Revised Thesis Proposal

2011-2012 AE Senior Thesis

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Justin Kovach – Structural Option
Dr. Boothby – Advisor

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Executive Summary

The UPMC Hamot Women's Hospital is a 5 story, 92 foot tall, healthcare facility located on the bay of Lake Erie. The steel framing system supports the lightweight concrete composite floor system and the lateral loads from wind and seismic forces are resisted by moment connected steel frames in the E-W plan direction and both a moment connected steel frame and a braced frame in the N-S plan direction.

This thesis proposal is intended to outline a course of learning for the Spring 2012 semester. This will be done through several investigations, with the depth concentration of the work being related to the buildings structure and then two breadth topics will investigate how that structure affects other components of the building.

The UPMC Hamot Women's Hospital was initially built as a two story structure, but was designed for a future two floors to be added. The hospital later decided that the additional 2 floors would not be sufficient, that they would require an additional 3 floors. From a structural point of view this posed a problem due to increased load accumulation as the structure approached the ground floor. Thus the decision was made to remove the current building, down to the first floor. The remaining elements were then reinforced, including beams, columns, and foundations.

The structural depth for this thesis will be split into three distinct investigations. An investigation on the new building code with a comparison to the previous edition and how it affects the structural weight and performance will be done. An investigation into the possibility of effectively utilizing braced frames rather than moment frames will be completed. Finally, an investigation into a complete building redesign without using the existing structure or grid will be done to determine if the correct decision was made by the construction team.

As these elements are completed two breadth studies will be undertaken. An architectural breadth will be done to analyze the impact on the architecture that the braced frame system has on the building. A construction management breadth will also be done to analyze the impact of not using the existing structure and grid to build from.

Introduction

Located on 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.

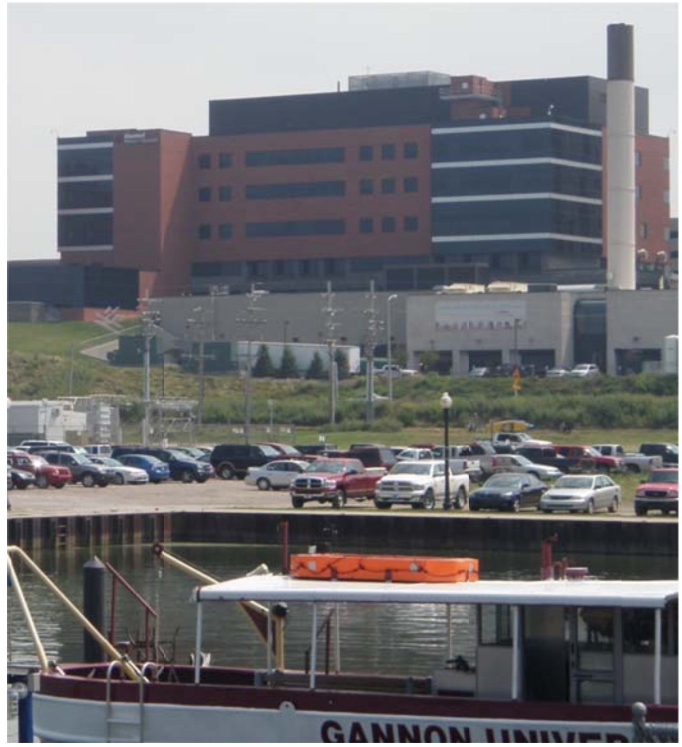


Figure 1: North Façade, Showing 2-D Escarpment

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 better 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.

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; 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 at each floor level; this opening is located on the west side of the building and measures approximately 27'-0"± by 30'-0"±.

Figure 2: Interior Water Wall

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 is intended to allow sufficient amounts of natural light into the building without being uncomfortable to the patients. The interior of the building was constructed to a very luxurious standard. 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 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, moreover, each room is 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 with 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 substructure is unusual 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 modifications to the spread footings are extensive in that many of the existing spread footings had to be increased in length, width, and depth. The minimum depth of the footings below grade is 3'-6". The typical foundation overbuild details can be found on sheet S403.

Figure 4: Foundation Excavation 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, W12x19 composite beams are likely to be used. 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 mechanical penthouse), 49' long braced frame along column line N, this is the only 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. In the E-W direction full height moment frames are utilized 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- 9th Edition)

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 to allow 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 + 20 (partitions)**
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 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 also unusual 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 due to the exposure factors and 2-D escarpment.

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

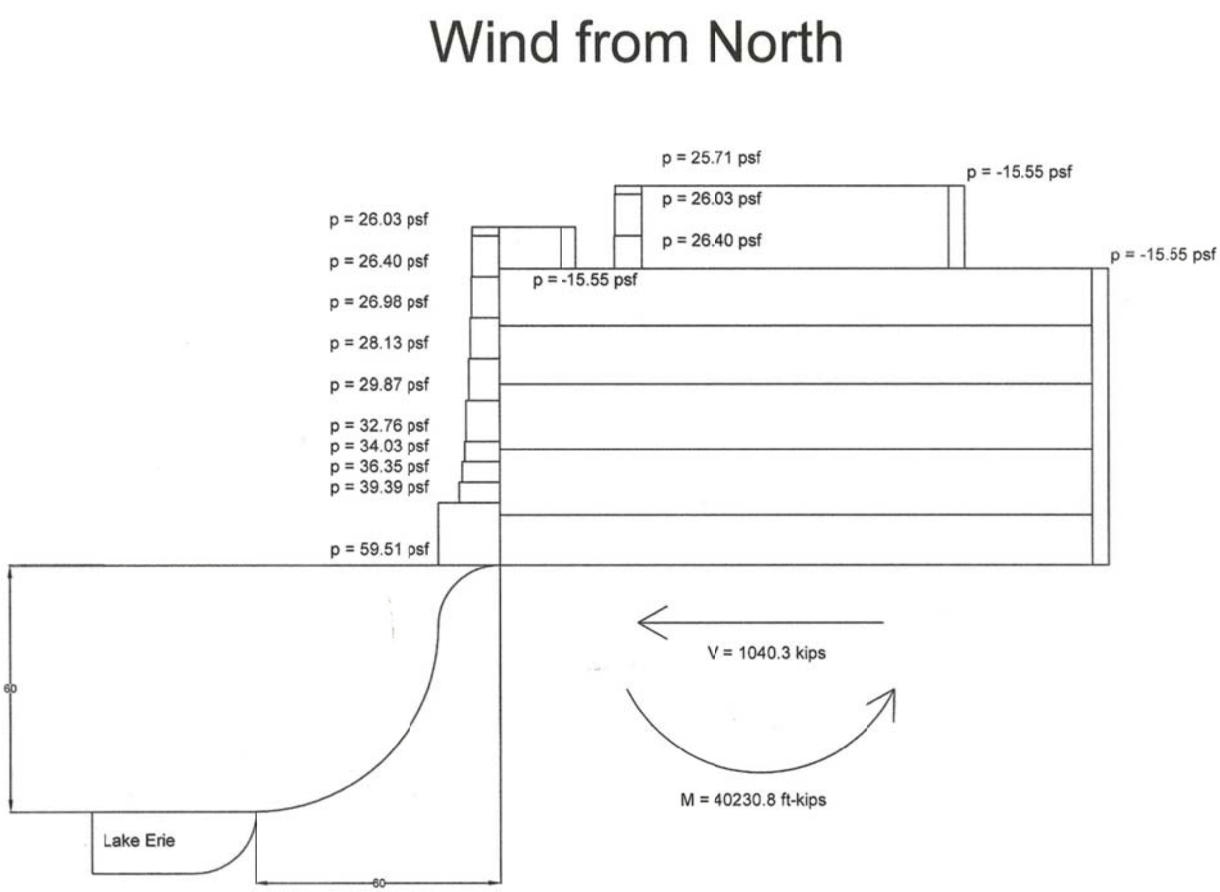


Figure 5: Wind Pressures in N-S Direction, showing 2-D Escarpment

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

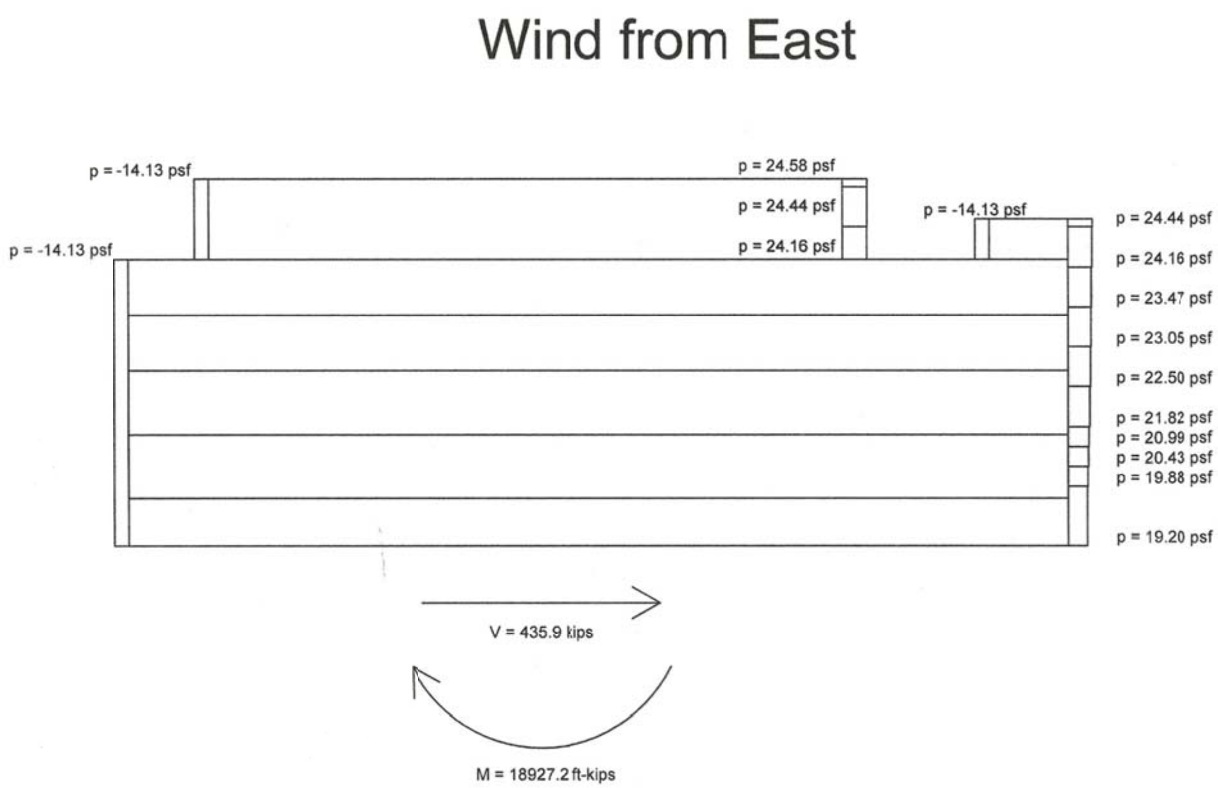


Figure 6: Wind Pressures in E-W Direction

- ## 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 an acceptable 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

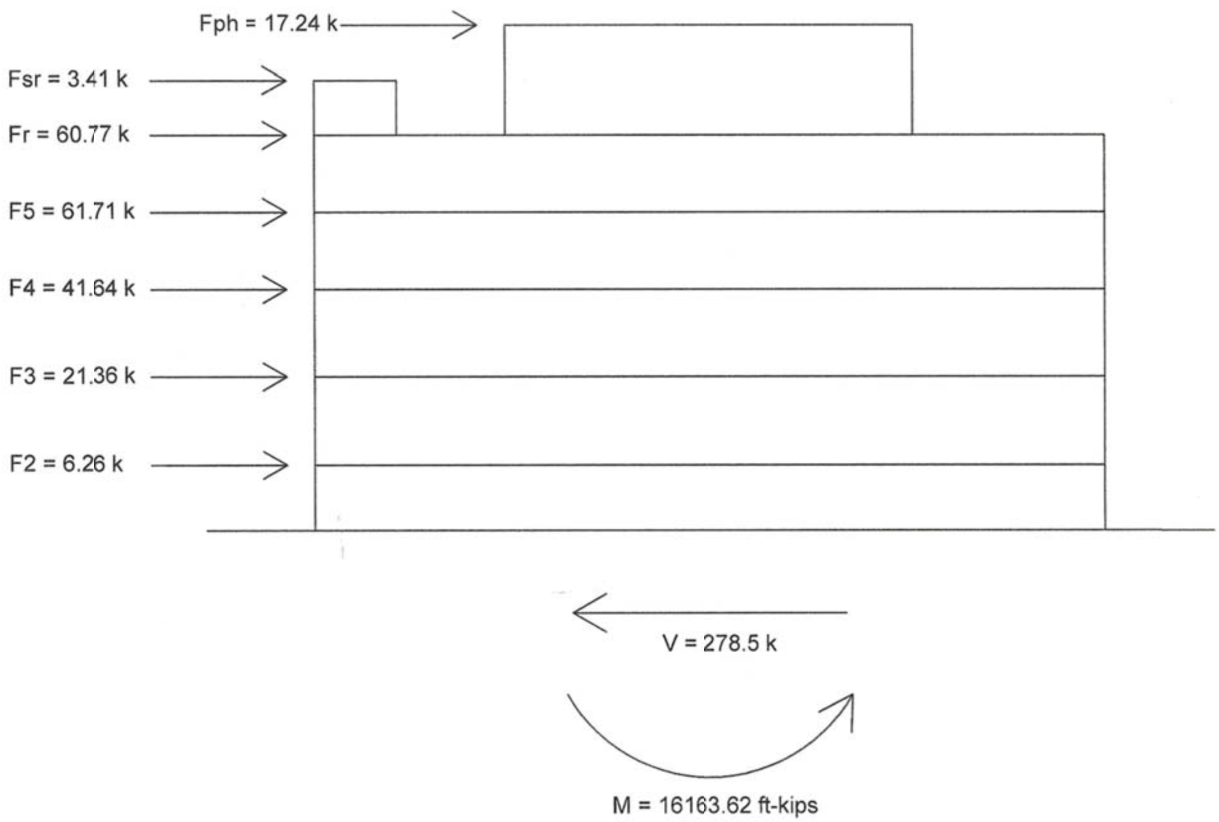


Figure 7: Earthquake Forces at Various Levels

Problem Statement

Technical Reports I, II, and III proved that the gravity and lateral systems utilized in the UPMC Hamot Women's Hospital are adequate for both strength and serviceability requirements. The major question throughout this project was based on the lateral system choice and the construction methods chosen with respect to tearing down the existing structure and starting over, or to do as the design team did and use the lower floors of the existing structure while reinforcing as needed. The decisions made for these issues were driven by various factors, primarily the architecture and building cost. The architect desired an open floor plan and was clear that the use of braced frames could not work with his visions for the spaces. Thus the use of very long moment frames was used; these connections are expensive and time consuming to produce. The construction team deemed that the use of the existing building floor plan would lead to the most cost effective building, although this would require almost all of the existing columns and beams to be reinforced, as well as several of the footings needing to be excavated and reinforced. A detailed cost analysis of this was never actually done, but the recommendation of the construction team was taken. The use of the existing system is also not desirable because many of the spans were deemed to not be as efficient, through using long-long-short spans. Designing a system that can incorporate with the architecture as well as be a more cost effective alternative is what is desired.

Problem Solution

Through the discoveries of these various Technical Reports and background knowledge of the building history various aspects of this project shall be analyzed. First a comparison of building codes (ASCE 7-05 vs. ASCE 7-10) will be done with special care being taken to analyze how the changes to the wind loading sections of the code affect this and other structures. This will be done because of two reasons. Primarily I feel that the new version of the code altered the occupancy category classifications, such that this building would change occupancy categories and thus be subject to a different loading. The other reason being that the student will be designing based on the new code upon graduation, so a more thorough investigation would be beneficial to the educational process. Secondly the existing moment frames will be redesigned as braced frames, with special care being taken to incorporate them with the current architectural theme, or conceal them within the structure as needed. This will be done using the loads determined through the use of ASCE 7-05 to allow for an equivalent comparison to the lateral system that is being utilized in the existing building. Adding braced frames where the current lateral system is located may prove to be difficult, although to move the frames in one column line will allow for the structure to be hidden as the architect requested. Then an analysis will be done to examine how a complete demolition of the existing facility could have affected the structure. This will be accomplished through finding new locations for columns (not using the existing grid), hopefully being able to find a more efficient layout. Constraints will be imposed to maintain the same building footprint and room areas, etc. This will allow for fewer construction cost variable and a more accurate final assessment. Obviously these alterations will affect other aspects of the building. For example placing braced frames inside of a wall will require a wider wall system and possible relocation of doors. These issues will be dealt with through various breadth topics discussed below.

Breadth Study I

The redesign of the buildings lateral system will undoubtedly have an impact on the buildings architecture. The architect was adamant about not having braces disrupt the floor plan of the hospital. Currently the lateral frames in the East-West direction are located at the far extremes of the building. So adding braced frames here may prove to be difficult, although to move the frames in one column line will allow for the structure to be hidden as the architect requested. Hiding the frames inside of the wall will also maintain the open rooms and sight lines, but may require the relocation of some of the doors within the building. Adjusting the building floor plan appropriately may be required. A study of the exterior façade and how braced frames along the exterior will affect the views that the hospital is famous for will also be done. Thus an architectural breadth will be required to analyze how these changes influence the architectural appearance of both the interior and exterior design of the UPMC Hamot Women's Hospital.

Breadth Study II

When redesigning the building as a 'new' structure and thus ignoring the existing column grid and starting from scratch poses a very interesting question. Will it cost more? This is unknown but was speculated by the construction team to be the case. A more in-depth study of this should be done before any conclusion can be drawn. Thus a construction management breadth will be done to analyze the cost and schedule differences associated with this alternative. This breadth will include a detailed schedule comparison, of both the existing schedule and the proposed new building schedule. Then a detailed cost analysis will be done to determine feasibility based on cost and schedule, with schedule implications being considered in the overall cost analysis.

MAE Course Related Study

Utilizing the knowledge gained through taking AE 534, Steel Connections, several typical connections will be analyzed for both the existing moment connections and the alternative braced connections.

Information gained through taking AE 597A, Computer Modeling of Building Structures, will also be utilized but adapted to this project. RAM Structural Systems will be the primary method of computer analysis. This platform was not explicitly taught as part of this course, but through teaching myself this platform and not blindly trusting the computer solution, the coursework becomes applicable.

Tasks and Tools

ASCE Load Comparison

- Comparing ASCE 7-05 and ASCE 7-10 Loads
    - Gravity Load Comparison
    - Lateral Load Comparison
        - Wind and Seismic
- Determine impact that the new code would have on this structure
    - Increase or decrease member sizes?
        - Columns Only
        - Beams Only
        - Both Columns and Beams
    - Increase or decrease in load results in heavier or lighter building?
- Discuss and compare results

Braced Frame Analysis and Design

- Locate potential places within the existing structure where a braced frame could be hidden from the building occupants (Sketch Elevations)
    - Using RAM Steel determine the required member sizes to support both gravity and lateral loads for both strength and serviceability
    - Analyze the structural impacts of changing the lateral system
    - Analyze the architectural impacts of changing the lateral system
        - Effects on both interior and exterior architectural considerations
- Design connection for the existing moment frame
- Design connections for the typical braced frame
- Discuss and compare results

New Building Analysis and Design

- Determine a feasible alternative column grid within the existing structure
- Use RAM Steel to design the gravity and lateral system for the new column grid
    - Compare the alternative structure to the existing structure based on structural weight and performance characteristics
- Determine cost comparisons between the as built and alternative building
- Determine schedule comparisons between the as built and alternative building
- Discuss and compare results

Thesis Schedule

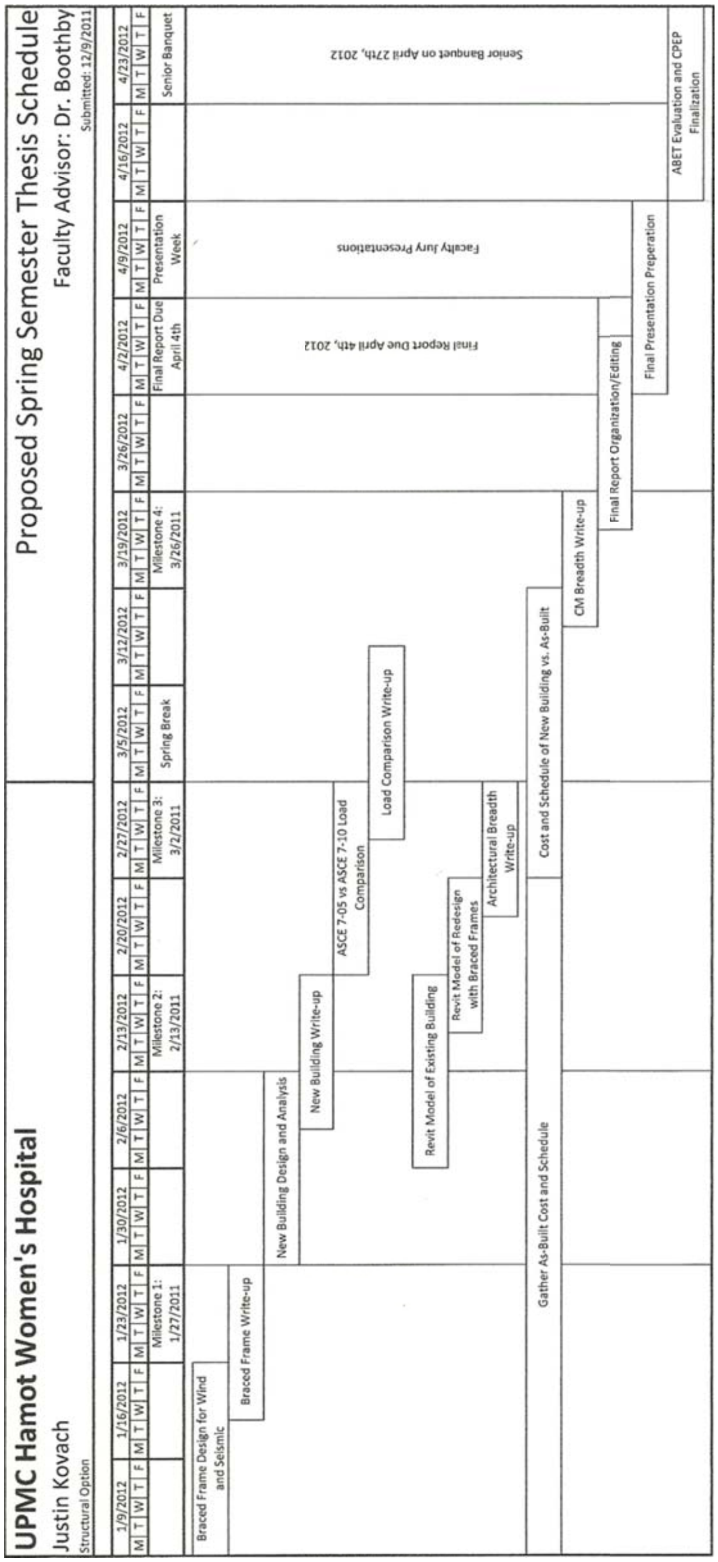


Figure 8: Proposed Thesis Schedule

Conclusion

The structural depth for this thesis will be split up into three distinct investigations. An investigation on the new building code with a comparison to the previous edition and how it affects the structural weight and performance will be done. An investigation into the possibility of effectively utilizing braced frames rather than moment frames will be completed. Finally, an investigation into a complete building redesign without using the existing structure or grid will be done to determine if the correct decision was made by the construction team.

As these elements are completed two breadth studies will be undertaken. An architectural breadth will be done to analyze the impact on the architecture that the braced frame system has on the building. A construction management breadth will also be done to analyze the impact of not using the existing structure and grid to build from.

Results from all of these studies will be summarized in a final report on or before April 4th, 2012.

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

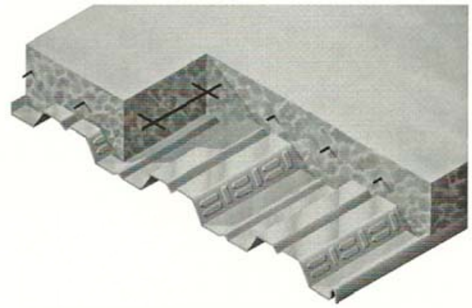
Roof Weight

1½" Galvanized Steel Roof Deck - 20 GA ↳ Wide Rib Deck	= 2 psf
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
4.00 (t=2.00) 30 PSF	2VLI22	8'-1	10'-3	10'-7	238	209	186	167	152	120	108	98	90	82	75	69	64	59	55
	2VLI20	9'-6	11'-8	12'-1	268	235	209	187	169	153	140	129	101	92	84	78	72	66	61
	2VLI19	10'-10	13'-0	13'-2	297	260	230	206	185	168	153	141	130	121	93	86	79	73	68
	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
4.50 (t=2.50) 35 PSF	2VLI22	7'-8	9'-10	10'-2	276	243	216	194	155	139	126	114	104	96	88	81	75	69	64
	2VLI20	9'-0	11'-3	11'-7	312	273	243	217	196	178	163	128	117	107	98	90	83	77	72
	2VLI19	10'-3	12'-5	12'-9	346	302	268	239	215	195	178	164	151	118	108	100	92	85	79
	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
5.00 (t=3.00) 39 PSF	2VLI22	7'-4	9'-5	9'-9	315	277	247	197	176	159	143	130	119	109	100	92	85	79	73
	2VLI20	8'-7	10'-9	11'-2	355	312	276	248	224	203	161	146	133	122	112	103	95	88	82
	2VLI19	9'-9	11'-11	12'-4	394	345	305	272	245	223	203	187	147	135	124	114	105	97	90
	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
5.25 (t=3.25) 42 PSF	2VLI22	7'-2	9'-3	9'-7	334	294	262	209	187	168	152	138	126	116	106	98	90	84	78
	2VLI20	8'-5	10'-7	10'-11	377	331	293	263	237	190	171	155	142	130	119	110	101	94	87
	2VLI19	9'-6	11'-8	12'-1	400	366	324	289	260	236	216	198	156	143	131	121	111	103	95
	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
5.50 (t=3.50) 44 PSF	2VLI22	7'-0	9'-1	9'-5	353	311	277	222	198	178	161	147	134	122	113	104	96	89	82
	2VLI20	8'-3	10'-4	10'-9	399	350	310	278	251	201	181	165	150	137	126	116	107	99	92
	2VLI19	9'-4	11'-6	11'-10	400	387	342	306	275	250	228	182	165	151	139	128	118	109	101
	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
6.25 (t=4.25) 51 PSF	2VLI22	6'-8	8'-7	8'-11	400	362	291	258	231	208	188	171	156	143	131	121	112	103	96
	2VLI20	7'-9	9'-10	10'-2	400	400	361	323	260	234	211	192	175	160	147	135	125	115	107
	2VLI19	8'-9	10'-11	11'-3	400	400	398	356	320	291	233	212	193	176	162	149	137	127	118
	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

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

VULCRAFT

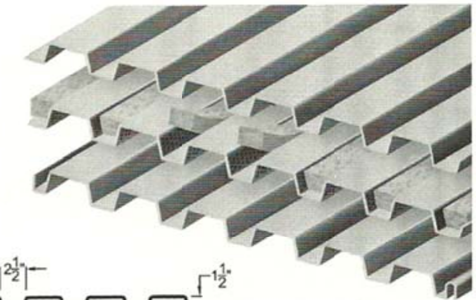
ROOF

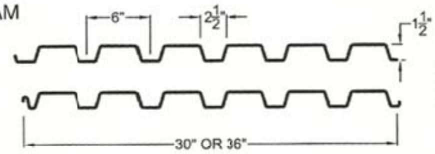
1.5 B, BI, BA, BIA

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

Allowable Total (PSF) / Load Causing Deflection of L/240 or 1 inch (PSF)
Span (ft.-in.) ctr to ctr of supports

No. of Spans	Deck Type	Max. SDI Const. Span	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 minimum lengths are not provided, web crippling must be checked.

NUCOR VULCRAFT GROUP

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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)

$$\gamma = 0.13 p_s + 14 = 0.13(40) + 14 = 19.2 \text{ psf}$$

N-S Drift

$\ell_u = 60'-0"$
$h_c = 20'-0"$

$\therefore h_d = 2.8'$

E-W Drift

$\ell_u = 140'-0"$
$h_c = 20'-0"$

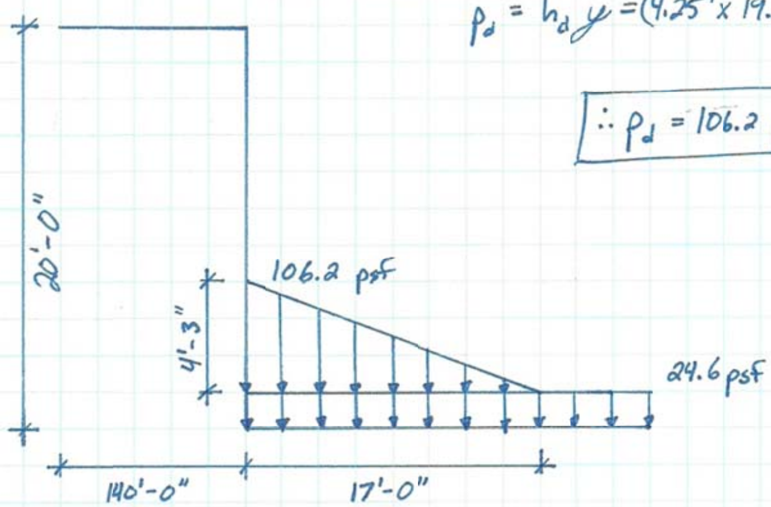
$\therefore h_d = 4.25'$

$\therefore$ Use $h_d = 4.25'$

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

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

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



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

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Stair Pop-Out)

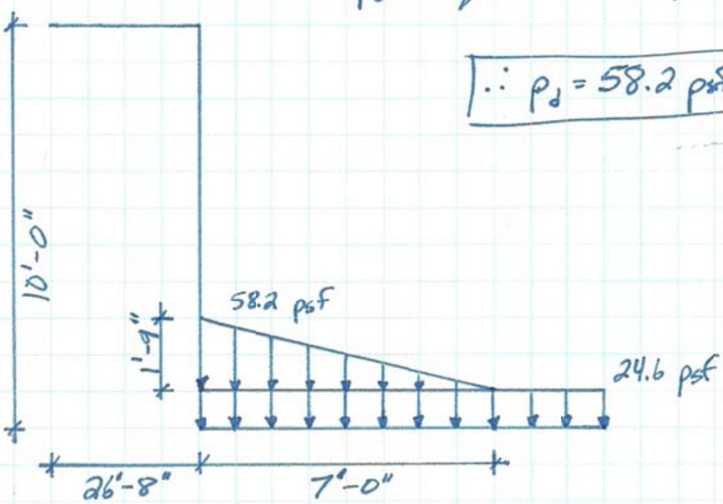
$$\gamma = 0.13 p_g + 14 = 0.13(40) + 14 = 19.2 \text{ pcf}$$

N-S Drift	E-W Drift
$\ell_u = 10'-10''$	$\ell_u = 26'-8''$
$h_c = 10'-0''$	$h_c = 10'-0''$
$\therefore h_d = 1.75'$	$\therefore h_d = 1.75'$

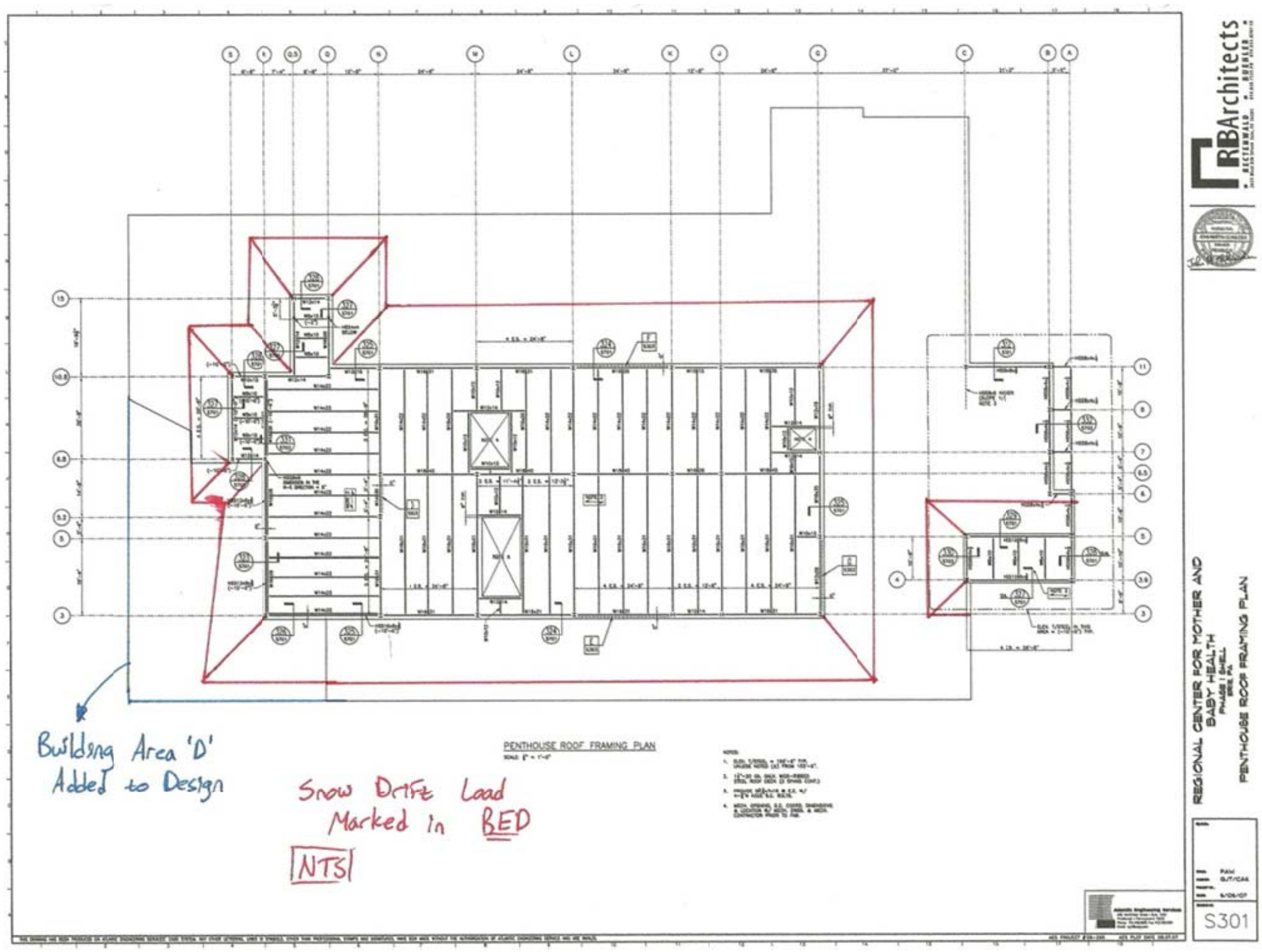
$$w = 4 h_d = 4(1.75') = 7'-0''$$

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

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



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

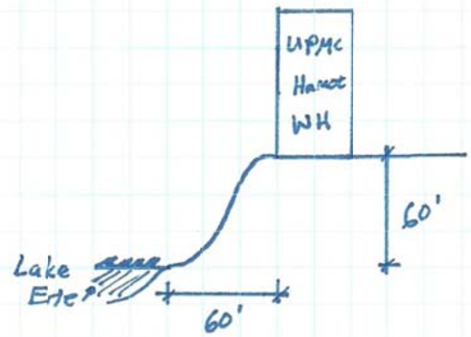
80'-90' = 1.40
90'-92' = 1.41 → Interpolated Value

C.2 – Wind Calculations (con't)

Wind Loads (cont)

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

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



$K_1 = 0.95(1.0)$

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

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

$= 1$

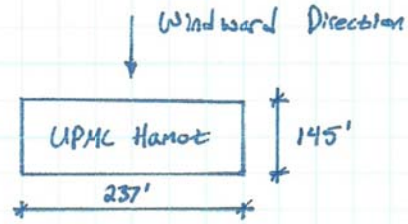
· 2D Escarpment
· Exposure D
· $H/L_H = 60/60 = 1.0$

$K_3 = e^{-\gamma z/L_H}$ $\quad \gamma = 2.5$

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 Bulding ⇒ 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

$G = 0.85$

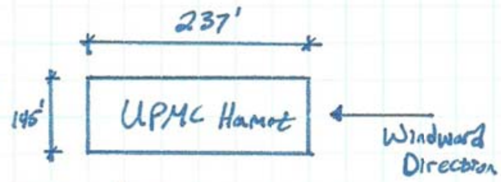
Enclosed Building ⇒ Figure 6-5

$GC_{pi} = \pm 0.18$

C.6 – Wind Calculations (con't)

Wind Loads (cont)

C_p Values ⇒ Figure 6-6



Plan View

$C_p = 0.8$ ⇒ 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 (con't)

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)

EQ Loads (con't)

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