Technical Report II

2011-2012 AE Senior Thesis

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Executive Summary

The following technical report analyzes four slab systems that are possible for use 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 systems were chosen and then analyzed using the IBC 2006 building code, which is the design code enforced on the building at its time of construction.

The 163,616 sq. ft. Women's Hospital was completed in early January of 2011. This structure has a 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.

The goal of this study of alternate floor systems was to examine and assess the feasibility of each of the systems. In the list that follows are the three floor systems that were researched, analyzed, and designed for this study.

- One Way Concrete Slab and Beam
- Precast Hollow Core Planks on Steel Beams
- Long Span Composite Steel Deck on Steel Beams

These systems were evaluated using both structural and non-structural criteria; a summary chart of these comparisons is presented near the end of this report. Each system's viability was analyzed based on the structural and non-structural criteria noted, and a decision was made. The concrete system was determined to be possible, but a further analysis would be required due to the increased weight of the system; the earthquake loading will increase and overturning may become an issue. If overturning becomes an issue then a different foundation system may need to be explored. The two other systems were determined to be feasible and viable options, with both of these systems having advantages and disadvantages.

Introduction

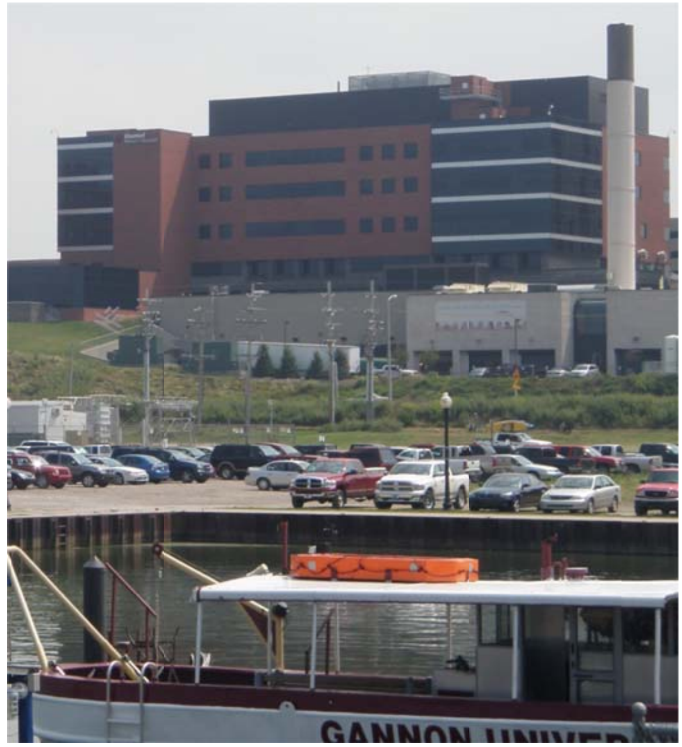


Figure 1: North Façade, Showing 2-D Escarpment

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.

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.

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.

Figure 2: Interior Water Wall

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 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 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 modifications to the spread footings are unique because many of the existing spread footings had to be increased in 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, 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 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

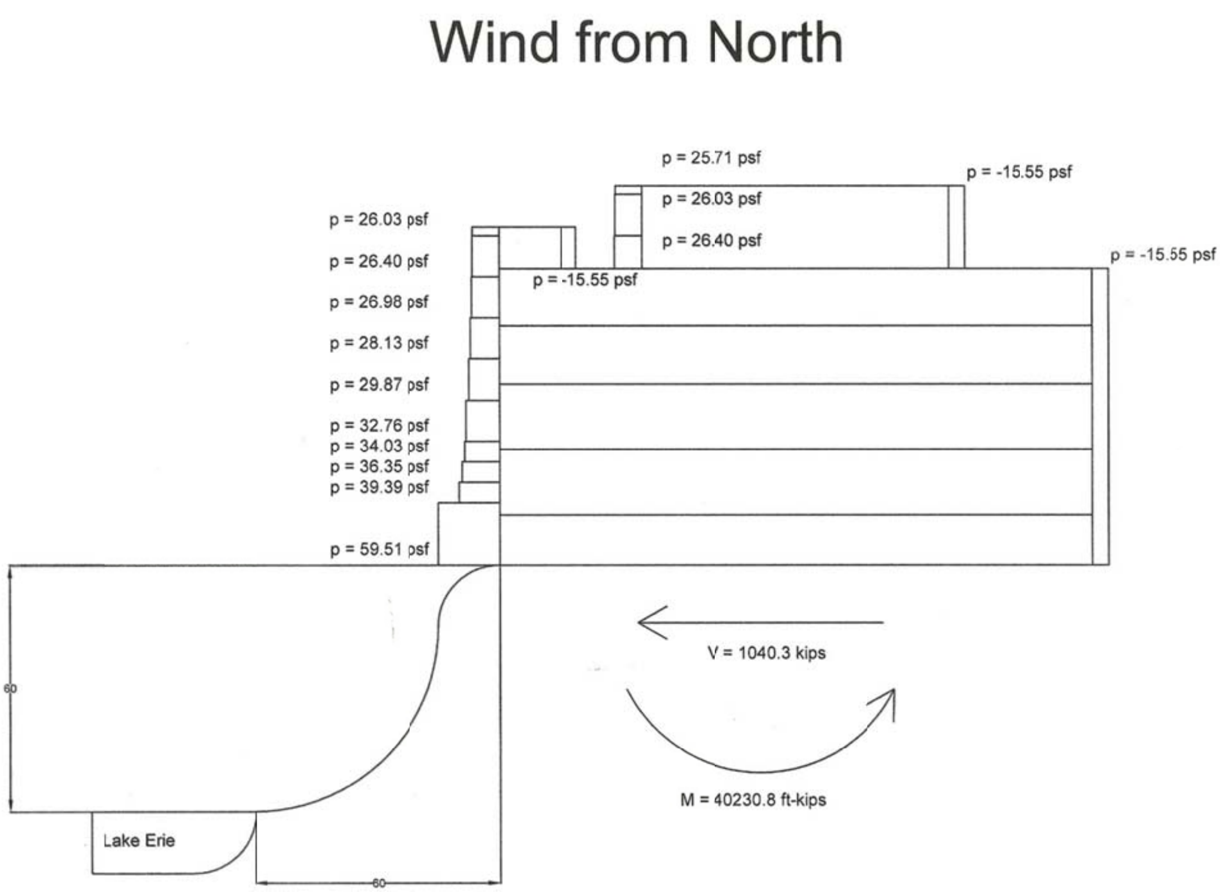


Figure 5: Wind Pressures in N-S Direction, showing 2-D Escarpment

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

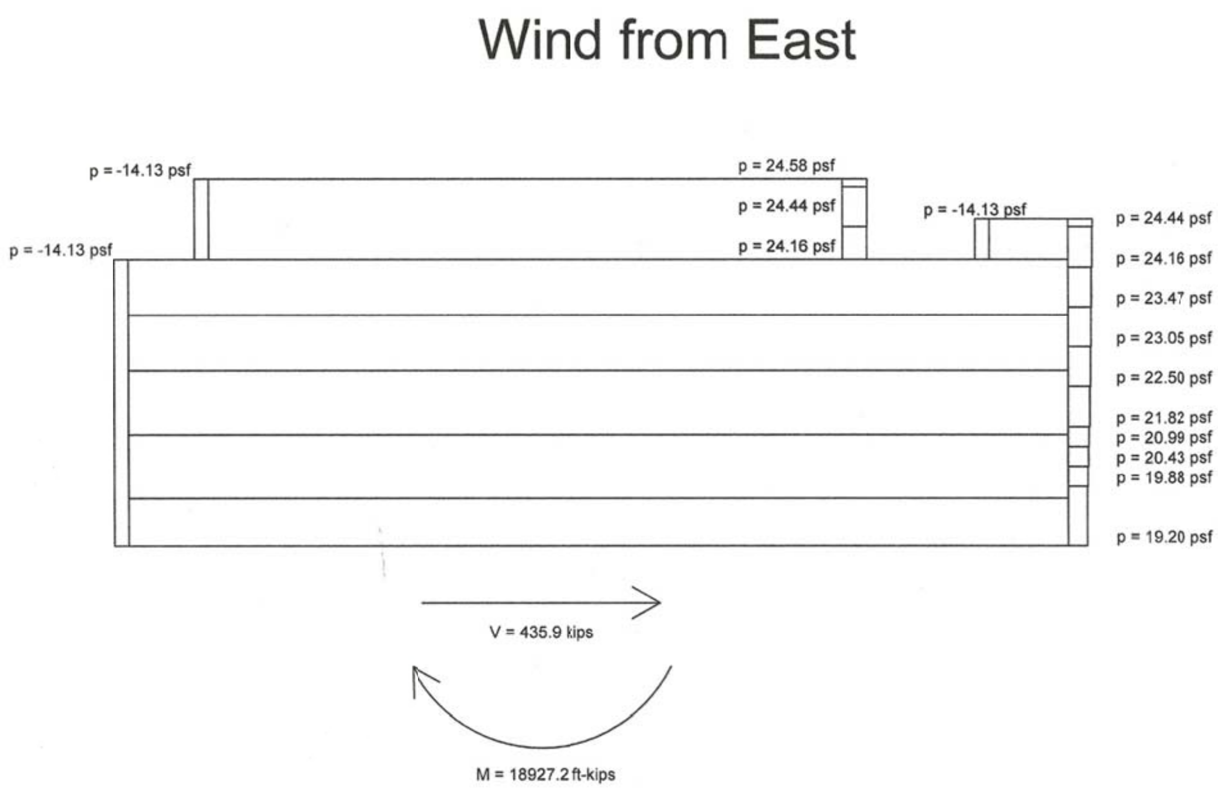


Figure 6: Wind Pressures in E-W Direction

• 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

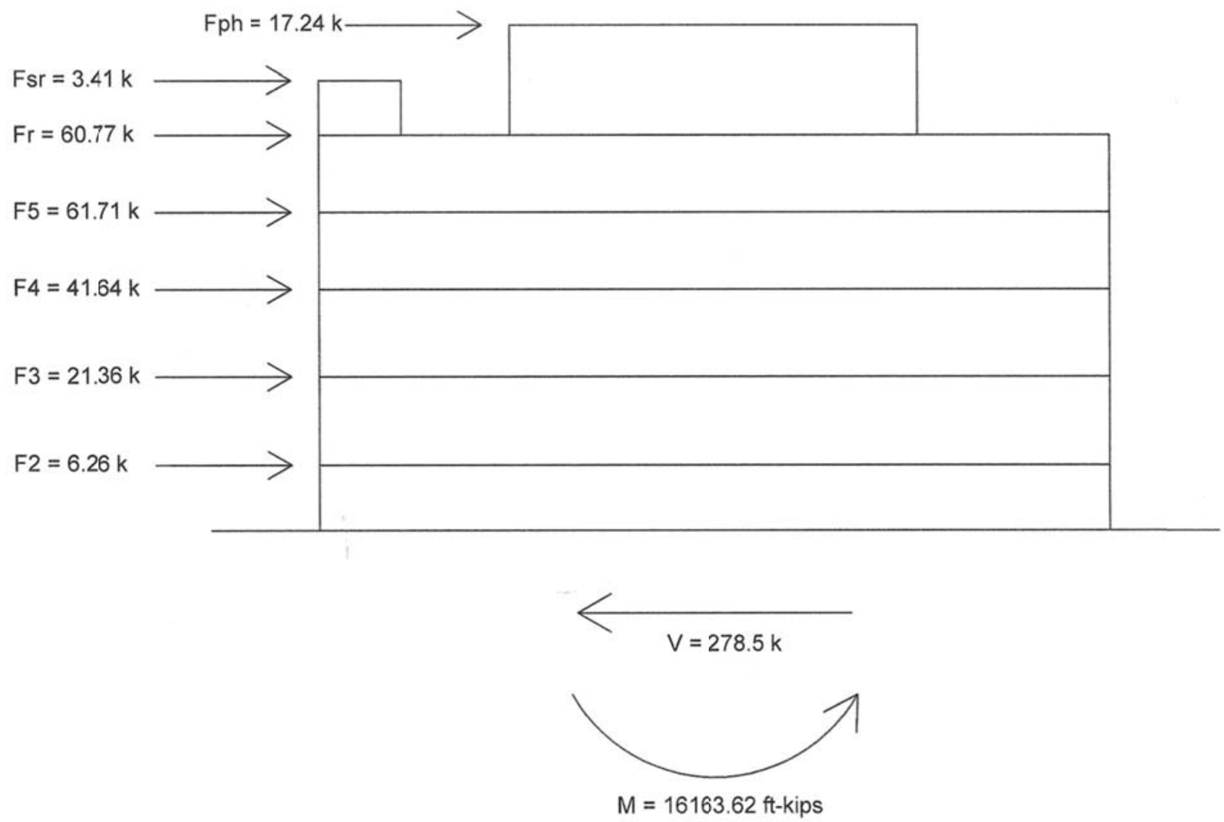
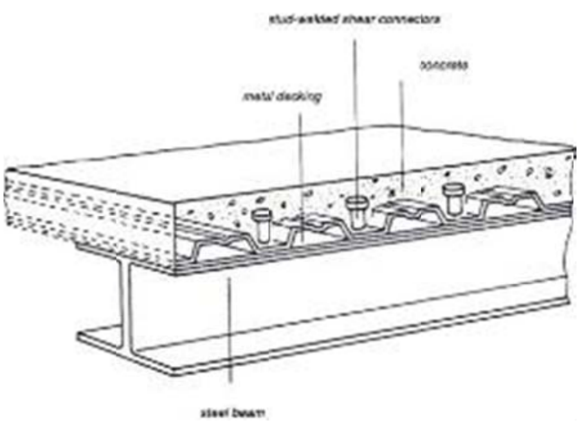


Figure 7: Earthquake Forces at Various Levels

Floor System Analysis

- ## Lightweight Concrete on Composite Metal Deck Calculations

A composite metal deck floor system consists of a high strength structural steel deck and a structural concrete slab, with reinforcement (typically just temperature and shrinkage). This floor system provides both economy and efficiency through taking advantage of the composite action between the steel deck and the concrete. By utilizing the lightweight concrete rather than the normal weight concrete, it is possible to lighten the floor system, which may decrease your beam and girder sizes, but will most definitely reduce your column and foundation sizes.



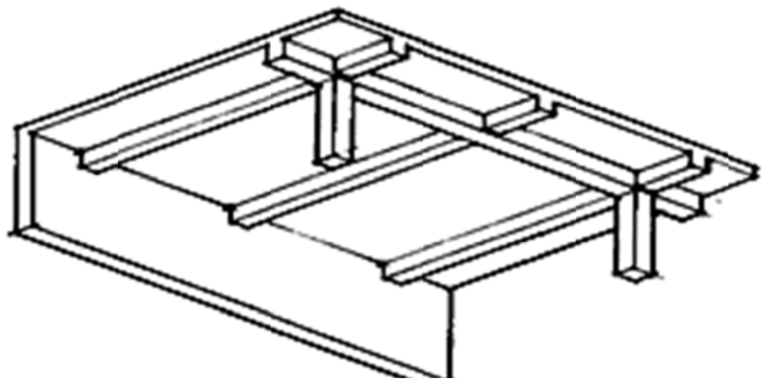
Calculations were performed on this system and these calculations yielded the need for 2VLI20 deck to achieve the 2 hour fire rating that is required. The composite beams were sized for to support the slab and deck self-weight, plus the superimposed dead and live loading. The composite beams were determined to be W14x22[10], thus requiring 10 shear studs spaced equally along the length of the beam. The composite girders were then sized to support the loads from the beams and its own self-weight. This design yielded a W16x26[18], thus requiring 18 shear studs placed evenly along the length of the girder. The loads from the floor system were then transferred through the girders and into the column. The column was assumed to be spliced 4'-0" above the 2nd and 4th floors. Live Load reduction was utilized and the column design yielded a W8x48 column above the 4th floor, and a W8x67 below the 4th floor. The details of these calculations can be found in Appendix F.

The effective weight of the structure was then determined for statistical purposes. This was only done for the typical bay, which is not truly representative of the entire structure, but will provide a basis for comparison. Determining the weight was done to allow me to grasp what impacts the gravity system may have on the lateral system, since earthquake loading is dependent on the effective weight of the structure; this structure weighed in at 229.74 kips.

This floor system, like all floor systems, has many advantages and disadvantages. This system is typically very light, which will allow for smaller members leading to a cheaper structure, the system also utilizes the floor slab when designing the beam, thus making the beam the most efficient. The system is also typically very quick to construct on site. The major disadvantage of this system is that it uses a lightweight concrete, which is more expensive than the normal weight concrete, this cost can be offset with the reduction in structural weight in the beams, girders, and columns; but this is dependent on the number of floors present in the building.

- **One-Way Slab Calculations**

A one-way concrete floor system consists of a slab with supporting beams and girders. For a bay to be analyzed as one bay it must meet the required aspect ratios. This system utilizes the one way slab and beams to allow for a shallower system, although overall structural weight becomes a concern due to increased seismic loading.



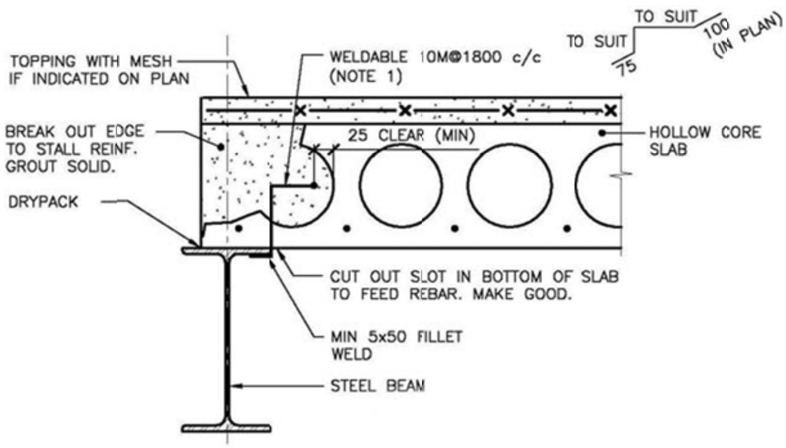
Calculations were performed on this system and these calculations yielded the need for a 6” thick concrete slab. The concrete columns were sized for to support the slab self-weight, plus the super imposed dead and live loading. The columns were determined to be 18” x 18” square with (8) - #6 bars spaced along the perimeter of the column. The concrete beams were then sized to support the loads from the slab and its own self-weight. The width of the beam was chosen based on the width of the column (18”), and the depth of the beam was chosen to attempt to keep the floor system at 16” deep (similar to the existing). The beams were analyzed to benefit from the T-beam behavior that one would expect from the slab. The design yielded the need for (8) - #5 bars in the bottom and (5) - #9 bars in the top of the section. The shear reinforcement was determined to require 3 legs of #3 bars spaced at 4” on center. The girder was then designed to support the beams and slab, yielding the need for (6) - #7 bars Top and Bottom, as well as standard #3 ties at 5” on center. The details of these calculations can be found in Appendix G.

The effective weight of the structure was then determined for statistical purposes. This was only done for the typical bay, which is not truly representative of the entire structure, but will provide a basis for comparison. Determining the weight was done to allow me to grasp what impacts the gravity system may have on the lateral system, since earthquake loading is dependent on the effective weight of the structure; this structure weighed in at 464.53kips.

This floor system, like all floor systems, has many advantages and disadvantages. This system is very versatile and adaptive to any shape that is desired, assuming that a form for the concrete can be made. Structural concrete systems typically yield large open bays, with a minimal floor system thickness. Thus potentially allowing for an extra floor in areas where height is a restriction. This system also works very well in controlling vibration issues, although that does not appear to be an issue with the current system. The drawbacks of structural concrete consist primarily of schedule and budget. The concrete requires curring time and with tall buildings with small footprints an issue of curring time can become an issue. Structural concrete is also very labor intensive to place and finish, which has the potential to drive up the cost of the project, especially if schedule delays occur.

• Hollow Core Plank Calculations

A hollow core concrete plank floor system consists of modular prestressed concrete members (or "planks") that are laid parallel to each other. This system provides a drastic improvement in the in span to depth ratio that you would expect with steel members. This system will typically bear on a steel system, but do to the minimal span to depth that is inherent with this system infill beams are typically not needed.



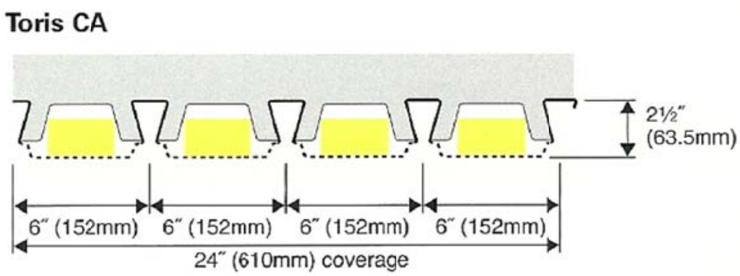
Calculations were performed on this system and these calculations yielded the need for a 6" thick hollow core system, reinforced with (2) - 7/16 and (2) – 3/8 strands The beams were sized to brace the columns, because in theory they carry no load based on their orientation to the floor system. The beams were sized to be W10x14, which was determined based on engineering judgment. The girders were then sized to support the loads from the precast concrete plank and its own self-weight. This design yielded a W14x74, which was determined based on a self-imposed depth limit of 16", to control excessive floor depths. The loads from the floor system were then transferred through the girders and into the column. The column was assumed to be spliced 4'-0" above the 2nd and 4th floors. Live Load reduction was utilized and the column design yielded a W8x31 column above the 4th floor, and a W8x40 below the 4th floor. The details of these calculations can be found in Appendix H.

The effective weight of the structure was then determined for statistical purposes. This was only done for the typical bay, which is not truly representative of the entire structure, but will provide a basis for comparison. Determining the weight was done to allow me to grasp what impacts the gravity system may have on the lateral system, since earthquake loading is dependent on the effective weight of the structure; this structure weighed in at 211.59 kips.

This floor system, like all floor systems, has many advantages and disadvantages. This system is typically very light, because it mixes the steel system with the concrete system, strategically placing the "voids" in the concrete slab where the concrete is not very efficient. The system is also typically very quick to construct on site as long as enough cranes are present. The major disadvantage of this system is that it requires the use of a crane to move it around the site; in order to keep up with this additional crane time required it would probably require a second crane. This costs extra money and may not be possible on a very tight site. This cost can be offset with the reduction in structural weight in the beams, girders, and columns; but this is dependent on the number of floors present in the building.

- **Long Span Deck Calculations**

The long span deck system offered by EPIC Metals Corporation is designed to be more than just a deck that can achieve long lengths without support from the structure. This system attempts to take an innovative approach to designing a modern, visually unobstructed interior with an architectural appeal. This is done through the deck itself, which is designed to be exposed, thus architectural acoustics becomes a concern. The Toris CA system utilizes noise reduction technology which isn't built into the deck, as well as a hanger system, which is utilized by attaching the fasteners to the dovetails in the decking.



Calculations were performed on this system and these calculations yielded the need for a 7.5" Toris CA slab with 3ksi concrete. The beams were sized to brace the columns and provide redundancy, because in theory they carry no load based on their orientation to the floor system. The beams were sized to be W10x14, which was determined based on engineering judgment. The girders were then sized to support the loads from the long span deck and slab as well as its own self-weight. This design yielded a W16x77, which was determined based on a self-imposed depth limit of 16", to control excessive floor depths. The loads from the floor system were then transferred through the girders and into the column. The column was assumed to be spliced 4'-0" above the 2nd and 4th floors. Live Load reduction was utilized and the column design yielded a W8x31 column above the 4th floor, and a W8x48 below the 4th floor. The details of these calculations can be found in Appendix I.

The effective weight of the structure was then determined for statistical purposes. This was only done for the typical bay, which is not truly representative of the entire structure, but will provide a basis for comparison. Determining the weight was done to allow me to grasp what impacts the gravity system may have on the lateral system, since earthquake loading is dependent on the effective weight of the structure; this structure weighed in at 267.89 kips.

This floor system, like all floor systems, has many advantages and disadvantages. This system utilizes the floor slab when designing the girders, thus making the girders more efficient, as well as utilizing the "long-span" aspect of the slab allows for the elimination of the infill beams. The systems best attributes are in the architectural area. This system is intended to be left exposed on the underside and also comes equipped with a hanger system which allows for mechanical systems, lights, etc. to be hung from the underside. This leads to a nice aesthetically pleasing ceiling system. The major disadvantage of this system is that it costs drastically more than the typical composite floor system. This cost can be offset with the reduction in structural weight through the elimination of the infill beams.

Floor System Summary

Floor System Summary				
	Existing	**Alternatives**		
	Composite Steel Deck	**One-Way Concrete Slab**	**Hollow Core Plank on Steel**	**Long Span Composite Deck on Steel**
Bay Size Changes	NO	NO	YES	NO
System Depth	24”	16”	24”	23.5”
System Cost (per Square Foot)	$19.95	$17.65	$10.39 + Structural Steel	Unknown, but more than Composite Steel on Deck
Additional Fire Protection	Yes (Structural Steel Only)	NO	Yes (Structural Steel Only)	Yes (Structural Steel Only)
Constructability	Moderate	Difficult	Easy	Moderate
Viability	YES	POSSIBLE	YES	YES

Foundations:

The foundations of the UPMC Hamot Women’s Hospital have been sized based on allowable bearing in most cases. This indicates that the foundation sizing is proportional to the weight of the building above, thus changing systems would undoubtedly have an impact on the foundations. Increasing the weight drastically may require a different foundation system all together, thus the viability of the concrete system should include a more detailed analysis of the implications on the foundation and the increased lateral loads.

Conclusion

As a result of this study, the feasibility of these alternative floor systems have been determined. Through the design of these systems with the same superimposed dead load and live loading and assessing the systems with both the structural and non-structural criteria allows a direct comparison with the existing floor construction.

The precast hollow core plank on steel beam system and the long span composite deck system were both determined to be feasible options for the UPMC Hamot Women's Hospital. The long span system would not require any changes to bay sizing, although the precast hollow core system would require the changing of the bay sizes to a 4'-0" increment. Changing the bay sizes could affect the size of the various rooms and hallways enclosed in the building, which could be an issue for the architect and owner.

The one-way concrete slab and beam system was determined to viable to this point, but the implication on the lateral loading and the foundation system of this much heavier option is still yet to be determined and could be considered as part of Technical Report 3 or as a proposal for the spring semester.

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

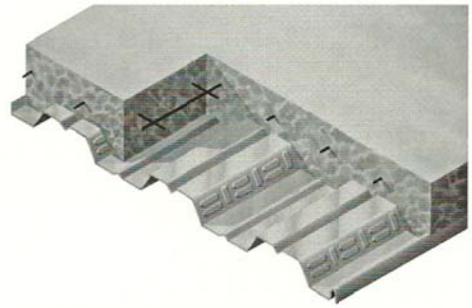
Roof Weight

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 Yd^3 / 100 ft^2	Theo. Concrete Volume ft^3 / ft^2	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

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

VULCRAFT

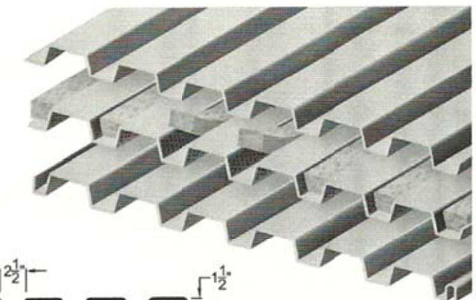
ROOF

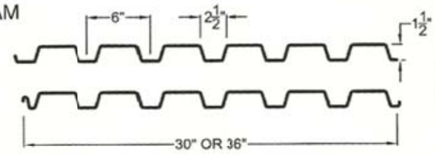
1.5 B, BI, BA, BIA

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)

No. of Spans	Deck Type	Max. SDI Const. Span	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

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

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

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

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Penthouse Roof)

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

N-S Drift

$l_u = 60'-0"$
$h_c = 20'-0"$

$\therefore h_d = 2.8'$

E-W Drift

$l_u = 140'-0"$
$h_c = 20'-0"$

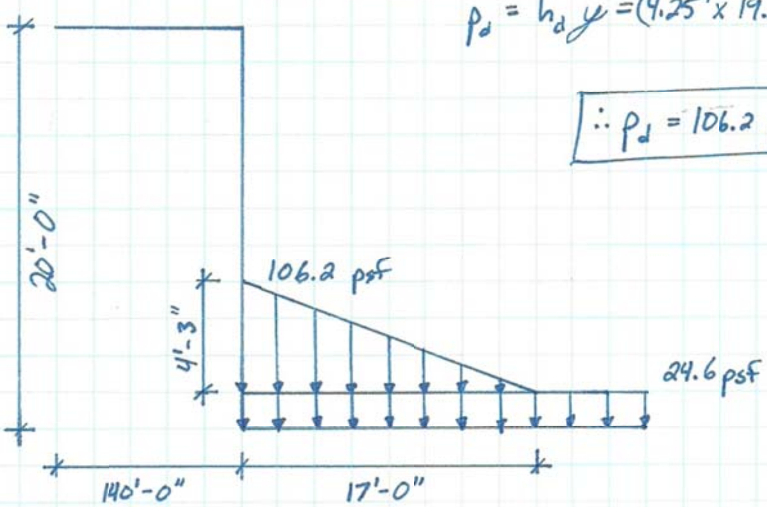
$\therefore h_d = 4.25'$

$\therefore$ 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

$\therefore p_d = 106.2$ psf



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

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Stair Pop-Out)

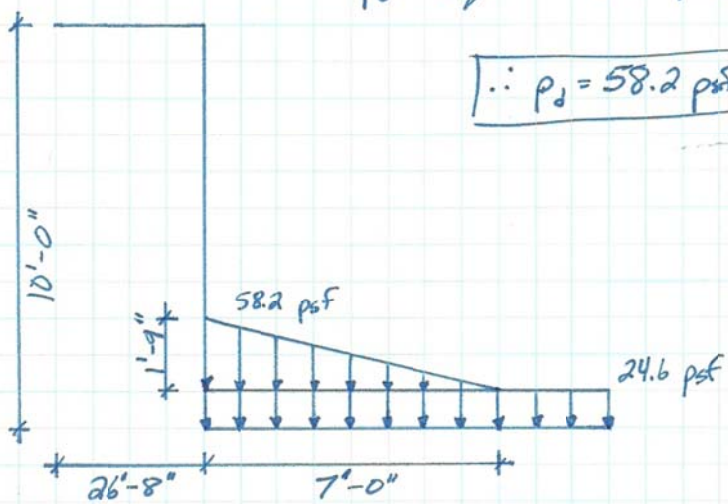
$\gamma = 0.13 p_g + 14 = 0.13(40) + 14 = 19.2$ pcf

N-S Drift	E-W Drift
$l_u = 10'\text{-}10''$	$l_u = 26'\text{-}8''$
$h_c = 10'\text{-}0''$	$h_c = 10'\text{-}0''$
$\therefore h_d = 1.75'$	$\therefore h_d = 1.75'$

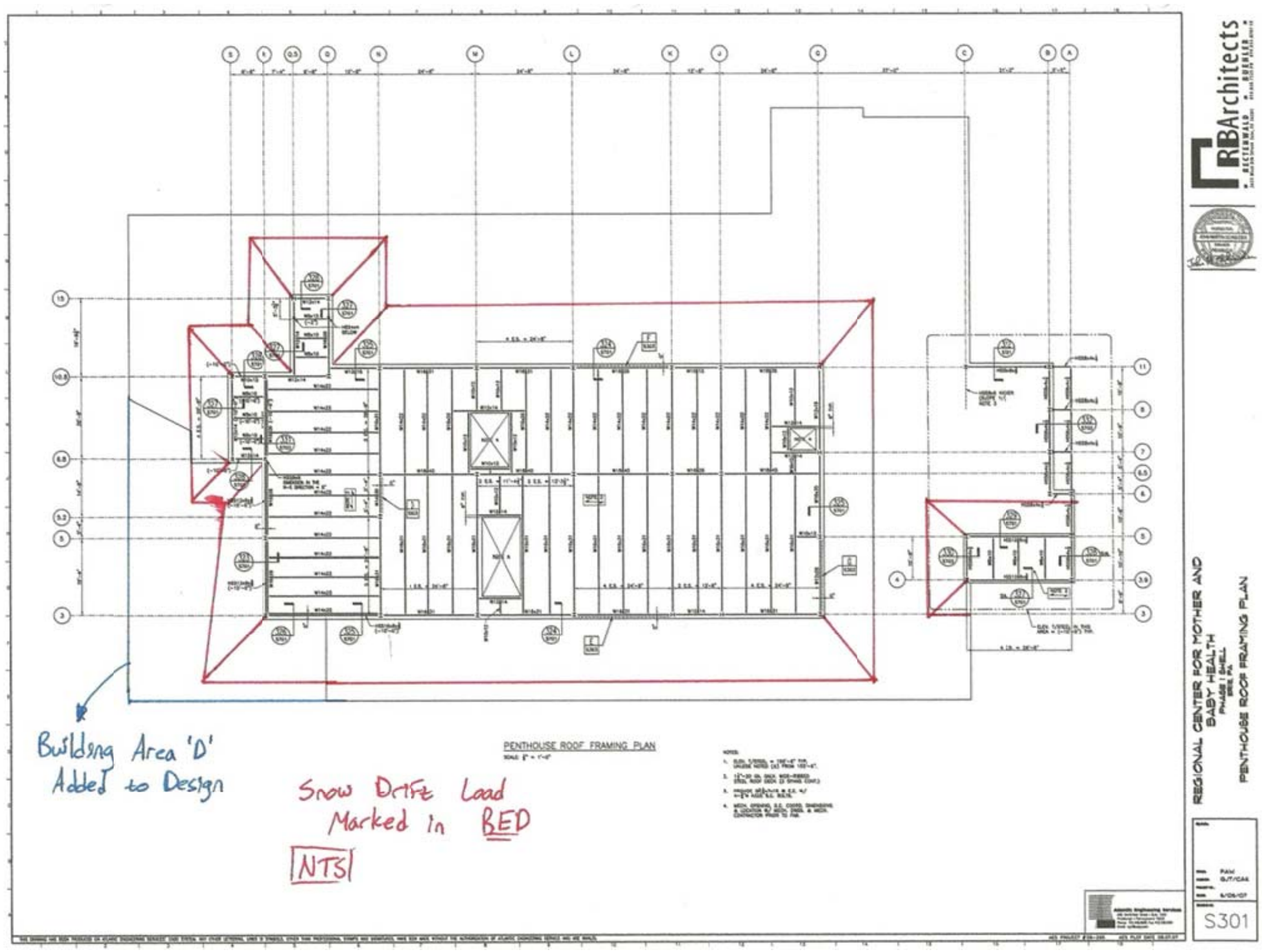
$w = 4 h_d = 4(1.75') = 7'\text{-}0''$

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

$\therefore p_d = 58.2$ 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

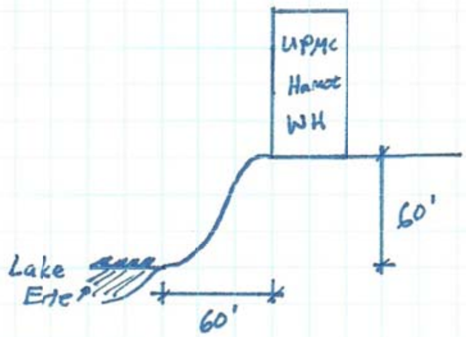
80'-90' = 1.40
90'-92' = 1.41 → Interpolated Value

C.2 – Wind Calculations (con't)

Wind Loads (cont)

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

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



$$K_1 = 0.95(1.0)$$

$$K_2 = \left(1 - \frac{|x|}{\mu L_H}\right) = \left(1 - \frac{0}{4(60)}\right) = 1$$

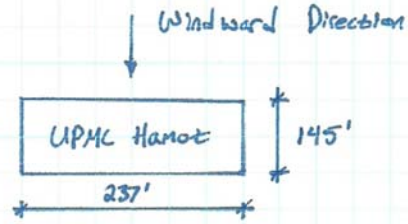
- 2D Escarpment
- Exposure D
- $H/L_H = 60/60 = 1.0$

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

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

$G = 0.85$

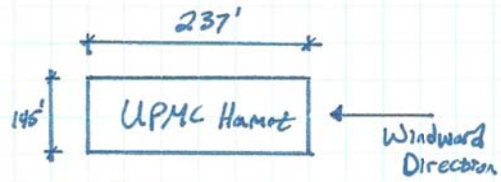
Enclosed Building ⇒ Figure 6-5

$GC_{pi} = \pm 0.18$

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'

Plan View

$L/B = 237/145 = 1.63$

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

$T = C_u T_a$

$T_L = 12$ ⇒ 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

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

$\therefore C_s = 0.0183$

$V = C_s W = 0.0183(11,606)$

$\therefore V = 212.39^k$

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)

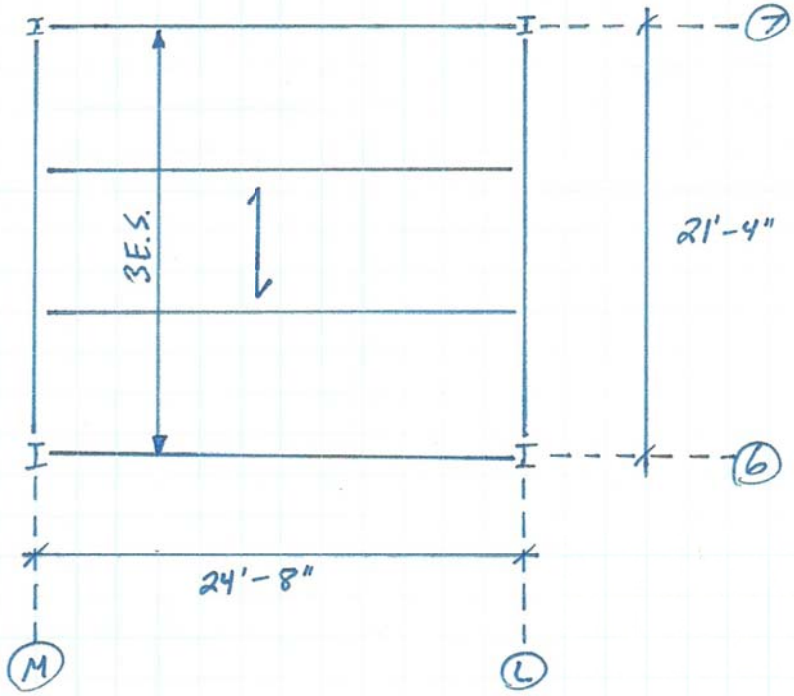
EQ Loads (cont)

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

Appendix E: Lightweight Concrete on Metal Deck Calculations

E.1 Lightweight Concrete on Metal Deck Caculations

Composite ⇒ Concrete on Metal Deck (Existing)



Loads

DL = 73 psf → Slab Weight = 48 psf
LL = 80 psf

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

t = 4" → Total Thickness = 6"

2VLI20 Deck

Max Unshored Clear Span (3 Span Condition)

$s = 7'\text{-}2" < 10'\text{-}9" = s_{max}$ ∴ OK

E.2 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ (Concrete on Metal Deck)

Decking

Loads

Dead = 73 psf
Live = 80 psf
153 psf
− 48 psf → Deck + Slab SW

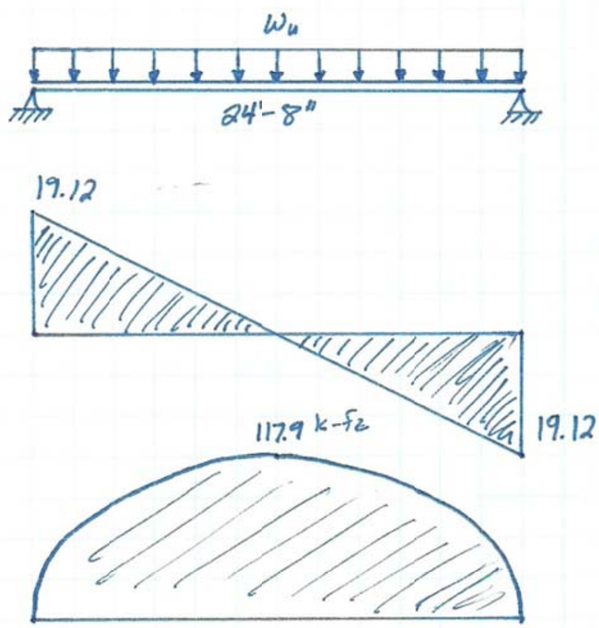
$w = 105$ psf < 278 psf (7'-6" clear; 2VLI) ∴ OK

Beam

Composite Beam: W 14x22 [10]

$A_g = 6.49$ in²
$I_x = 199$ in²
$F_y = 50$ ksi
$d = 13.7''$

Trib Width: 7'-2"
Span: 24'-8"



$w_D = 73$ psf $= 524$ plf
$w_L = 80$ psf $= 574$ plf

$w_u = 1.2w_D + 1.6w_L$

$= 1.2(0.524) + 1.6(0.574)$

∴ $w_u = 1.55$ ksf

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

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

E.3 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Beam (con't)

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

$\therefore b_{eff} = 74''$

$V'_c = 0.85 f'_c b_{eff} t = 0.85(4)(74)(6) = 1509.6^k$

$V'_s = F_y A_s = 50(6.49) = 324.5^k$

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

$V'_s = 0.85 f'_c (b_{eff})(a) \rightarrow \therefore a = \frac{V'_s}{0.85 f'_c b_{eff}} = \frac{324.5}{0.85(4)(74)}$

$a = 1.29''$

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

$\phi M_n = 0.9 M_n = 0.9(330.0)$

$\phi M_n = 297.0$ ft-k

$\phi M_n > M_u$ $\therefore$ OK

$\phi V_n = 94.5^k > V_u = 19.12^k$ $\therefore$ OK
↳ Table 3-2

E.4 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⟹ Concrete on Metal Deck

Beam (con't)

$$\Delta_{LL} = \ell/360 = \frac{24.67(12)}{360} = 0.822''$$

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

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

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

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

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

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

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

$$\Delta_{LL} = \frac{5 w_{LL} \ell^4}{384 E I} = \frac{5(0.574)(24.67)^4(1728)}{384(29{,}000)(682.8)} = 0.241''$$

$$\Delta_{LL} = 0.241'' < 0.822'' \quad \therefore \underline{OK}$$

E.5 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Beam (con't)

Wet Concrete Deflection

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

(slab self + 20)

$$w = 68(7.167') + 22\,plf = 0.510\,klf$$

$$I_{req} = \frac{5 w \ell^4}{384 E \Delta_{max}} = \frac{5(0.510)(24.67)^4(1728)}{384(29{,}000)(1.233)}$$

$$I_{req} = 118.7\,in^4 < 199\,in^4$$

W14x22

E.6 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Girder

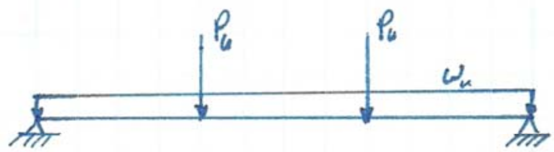
Composite: W16x26 [18]

$I_x = 301\ in^4$

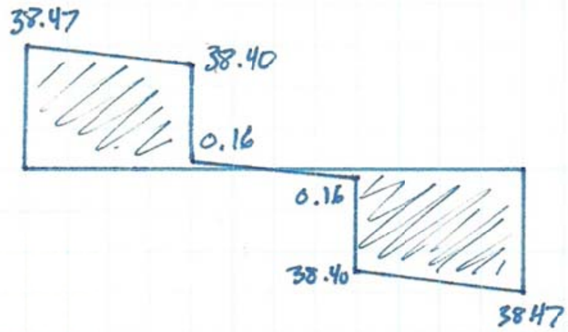
$A_s = 7.68\ in^2$

$F_y = 50\ ksi$

$d = 15.7"$



$w_u = 1.2(0.026) = 0.031\ klf$
↳ self weight

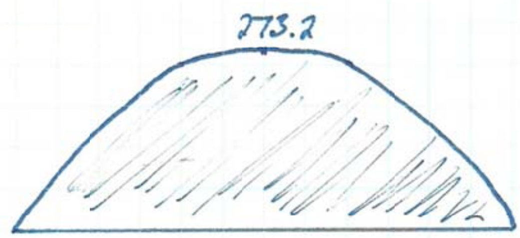


$P_u = 2(19.12^k) = 38.24^k$

$V_u = 38.47^k$

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

$b_{eff} = \min \begin{cases} span/4 = 64" \Rightarrow \text{controls} \\ spacing = 296" \end{cases}$



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

$\therefore V'_c = 1305.6^k$

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

$\therefore V'_s = 384^k$

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

$V'_s = 0.85 f'_c\, b_{eff}\, a \Rightarrow a = \dfrac{V'_s}{0.85 f'_c\, b_{eff}} = \dfrac{384}{0.85(4)(64)} = 1.76"$

E.7 Lightweight Concrete on Metal Deck Caculations (con't)

Composite → Concrete on Metal Deck

Girder (con't)

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

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

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

$$\phi M_n > M_u \qquad \therefore OK$$

$$\phi V_n = 106^k > V_u = 38.5^k$$
↳ Table 3-2 $\therefore OK$

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

$$\Sigma Q_n = \min \begin{cases} 0.85 F'_c b_{eff} t = 1305.6 \\ A_s F_y = 384 \rightarrow \text{controls} \end{cases}$$

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

$$\bar{y} = \frac{A_s(d/2) + \frac{\Sigma Q_n}{F_y}(d + Y2)}{A_s + \frac{\Sigma Q_n}{F_y}} = \frac{7.68(15.7/2) + \frac{384}{50}(15.7 + 5.12)}{7.68 + \frac{384}{50}} = 14.335''$$

$$I_{LB} = I_{x(s)} + A_s(\bar{y} - d/2)^2 + \frac{\Sigma Q_n}{F_y}(d + Y2 - \bar{y})^2$$
$$= 301 + 7.68(14.335 - 15.7/2)^2 + \frac{384}{50}(15.7 + 5.12 - 14.335)^2 = 997.0 \text{ in}^4$$

$$\Delta_{LL} = \frac{P_{LL}(a)}{24EI_{LB}}(3\ell^2 - 4a^2) \qquad a = 21'\text{-}4''/3 = 85.33''$$
$$\ell = 21'\text{-}4'' = 256''$$
$$= 0.314'' \qquad P_{LL} = 14.48^k$$

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

$\therefore OK$

E.8 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Girder (con't)

Wet Concrete Deflection

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

$$I_{req} = \frac{P_u a}{24 E \Delta_{max}}\left(3l^2 - 4a^2\right) = \frac{14.48(85.33)}{24(29{,}000)(1.07)}\left(3(256)^2 - 4(85.33)^2\right)$$

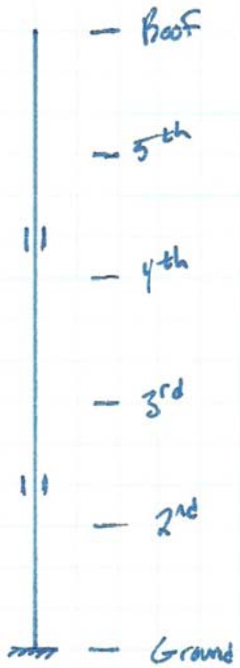
$$\therefore I_{req} = 277.9\ in^4 < I_x = 301\ in^4 \qquad \therefore OK$$

W16 x 26

E.9 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Column



= Influence Area

= Tributary Area

Influence Area

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

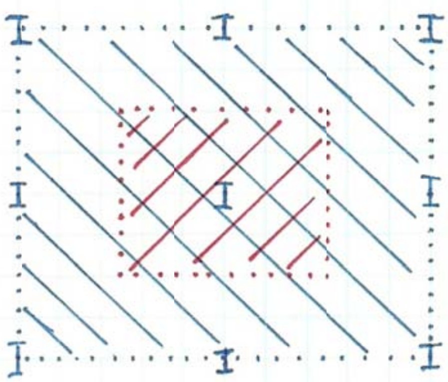
$$\therefore A_i = 2006.2\ ft^2$$

Tributary Area

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

$$\therefore A_t = 501.6\ ft^2$$

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



E.10 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Column (con't)

Loads

Below 5th

$P_D = 501.6(20 + 73) = 46.65^k$
$P_S = 24.64(501.6) = 12.36^k$
$P_L = 0.585(80)(501.6) = 23.47$
$L_{red} = 0.25 + \frac{15}{\sqrt{4 \times 501.6}} = 0.585$
$P_{u5} = 1.2(46.65) + 1.6(23.47) + 0.5(12.36) = 99.7^k$

Below 3rd

$P_D = 501.6(20 + 3(73)) = 119.88^k$
$P_S = 12.36^k$
$P_L = 0.443(3)(80)(501.6) = 53.33^k$
$L_{red} = 0.25 + \frac{15}{\sqrt{4(3)(501.6)}} = 0.443$

$P_{u3} = 1.2(119.88) + 1.6(53.33) + 0.5(12.36) = 235.4^k$

Below 2nd

$P_D = 501.6(20 + 4(73)) = 156.50^k$
$P_S = 12.36^k$
$P_L = 0.417(4)(80)(501.6) = 66.93^k$
$L_{red} = 0.25 + \frac{15}{\sqrt{4(4)(501.6)}} = 0.417$

$P_{u2} = 1.2(156.50) + 1.6(66.93) + 0.5(12.36) = 301.1^k$

E.11 Lightweight Concrete on Metal Deck Caculations (con't)

Composite ⇒ Concrete on Metal Deck

Column (con't)

Below 5th Floor

$P_{u5} = 99.7^k$

Table 4-1 (Steel Manual): $kL = 14'$

W8x48 $\quad \phi P_n = 394^k \geq 99.7^k \quad \therefore$ OK

Below 3rd

$P_{u3} = 235.4^k$

Table 4-1 (Steel Manual): $kL = 16'$

W8x67 $\quad \phi P_n = 487^k \geq 235.4^k \quad \therefore$ OK

Below 2nd

$P_{u2} = 301.1^k$

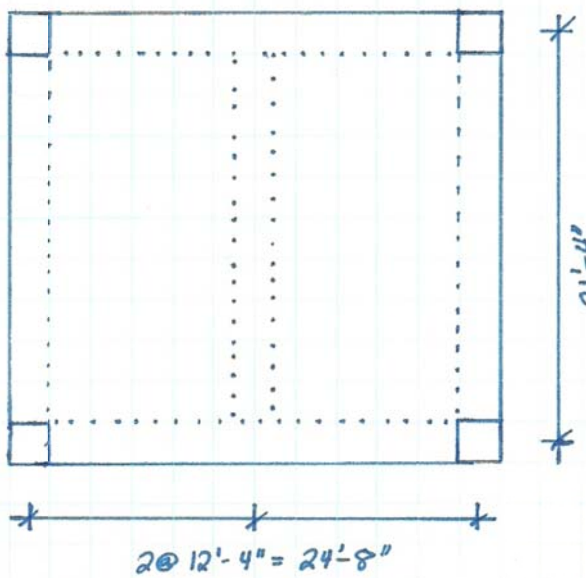
Table 4-1 (Steel Manual): $kL = 12'$

W8x67 $\quad \phi P_n = 487^k \geq 301.1^k \quad \therefore$ OK

Appendix F: One-Way Slab Calculations

F.1 One-Way Slab Calculations

One Way Concrete Slab



All ends continuous

$F'_c = 4000$ psi $\Rightarrow$ Normal Weight
$F_y = 60{,}000$ psi

$\beta_1 = 0.85$

Slab

Minimum Thickness

$h = \ell/28 = 12.33(12)/28 = 5.29" \Rightarrow$ Use $h = 6"$
↳ Table 9.5a

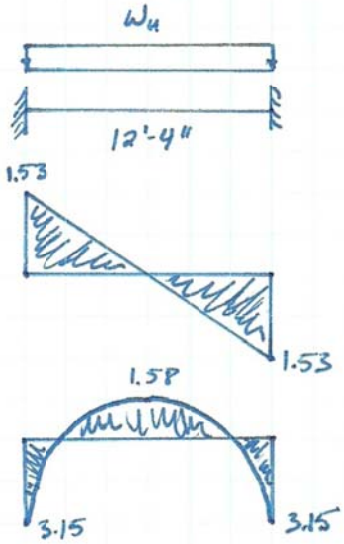
$DL_{self} = \frac{6"}{12}(150\ pcf) = 75$ psf
$SDL = 25$ psf
$LL = 80$ psf

$C_c = 3/4" \Rightarrow \therefore$ Assume $d = 5"$

F.2 One-Way Slab Calculations (con't)

One Way Concrete Slab

Slab (cont)



$w_u = 1.2w_D + 1.6w_L = 1.2(100) + 1.6(80)$

$\therefore w_u = 0.248$ klf

$V_u = \frac{w_u l}{2} = \frac{0.248(12.33)}{2} = 1.53^k$

$M_{u\,end} = \frac{w_u l^2}{12} = \frac{0.248(12.33)^2}{12} = 3.15$ ft-k

$M_{u\,mid} = \frac{w_u l^2}{24} = \frac{0.248(12.33)^2}{24} = 1.58$ ft-k

End Span Reinforcement

Since $f'_c = 4000$ psi; $f_y = 60{,}000$ psi $\rightarrow$ Assume $\rho = 1.25\%$

$A_s = \frac{M_u}{4d} = \frac{3.15(12)}{4(3)} = 0.2625$ in² $\Rightarrow \therefore$ Use W10x4 WWF

$A_s = 0.30$ in²

will also work for midspan

F.3 One-Way Slab Calculations (con't)

One Way Concrete Slab

Column $A_t = 526.2\ ft^2$

@Base

$P_D = 526.2(100)(5) = 263.1^k$

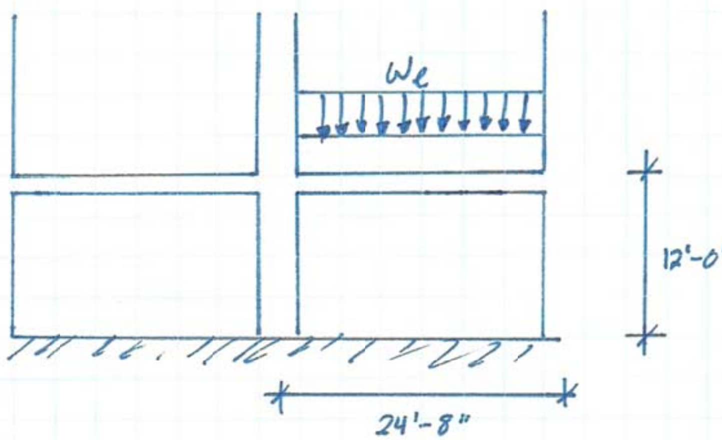
$P_S = 24.64(526.2) = 12.97^k$

$P_L = 0.413(4)(80)(526.2) = 69.62^k$

$LL_{red} = 0.25 + \frac{15}{\sqrt{4(4)(526.6)}} = 0.413$

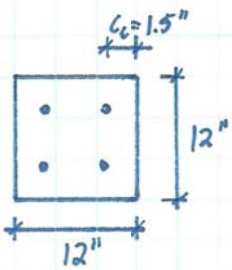
$\therefore P_u = 1.2P_D + 1.6P_L + 0.5P_S$
$= 1.2(263.1) + 1.6(69.62) + 0.5(12.97)$

$\therefore P_u = 433.6^k$



$M_{\ell ens} = \frac{w_\ell \ell^2}{12} = 86.53\ ft\text{-}k$

$\frac{\phi P_n}{b^2} = \frac{\phi M_n}{b^3} \Rightarrow b_{min} = \frac{\phi M_n}{\phi P_n} = 2.4" \Rightarrow$ use $b = 12"$



$\gamma = \frac{h - 2d'}{h} = \frac{12 - 2(1.5)}{12} = 0.75$

$\frac{\phi P_n}{bh} = \frac{433.6^k}{12(12)} = 3.01\ ksi$

$\frac{\phi M_n}{bh^2} = \frac{86.53(12)}{12(12)^2} = 0.60\ ksi$

Table R-4-60-0.75, Pg 1066 Wight & MacGregor

Does Not Work

F.4 One-Way Slab Calculations (con't)

One Way Concrete Slab

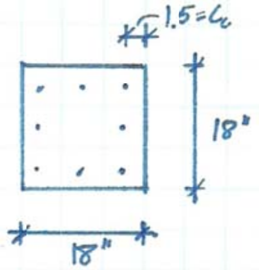
Column

@ Base

$P_u = 433.6^k$

$M_u = 86.53$ ft-k

Try 18" square column



Assume $\gamma = 0.75$

$\frac{\phi P_n}{bh} = \frac{433.6}{18(18)} = 1.34$ ksi

$\frac{\phi M_n}{bh^2} = \frac{86.53(12)}{18(18)^2} = 0.178$

} Table R-4-60-0.75 pg 1066 Wight + MacGregor

works for $\rho_g = 0.01$

$\rho_g = 0.01 = \frac{A_s}{bh} \Rightarrow \therefore A_{s,min} = 3.24\ in^2$

$\therefore$ Use (8) - #6 → As Shown

F.5 One-Way Slab Calculations (con't)

One Way Concrete Slab

Beams

$b = 18'' \Rightarrow$ to match columns

$w_D = 21.33\text{ ft}(100\text{ psf}) = 2.13\text{ klf}$

$w_L = 21.33\text{ ft}(80\text{ psf}) = 1.71\text{ klf}$

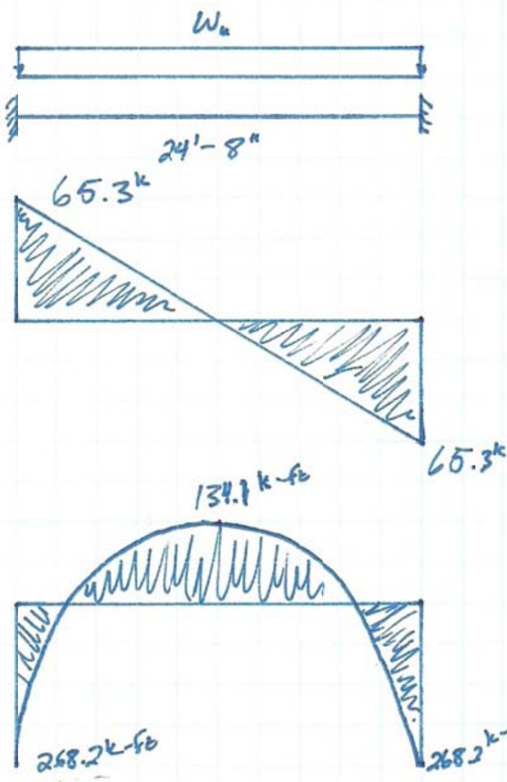
$w_u = 1.2w_D + 1.6w_L = 1.2(2.13) + 1.6(1.71)$

$\therefore w_u = 5.29\text{ klf}$

$V_u = \frac{w_u \ell}{2} = 65.3^k$

$M_{u_{end}} = \frac{w_u \ell^2}{12} = \frac{5.29(24.66)^2}{12} = 268.2\text{ ft-k}$

$M_{mid} = \frac{w_u \ell^2}{24} = \frac{5.29(24.66)^2}{24} = 134.1\text{ ft-k}$



$h = 16'' \Rightarrow$ Max depth desired

$h_f = 6''$

$b_{eff} = \min \begin{cases} b_w + 16h_f = 18'' + 16(6'') = 114'' \\ b_w + 2(\tfrac{1}{2} c_d) = 18'' + 2(\tfrac{1}{2}(140'')) = 88'' \Rightarrow \text{controls} \\ \tfrac{1}{4}\ell_{span} = 238'' \end{cases}$

F.6 One-Way Slab Calculations (con't)

One Way Concrete Slab

Beams (cont)

Assume $a \leq h_f$ (T-Beam Behavior)

$$\phi M_n = 134.1(12) = \phi A_s f_y\left(d - \frac{A_s f_y}{1.7 f'_c b}\right) = 0.9 A_s (60)\left(14" - \frac{A_s(60)}{1.7(4)(88)}\right)$$

$$1609.2 = 756 A_s - 5.41 A_s^2$$

$$\therefore A_s = \frac{M_u}{4\,8d} = \frac{1609.2}{48(14)} = 2.40 \text{ in}^2$$

Try (8) – #5 $\Rightarrow A_{s\,prov} = 2.48 \text{ in}^2$

Spacing Min = 1" $b_w \geq 2C_c + 8d_b + 7s_{min} = 2(1.5) + 8(.625) + 7(1) = 15"$ $\therefore$ OK

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.48(60)}{0.85(4)(88)} = 0.5" < 6" \quad \therefore a \leq h_f \Rightarrow \text{Good Assumption}$$

Top Steel

$$\phi M_n = 268.2 \text{ ft-k} = 3218.4 \text{ in-k}$$

$$A_s = \frac{M_u}{48d} = \frac{3218.4}{48(14)} = 4.79 \text{ in}^2$$

Try (5) – #9 $\Rightarrow A_{s\,prov} = 5 \text{ in}^2$

Spacing Min = 1" $b_w \geq 5d_b + 4s_{min} = 9"$ $\therefore$ OK

F.7 One-Way Slab Calculations (con't)

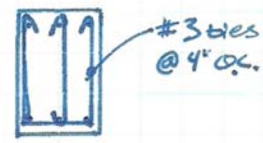
One Way Concrete Slab

Beams (cont)

Shear Reinforcement

$V_s = 65.3^k = A_v f_y \frac{d}{s} = 2(0.11)(60)\frac{14}{s}$

$\therefore s = 2.83'' \Rightarrow$ too close



Try 3 legs

$V_s = 65.3^k = 3(0.11)(60)\frac{14}{s}$

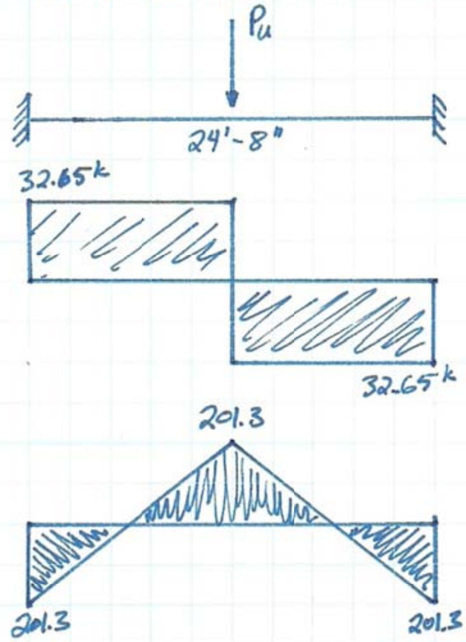
$\therefore s = 4.25'' \Rightarrow$ Use $s = 4''$

F.8 One-Way Slab Calculations (con't)

One Way Concrete Slab

Girder

Try to maintain same dims as beam ⇒ $h = 16"$ $b = 18"$



$P_u = 65.3^k$

$V_u = \frac{P_u}{2} = \frac{65.3^k}{2}$

$\therefore V_u = 32.65^k$

$M_u = \frac{P_u l}{8} = \frac{65.3^k(24.67)}{8}$

$\therefore M_u = 201.3\ ft\text{-}k$

$A_s = \frac{M_u}{48d} = \frac{201.3(12)}{48(14)}$

$\therefore A_s = 3.59 in^2$

Try 6-#7 Bars ⇒ $As_{prov} = 3.6 in^2$

Spacing Min = 1"

$b_w \geq 2C_c + 6d_b + 5s = 2(1.5) + 6(0.875) + 5(1) = 13.25" < 18"$ ∴ OK

∴ Use (6)-#7 Bars T+B

F.9 One-Way Slab Calculations (con't)

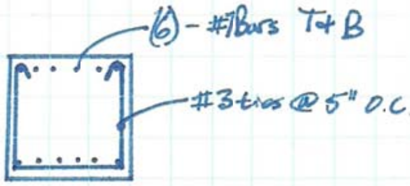
One Way Concrete Slab

Girder (cont)

Shear Reinforcement

$V_s = 32.65^k = A_v f_v \frac{d}{s} = 2(0.11)(60)\frac{14}{s}$

$\therefore S = 5.66'' \Rightarrow$ Use $S = 5''$ along entire length



Appendix G: Hollow Core Plank Calculations

G.1 Hollow Core Plank Calculations

Flexicore
Hollow Core
PRESTRESSED CONCRETE SLAB

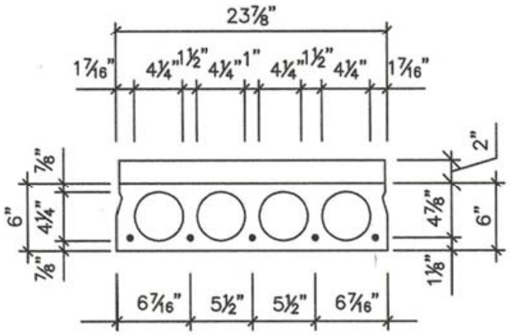
6" X 24" Section
2" Structural Topping

Safe Load Table

UNIFORMLY DISTRIBUTED SUPERIMPOSED SERVICE LOAD IN PSF

Standard Designation	Strands No. & Size	Strand Area Sq. In.	M Ft.-Kips per Unit	ϕM_n Ft.-Kips Per Unit	Span Length (l) in Ft. 10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27
T624A-38	2-7/16 & 2-3/8	0.376	29.4	40.7	583	525	476	435	400	368	316	274	238	207	182	159	137	117	99	83	68	56
T624A-33	3-7/16	0.324	26.3	35.9	583	525	476	435	374	318	272	235	203	176	154	133	112	93	77	63	50	
T624A-30	2-7/16 & 1-3/8	0.296	24.6	33.2	583	525	476	405	341	290	248	213	184	159	138	118	98	81	66	52		
T624A-27	1-7/16 & 2-3/8	0.268	23.0	30.5	583	525	441	367	308	261	223	191	164	141	122	103	84	68	54			
T624A-24	3-3/8	0.240	21.4	27.7	583	480	394	328	275	232	197	168	143	123	105	88	71	56				

controlled by: ultimate shear service



A = 86.5 in^2		f'_c = 6000	psi
A_c = 120.43 in		f'_{ci} = 3500	psi
b_w = 6.875 in^2		f'_{ct} = 3000	psi
I_g = 365.7 in^4		f_{pu} = 250	ksi
I_{gc} = 766.982 in^4			
y_b = 3.000 in			
y_{bc} = 4.17 in			

NOTES:

1) Grouted weight of topped structural unit is 70 psf or 140 plf based on concrete unit weight of 150 pcf.
2) Design is based on ACI Standard, "Building Code Requirements for Reinforced Concrete (ACI318)."
3) No shear reinforcement is required for the tabulated loads.
4) Tabulated loads are based on U=1.2D+1.6L and with all load superimposed on the structural section considered as live load.
5) Tabulated loads in the blue area are controlled by shear strength of the concrete.
6) Tabulated loads in yellow are controlled by permissible flexural tension at service loads.
7) All strand stressed to 70% of ultimate.
8) For longer spans and conditions not covered in the load table, consult Molin.

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PCI

Hollow Core - Flexicore 6" X 24" Section, 2" Structural Topping

G.2 Hollow Core Plank Calculations (con't)

Instructions For Using Hollow Core Safe Load Table

A. NOTATION

- A = cross sectional area of Hollow Core sections.
- A_C = cross sectional area of composite hollow core section.
- b_w = minimum web width.
- D = dead loads or related internal moments and forces
- f'_c = specified compressive strength of concrete.
- f'_{ci} = compressive strength of concrete at transfer of prestress
- f_{pe} = compressive stress in concrete due to prestress only (after all losses) at bottom fiber of the section
- f_{pu} = specified tensile strength of prestressing steel.
- f_{ps} = stress in prestressing steel at nominal strength.
- f_{si} = initial or tensioning stress in prestressing steel
- I_g = moment of inertia of the gross Hollow Core section.
- I_{gc} = moment of inertia of the gross composite section.
- l = span length
- L = live loads or related internal moments and forces
- M = service load moment causing flexural tension of $7.5\sqrt{f'_c} = (7.5\sqrt{f'_c} + f_{pe})$
- M_d = moment due to service dead load (including weight of the structural unit)
- M_l = moment due to service live load
- M_s = moment due to service loads modified to correspond the composite section $= M_w \frac{I_{gc} y_b}{I_g y_{bc}} + M_{sd} + M_l$
- M_{sd} = moment due to superimposed deadloads (resisted by composite section)
- M_n = nominal moment strength, assuming fully developed strands
- M_u = applied factored moment = $1.2M_d + 1.6M_l$
- M_w = moment due to weight of Hollow Core slab and topping (resisted by Hollow Core section only)
- U = required strength to resist factored loads or related internal moments and forces
- w_l = uniform service live load
- w_s = uniform superimposed load = wsd + w_l
- w_{sd} = uniform dead load due to superimposed loading
- y_b = distance from bottom fiber to center of gravity of the Hollow Core section
- y_{bc} = distance from bottom fiber to center of gravity of the composite section
- ϕ = strength reduction factor
- ϕM_n = design moment strength, assuming fully developed strands

B. UNIFORM LOADING - Whan all superimposed loads are considered to be live loads. ($w_{sd} = 0; w_s = w_l$).
For the given l & w_s select the required standard designation directly from the load table

C. UNIFORM LOADING - When superimposed load consists of both dead and live loads. ($w_s = w_{sd} + w_l$).

a. Calculate modified $w_s = \frac{1.2}{1.6} w_{sd} + w_l$.
b. Enter the table with the given l and modified w_s, and select the standard designation.

D. NON-UNIFORM LOADING

1. Calculate maximum $Mu = 1.2\ M_d + 1.6\ M_l$
2. Enter the column in the load table entitled "ϕM_n" and select standard designation having $\phi M_n \geq M_u$.
3. Check development requirements of prestressing strands in accordance with Section 12.9 of ACI 318.
4. Check flexural stresses at service loads:
   a. Calculate maximum M_s.
   b. Enter the column in the load table entitled "M". For the standard designation selected in Step 2, M should be $\geq M_s$.
   c. If $M < M_s$, select standard designation having $M \geq Ms$.
5. Check shear strength of concrete to determine if any shear reinforcement is required.

E. CAMBER AND DEFLECTION

1. The table indicates maximum safe loads, however, camber and deflection may limit the use of a prestressed unit even though the load carrying capacity is satisfactory.
2. Camber and deflection must always be investigated for the contemplated loading condition and span so that these factors are compatible with abutting materials in the proposed building. Consult your local manufacturer, Molin Concrete Products Company.

DESIGN CRITERIA

Principal design criteria used for development of the load table are:

1. f_{ps} calculated by Section 18.7.2 of ACI 318.
2. Total loss of prestress assumed = 18% of f_{si} with initial loss at transfer of prestress assumed = 10% of f_{si}.
3. Premissible flexural stresses in concrete at service loads: Compression = $0.45\ f'_c$ Tension = $7.5\sqrt{f'_c}$.
4. Shear strength conservatively assumed to be limited to $3.5\sqrt{f'_c}$ in accordance with ACI 318 Section 11.4.2. Additional shear strength may be available with more detailed analysis.

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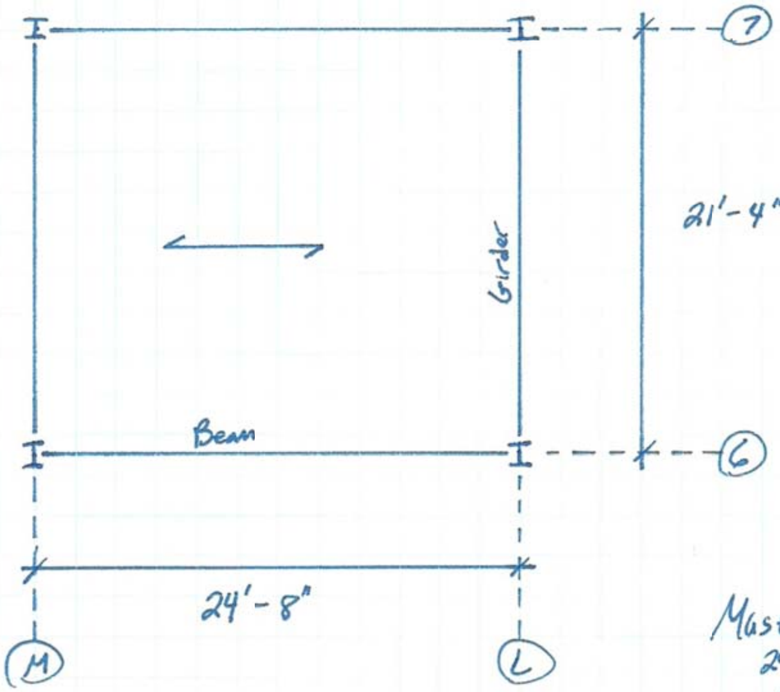
January 2006

G.3 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams



Must Shorten 24'-8" span to 24'-0"

Loads

DL = 70 psf ⇒ Flexicore 6"x24" w/ 2" topping ⇒ Self-Weight
LL = 80 psf
SDL = 25 psf

Superimposed Service Loads $= 1.2(25) + 1.6(80) = 158 = U_{req'd}$

Try T624A-38

$U = 1.2D^{0} + 1.6L = 1.6(99) = 158.4 > 158$ ∴ OK

Controlled by Service Limitations

G.4 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Beam Design.

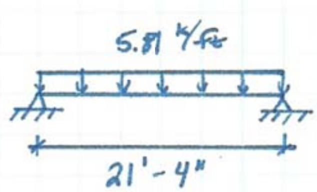
In theory the beams carry no load, therefore many would wonder why they are needed. They serve the primary job of bracing the columns in their weak axis. So the decision was made to use W10x12 beams. Although W8x10's would work for gravity loads, I decided to use my engineering judgement and impose a self constraint. This is primarily due to the fact that a difference of 2 plf between beams is nearly negligable, as well as a 10" deep section would not appear to "flimsy" to a non-structural engineer (ie the owner or architect).

Girder Design

$DL = 70\ psf\ (24') = 1680\ plf$
$SDL = 25\ psf\ (24') = 600\ plf$
$LL = 80\ psf\ (24') = 1920\ plf$

$$w_u = 1.2(1.68 + 0.6) + 1.6(1.92) = 5.81\ klf$$

$$w_\ell = 1.92\ klf$$



$$V_u = \frac{5.81(21.33)}{2}$$

$$\therefore V_u = 61.95\ ^k$$

$$M_u = \frac{w\ell^2}{8} = \frac{5.81(21.33)^2}{8}$$

$$\therefore M_u = 330.4\ ft\text{-}k$$

G.5 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Girder Design (cont)

Using Table 3-10 (Steel Manual):

UBL = 21.5'

Use W14x74 $\Rightarrow \phi M_n = 370$ Ft-k > 330.4 $\therefore$ OK

Shear Check

$\phi V_n = 1.0(0.6) F_y A_w C_v = 1.0(0.6)(50)(14.2 \times 0.45)(1.0)$

$\therefore \phi V_n = 191.7^k > 61.95^k$ $\therefore$ OK

LL Deflection

$\Delta_{LL} \leq \ell/360 = 0.711''$

$\Delta_{LL} = \frac{5 w \ell^4}{384 EI} = \frac{5(1.92)(21.33)^4(1728)}{384(29{,}000)(795)}$

$\Delta_{LL} = 0.388'' \leq 0.711''$ $\therefore$ OK

Total Load Deflection

$\Delta_{TL} = \ell/240 = 1.0667''$

$\Delta_{TL} = \frac{5 w \ell^4}{384 EI} = \frac{5(5.81)(21.33)^4(1728)}{384(29{,}000)(795)}$

$\Delta_{TL} = 1.17'' > 1.067''$ $\therefore$ No Good

G.6 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Girder Design (cont)

Total Load Deflection (cont)

$$\ell/240 = \frac{5w\ell^4}{384EI} \Rightarrow 1.0667 = \frac{5(5.81)(21.33)^4(1728)}{384(29{,}000)I_{req'd}}$$

$$\therefore I_{req'd} = 875.3\ in^4$$

Table 3-3 (Steel Manual)

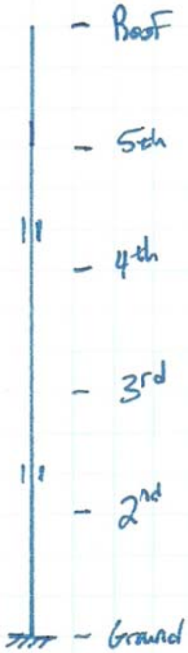
Use W16x67 $\Rightarrow I_x = 954\ in^4 > 875\ in^4$ $\therefore$ OK

Comes from a self imposed depth limit of ≈ 16"

G.7 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Column Design



Influence Area

$A_i = 4(24')(20')$

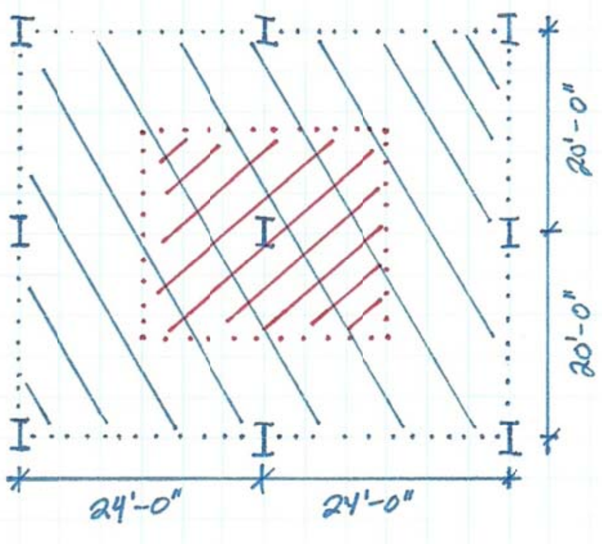
$\therefore A_i = 1920\ ft^2$

Tributary Area

$A_t = (24')(20')$

$\therefore A_t = 480\ ft^2$

$\therefore K_u = 4$



= Influence Area

= Tributary Area

G.8 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Column Design (con't)

Load Below 5^{th}

$P_S = 24.64\text{ psf}(480\text{ ft}^2) = 11.83^k$
$P_L = 0.592(80)(480\text{ ft}^2) = 22.75^k$

$LL_{red} = 0.25 + \frac{15}{\sqrt{4(480)}} = 0.592$

$P_D = 20(480) + 95(480) = 55.2^k$

$P_u = 1.2(55.2) + 1.6(22.75) + 0.5(11.83)$

$\therefore P_{u5} = 108.6^k$

Load Below 3^{rd}

$P_S = 11.83^k$
$P_D = 480[20 + 3(95)] = 146.4^k$
$P_L = 0.448(3)(80)(480) = 51.57^k$
$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 3(480)}} = 0.448$
$P_{u3} = 1.2(146.4) + 1.6(51.57) + 0.5(11.83)^k = 264.1^k$

Load Below 2^{nd}

$P_S = 11.83^k$
$P_D = 480[20 + 4(95)] = 192^k$
$P_L = 0.421(4)(80)(480) = 64.69^k$
$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 4(480)}} = 0.421$

$P_{u2} = 1.2(192) + 1.6(64.69) + 0.5(11.83) = 339.8^k$

G.9 Hollow Core Plank Calculations (con't)

Hollow Core Concrete on Steel Beams

Column Designs (cont)

Below 5th Floor

$P_{u5} = 108.6^k$

Table 4-1 (Steel Manual):

$kL = 14'$

Use W8x31

$\phi P_n = 248^k \geq 108.6^k$

Below 3rd Floor

$P_{u3} = 264.1^k$

$KL = 16'$

Use W8x40

$\phi P_n = 275^k \geq 264.1^k$

Below 2nd Floor

$P_{u2} = 339.8^k$

$KL = 12'$

Use W8x40

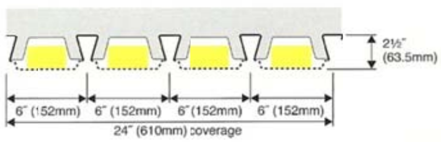
$\phi P_n = 366^k \geq 339.8^k$

Appendix H: Long Span Deck Calculations

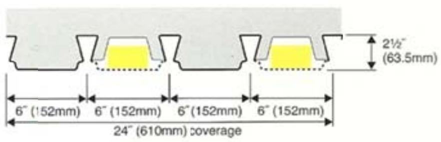
H.1 Long Span Deck Calculations

10 TORIS® CA & C COMPOSITE FLOOR DECK CEILING SYSTEM TECHNICAL TABLES

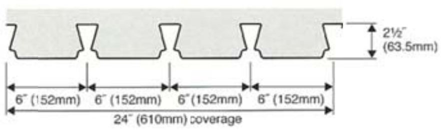
Toris CA



Toris CA 50%



Toris C



Superior Fire Ratings

The Toris CA Acoustical Floor Deck Ceiling System has an efficient unprotected fire rating listed in the table to the right.

Toris C Composite Floor Deck Fire Ratings under U.L. Design Numbers D971 is superior to fire ratings of generic composite floor decks. In most instances, the fire ratings of Toris C Composite Floor Deck slabs require from ½-inch to 1¼-inch less slab depth than generic profile slabs.

For the unprotected fire ratings shown on the following tables, no spray-applied fireproofing is required on the deck.

Toris CA Fire Ratings (U.L. Design Number D971)

Restrained Fire Rating	Total Slab Depth (in.)	Type and Density of Concrete (pcf)
1 hour	6.25	RW (147)
1 hour	5	LW (110)
1½ hours	6.75	RW (147)
2 hours	7	RW (147)
2 hours	5.75	LW (110)
3 hours	7.75	RW (147)
3 hours	6.75	LW (110)

NOTE: Toris CA can achieve the loads shown on page 11 with the fire ratings indicated above.

Toris C Fire Ratings (U.L. Design Number D971)

Restrained Fire Rating	Total Slab Depth (in.)	Type and Density of Concrete (pcf)
1 hour	4.5	RW (147)
1½ hours	5	RW (147)
2 hours	5.5	RW (147)
2 hours	4.75	LW (110)
3 hours	6.75	RW (147)
3 hours	5.5	LW (110)

NOTE: Toris C can achieve the loads shown on page 11 with the fire ratings indicated above.

RW = Regular Weight Concrete
LW = Lightweight Concrete

Suggested Temperature and Shrinkage Reinforcement

Slab Depth (in.)	Welded Wire Fabric Mesh
4	6 x 6 – W1.4 x W1.4
4½ – 5	6 x 6 – W2.1 x W2.1
5½ – 8	6 x 6 – W2.9 x W2.9

See U.L. Fire Resistance Directory for temperature and shrinkage reinforcement of fire rated assemblies. U.L. Fire Rated Slabs require 6 x 6 – W2.9 x W2.9 mesh.

Toris CA Noise Reduction Coefficients*

	Absorption Coeficients						NRC
	125Hz	250Hz	500Hz	1000Hz	2000Hz	4000Hz	
100% A	0.15	0.67	0.86	0.88	0.91	0.81	0.85
50% A**	0.21	0.68	0.74	0.75	0.54	0.40	0.70

* In accordance with ASTM C423 and E795. Consult EPIC Metals Corporation for other test results and individual reports. The NRC is the average of the absorption coefficients at 250, 500, 1000, and 2000 Hz., rounded off to the nearest .05.
** Estimates

Toris CA & Toris C Section Properties

Design Thickness		Weight		A_S		I_P		S_P		S_N	
(in.)	(mm)	(psf)	(kg/m²)	(in.²/ft.)	(mm²/m)	(in.⁴/ft.)	(mm⁴/m)(10⁶)	(in.³/ft.)	(mm³/m)(10³)	(in.³/ft.)	(mm³/m)(10³)
0.0358	0.91	2.9	14.2	0.82	1734	0.766	1.052	0.466	25.054	0.428	23.011
0.0474	1.20	3.8	18.5	1.08	2283	1.010	1.380	0.621	33.387	0.581	31.237
0.0600	1.52	4.8	24.4	1.37	2896	1.274	1.740	0.778	41.828	0.749	40.269

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H.2 Long Span Deck Calculations (con't)

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Toris CA* & C Composite Floor Deck Systems

Concrete	Slab Depth and Weight	Design Thickness (in.)	Maximum Clear Span Without Shoring (ft.-in.)			Uniform Service Load Slab Capacity, PSF/Spans (C-C of Supports)															
						Simple Span Condition (see Note 2)											Continuous Span Condition Negative Moment Steel Reinforcing REQUIRED (see Note 5)				
			Simple Span	Double Span	Triple Span	6'0"	8'0"	10'0"	12'0"	14'0"	15'0"	16'0"	17'0"	18'0"	19'0"	20'0"	16'0"	18'0"	20'0"	22'0"	24'0"
3 ksi Regular Weight Concrete, (145 pcf)	4.5" 54 PSF	0.0358	9-8	9-11	10-3	500	362	252	153	77	53	—	—	—	—	—	111	62	—	—	—
		0.0474	11-5	11-6	11-11	500	492	345	178	92	65	44	—	—	—	—	130	76	41	—	—
		0.0600	12-4	13-0	13-5	500	492	354	197	104	74	52	—	—	—	—	145	86	48	—	—
	5" 60 PSF	0.0358	9-3	9-6	9-10	500	413	287	215	116	83	58	—	—	—	—	161	96	54	—	—
		0.0474	10-11	11-0	11-5	500	500	404	247	134	98	70	49	—	—	—	185	112	66	—	—
		0.0600	12-0	12-5	12-10	500	500	404	274	151	111	81	58	—	—	—	206	127	76	42	—
	5.5" 66 PSF	0.0358	8-11	9-2	9-5	500	464	322	241	162	120	88	62	42	—	—	202	137	82	46	—
		0.0474	10-6	10-7	11-0	500	500	454	333	185	139	103	75	53	—	—	251	157	97	57	—
		0.0600	11-9	12-0	12-5	500	500	454	350	208	157	118	87	63	44	—	280	177	111	67	—
	6" 72 PSF	0.0358	8-7	8-10	9-1	500	500	358	268	211	163	122	90	65	45	—	229	170	115	69	—
		0.0474	10-1	10-3	10-7	500	500	500	388	248	188	143	107	79	57	—	305	212	135	84	48
		0.0600	11-5	11-7	12-0	500	500	500	388	277	212	162	123	92	68	48	330	238	154	98	59
	6.5" 78 PSF	0.0358	8-4	8-6	8-10	500	500	393	294	232	202	165	125	93	68	47	256	190	143	99	59
		0.0474	9-9	9-11	10-3	500	500	500	426	322	247	190	146	110	82	60	342	258	181	116	72
		0.0600	11-1	11-2	11-7	500	500	500	426	344	276	214	165	127	96	71	363	308	203	133	85
	7" 84 PSF	0.0358	8-0	8-3	8-7	500	500	428	321	253	223	188	160	126	94	69	282	210	158	120	83
		0.0474	9-5	9-7	9-11	500	500	500	465	356	302	244	189	146	112	84	378	286	220	154	99
		0.0600	10-8	10-10	11-3	500	500	500	465	375	342	274	214	167	130	99	397	346	261	175	116
	7.5" 90 PSF	0.0358	7-10	8-0	8-4	500	500	464	347	274	244	206	175	149	127	96	309	230	174	132	100
		0.0474	9-2	9-4	9-8	500	500	500	500	391	332	284	241	189	148	114	415	314	242	188	132
		0.0600	10-4	10-7	10-11	500	500	500	500	406	370	339	271	214	168	131	430	375	310	223	151
3 ksi Lightweight Concrete, (110 pcf)	4.5" 42 PSF	0.0358	10-9	10-11	11-3	500	344	236	119	60	42	—	—	—	—	—	87	49	—	—	—
		0.0474	12-6	12-7	13-1	500	468	270	139	72	51	—	—	—	—	—	102	59	—	—	—
		0.0600	13-2	14-3	14-9	500	468	299	156	82	59	41	—	—	—	—	115	68	—	—	—
	5" 47 PSF	0.0358	10-3	10-6	10-10	500	392	273	168	89	64	45	—	—	—	—	124	74	42	—	—
		0.0474	12-2	12-2	12-7	500	500	366	192	104	76	55	—	—	—	—	144	87	51	—	—
		0.0600	12-10	13-9	14-2	500	500	384	217	119	88	65	46	—	—	—	163	100	61	—	—
	5.5" 51 PSF	0.0358	9-11	10-1	10-5	500	441	306	227	125	92	67	48	—	—	—	170	105	63	—	—
		0.0474	11-8	11-9	12-1	500	500	431	259	144	108	80	58	41	—	—	195	122	76	44	—
		0.0600	12-6	13-3	13-8	500	500	431	291	164	124	93	69	50	—	—	221	140	88	53	—
	6" 56 PSF	0.0358	9-7	9-9	10-1	500	489	340	254	168	126	95	70	50	—	—	226	143	89	54	—
		0.0474	11-3	11-4	11-9	500	500	478	338	192	146	111	83	62	44	—	257	164	105	65	—
		0.0600	12-3	12-10	13-3	500	500	478	369	217	166	127	97	73	53	—	288	186	120	77	46
	6.5" 60 PSF	0.0358	9-3	9-6	9-10	500	500	373	280	219	167	127	97	72	53	—	269	188	121	76	45
		0.0474	10-11	11-0	11-4	500	500	500	405	250	192	148	113	86	64	47	331	215	141	91	56
		0.0600	12-0	12-5	12-10	500	500	500	405	279	216	167	129	99	76	56	368	241	159	105	67
	7" 65 PSF	0.0358	9-0	9-2	9-6	500	500	407	305	240	213	165	127	97	73	53	296	224	156	102	64
		0.0474	10-7	10-8	11-0	500	500	500	441	316	245	190	148	115	88	66	393	273	181	120	78
		0.0600	11-9	12-1	12-6	500	500	500	441	351	274	214	168	131	102	78	411	304	204	137	91
	7.5" 70 PSF	0.0358	8-9	8-11	9-3	500	500	440	330	260	234	208	162	126	97	73	324	245	189	132	86
		0.0474	10-3	10-5	10-9	500	500	500	478	386	304	238	187	147	115	88	431	329	227	154	103
		0.0600	11-7	11-9	12-2	500	500	500	478	386	340	268	212	168	132	103	446	377	256	175	119

No Shoring — Shoring Required in Shaded Areas

COMPOSITE SLAB DESIGN NOTES:

1. All loads are assumed to be statically applied. For dynamic loads, consult EPIC Metals Corporation.
2. Simple span conditions are based on simple span composite design.
3. Deflection limit of the composite slab is L/360 under total load.
4. Loads appearing in shaded areas require shoring.
5. Continuous span conditions are based on continuous span composite design and require appropriate negative moment reinforcing steel over supports.
6. Composite slab design is based on LRFD.
7. The slab weight has already been accounted for in the service loads listed above.

* Reduce loads by 20% for Toris CA.

DECK DESIGN AS A FORM:

a. Maximum clear spans without shoring are based on the Steel Deck Institute recommendations for sequential loading and load resistance factor design. The table is based on 40 ksi steel yield stress and deflection limits of L/180 or .75 inches, whichever is less. If heavier construction loads or less form deflection are required, spans must be reduced. Consult Epic for recommendations.
b. Runways and planking must be used for all concrete placement.
c. Minimum bearing is 2" at end supports and 4" at interior supports.
d. Slab weight includes 4.8 psf for deck weight.
e. Deduct 12 psf from slab weights shown above for Epicore Toris CA, lightweight concrete.
f. Deduct 16 psf from slab weights shown above for Epicore Toris CA, normal weight concrete.

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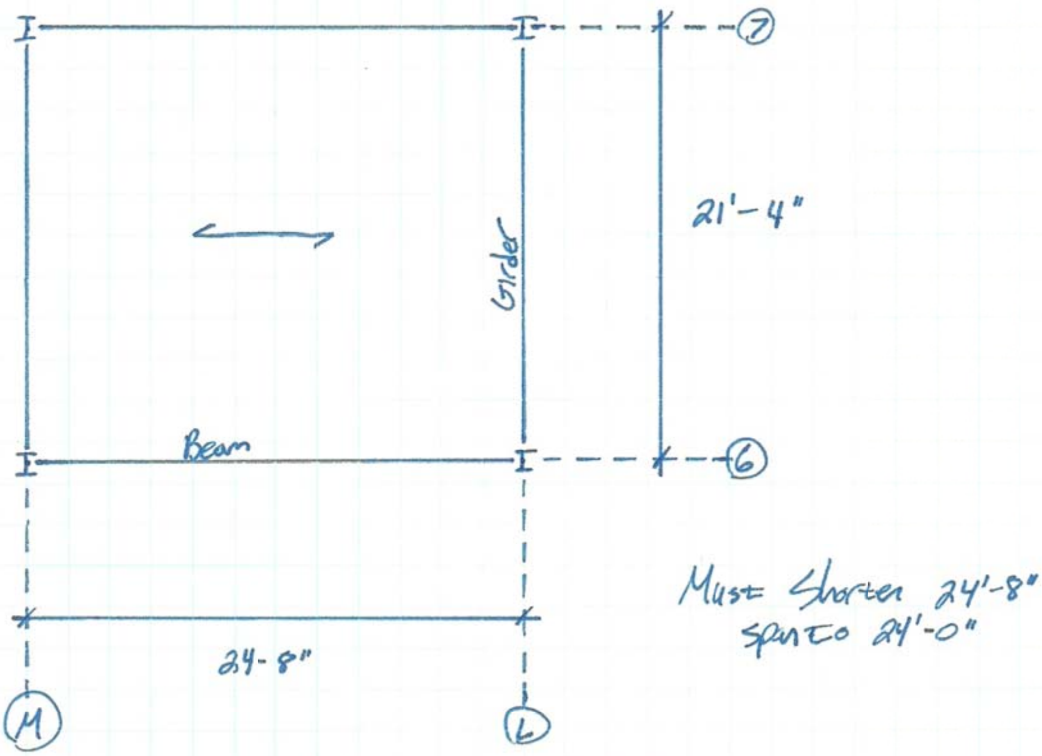
H.3 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams



Loads

SDL = 25 psf
LL = 80 psf

∴ Service Load = 105 psf

Try Toris CA Composite Floor Deck System (EPIC Metals)

24'-0" Span

Use 7.5" slab w/ 3 ksi Normal Weight Concrete

$\frac{132 \text{ psf}}{1.2} = 110 \text{ psf} > 105 \text{ psf}$ ∴ Deck OK

↳ Reduction For CA Deck

H.4 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Beam Design

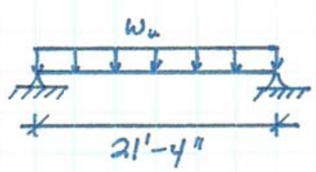
In theory the beams carry no load because the deck runs parallel to the beam, therefore many would wander why they are needed. They serve the primary job of bracing the columns in their weak axis. So the decision was made to use W10x12 beams. Although W8x10's would work for gravity loads, I decided to use my engineering judgement and impose a self constraint. This is primarily due to the fact that a difference of 2 plf between beams is nearly negligable, as well as a 10" deep section would not appear to "flimsy" to a person not trained in structural engineering. (i.e. the owner or architect)

Girder Design

DL = 90 psf (24') = 2160 plf
SDL = 25 psf (24') = 600 plf
LL = 80 psf (24') = 1920 plf

$$w_u = 1.2(2.16 + 0.6) + 1.6(1.92) = 6.38 \text{ klf}$$

$$w_e = 1.92 \text{ klf}$$



$$V_u = \frac{6.38(21.33)}{2}$$

$$\therefore V_u = 68.1^k$$

$$M_u = \frac{w l^2}{8} = \frac{6.38(21.33)^2}{8}$$

$$\therefore M_u = 363.0 \text{ ft-k}$$

H.5 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Girder Design (cont)

Using Table 3-10 (Steel Manual):

$UBL = 21.5'$

Use W14x74 $\Rightarrow \phi M_n = 370$ ft-k > 363.0 ft-k ∴ OK

Shear Check

$\phi V_n = 1.0(0.6) F_y A_w C_v = 1.0(0.6)(50)(14.2 \times 0.45)(1.0)$

$\therefore \phi V_n = 191.7^k > 68.1^k$ ∴ OK

Live Load Deflection

$\Delta_{LL} \leq \ell/360 = 0.711''$

$$\Delta_{LL} = \frac{5w\ell^4}{384EI} = \frac{5(1.92)(21.33)^4(1728)}{384(29{,}000)(795)}$$

$\Delta_{LL} = 0.388'' < 0.711''$ ∴ OK

Total Load Deflection

$\Delta_{TL} \leq \ell/240 = 1.067''$

$$\Delta_{TL} = \frac{5w\ell^4}{384EI} = \frac{5(6.38)(21.33)^4(1728)}{384(29{,}000)(795)}$$

$\Delta_{TL} = 1.290'' > 1.067''$ ∴ No Good

H.6 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Girder Design (cont)

Total Load Deflection (cont)

$$\ell/240 = \frac{5w\ell^4}{384EI} \Rightarrow 1.067'' = \frac{5(6.38)(21.33)^4(1728)}{384(29{,}000)\,I_{req'd}}$$

$$\therefore I_{req'd} = 961.2\ in^4$$

Table 3-3 (Steel Manual)

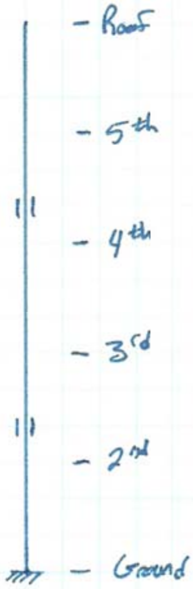
Use W16x77 $\Rightarrow I_x = 1110\ in^4 > 961.2\ in^4$ $\therefore$ OK

Comes from a self imposed depth limit of ≈ 16"

H.7 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Column Design



Tributary Area

$$A_t = \left(\frac{19'-4"}{2} + \frac{21'-4"}{2}\right)(24'-0")$$

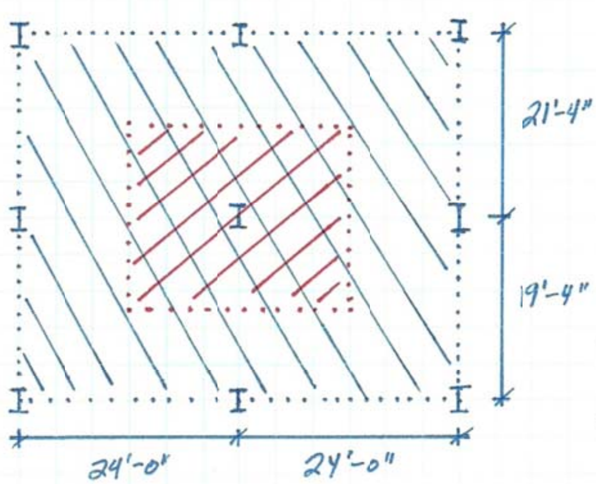
$\therefore A_t = 488\ ft^2$

Influence Area

$$A_i = (19'-4" + 21'-4")(48'-0")$$

$\therefore A_i = 1952\ ft^2$

$\therefore K_{LL} = 4$



= Influence Area

= Tributary Area

H.8 Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Column Design (cont)

Load Below 5th

$P_S = 24.64\ psf(488\ ft^2) = 12.02^k$
$P_D = 488[20 + 105] = 61.0^k$
$P_L = 0.590(80)(488) = 23.01^k$

$U_{red} = 0.25 + \frac{15}{\sqrt{4(488)}} = 0.590$

$P_{u5} = 1.2(61.0) + 1.6(23.01) + 0.5(12.02) = 116.0^k$

Load Below 3rd

$P_S = 12.02^k$
$P_D = 488[20 + 3(105)] = 163.48^k$
$P_L = 0.446(3)(80)(488) = 52.24^k$

$U_{red} = 0.25 + \frac{15}{\sqrt{4(488)(3)}} = 0.446$

$P_{u3} = 1.2(163.48) + 1.6(52.24) + 0.5(12.02) = 285.77^k$

Load Below 2nd

$P_S = 12.02^k$
$P_D = 488[20 + 4(105)] = 214.72^k$
$P_L = 0.420(4)(80)(488) = 65.55^k$

$U_{red} = 0.25 + \frac{15}{\sqrt{4(4)(488)}} = 0.420$

$P_{u2} = 1.2(214.72) + 1.6(65.55) + 0.5(12.02) = 368.6^k$

H.9- Long Span Deck Calculations (con't)

Long Span Deck (EPIC Deck) on Steel Beams

Column Design (cont)

Below 5th Floor

$P_{u5} = 116.0^k$

Table 4-1 (Steel Manual): $kL = 14'$

Use W8x31

$\phi P_n = 248^k \geq 116.0^k$ ∴ OK

Below 3rd Floor

$P_{u3} = 285.8^k$

Table 4-1 (Steel Manual): $kL = 16'$

Use W8x48

$\phi P_n = 340^k \geq 285.8^k$ ∴ OK

Below 2nd Floor

$P_{u2} = 368.6^k$

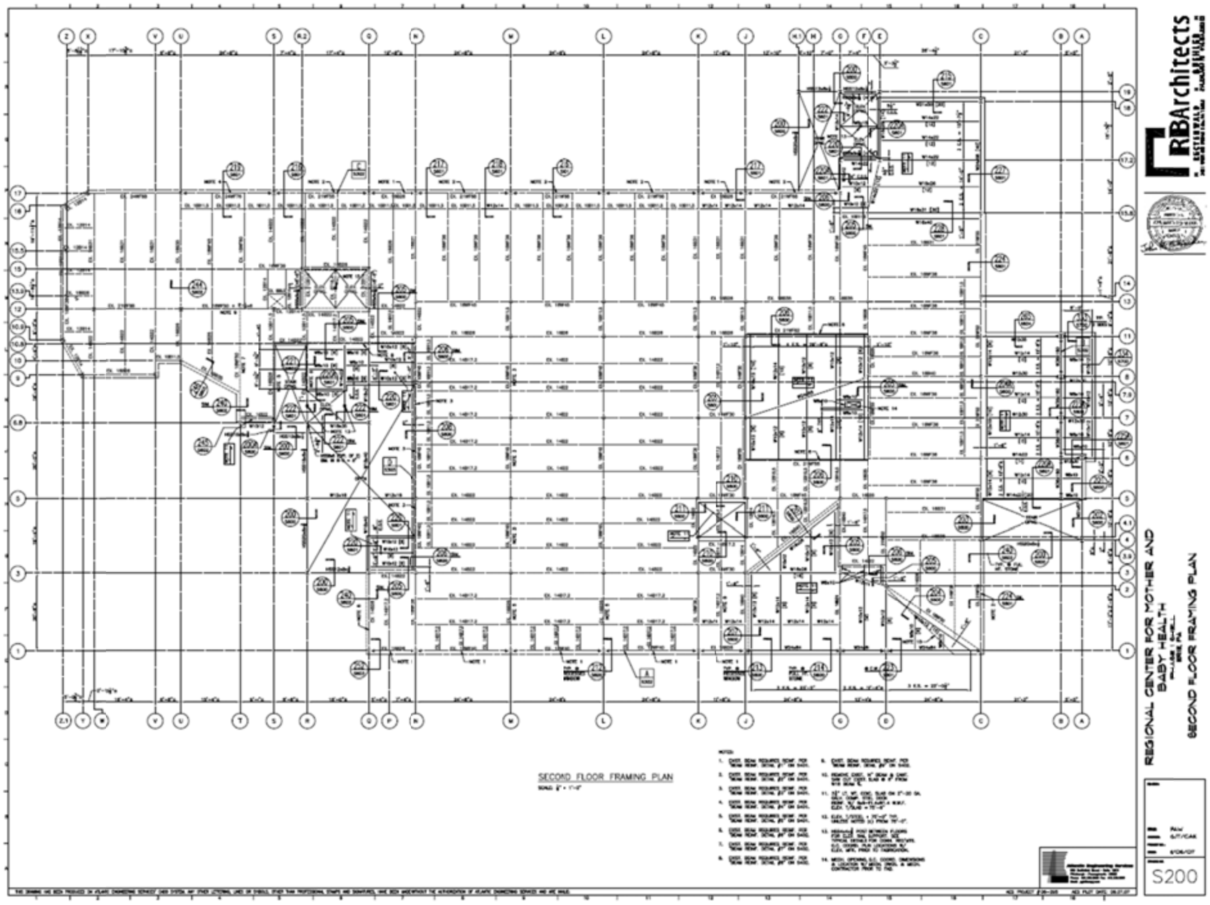
Table 4-1 (Steel Manual): $kL = 12'$

Use W8x48

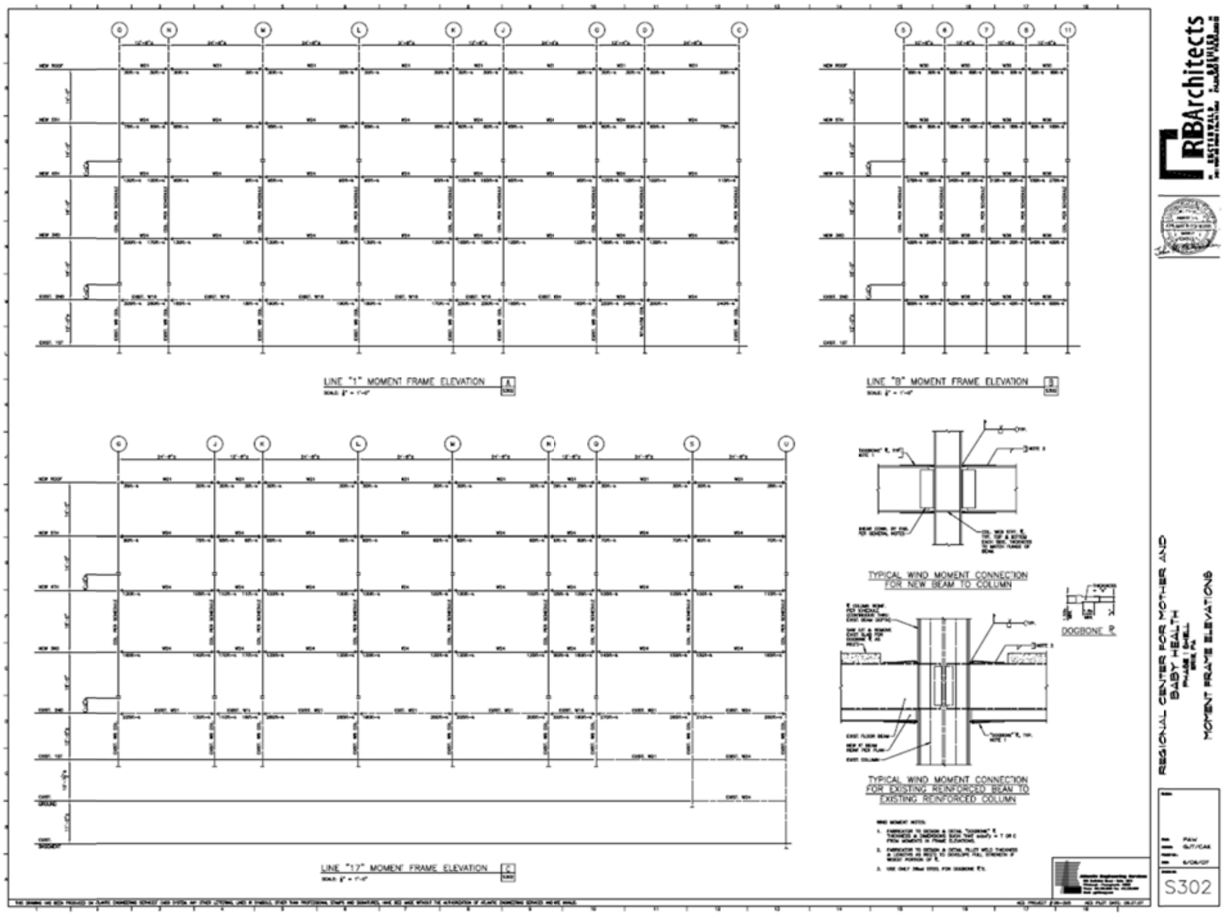
$\phi P_n = 447^k \geq 368.6^k$ ∴ OK

Appendix I: Relevant Building Plans

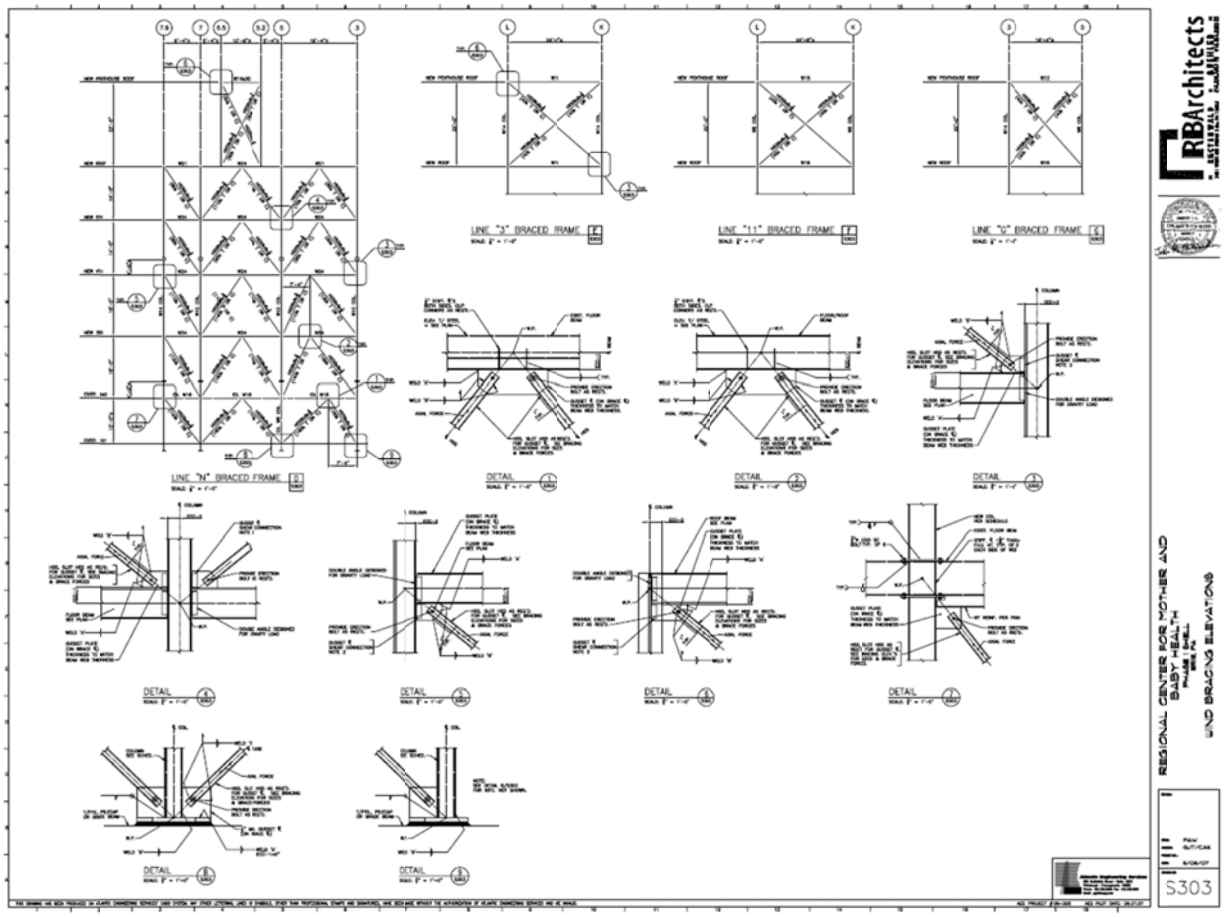
I.1 – S200 - Second Floor Structural Plan



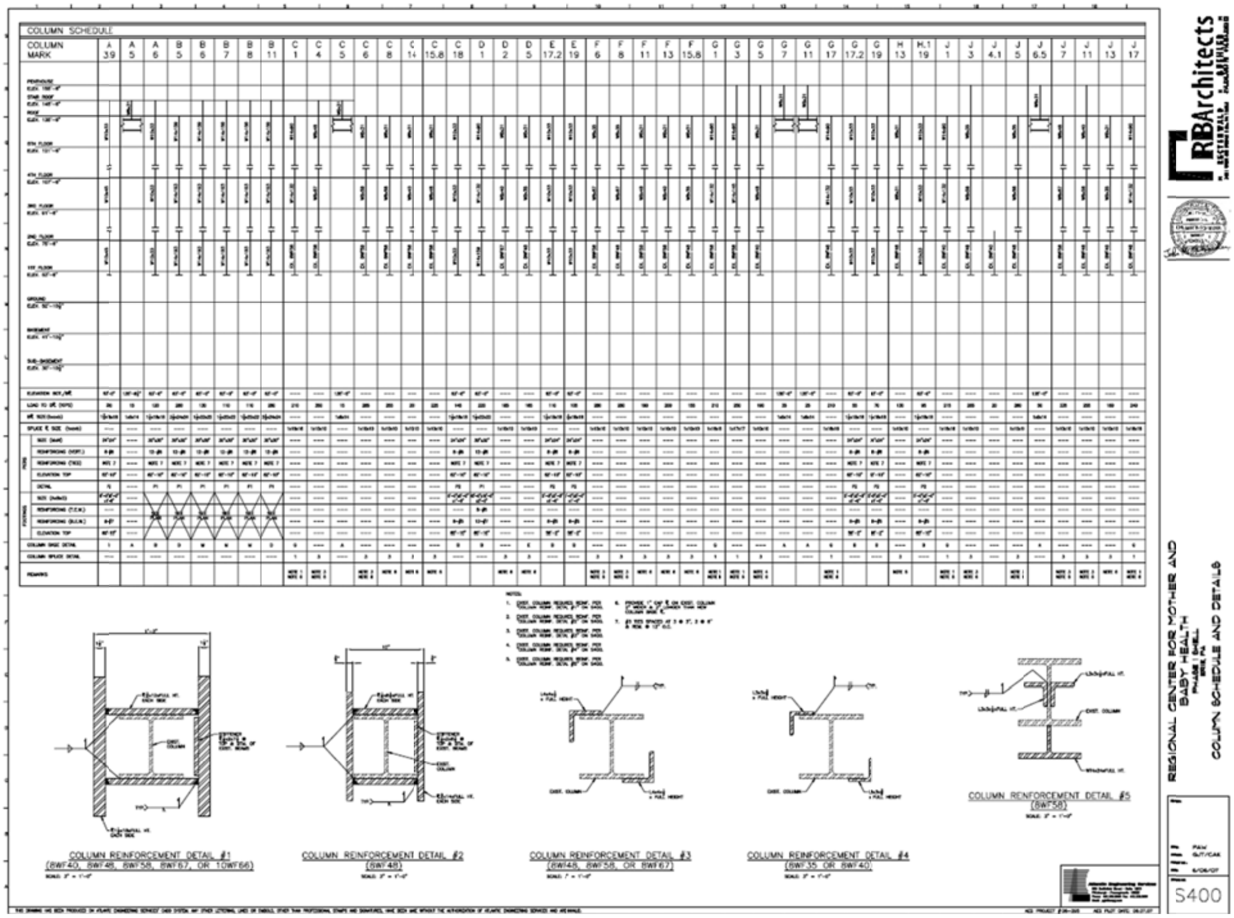
I.2 – S302 - Moment Frame Elevations



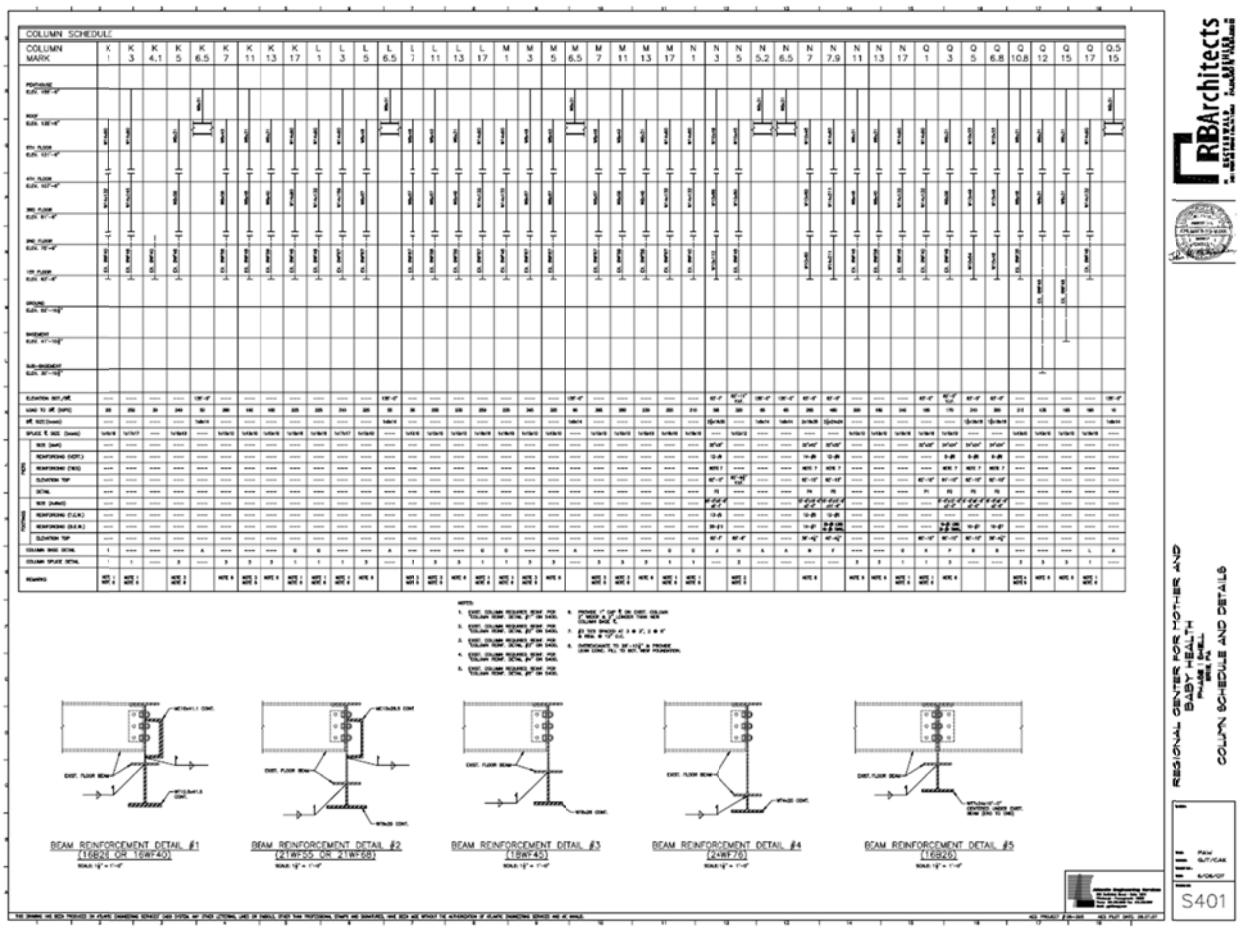
I.3 – S303 - Braced Frame Elevations



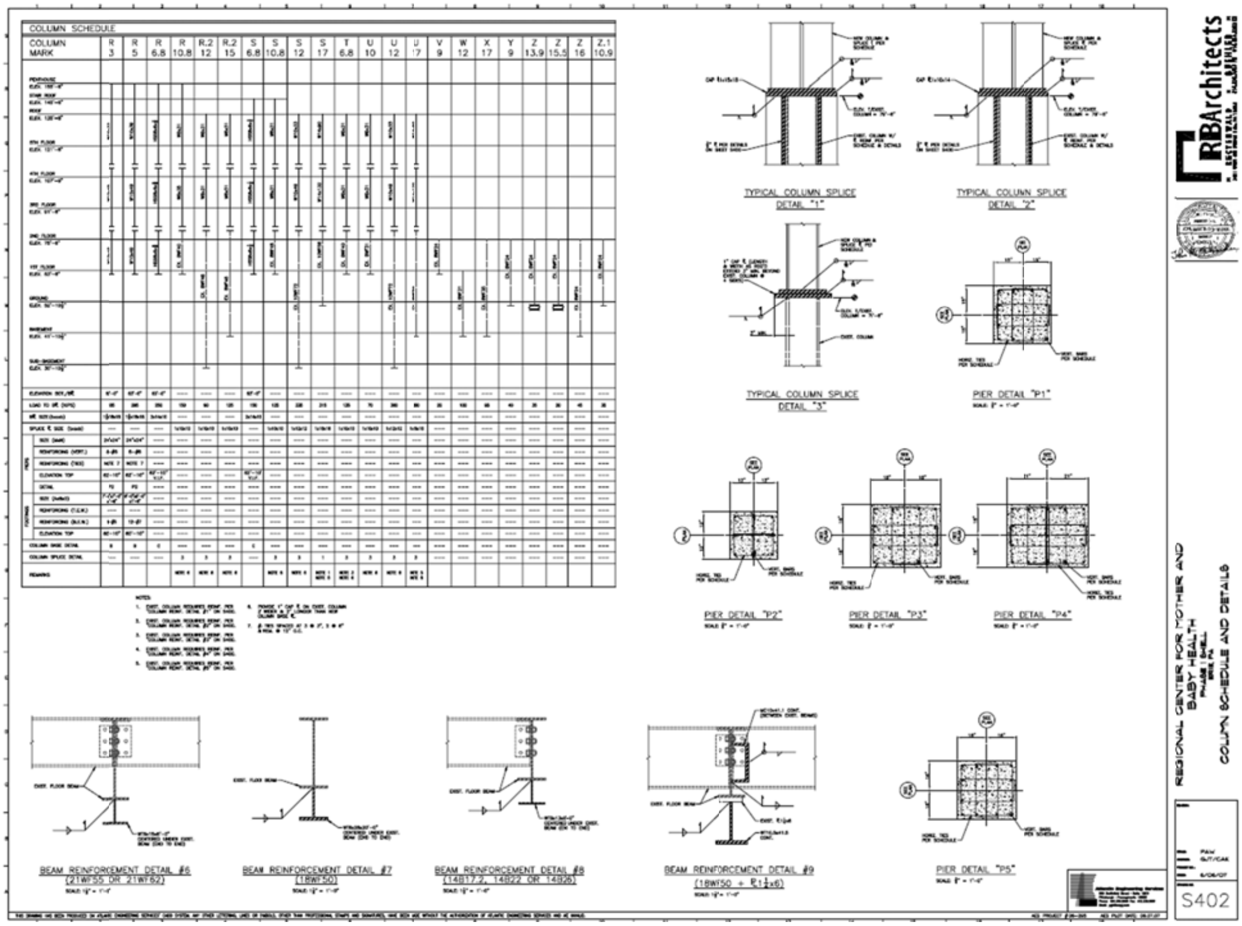
I.4 – S400 - Column Schedule and Typical Column Reinforcements



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



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



I.7 – S403 - Foundation Overbuilds