41.64^k$
$F_3 = C_{v3}\ V = 21.36^k$
$F_2 = C_{v2}\ V = 6.26^k$

Appendix E: Spot Checks

E.1 – Decking Check

Decking

Span = 21'-4"/3 = 7'-2"

t = 4" → total thickness = 6"

2VLI20 Deck

Loads

Dead = 69 psf
Live = 80 psf
149 psf
− 48 psf → Deck + Slab SW

w = 101 psf < 278 psf (7'-6" clear; 2VLI) ∴ OK

Max Unshored Clear Span

− 3 Span Condition meet

$S = 7'\text{-}2'' < 10'\text{-}9'' = S_{max}$ ∴ OK

E.2 – Beam Check

Beam

Composite Beam: W 14 x 22 [10]

$A_g = 6.49\ in^2$

$I_x = 199\ in^4$

$F_y = 50$ ksi

$d = 13.7"$

Trib width: 7'-2"

Span : 24'-8"

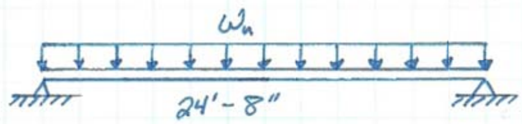
$w_d = 69$ psf $= 506$ plf

$w_\ell = 80$ psf $= 586.67$ plf

$w_u = 1.2 w_d + 1.6 w_\ell$

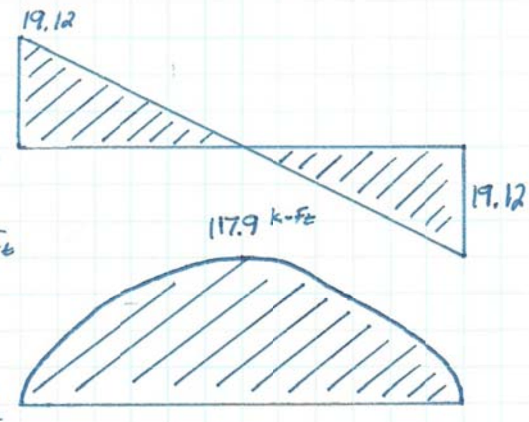
$= 1.2(.506) + 1.6(.587)$

$\therefore w_u = 1.55$ klf



$V_u = \frac{w_u \ell}{2} = \frac{1.55(24.67)}{2} = 19.12$ k

$M_u = \frac{w_u \ell^2}{8} = \frac{1.55(24.67)^2}{8} = 117.9$ k-ft



$b_{eff} = \min \begin{cases} \text{span}/4 = 6.17' \Rightarrow \text{controls} \\ \text{spacing} = 7.17' \end{cases}$

$\therefore b_{eff} = 74"$

$V_c' = 0.85 f_c' b_{eff} t = 0.85(4)(74)(6) = 1509.6$ k

$V_s' = F_y A_s = 50(6.49) = 324.5$

$V_c' > V_s' \Rightarrow \therefore$ NA in concrete

E.3 – Beam Check (con't)

Beam (cont)

$$V_s' = 0.85 f'_c (b_{eff})(a) \implies \therefore a = \frac{V_s'}{0.85 f'_c b_{eff}} = \frac{324.5}{0.85(4)(.74)}$$

$$a = 1.29''$$

$$M_n = \frac{V_s'(d/2 + t - a/2)}{12} = \frac{324.5(13.7/2 + 6 - 1.29/2)}{12} = 330.0 \text{ ft-k}$$

$$\phi M_n = 0.9 M_n = 0.9(330.0 \text{ ft-k})$$

$$\boxed{\phi M_n = 297.0 \text{ ft-k}}$$

$$\phi M_n > M_u \implies \therefore ok$$

$$\phi V_n = 94.5^k > V_u = 19.12^k \qquad \therefore ok$$

↳ Table 3-2

$$\Delta_{LL} = \ell/360 = \frac{24.67(12)}{360} = 0.8222 \text{ in}$$

$$\Sigma Q_n = \min \begin{vmatrix} 0.85 f'_c b_{eff} t = 1509.6^k \\ A_s F_y = 324.5^k \end{vmatrix} \implies \text{controls}$$

$$Y2 = t_{slab} - a/2 = 6 - 1.29/2 = 5.36$$

$$\bar{Y} = \frac{A_s(d/2) + \frac{\Sigma Q_n}{F_y}(d + Y2)}{A_s + \frac{\Sigma Q_n}{F_y}} = \frac{6.49(13.7/2) + \frac{324.5}{50}(13.7 + 5.36)}{6.49 + \frac{324.5}{50}}$$

$$\therefore \bar{Y} = 12.955''$$

E.4 – Beam Check (con't)

Beam (cont)

$$I_{LB} = I_{x(s)} + A_s\left(\bar{y} - d/2\right)^2 + \frac{\Sigma Q_n}{F_y}\left(d + Y2 - \bar{y}\right)^2$$

$$= 199 + 6.49\left(12.955 - 13.7/2\right)^2 + \frac{324.5}{50}\left(13.7 + 5.36 - 12.955\right)^2$$

$$\therefore I_{LB} = 682.8 \text{ in}^4$$

$$\Delta_{LL} = \frac{5 w_{LL}\, l^4}{384 E I} = \frac{5(.587)(24.67)^4(1728)}{384(29000)(682.8)} = 0.247 \text{ in}$$

$$\Delta_{LL} = 0.247'' < 0.8222'' \qquad \therefore OK$$

Wet Concrete Deflection

$$\Delta_{max} = l/240 = \frac{24.67(12)}{240} = 1.233''$$

$$w = 69(7.167) + 22\text{plf} = 0.517 \text{ klf}$$

$$I_{req} = \frac{5 w l^4}{384 E \Delta_{max}} = \frac{5(0.517)(24.67)^4(1728)}{384(29000)(1.233'')}$$

$$I_{req} = 120.3 \text{ in}^4 < 199 \text{ in}^4 \qquad \therefore OK$$

$$\boxed{\text{W14x22 Works}}$$

E.5 – Girder Check

Girder

Composite: W16 x 26 [18]

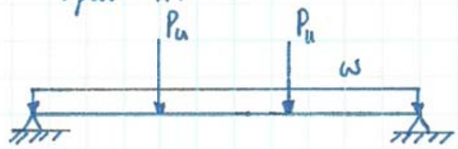
$I_x = 301 \text{ in}^4$

$A_g = 7.68 \text{ in}^2$

$F_y = 50 \text{ ksi}$

$d = 15.7"$

Span = 21'-4"

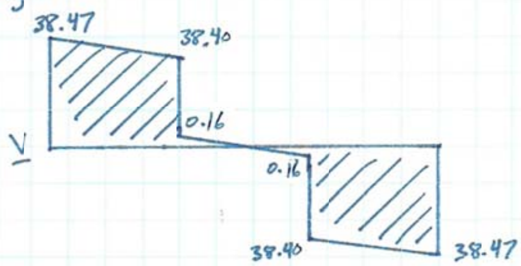


$w = 26 \text{ plf} \Rightarrow$ Girder Self Weight

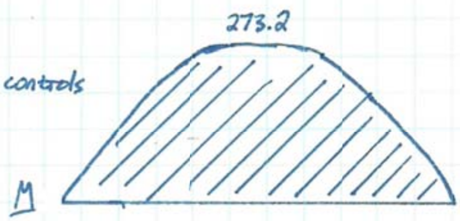
$P_u = 2(19.12^k) = 38.24^k$

$V_u = 38.47^k$

$M_u = 273.2 \text{ k-ft}$



$b_{eff} = \min \begin{cases} \text{span}/4 = 21'\text{-}4"/4 = 64" \Rightarrow \text{controls} \\ \text{Spacing} = 24'\text{-}8" = 296" \end{cases}$



$V'_c = 0.85 f'_c \, b_{eff} \, t$
$= 0.85(4)(64)(6)$

$\therefore V'_c = 1305.6^k$

$V'_s = F_y A_s = 50(7.68)$

$\therefore V'_s = 384^k$

$V'_c > V'_s \quad \therefore$ NA in concrete

E.6 – Girder Check (con't)

Girder (cont)

$$V_s' = 0.85f_c'(b_{eff})(a) \Rightarrow a = \frac{V_s'}{0.85 f_c' b_{eff}} = \frac{384}{0.85(4)(64)}$$

$$a = 1.76''$$

$$M_n = \frac{V_s'(d/2 + t - a/2)}{12} = \frac{384(15.7/2 + 6 - 1.76/2)}{12} = 415.0 \text{ k-ft}$$

$$\phi M_n = 0.9 M_n = 0.9(415.0)$$

$$\boxed{\therefore \phi M_n = 373.5 \text{ ft-k}}$$

$$\phi M_n > M_u \quad \therefore \underline{OK}$$

$$\phi V_n = \underset{\hookrightarrow \text{Table 3-2}}{106^k} > V_u = 38.5^k \qquad \therefore \underline{ok}$$

$$\Delta_{LL} = \ell/360 = 21.33(12)/360 = 0.711''$$

$$\Sigma Q_n = \min \left| \begin{array}{ll} 0.85 f_c' b_{eff} t & = 1305.6 \\ A_s F_y & = 384 \Rightarrow \text{controls} \end{array} \right.$$

$$Y2 = t_{slab} - a/2 = 6 - 1.76/2 = 5.12''$$

$$\bar{y} = \frac{A_s(d/2) + \frac{\Sigma Q_n}{F_y}(d + Y2)}{A_s + \frac{\Sigma Q_n}{F_y}} = \frac{7.68(15.7/2) + \frac{384}{50}(15.7 + 5.12)}{7.68 + \frac{384}{50}}$$

$$\therefore \bar{y} = 14.335''$$

E.7 – Girder Check (con't)

Girder (cont)

$$I_{LB} = I_{x(s)} + A_s(\bar{y} - d/2)^2 + \frac{\Sigma Q_n}{F_y}(d + Y2 - \bar{y})^2$$

$$= 301 + 7.68(14.335 - 15.7/2)^2 + \frac{384}{50}(15.7 + 5.12 - 14.335)^2$$

$$I_{LB} = 947.0 \text{ in}^4$$

$$\Delta_{LL} = \frac{P_{LL}(a)}{24EI_{LB}}(3l^2 - 4a^2) \qquad a = 21'\text{-}4''/3 = 85.33'' \qquad l = 21'\text{-}4'' = 256''$$

$$= 0.314'' \qquad P_{LL} = 14.48^k$$

$$\Delta_{LL} = 0.314'' > \Delta_{LL\,max} = 0.711''$$

$\therefore$ OK

Wet Concrete Deflection

$$\Delta_{max} = l/240 = 21'\text{-}4''(12)/240 = 1.07''$$

$$I_{req} = \frac{P_{LL}\,a}{24E\Delta_{max}}(3l^2 - 4a^2)$$

$$= \frac{14.48(85.33)}{24(29000)(1.07)}(3(256)^2 - 4(85.33)^2)$$

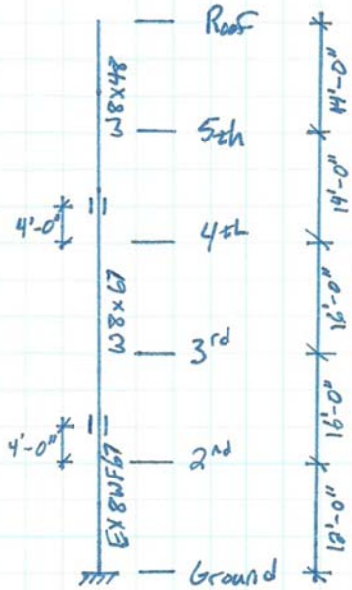
$$\therefore I_{req} = 277.9 \text{ in}^4 < I_x = 301 \text{ in}^4 \qquad \therefore OK$$

$$\boxed{\therefore \text{W16x26 Works}}$$

E.8 – Column Check

Column

Column L-5



= Influence Area

= Tributary Area

Influence Area

$$A_i = (19'\text{-}4" + 21'\text{-}4")(24'\text{-}8" + 24'\text{-}8")$$

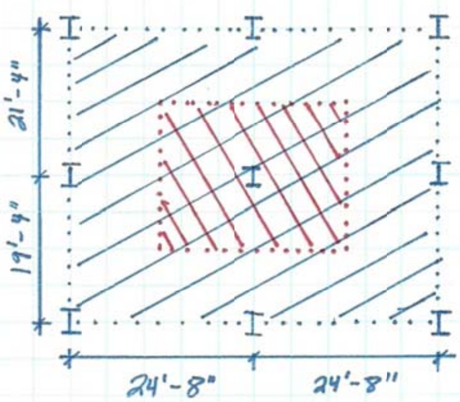
$$\therefore A_i = 2006.2 \text{ ft}^2$$

Tributary Area

$$A_t = \left(\frac{19'\text{-}4"}{2} + \frac{21'\text{-}4"}{2}\right)(24'\text{-}8")$$

$$\therefore A_t = 501.6 \text{ ft}^2$$

$$\therefore K_{LL} = 4$$



E.9 – Column Check (con't)

Column (cont)

Column Loads

Below 5th (1 Reducible Floor, Roof Live Unreducible)

$P_S = 24.64\text{ psf}(501.6\text{ ft}^2) = 12.36^k \Rightarrow$ controls over roof live load
$P_L = 0.585(80)(501.6\text{ ft}^2) = 23.47^k$

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 501.6}} = 0.585$$

$$P_D = \overset{\text{roof DL}}{20}(501.6) + \overset{5^{th}\text{ DL}}{69}(501.6) = 44.62^k$$

$$P_u = 1.2(44.62) + 1.6(23.47) + 0.5(12.36)$$

$$\therefore P_{u5} = 97.28^k$$

Below 3rd (3 Floors of Reducible Area)

$P_S = 12.36^k$
$P_D = 20(501.6) + 3(69)(501.6) = 113.86^k$
$P_L = 0.443(3)(80)(501.6) = 53.33^k$

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 3(501.6)}} = 0.443$$

$$P_u = 1.2(113.86) + 1.6(53.33) + 0.5(12.36)$$

$$\therefore P_{u3} = 228.14^k$$

E.10 – Column Check (con't)

Column (con't)

Column Loads

Below 2^{nd} (4 Floors of Reducible Area)

$$P_S = 12.36^k$$
$$P_D = 20(501.6) + 4(67)(501.6) = 148.47^k$$
$$P_L = 0.417(4)(80)(501.6) = 66.93^k$$

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 4(501.6)}} = 0.417$$

$$P_u = 1.2(148.47) + 1.6(66.93) + 0.5(12.36)$$

$$\therefore P_{u2} = 291.43^k$$

E.11 – Column Check (con't)

Column (cont)

Max Column Capacity (Assume k = 1.0 ⇒ conservative)

W8 × 48
$A_g = 14.1\ in^2$
$r_y = 2.08$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r_y}\right)^2} = \frac{\pi^2 (29000)}{\left(\frac{1.0(14\times12)}{2.08}\right)^2} = 43.87$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[0.658^{(50/43.87)}\right] 50$$

$$= 31.03$$

$$P_n = F_{cr} A_g = 31.03 (14.1) = 437.56^k$$

$$\phi P_n = 0.9(P_n) = 0.9(437.56)$$

$$\phi P_n = 393.8^k > P_{us} = 97.28^k$$

Slenderness Limit

$$\frac{kL}{r_y} \le 200$$

$$\frac{kL}{r_y} = \frac{1.0(14\times12)}{2.08} = 80.77 \le 200$$

$$\boxed{\therefore \text{W8} \times 48 \text{ Works}}$$

E.12 – Column Check (con't)

Column (cont)

Max Column Capacity (Assume $k = 1.0 \Rightarrow$ conservative)

W8x67

$A_g = 19.7\,in^2$

$r_x = 3.72$

$r_y = 2.12$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r_x}\right)^2} = \frac{\pi^2 (29000)}{\left(\frac{1.0(14 \times 12)}{3.72}\right)^2} = 140.33$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[0.658^{(50/140.33)}\right] 50 = 43.07$$

$$P_n = F_{cr} A_g = 43.07(19.7) = 848.48^k$$

$$\phi P_n = 0.9(848.48)$$

$$\phi P_n = 763.63^k$$

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r_y}\right)^2} = \frac{\pi^2 (29000)}{\left(\frac{1.0(16 \times 12)}{2.12}\right)^2} = 34.89$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[0.658^{(50/34.89)}\right] 50 = 27.45$$

$$P_n = F_{cr} A_g = 27.45(19.7) = 540.77^k$$

$$\phi P_n = 0.9(540.77)$$

$$\phi P_n = 486.69^k > P_{u3} = 228.14^k$$

Controls

Check Slenderness

$$\frac{kL}{r} \le 200 \qquad \frac{kL}{r} = \frac{1.0(16 \times 12)}{2.12} = 90.57 \le 200$$

$\therefore$ OK

$\boxed{\therefore \text{W8x67 OK}}$

E.13 – Column Check (con't)

Column (cont)

Max Column Capacity (Assume $k=1.0 \Rightarrow$ Conservative)

Ex 8WF67
$A_g = 19.7\ in^2$
$r_y = 2.12$
} Found on Supplemental CD

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r_y}\right)^2} = \frac{\pi^2(29000)}{\left(\frac{1.0(12\times12)}{2.12}\right)^2} = 62.04$$

$$F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[0.658^{(50/62.04)}\right] 50 = 35.68$$

$$P_n = F_{cr} A_g = 35.68(19.7) = 702.96\ k$$

$$\phi P_n = 0.9(702.96)$$

$$\therefore \phi P_n = 632.66 > 291.43^k = P_{u2} \qquad \therefore \underline{ok}$$

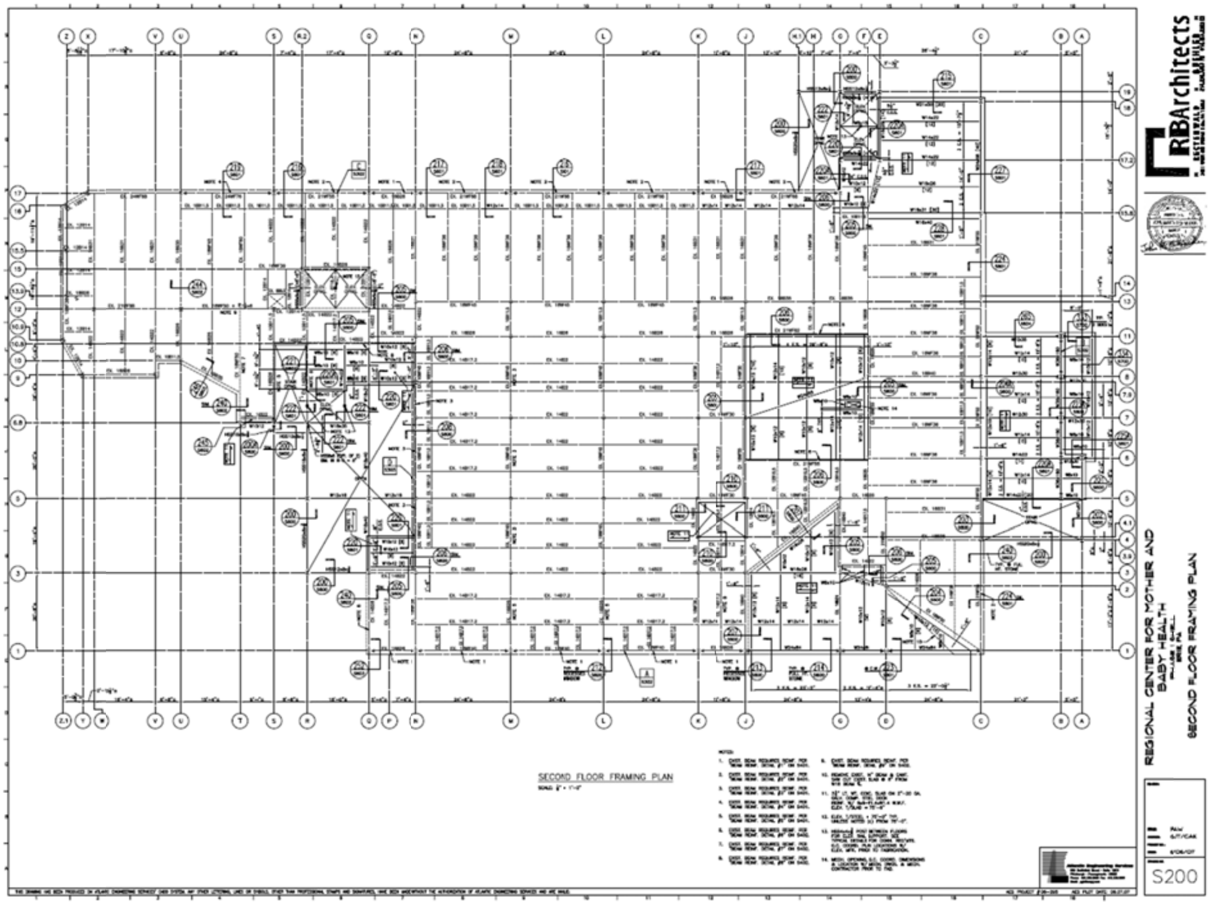
Slenderness Limit

$$\frac{kL}{r} \le 200 \qquad \frac{kL}{r_y} = \frac{1.0(12\times12)}{2.12} = 67.92 \le 200 \quad \therefore \underline{ok}$$

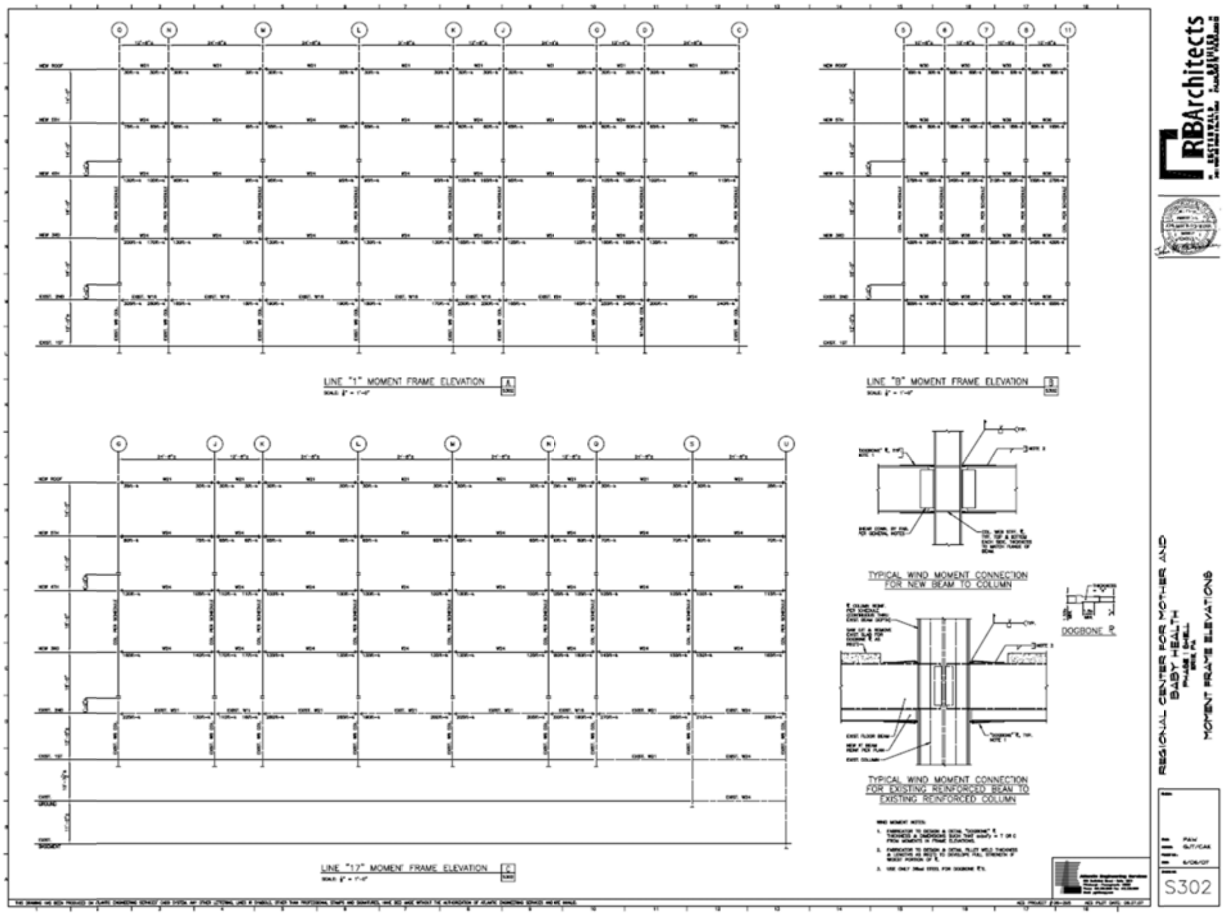
$$\boxed{\therefore \text{EX8WF67 Works}}$$

Appendix F: Relevant Building Plans

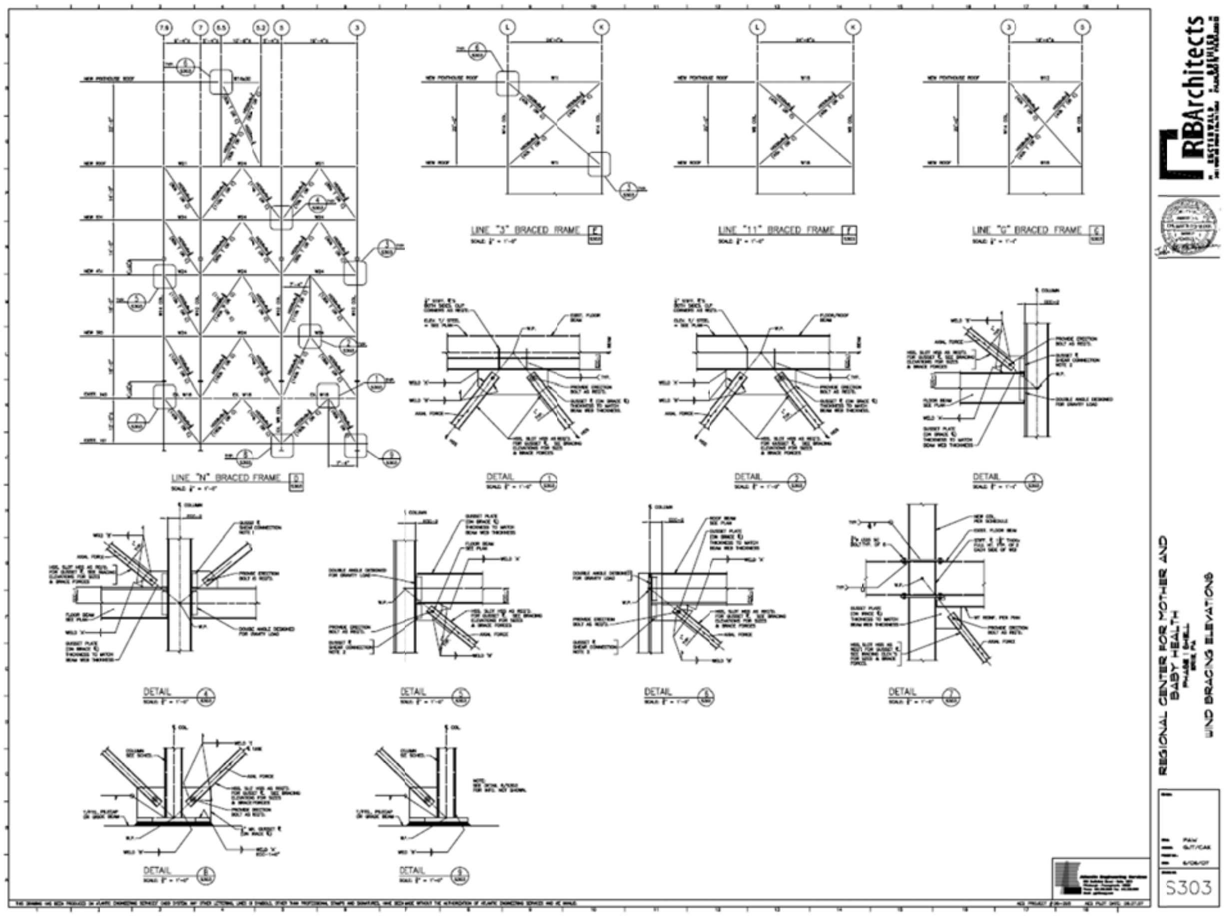
F.1 – S200 - Second Floor Structural Plan



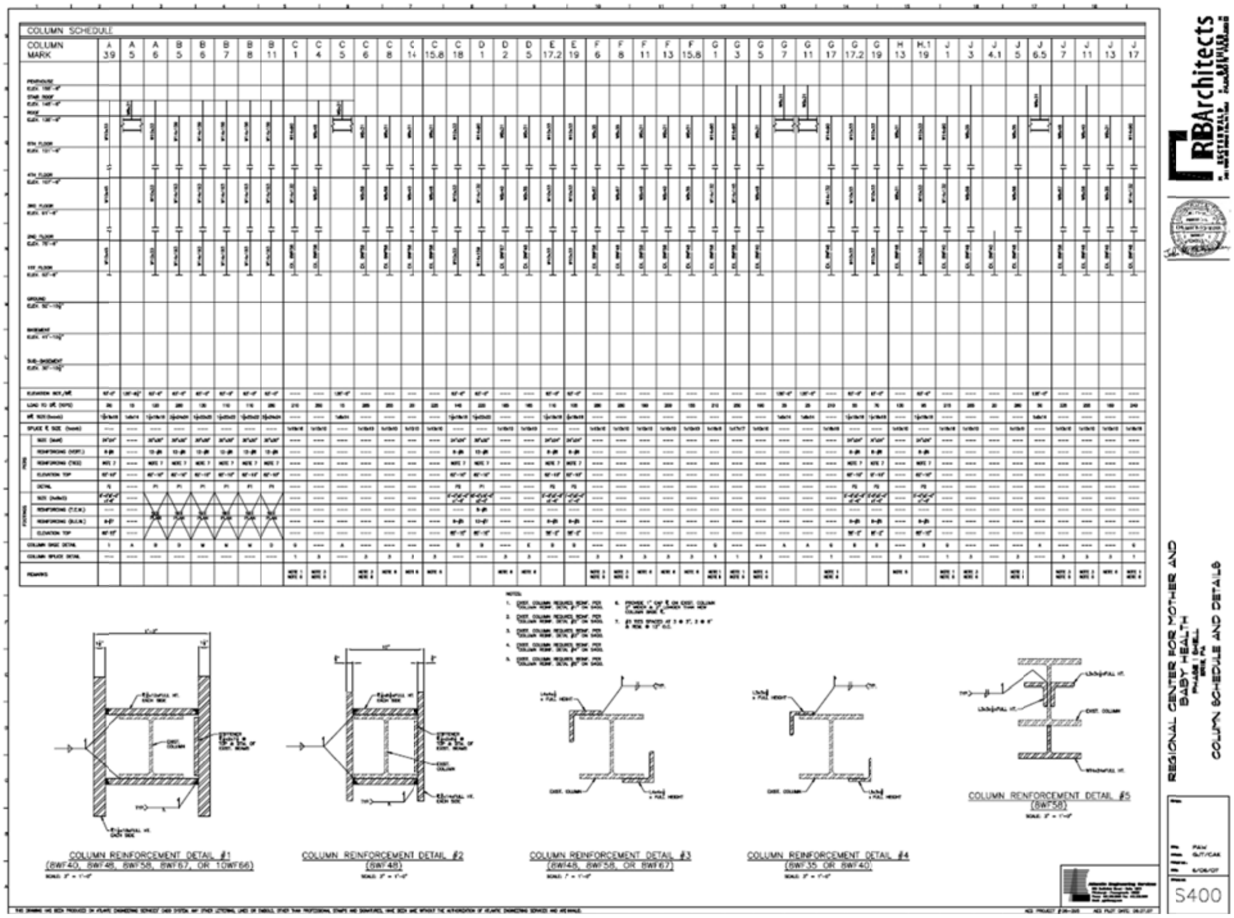
F.2 – S302 - Moment Frame Elevations



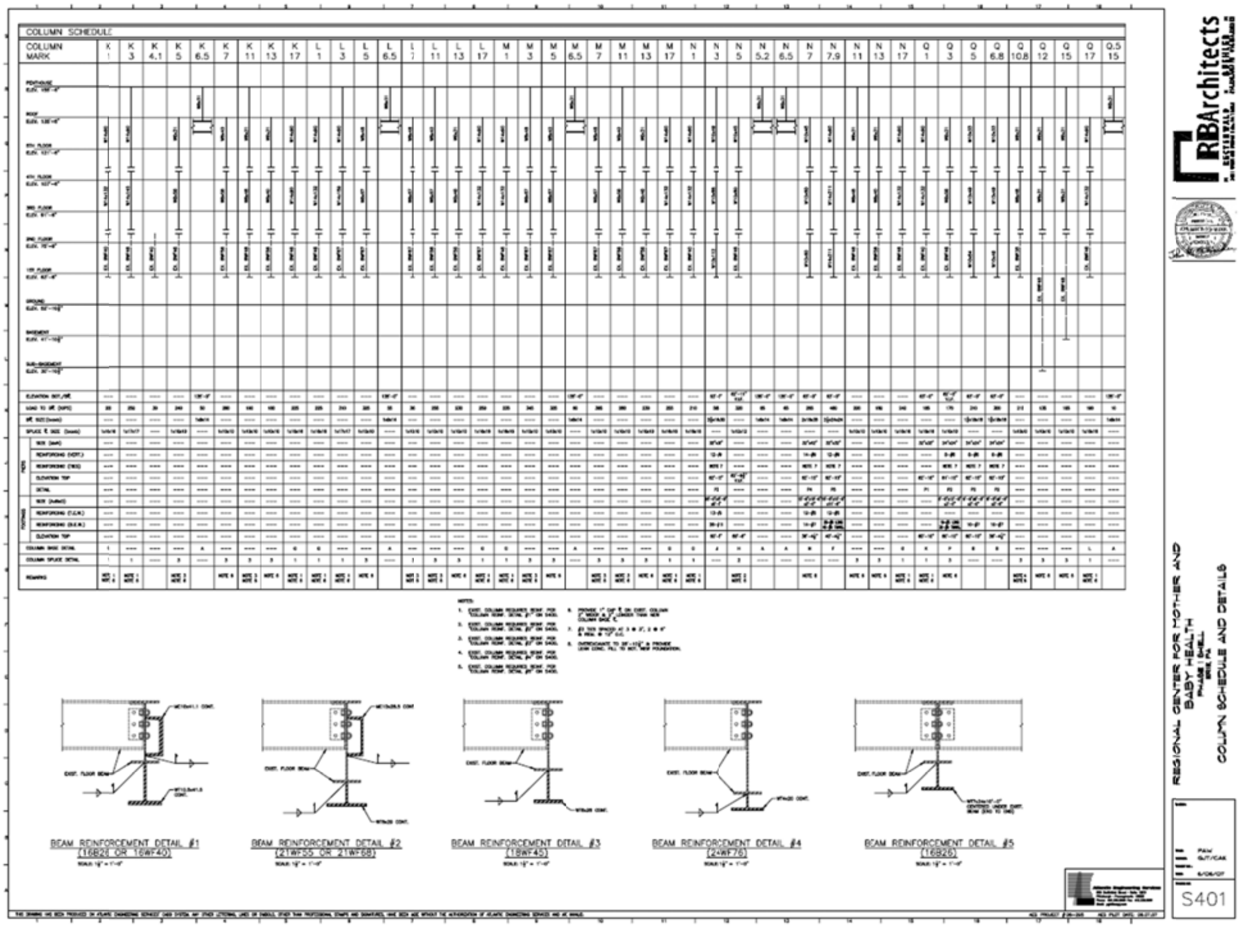
F.3 – S303 - Braced Frame Elevations



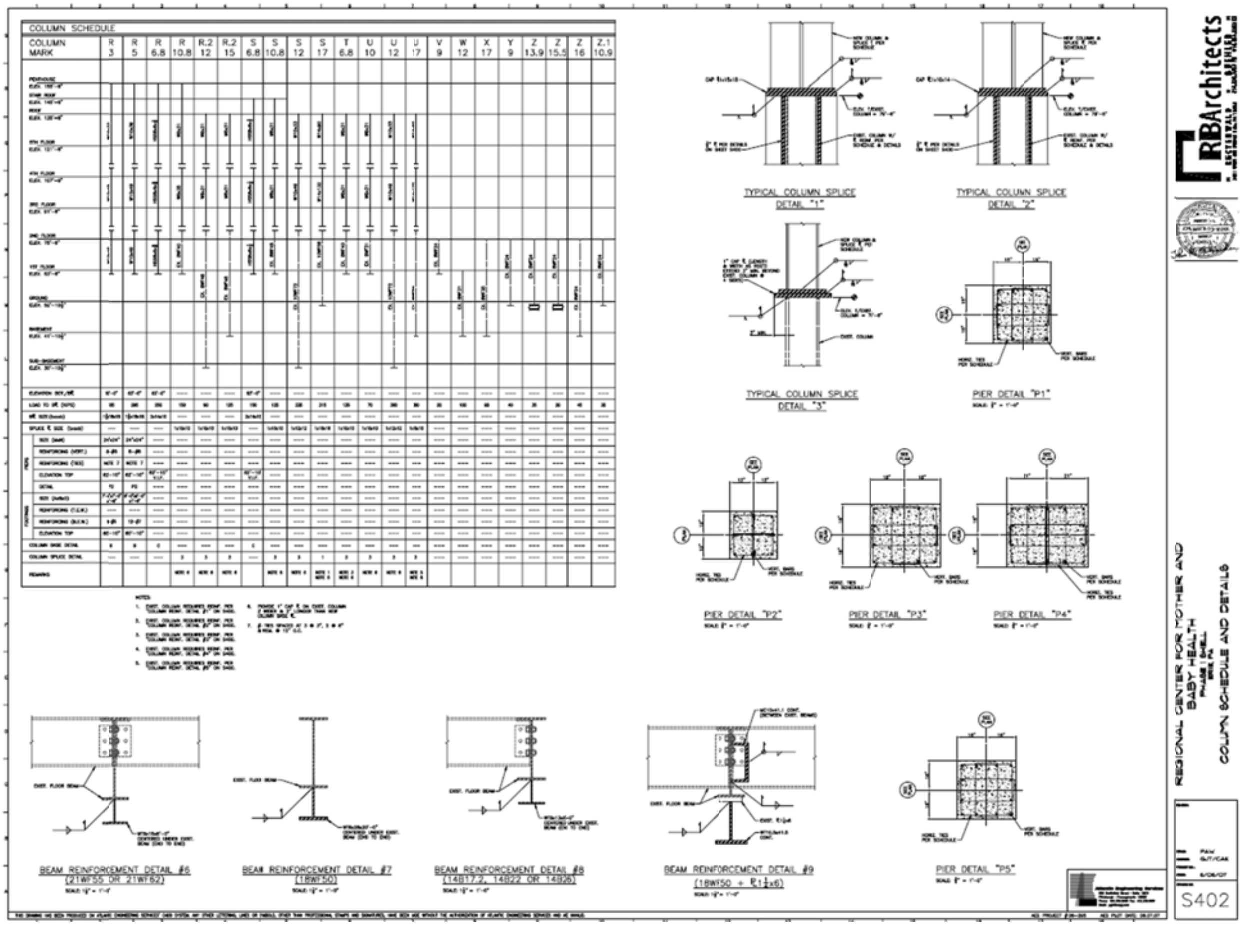
F.4 – S400 - Column Schedule and Typical Column Reinforcements



F.5 – S401 - Column Schedule and Typical Beam Reinforcements



F.6 – S402 – Column Schedule and Typical Beam Reinforcements



F.7 – S403 - Foundation Overbuilds