# Technical Report III

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# Executive Summary

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

The goal of this technical report entitled, "Lateral System Analysis and Confirmation Design," is to evaluate the existing lateral system as built in the UPMC Hamot Women's Hospital and confirm that it has sufficient strength and meets all serviceability requirements.  Using a rigid diaphragm assumption and lateral force distribution based on load follows stiffness.  STAAD models were created for the 2-D frames in order to determine frame stiffness and distribute the lateral forces appropriately.  The force distribution was determined based on the stiffness and the forces were input into STAAD to try and get a rough estimate of the frame reaction.  The results were slightly unsettling, revealing the need for a more detailed analysis.  This need would lead to the creation of a RAM model, which was utilized to analyze the lateral system and include more detailed calculations such as torsional analysis.  Drift limits of h/400 for wind and 0.025h for seismic were imposed on the structure.  The structure was deemed to meet all strength and serviceability requirements imposed.  Overturning moment and uplift forces were then analyzed and evaluated conservatively, by ignoring the soil weight above the footing and only relying on the weight of the footing itself.  No issues with this were found.

After the analysis of lateral systems present at the UPMC Hamot Women's Hospital was fully evaluated, no issues for either strength or serviceability requirements were found.

## Introduction

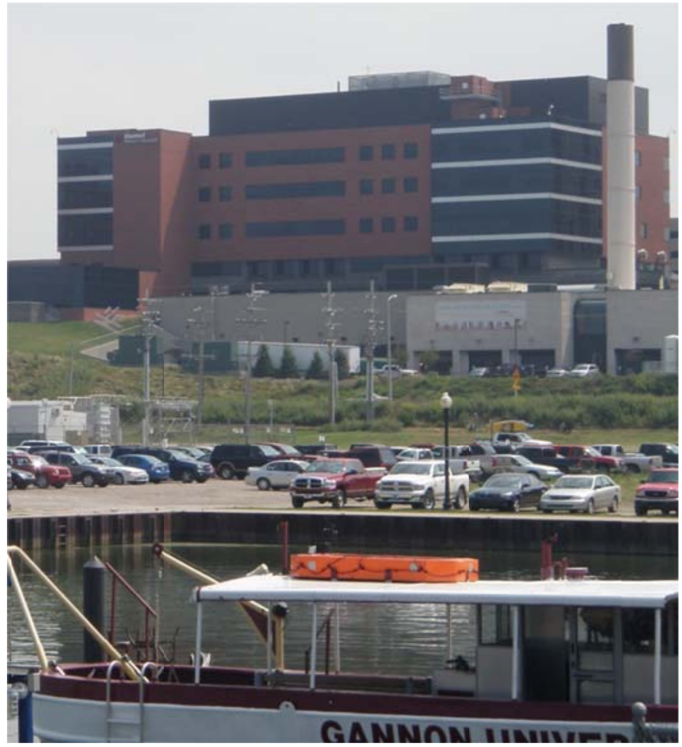


Figure 1: North Façade, Showing 2-D Escarpment

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 4$^{th}$ 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

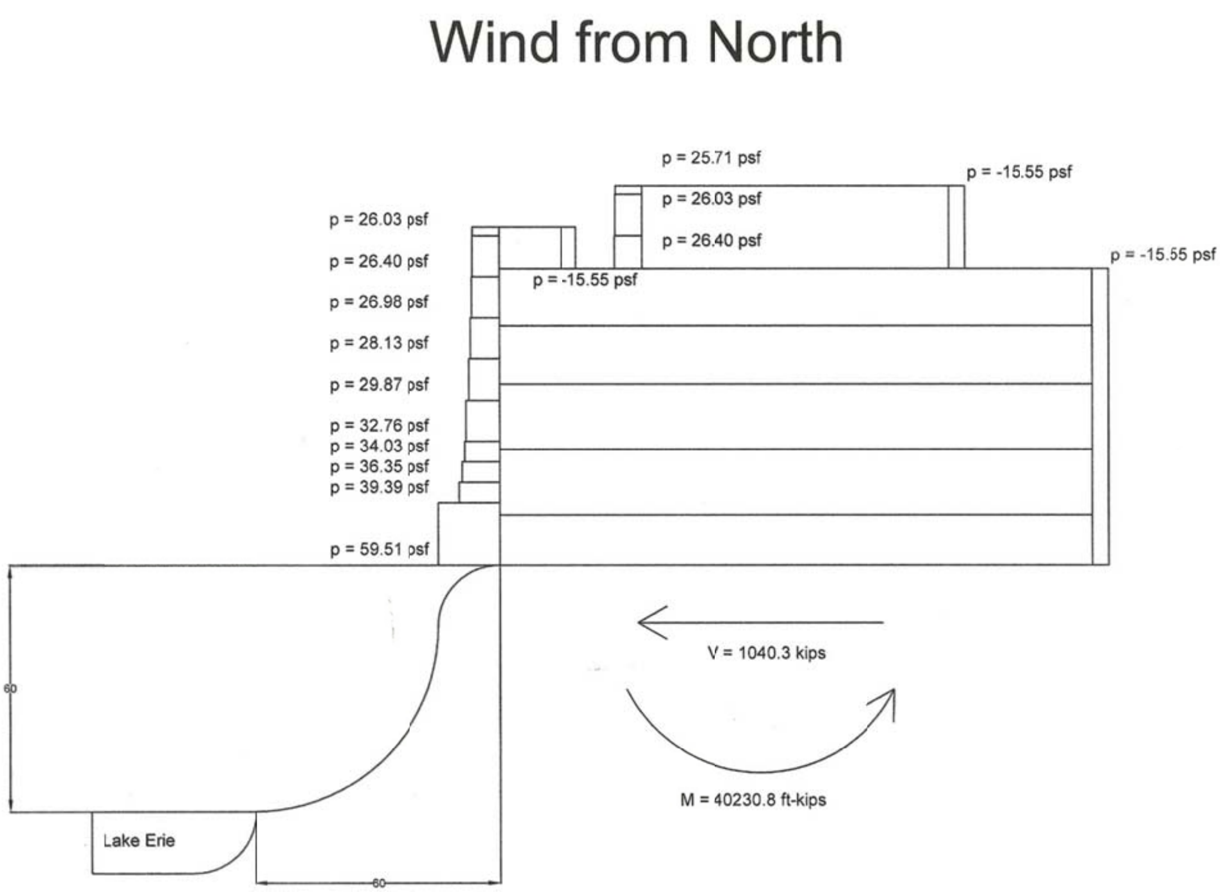



Figure 5: Wind Pressures in N-S Direction, showing 2-D Escarpment

| 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

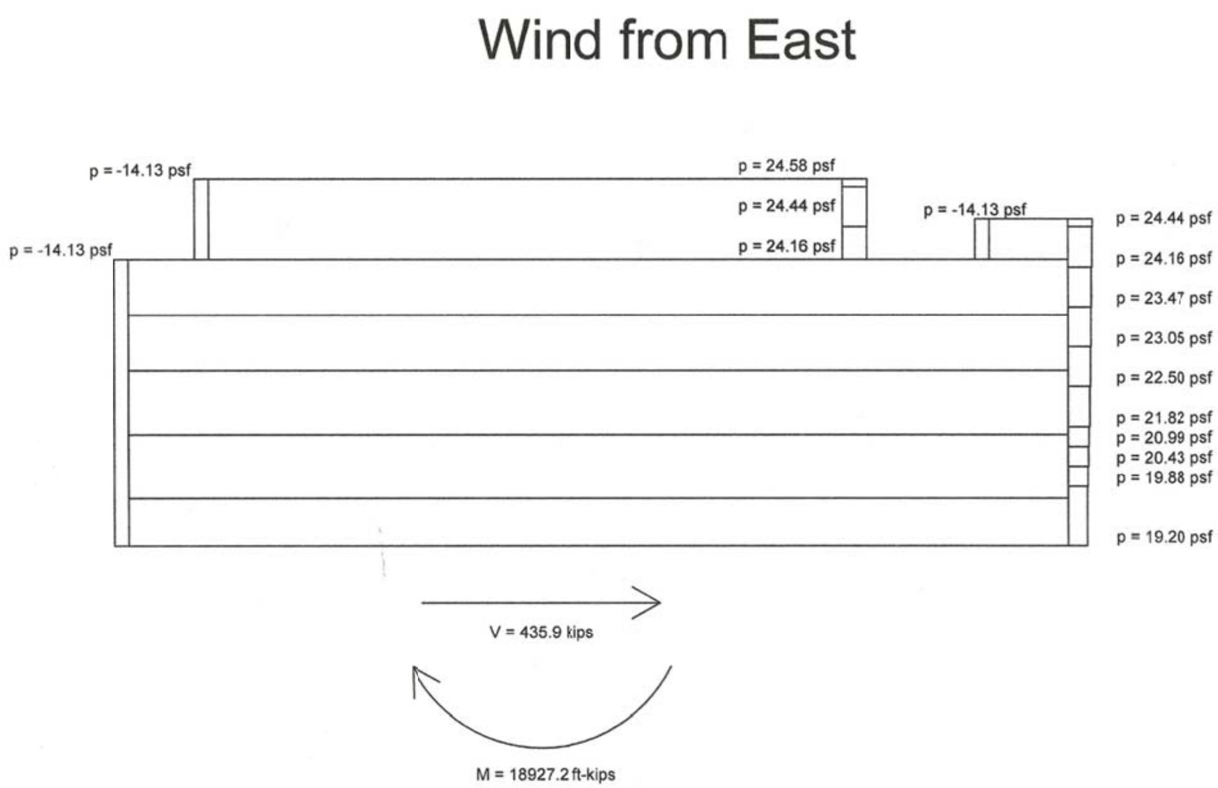



Figure 6: Wind Pressures in E-W Direction

## • 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

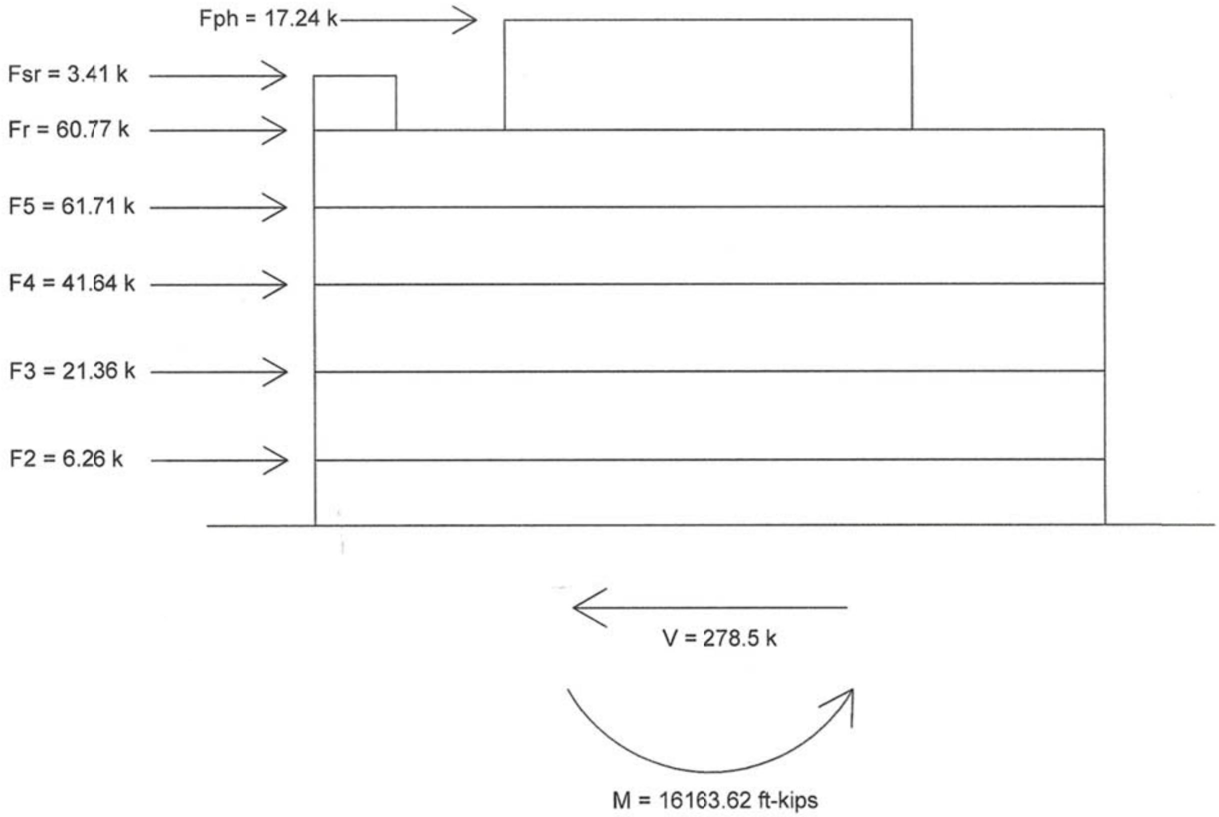



Figure 7: Earthquake Forces at Various Levels

# 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.

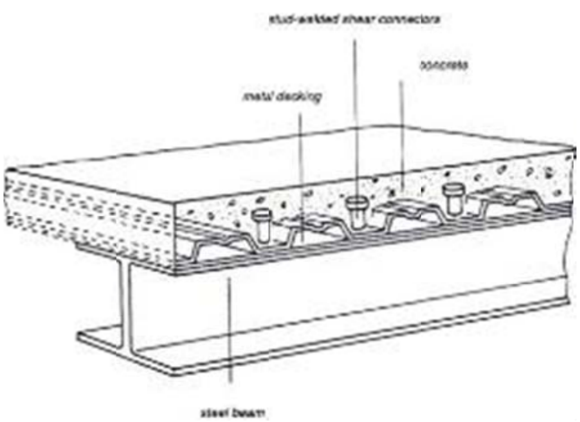



Figure 8: Lightweight Concrete on Metal Deck

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 2$^{nd}$ and 4$^{th}$ floors. Live Load reduction was utilized and the column design yielded a W8x48 column above the 4$^{th}$ floor, and a W8x67 below the 4$^{th}$ 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.

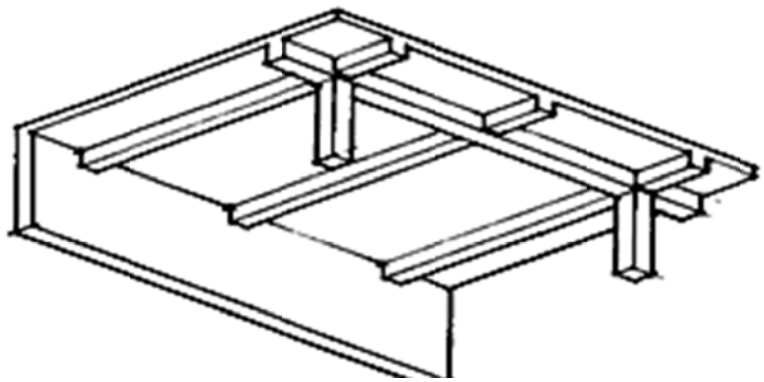


Figure 9: One Way Concrete Slab

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.

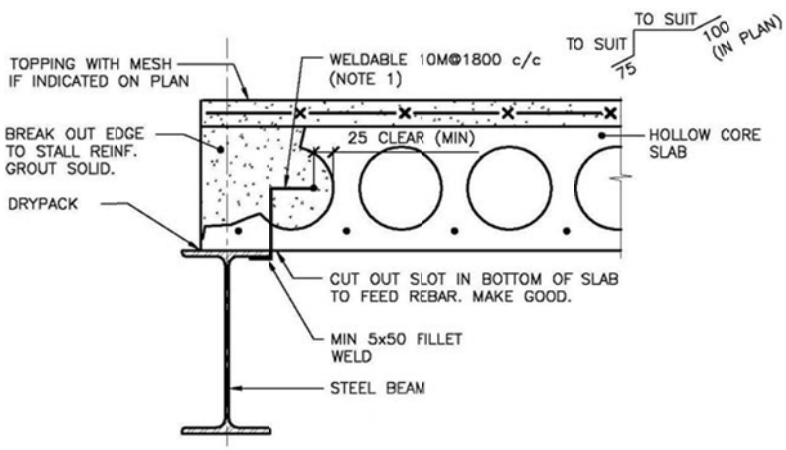



Figure 10: Hollow Core Plank on Steel Beam

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 2$^{nd}$ and 4$^{th}$ floors. Live Load reduction was utilized and the column design yielded a W8x31 column above the 4$^{th}$ floor, and a W8x40 below the 4$^{th}$ 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.

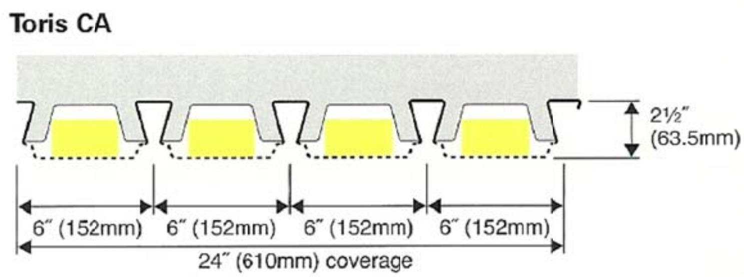



Figure 11: Lond Span (EPIC Metals) Deck on Steel Beams

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 $2^{nd}$ and $4^{th}$ floors. Live Load reduction was utilized and the column design yielded a W8x31 column above the $4^{th}$ floor, and a W8x48 below the $4^{th}$ 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 |

Table 6: Floor System Comparison

### 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.

# Lateral System Analysis

Distribution of Lateral Forces

The lateral forces of the UPMC Hamot Women's Hospital are assumed to be distributed by the composite floor slab. This element is assumed to be rigid and thus distributes the forces based on the relative stiffness of the various frames in each direction.

- ## STAAD Frame Analysis

A 2-D frame analysis was completed with the intention to analyze the frames individually. This is important to this project due to the uncertainty of how the Engineer of Record determined the lateral forces for design. When analyzing the wind forces as done previously, the question of how the 2-D escarpment along the North wall of the structure was or was not accounted for. The use of this simplified computer modeling technique will allow the author of this report to determine relative frame stiffness in both the N-S and E-W directions and then distribute the lateral loads based on this relative stiffness. This was done for both the earthquake and wind loads as determined above, based on ASCE 7-05. Details of these calculations and distribution of forces can be found in Appendix XXX.

The STAAD parameters feature was utilized for various aspects, such as:

- Stress Level Tracker
- Member Yield and Rupture Strengths based on Shape
  - W-Shapes (A992)
    - $F_y$ = 50 ksi
    - $F_u$ = 65 ksi
  - HSS Shapes (A500 Gr. B)
    - $F_y$ = 46 ksi
    - $F_u$ = 58 ksi
- Fully Braced Composite Floor Members where Applicable

The member strengths parameter is important to note and change within STAAD because the STAAD default for member strengths is A36 steel, which is not used in any of these frames. The fully braced composite floor members is important to utilize when analyzing the floor beams because a beam that is being analyzed as unbraced is overly conservative and not representative of the behavior that is expected.

Utilizing the stress level tracker available within STAAD allows for a much quicker analysis as to if a member fails applicable strength limit states as detailed in the AISC Specification. The stress level tracker output was done through the graphical user interface (GUI), set at the following parameters; stress level below 80% is indicated in green, stress levels between 80% and 100% is indicated in blue, and stress levels above 100% is indicated in red and would fail under the applied loads.

Note that the first level columns and beams were reinforced as part of the renovation of this structure, modeling these reinforcements to the tenth degree of accuracy was determined to be too time consuming, and thus was omitted. In substitution the frames were modeled as if NO column or beam reinforcement was done. This also allows the author of this report to get an idea for how necessary the reinforcements are for the lateral system.

Frame 1 Results

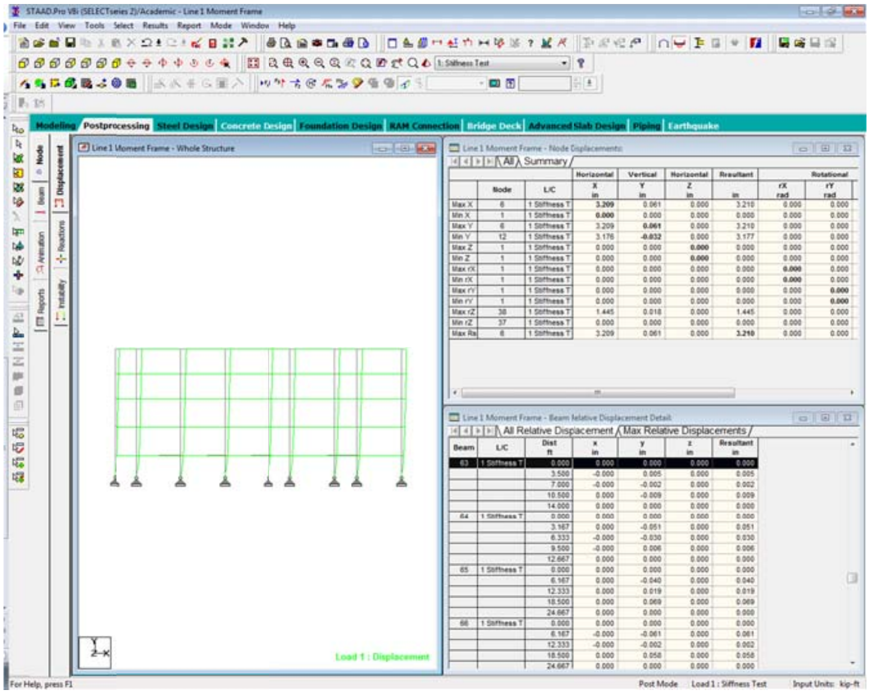


Figure 12: Displacement Analysis for Frame 1

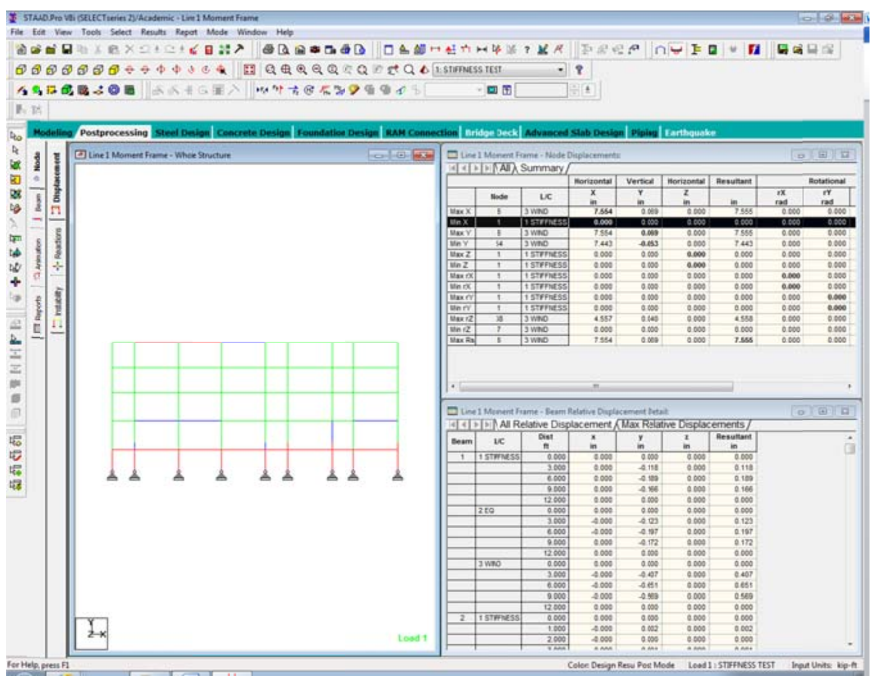


Figure 13: Stress Level Tracker for Frame 1

Frame 17 Results

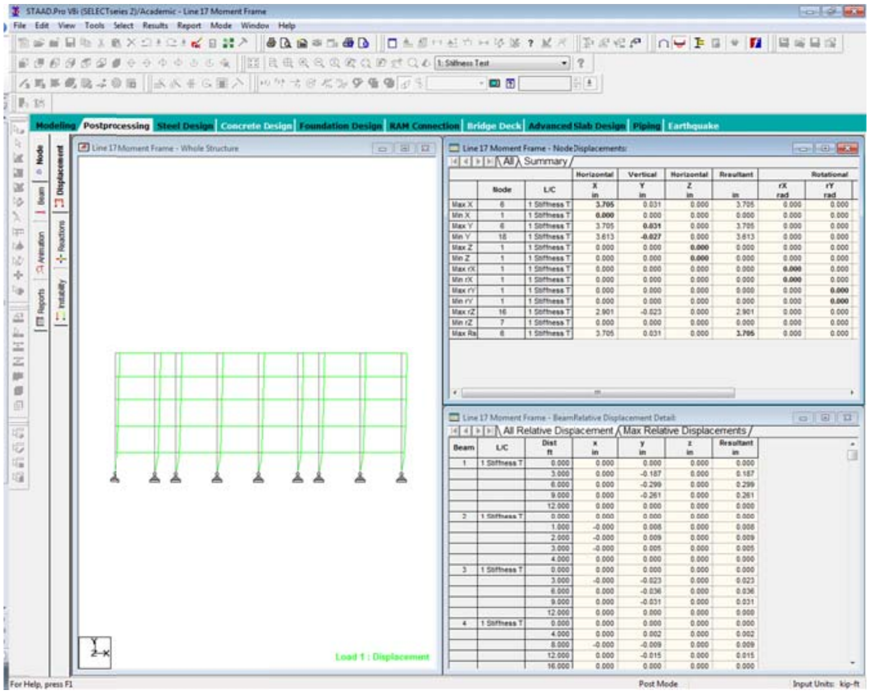


Figure 14: Displacement Analysis for Frame 17

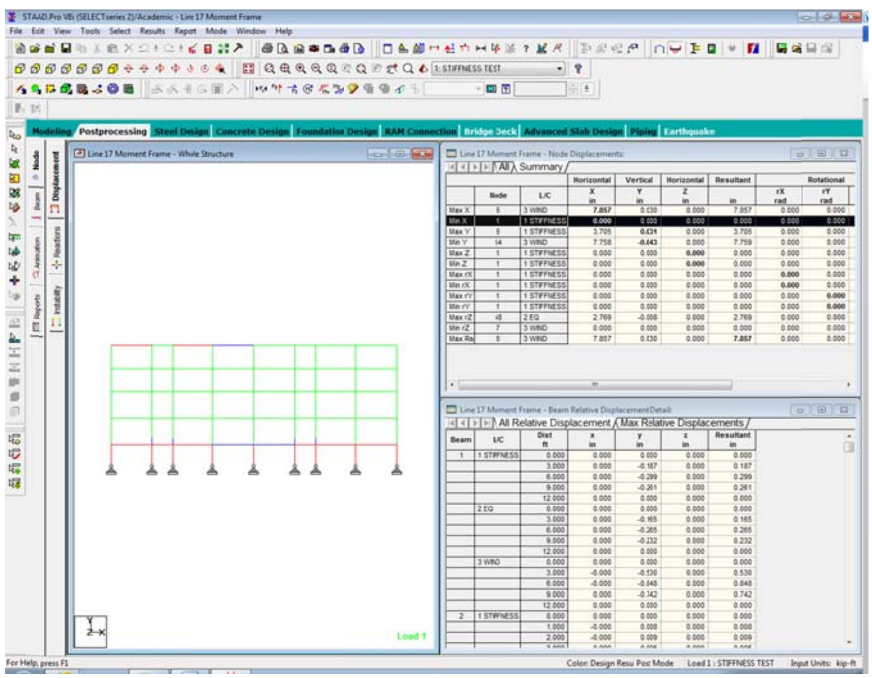


Figure 15: Stress Level Tracker for Frame 17

Frame B Results

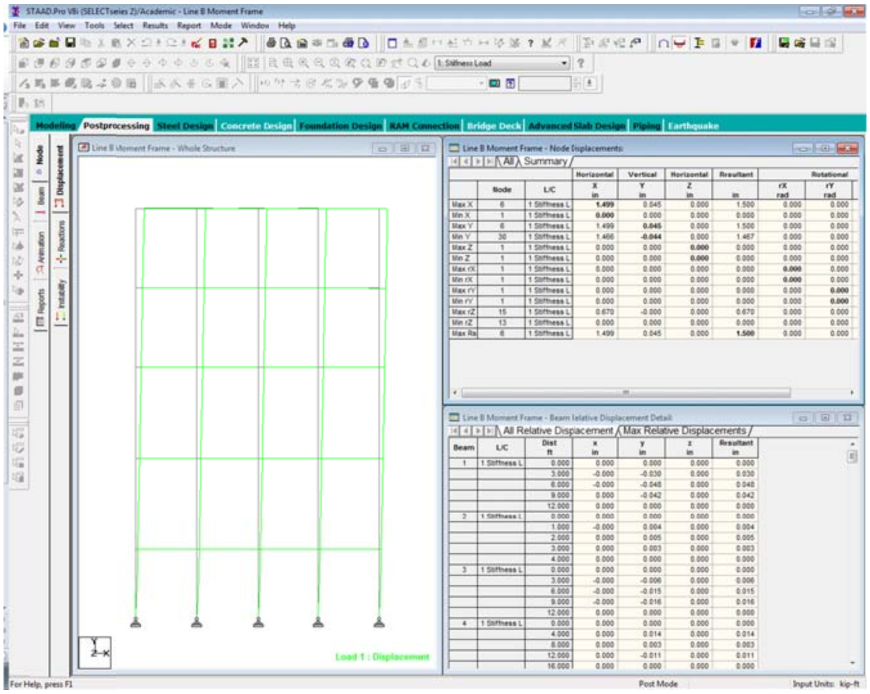


Figure 16: Displacement Analysis for Frame B

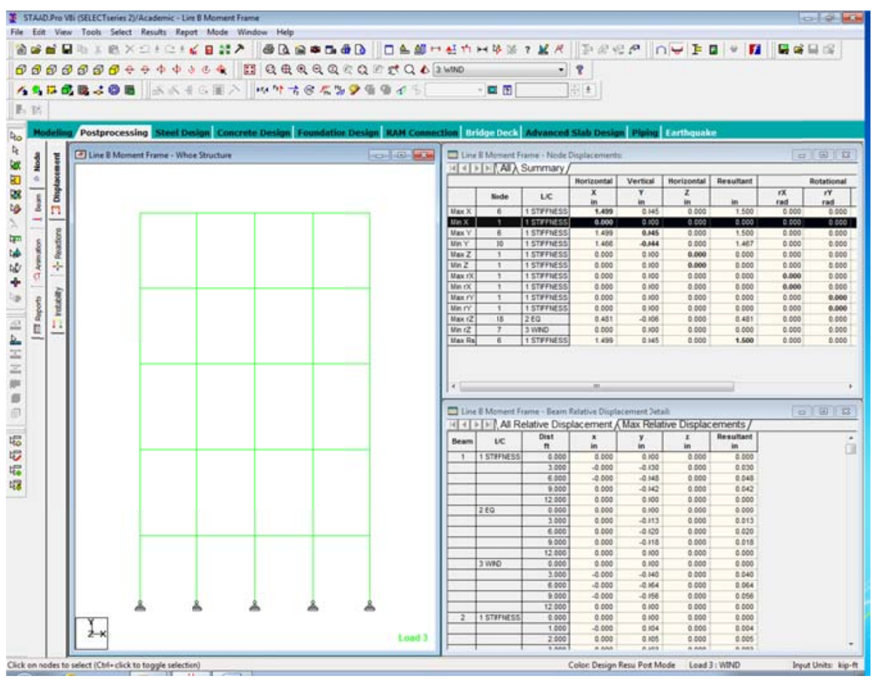


Figure 17: Stress Level Tracker for Frame B

Frame N Results

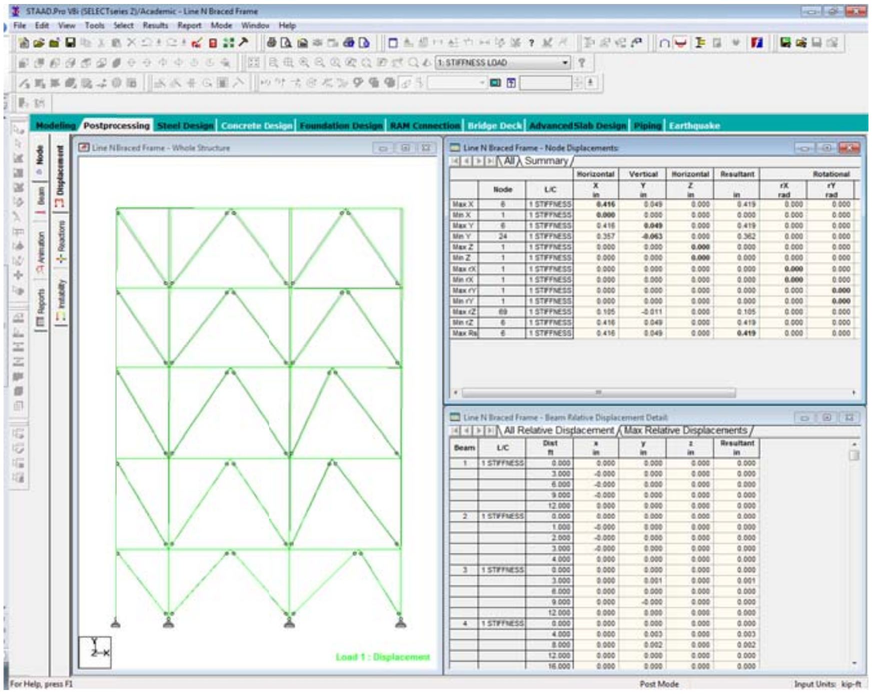


Figure 18: Displacement Analysis for Frame N

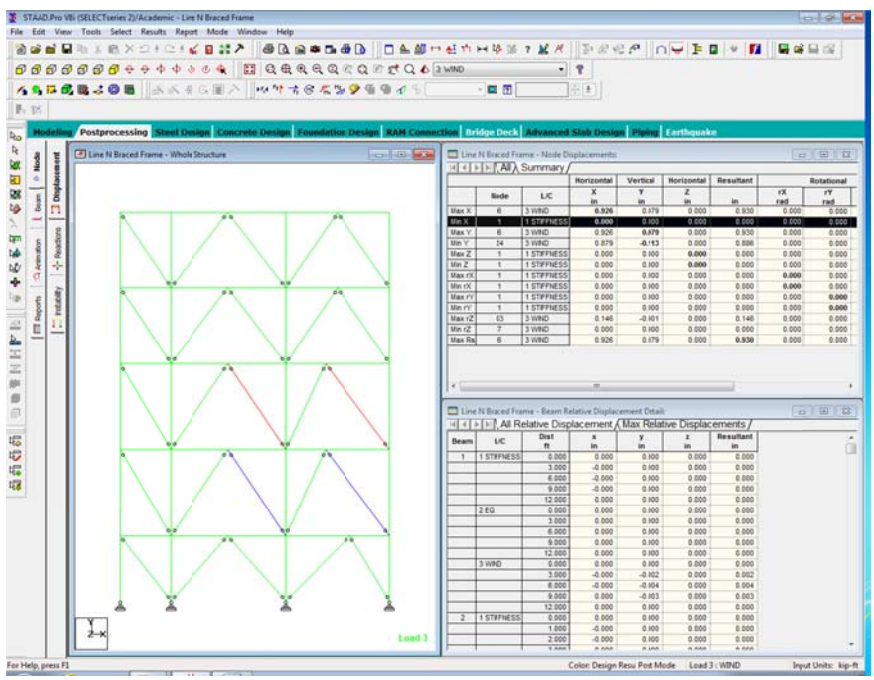


Figure 19: Stress Level Tracker for Frame N

Analyzing these frames as detailed above allowed for several conclusions to be drawn.

The N-S Lateral System proved to be quite representative of what would be expected. The moment frame along column line B was determined to be much less rigid than the braced frame along column line N. This led to the braced frame taking much more of the lateral loads in this direction, once again as expected. The stresses in members along Column Line B were less than 80% of capacity in all members, where the stresses in members along Column Line N had a 2 bracing members that were not deemed adequate to carry the applied loads. This may or may not be a concern because unfortunately the way these frames were modeled did not allow for the bracing to be designed as a tension only member, which it may have been designed for.

The E-W Lateral System also proved to be quite representative of what would be expected; both frames in this direction had similar stiffness, so based on the force distribution method chosen they would both receive similar forces. Many of the first floor beams and columns ended up being overstressed, which as before is going to be ignored for the time being. The only concern with the lateral frames in this direction is that each frame has two roof beams that are overstressed. This is a concern but will need further analysis to determine if the members designed were not adequate.

- **RAM Steel Frame Analysis**

A RAM Model was created for the analysis of the building lateral system. This model, as shown below was intended to include as much detail as possible in order to replicate the as-built construction present at the UPMC Women's Hospital.

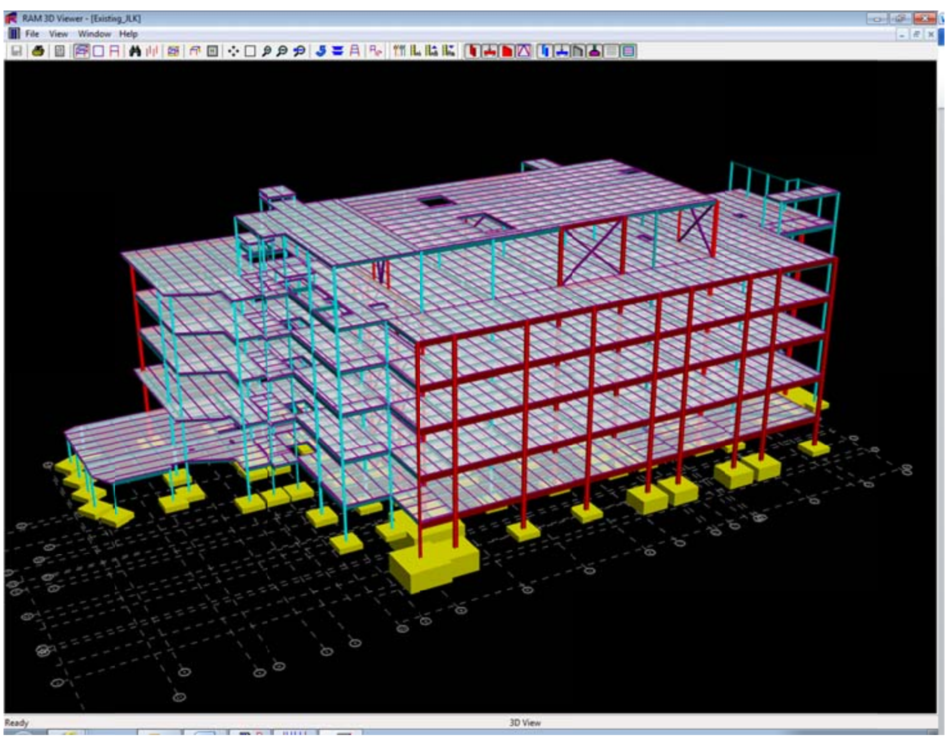



Figure 20: 3-D View of RAM Steel Model

Load Combinations

D = Dead Load
Lp = Positive Live Load
Ln = Negative Live Load
Sp = Positive Snow Load
Sn = Negative Snow Load
Ex = Earthquake X-Direction Load
Ey = Earthquake Y-Direction Load
Wx = Wind X-Direction Load
Wy = Wind Y-Direction Load

E = Eccentricity

D = 1.0 D
Lp = 1.0 D +1.0 Lp
Ln = 1.0 D + 1.0 Ln
Sp = 1.0 D + 1.0 Sp
Sn = 1.0 D + 1.0 Sn
E1 = Ex + E
E2 = Ex – E
E3 = Ey + E
E4 = Ey – E
W1 = Wx
W2 = Wy
W3 = Wx + E
W4 = Wx – E
W5 = Wy + E
W6 = Wy – E
W7 = Wx + Wy
W8 = Wx - Wy
W9 = Wx + Wy (CW)
W10 = Wx + Wy (CCW)
W11 = Wx – Wy (CW)
W12 = Wx – Wy (CCW)

The Load Cases Listed above are the “major” load cases. In total RAM Frame analyzed 462 different load cases that encompass every possible direction and load factor allowed under ASCE 7-05. This proves to be the major benefit of RAM Steel, because a hand analysis of 462 load cases is not feasible.

ASCE 7-05 defines wind load in 4 cases, RAM Frame considers these 4 cases accordingly. Case 1 is covered with load combinations W1 and W2. Case 2 is covered with load combinations W3, W4, W5 and W6. Case 3 is covered with load combinations W7 and W8. Case 4 is covered with load combinations W9, W10, W11 and W12.

In the analysis of the lateral system, RAM Steel was used to calculate the lateral loads and then apply them appropriately to the floor diaphragms. This is the most realistic application of loads possible, because RAM Steel is very efficient at finding the diaphragm center of mass and corresponding floor center of rigidities; thus accounting for torsion and the additional forces that the respective frames would take in a torsional analysis, as shown below in the difference in location from the Center of Rigidity (blue) and the Center of Mass (Red) at the various floor levels

.

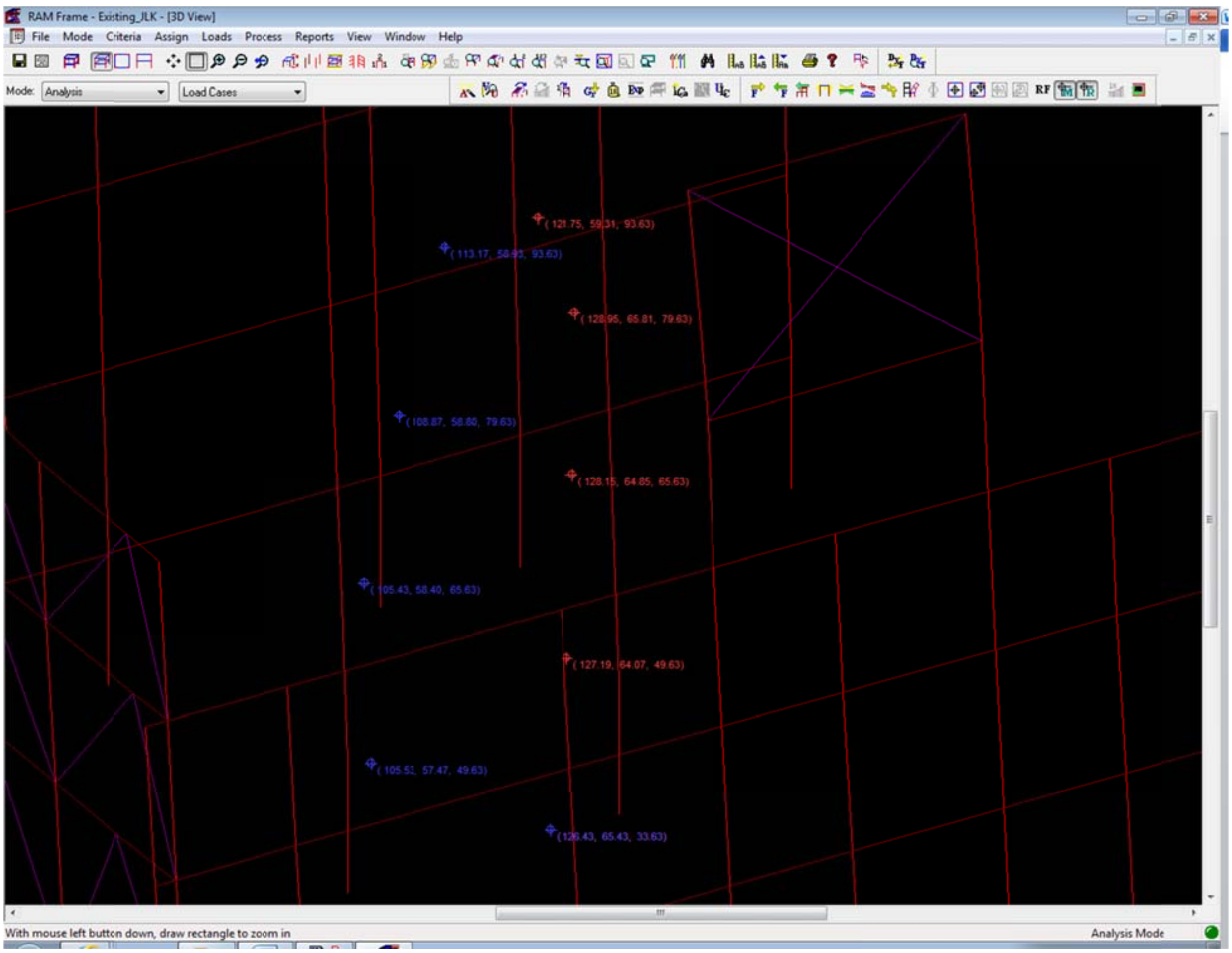



Figure 21: Center of Mass to Center of Rigidity Comparison

RAM Steel was also utilized to calculate drift with torsional effects. Below are the tabulated values at the roof level.

| Story | Load Combo | Displacement | | Story Drift | | Drift Ratio | |
|---|---|---|---|---|---|---|---|
| | | X-Dir | Y-Dir | X-Dir | Y-Dir | X-Dir | Y-Dir |
| Roof | D | -0.0048 | -0.0308 | -0.0032 | -0.0157 | 0.0000 | 0.0001 |
| | Lp | -0.0107 | -0.0278 | -0.0035 | -0.0151 | 0.0000 | 0.0001 |
| | Ln | 0.0001 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| | Sp | -0.0055 | -0.0225 | -0.0032 | -0.0105 | 0.0000 | 0.0001 |
| | Sn | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| | E1 | 0.8368 | 0.0524 | 0.1511 | 0.0086 | 0.0009 | 0.0001 |
| | E2 | 0.7920 | -0.0246 | 0.1433 | -0.0062 | 0.0009 | 0.0000 |
| | E3 | -0.1149 | 0.3372 | -0.0084 | 0.0870 | 0.0001 | 0.0005 |
| | E4 | -0.0030 | 0.5295 | 0.0104 | 0.1229 | 0.0001 | 0.0007 |
| | W1 | 0.9611 | 0.0663 | 0.1370 | 0.0080 | 0.0008 | 0.0000 |
| | W2 | -0.0702 | 0.6105 | 0.0034 | 0.1365 | 0.0000 | 0.0008 |
| | W3 | 0.7713 | 0.1361 | 0.1093 | 0.0190 | 0.0007 | 0.0001 |
| | W4 | 0.6704 | -0.0366 | 0.0962 | -0.0070 | 0.0006 | 0.0000 |
| | W5 | -0.2129 | 0.1829 | -0.0202 | 0.0576 | 0.0001 | 0.0003 |
| | W6 | 0.1077 | 0.7329 | 0.0253 | 0.1472 | 0.0002 | 0.0009 |
| | W7 | 0.6674 | 0.5081 | 0.1051 | 0.1085 | 0.0006 | 0.0006 |
| | W8 | 0.7723 | -0.4090 | 0.0999 | -0.0966 | 0.0006 | 0.0006 |
| | W9 | 0.6586 | 0.6519 | 0.1007 | 0.1247 | 0.0006 | 0.0007 |
| | W10 | 0.3425 | 0.1102 | 0.0569 | 0.0380 | 0.0003 | 0.0002 |
| | W11 | 0.7373 | -0.0359 | 0.0969 | -0.0292 | 0.0006 | 0.0002 |
| | W12 | 0.4212 | -0.5776 | 0.0530 | -0.1158 | 0.0003 | 0.0007 |

Figure 22: Displacement Results at the Roof Level

RAM Frame Drift results were then taken and compared against allowable drift results given in ASCE 7-05. Which states that maximum drift for wind is h/400 and the maximum allowable drift for seismic is 0.025h.

| Drift Analysis | | | | |
|---|---|---|---|---|
| | Wind Analysis | | Earthquake Analysis | |
| | X-Direction | Y-Direction | X-Direction | Y-Direction |
| **Max Δ** | 0.9611” | 0.7329” | 0.8369” | 0.5295” |
| **Allowable Δ** | 2.16” | 2.16” | 21.6” | 21.6” |
| **Results** | Acceptable | Acceptable | Acceptable | Acceptable |

Table 7: Comparison of Displacement Values

As expected the drift for wind was the controlling factor. This is due to the relatively low seismic region and the 2-D escarpment present on the North side of the building.

Story Drifts were also analyzed in this manner and were deemed to be well within the allowable limits.

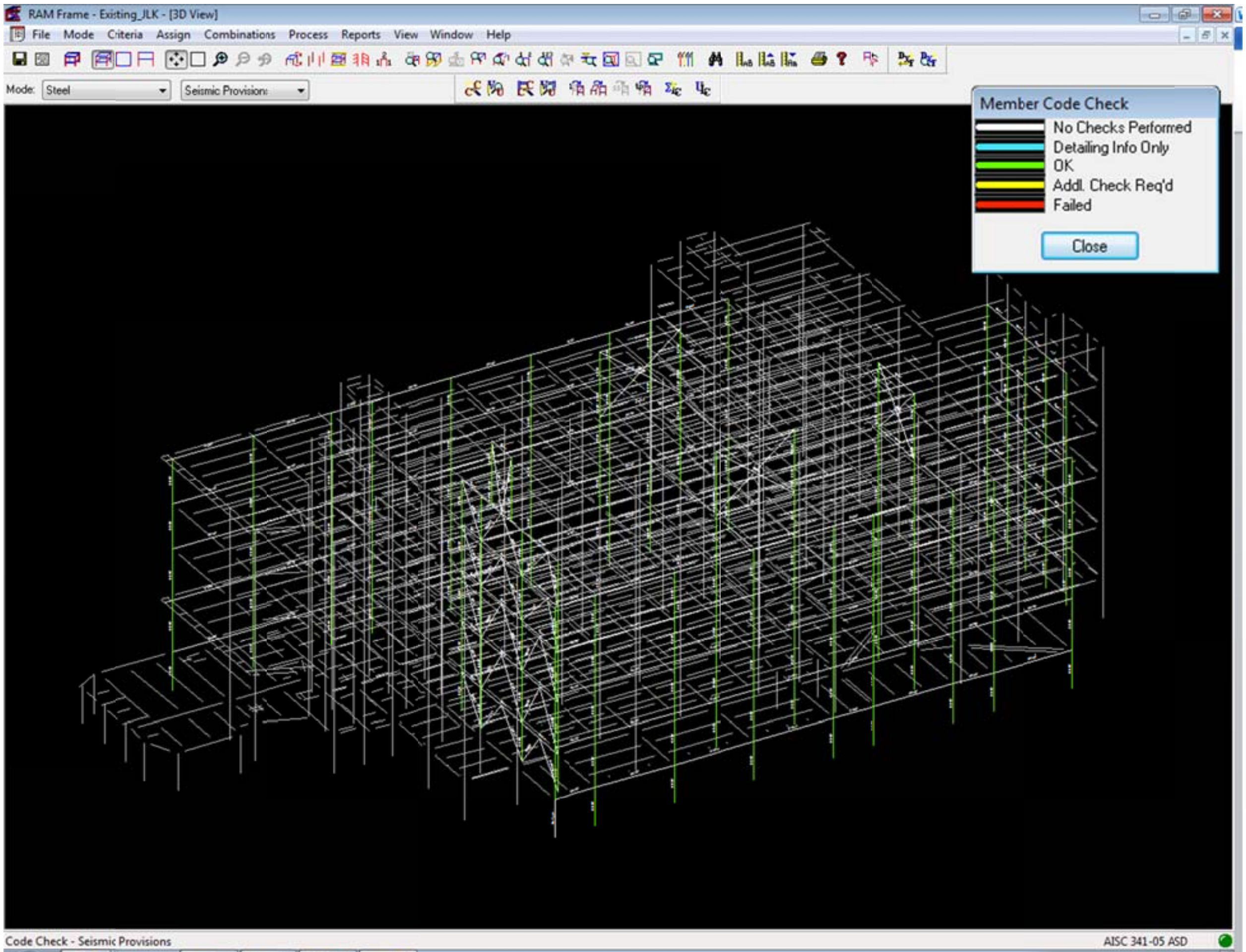



Figure 23: Member Code Check

After analyzing the Lateral Force Resisting System present in the UPMC Hamot Women's Hospital all members were found to be adequate as designed.

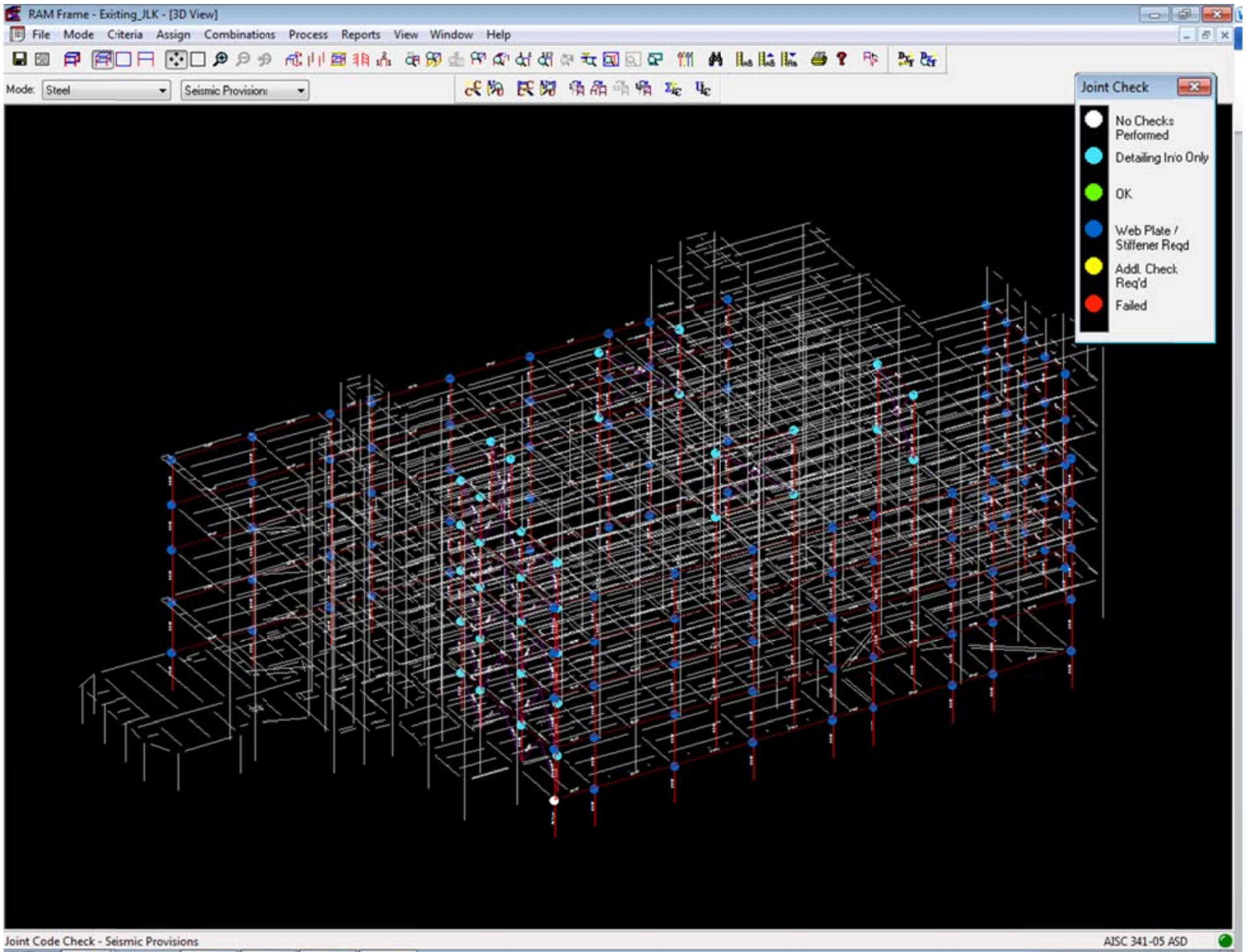



Figure 24: Member Joint Check

After using RAM Frame to analyze the need for member stiffeners, most members were found to require a stiffener. This is acceptable because the members in question were designed with stiffeners.

## Lateral System Summary

The use of RAM Steel and RAM Frame is for purely computational power. The computer can analyze the structure at a much faster rate of speed than a human with a calculator could ever dream of doing this work. This benefit comes with the disadvantage of not being able to see exactly what the computer is doing. In order to account for that several spot checks were done as indicated and can be found in Appendix I. The spot checks for the beam, column, and brace were all completed based on tabulated values in the AISC steel manual. The interaction equations for combined loading were used where applicable. This analysis supported the RAM calculations found through computer modeling. Knowing the options and defaults of the computer software is the key to being able to safely utilize the computational power that any structural design software can provide.

| Frame Summary | | | | |
|---|---|---|---|---|
| | N-S Lateral Force Resisting System | | E-W Lateral Force Resisting System | |
| | Column Line B | Column Line N | Column Line 1 | Column Line 17 |
| **Type** | Moment Frame | Braced Frame | Moment Frame | Moment Frame |
| **Length** | 42.67’ | 50’ | 161.33’ | 173.33’ |
| **Frame Stiffness** | 66.7 k/in | 240.4 k/in | 31.2 k/in | 27.0 k/in |
| **Relative Stiffness** | 0.217 | 0.783 | 0.536 | 0.464 |

Table 8: Lateral System Summary

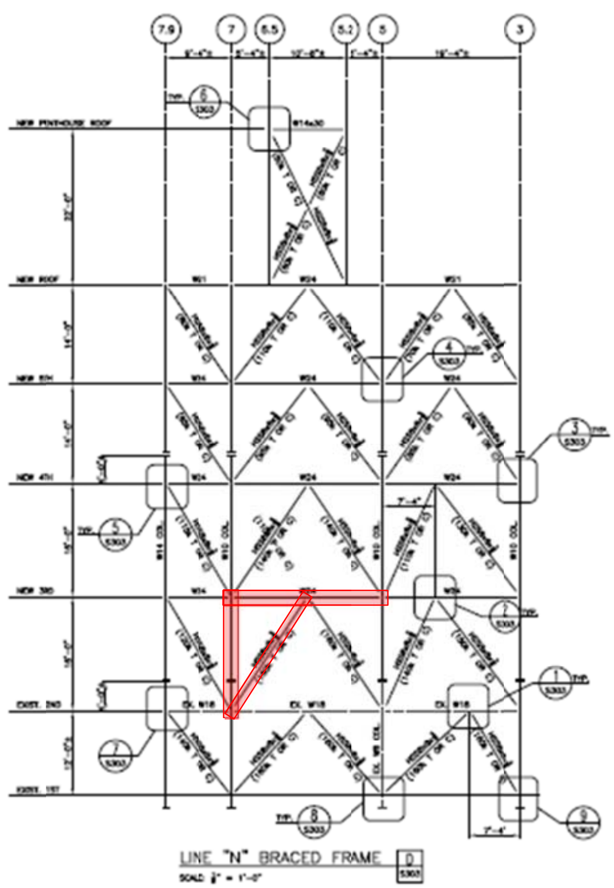



Figure 25: Members Used for Spot Checks

## Conclusion

This lateral system analysis and confirmation concludes that the lateral system of the UPMC Hamot Women's Hospital is adequate for both strength and serviceability requirements. This was determined through a STAAD and RAM Model and various hand calculations to confirm the results found in the computer analysis. The analysis of the lateral system involved both wind and earthquake criteria found in ASCE 7-05, with wind found to be the controlling load case due to the 2-D Escarpment present on the north side of the building and the high exposure factor.

Both strength and serviceability requirement for the lateral system were found to be well within the limits required by the code. The drift was calculated through the computer analysis and deemed to be representative of both the expected drift.

The use of the computer models was done to expedite the process of lateral frame analysis and individual member design. As well as to analyze drift and loads on the structure quickly and efficiently. Three spot checks (a beam, column, and brace) were done to confirm the strength requirements found were accurate and reliable, which they were determined to be.

# Appendix A: Gravity Load Calculations

A.1 – Dead Load Calculations

Dead Loads

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

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

Total Slab Thickness = 6"
Theoretical Concrete Volume = $0.417\ ft^3/ft^2 \times 110\ lb/ft^3 = 46\ lb/ft^2$
Deck Weight = 2 psf

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

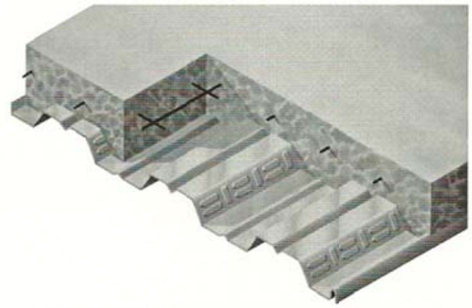
Roof Weight

1½" Galvanized Steel Roof Deck - 20 GA = 2 psf
↳ Wide Rib Deck
Roofing 3 psf
Insulation 5 psf
Ceiling / MEP 5 psf
15 psf

∴ Use 20 psf total
↳ Includes 5 psf Superimposed DL

A.2 – Vulcraft Manual Page for 2VLI Decks

VULCRAFT



## SLAB INFORMATION

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

## (N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

| TOTAL SLAB DEPTH | DECK TYPE | SDI Max. Unshored Clear Span | | | Superimposed Live Load, PSF Clear Span (ft.-in.) | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 SPAN | 2 SPAN | 3 SPAN | 6'-0 | 6'-6 | 7'-0 | 7'-6 | 8'-0 | 8'-6 | 9'-0 | 9'-6 | 10'-0 | 10'-6 | 11'-0 | 11'-6 | 12'-0 | 12'-6 | 13'-0 |
| 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

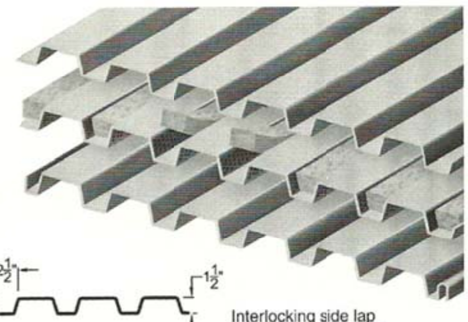
ROOF

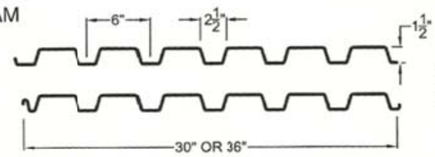
# 1.5 B, BI, BA, BIA

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 $I_p$ in$^4$/ft | $S_p$ in$^3$/ft | $I_n$ in$^4$/ft | $S_n$ in$^3$/ft | $V_a$ lbs/ft | $F_y$ ksi |
|---|---|---|---|---|---|---|---|---|
| B24 | 0.0239 | 1.46 | 0.107 | 0.120 | 0.135 | 0.131 | 2634 | 60 |
| B22 | 0.0295 | 1.78 | 0.155 | 0.186 | 0.183 | 0.192 | 1818 | 33 |
| B20 | 0.0358 | 2.14 | 0.201 | 0.234 | 0.222 | 0.247 | 2193 | 33 |
| B19 | 0.0418 | 2.49 | 0.246 | 0.277 | 0.260 | 0.289 | 2546 | 33 |
| B18 | 0.0474 | 2.82 | 0.289 | 0.318 | 0.295 | 0.327 | 2870 | 33 |
| B16 | 0.0598 | 3.54 | 0.373 | 0.408 | 0.373 | 0.411 | 3578 | 33 |

## ACOUSTICAL INFORMATION

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

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

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

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

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

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

Batts are field installed and may require separation.

## VERTICAL LOADS FOR TYPE 1.5B

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

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

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

NUCOR VULCRAFT GROUP

7

A.4 – Live Loads from ASCE 7-05

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

## Appendix B: Snow Load & Drift Calculations

B.1 - Snow Load and Drift Calculations

Snow Loads

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

ASCE 7-05

Flat Roof Snow Load

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

$p_g = 40$ psf, see note above

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

$C_t = 1.0$ ⇒ Table 7-3 (ASCE 7-05)

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

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

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

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

## Snow Loads (cont)

### ASCE 7-05

Drift Snow Load (Penthouse Roof)

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

N-S Drift

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

$\therefore h_d = 2.8'$

E-W Drift

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

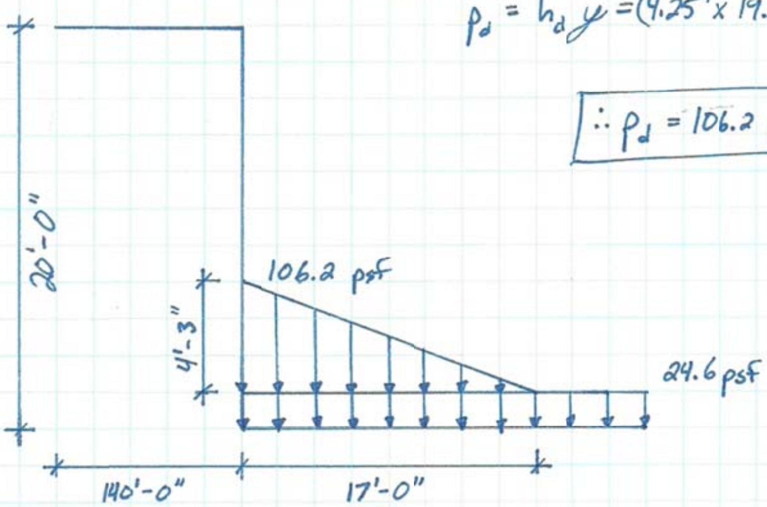
$\therefore h_d = 4.25'$

$\therefore$ 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

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



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

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Stair Pop-Out)

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

N-S Drift

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

$\therefore h_d = 1.75'$

E-W Drift

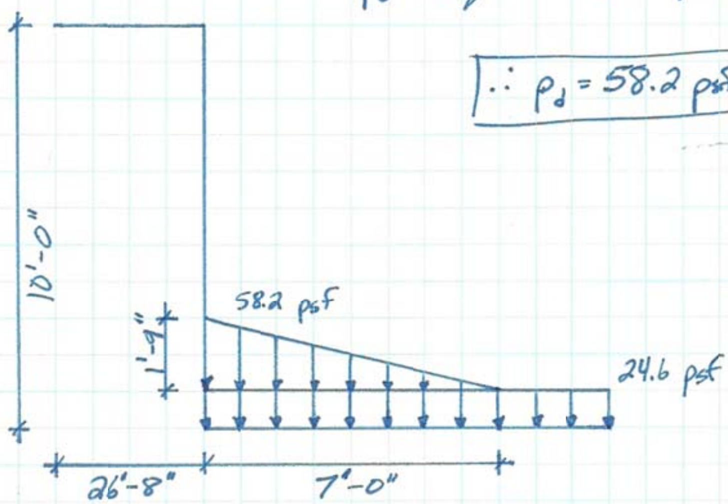
$l_u = 26'-8"$
$h_c = 10'-0"$

$\therefore h_d = 1.75'$

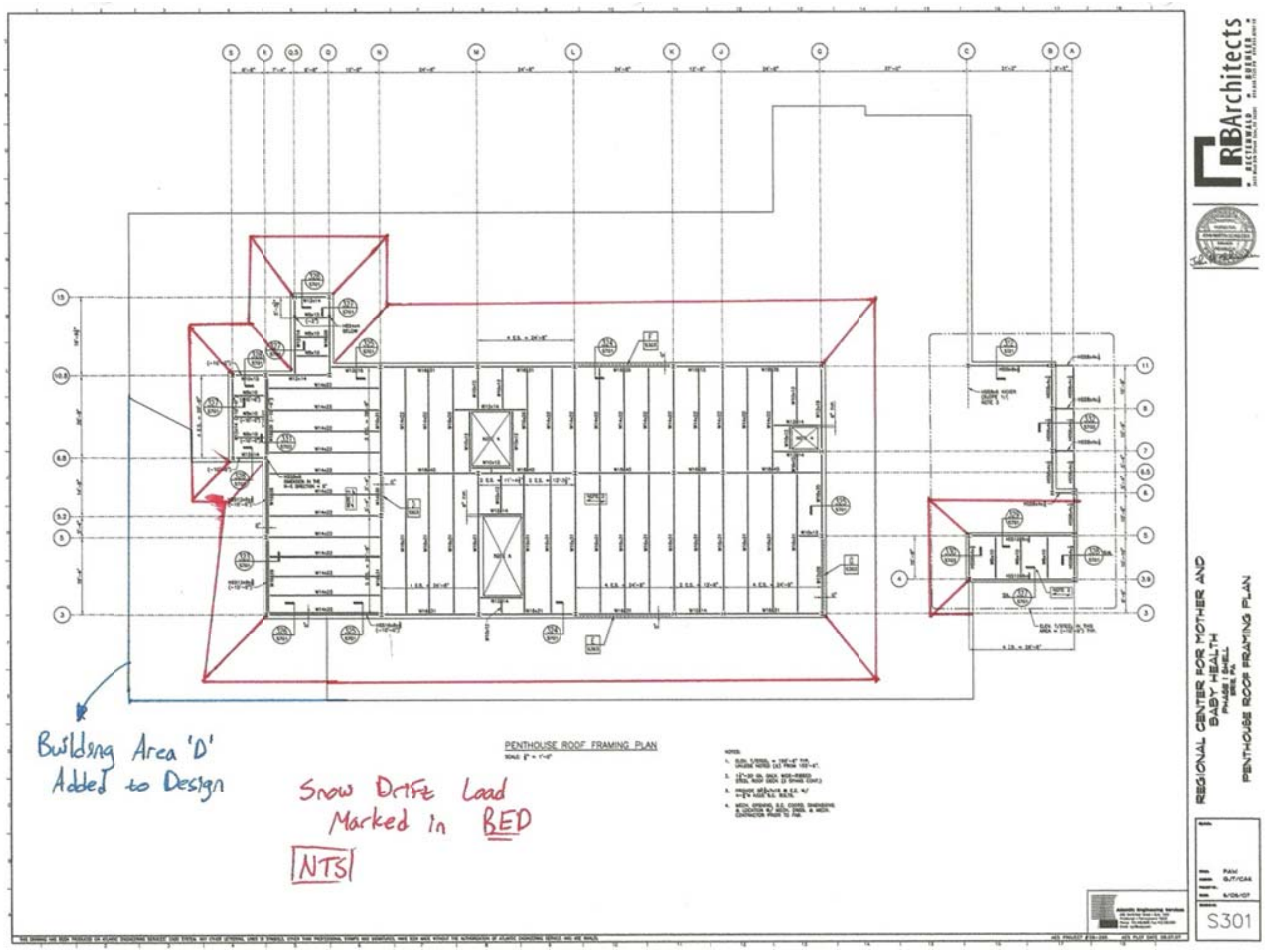
$w = 4 h_d = 4(1.75') = 7'-0"$

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

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



B.4 - Drift Plan



## Appendix C: Wind Load Calculations

C.1 – Wind Calculations

Wind Loads

ASCE 7-05

Method 2 – Analytical Procedure

Assume: Enclosed Building
Rigid Building

Wind From North

$V = 90$ mph → Figure 6-1

$K_d = 0.85$ → Table 6-4

$I = 1.15$ → Table 6-1

Occupancy Category = III → Table 1-1

$K_h$ & $K_z$ → Table 6-3 → Case 2

Surface Roughness D → Exposure D

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

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

C.2 – Wind Calculations (con't)

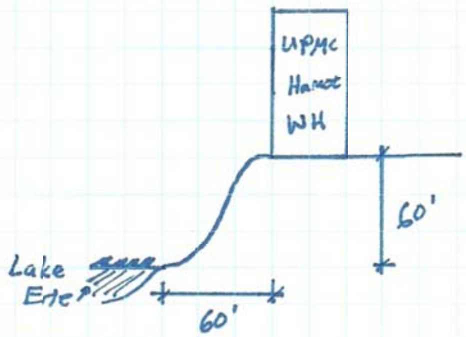
<u>Wind Loads</u> (cont)

$K_{zt} \Rightarrow$ 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
- $H/L_H = 60/60 = 1.0$

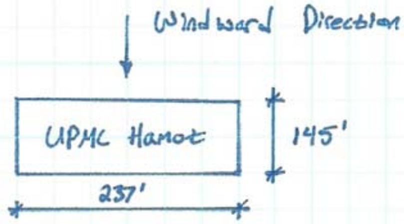
$$K_3 = e^{-\gamma z/L_H} \qquad \gamma = 2.5$$

| | |
|---|---|
| z = 80 | = 0.036 |
| z = 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 |

z = 90 = 0.024

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 $\Rightarrow$ 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 $\Rightarrow$ 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 $\Rightarrow$ 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

$G = 0.85$

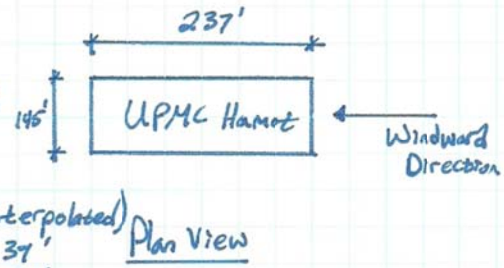
Enclosed Building ⇒ Figure 6-5

$GC_{pi} = \pm 0.18$

C.6 – Wind Calculations (con't)

Wind Loads (con't)

$C_p$ Values $\Rightarrow$ Figure 6-6




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

$L/B = 237/145 = 1.63$

$q_z$ Values $\Rightarrow$ 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
$M_{tot}$ = 40230.8 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 Wind | 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

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

$\therefore C_s = 0.0183$

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

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

D.2 – Seismic Calculations (con't)

EQ Loads (cont)

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

$k = 1.5265 \Rightarrow$ Interpolation

| | | |
|---|---|---|
| PH | $W_{PH} h_{PH}^k$ = | 313,750 |
| SR | $W_{SR} h_{SR}^k$ = | 62,005 |
| R | $W_R h_R^k$ = | 1,105,756 |
| 5 | $W_5 h_5^k$ = | 1,122,849 |
| 4 | $W_4 h_4^k$ = | 757,774 |
| 3 | $W_3 h_3^k$ = | 388,724 |
| 2 | $W_2 h_2^k$ = | 113,976 |
| | | 3,864,834 |

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

D.3 – Seismic Calculations (con't)

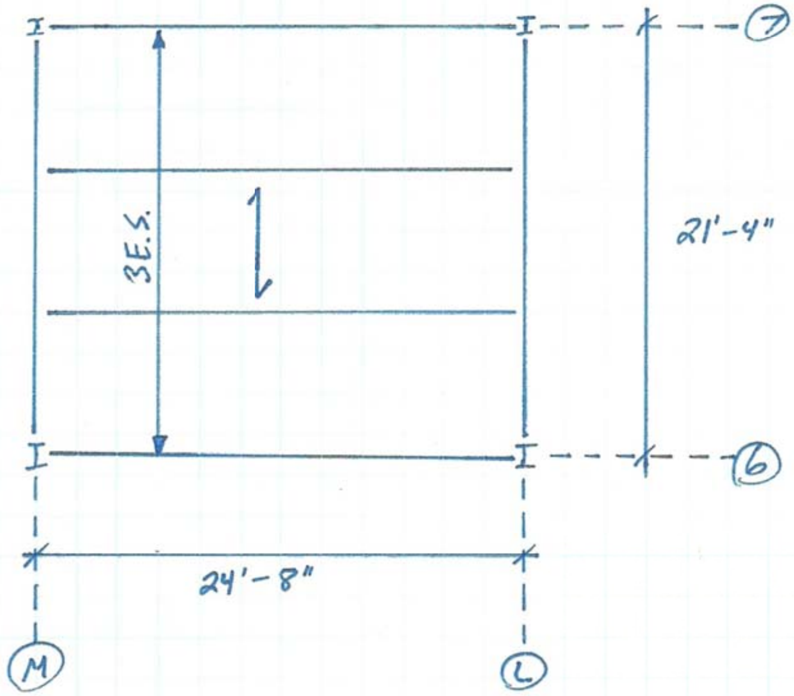
EQ Loads (cont)

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

# 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

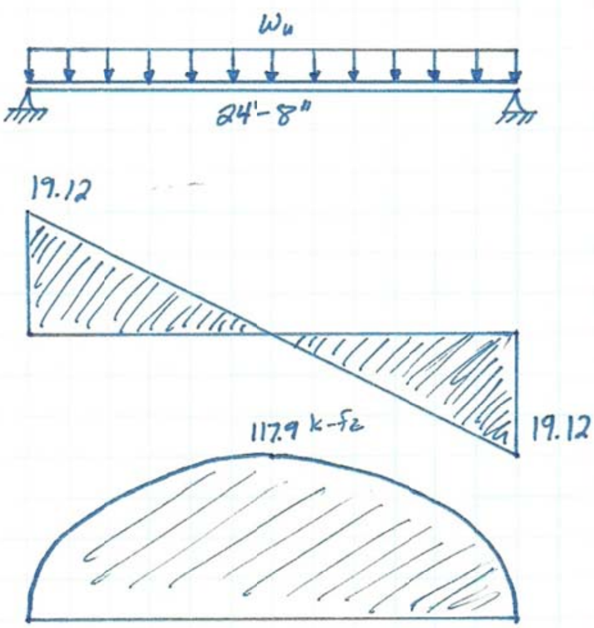
$w = 105$ psf < 278 psf (7'-6" clear; 2VLI) ∴ ok

Beam

Composite Beam: W14x22 [10]

$A_g = 6.49 \text{ in}^2$
$I_x = 199 \text{ in}^2$
$F_y = 50$ ksi
$d = 13.7"$

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




$w_D = 73 \text{ psf} = 524 \text{ plf}$
$w_\ell = 80 \text{ psf} = 574 \text{ plf}$

$w_u = 1.2 w_D + 1.6 w_\ell$

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

$\therefore w_u = 1.55 \text{ ksf}$

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

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

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

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

$\phi M_n > M_u$ $\therefore$ 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)

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

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

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

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

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

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

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

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

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

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

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

Composite ⇒ Concrete on Metal Deck

Beam (con't)

Wet Concrete Deflection

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

$$w = \overset{\text{slab self} + 20}{68}(7.167') + 22\,plf = 0.510\,klf$$

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

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

$$\boxed{W14 \times 22}$$

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

Composite $\Rightarrow$ Concrete on Metal Deck

Girder

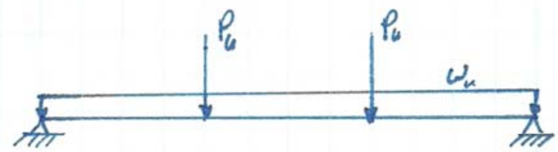
Composite: W16x26 [18]

$I_x = 301\ in^4$

$A_s = 7.68\ in^2$

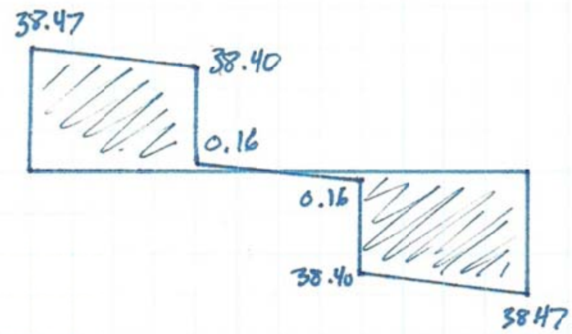
$F_y = 50\ ksi$

$d = 15.7''$




$w_u = 1.2(0.026) = 0.031\ klf$

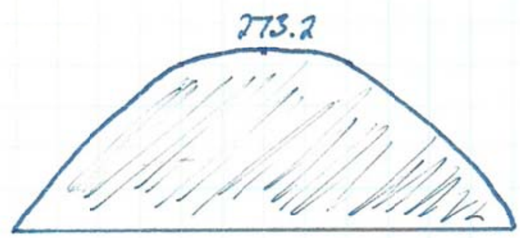
↳ self weight




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

$V_u = 38.47^k$

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




$b_{eff} = \min \left| \begin{array}{l} span/4 = 64'' \Rightarrow \text{controls} \\ spacing = 296'' \end{array} \right.$

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

$\therefore V'_c = 1305.6^k$

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

$\therefore V'_s = 384^k$

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

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

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

Composite → Concrete on Metal Deck

Girder (con't)

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

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

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

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

$\phi V_n = 106^k > V_u = 38.5^k$

↳ Table 3-2 $\therefore$ OK

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

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

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

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

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

$$\Delta_{LL} = \frac{P_{LL}(a)}{24 E I_{LB}}(3\ell^2 - 4a^2)$$
$$= 0.314"$$

$a = 21'\text{-}4"/3 = 85.33"$
$\ell = 21'\text{-}4" = 256"$
$P_{LL} = 14.48^k$

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

$\therefore$ OK

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

Composite ⇒ Concrete on Metal Deck

Girder (con't)

Wet Concrete Deflection

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

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

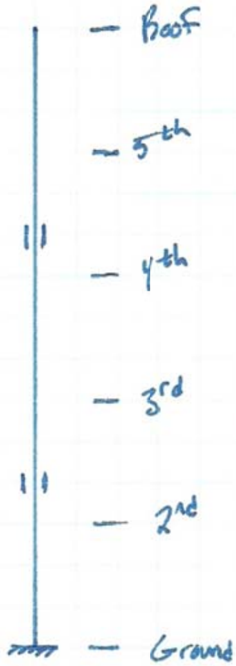
$$\therefore I_{req} = 277.9\,in^4 < I_x = 301\,in^4 \qquad \therefore OK$$

$\boxed{W16 \times 26}$

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

Composite ⇒ Concrete on Metal Deck

Column




= Influence Area

= Tributary Area

Influence Area

$$A_i = (19'-4" + 21'-4")(24'-8" + 24'-8")$$

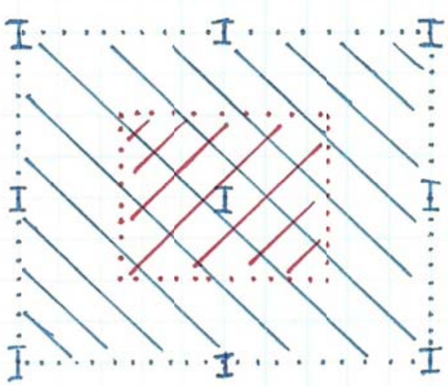
$$\therefore A_i = 2006.2 \text{ ft}^2$$

Tributary Area

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

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

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



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

Composite ⇒ Concrete on Metal Deck

Column (con't)

Loads

Below 5th

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

Below 3rd

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

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

Below 2nd

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

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

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

$P_{u2} = 301.1^k$

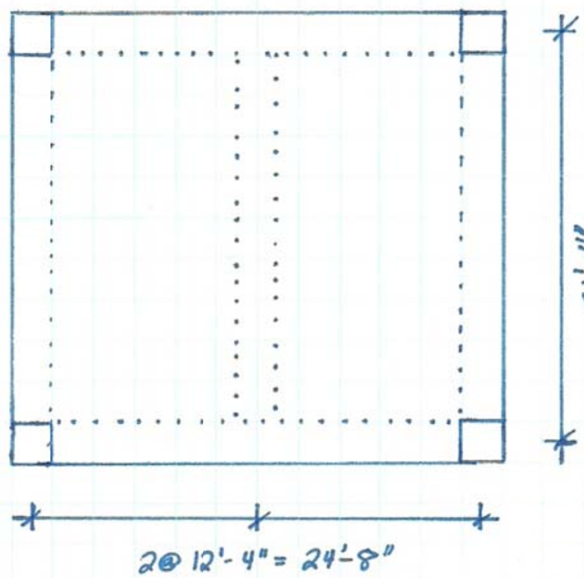
Table 4-1 (Steel Manual): $kL = 12'$

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

$$h = \ell/28 = 12.33(12)/28 = 5.29" \Rightarrow \text{Use } h = 6"$$

↳ Table 9.5a

$DL_{self} = \frac{6"}{12}(150 \text{ pcf}) = 75$ 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)

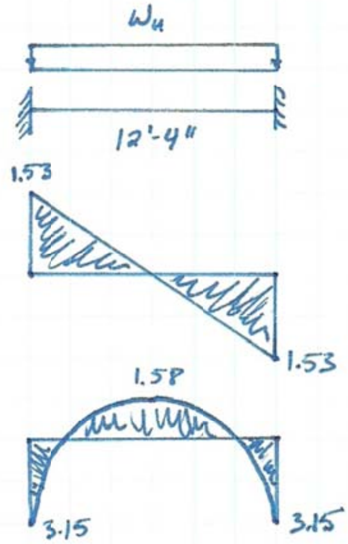
$w_u = 1.2w_D + 1.6w_L = 1.2(100) + 1.6(80)$

$\therefore w_u = 0.248 \text{ klf}$

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

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

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




End Span Reinforcement

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

$A_s = \frac{M_u}{4d} = \frac{3.15(12)}{4(3)} = 0.2625 \text{ in}^2 \Rightarrow \therefore$ Use W10x4 WWF $A_s = 0.30 \text{ in}^2$

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

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

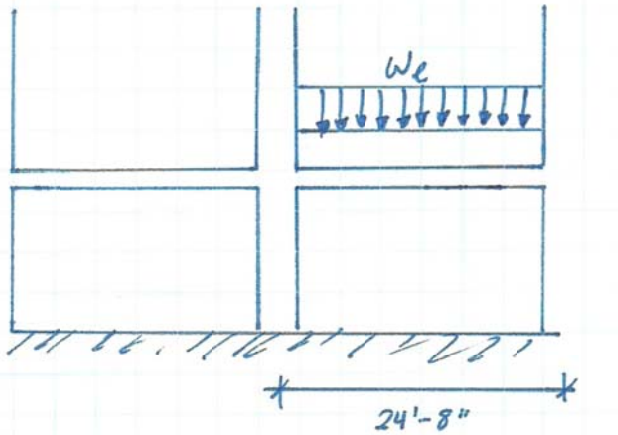
$P_s = 24.64(526.2) = 12.97^k$

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

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

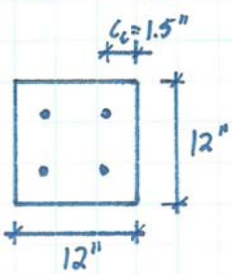
$\therefore P_u = 1.2P_D + 1.6P_L + 0.5P_s$
$= 1.2(263.1) + 1.6(69.62) + 0.5(12.97)$

$\therefore P_u = 433.6^k$




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

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

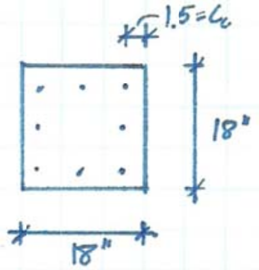
Column

@ Base

$P_u = 433.6^k$

$M_u = 86.53$ ft-k

Try 18" square column




Assume $\gamma = 0.75$

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

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

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

works for $\rho_g = 0.01$

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

$\boxed{\therefore \text{Use } (8) - \#6 \Rightarrow A_s \text{ Shown}}$

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

# One Way Concrete Slab

## Beams

$b = 18''$ ⇒ to match columns

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

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

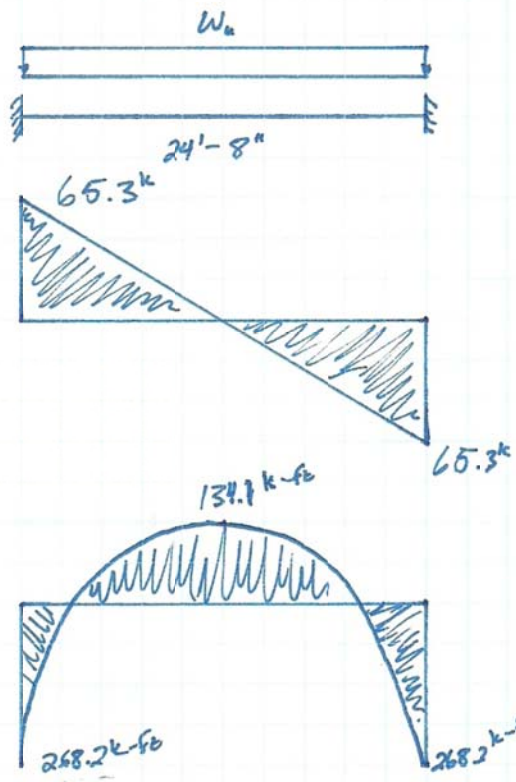
$w_u = 1.2w_D + 1.6w_L = 1.2(2.13) + 1.6(1.71)$

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

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

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

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




$h = 16''$ ⇒ Max depth desired

$h_f = 6''$

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

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

One Way Concrete Slab

Beams (cont)

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

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

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

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

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

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

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

Top Steel

$\phi M_n = 268.2$ ft-k $= 3218.4$ in-k

$$A_s = \frac{M_u}{4d} = \frac{3218.4}{48(14)} = 4.79\ in^2$$

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

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

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

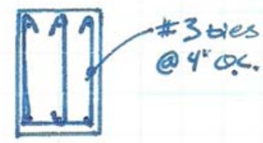
One Way Concrete Slab

Beams (con't)

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 ⇒ $h = 16''$ $b = 18''$

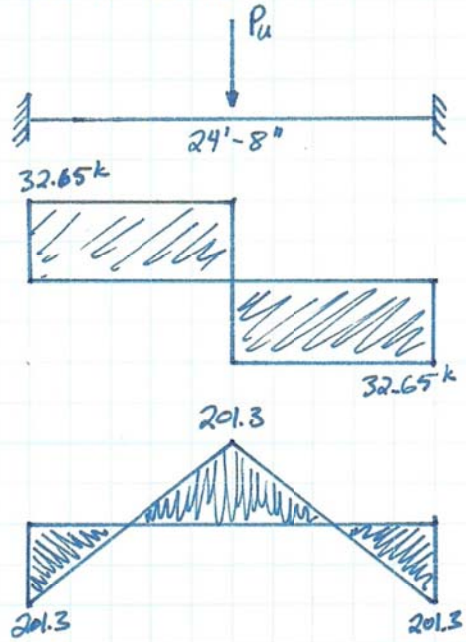
$P_u = 65.3^k$

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

$\therefore V_u = 32.65^k$

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

$\therefore M_u = 201.3$ ft-k




$A_s = \frac{M_u}{4\phi d} = \frac{201.3(12)}{48(14)}$

$\therefore A_s = 3.59 in^2$

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

Spacing Min = 1"

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

$\therefore$ Use (6)-#7 Bars T+B

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

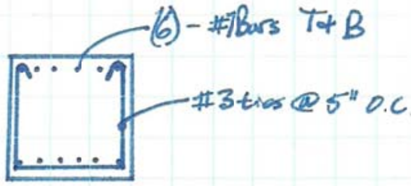
One Way Concrete Slab

Girder (cont)

Shear Reinforcement

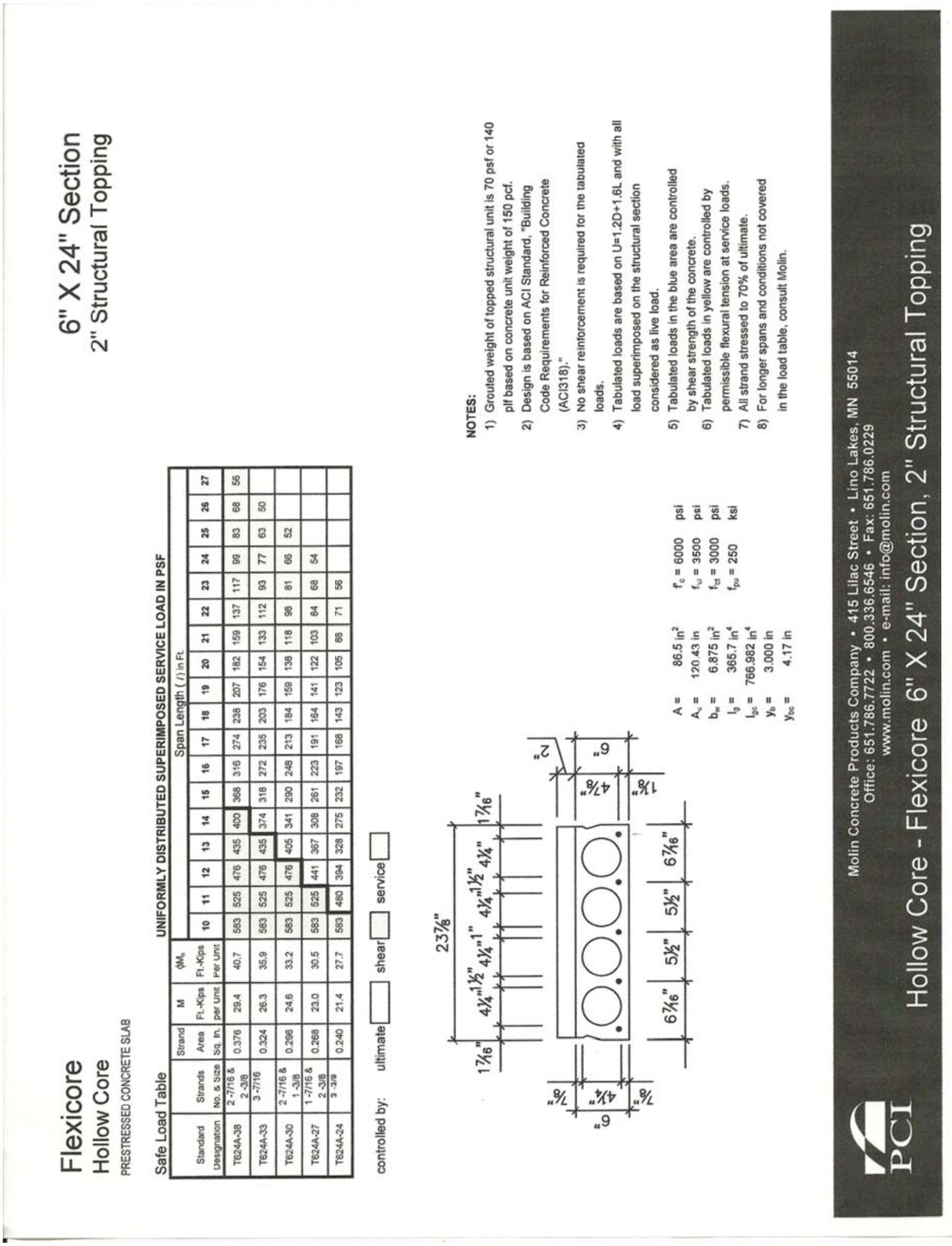
$V_s = 32.65^k = A_v f_v \frac{d}{s} = 2(0.11)(60)\frac{14}{s}$

$\therefore s = 5.66''$ ⇒ 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 | $\phi M_n$ 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
- $\phi$ = strength reduction factor
- $\phi M_n$ = 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 "$\phi M_n$" 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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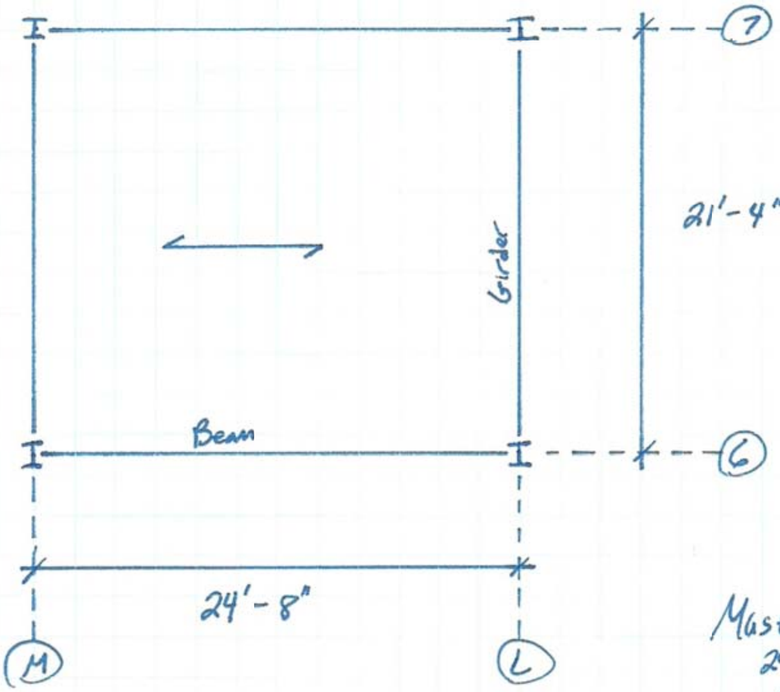
January 2006

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

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

Controlled by Service Limitations

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

# Hollow Core Concrete on Steel Beams

## Beam Design.

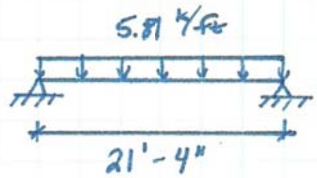
In theory the beams carry no load, therefore many would wonder why they are needed. They serve the primary job of bracing the columns in their weak axis. So the decision was made to use W10x12 beams. Although W8x10's would work for gravity loads, I decided to use my engineering judgement and impose a 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

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

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

$$w_\ell = 1.92\ klf$$




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

$$\therefore V_u = 61.95\ k$$

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

$$\therefore M_u = 330.4\ ft-k$$

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 $> 330.4$ ∴ OK

Shear Check

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

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

LL Deflection

$\Delta_{LL} \leq l/360 = 0.711"$

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

$\Delta_{LL} = 0.388" \leq 0.711"$ ∴ OK

Total Load Deflection

$\Delta_{TL} = l/240 = 1.0667"$

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

$\Delta_{TL} = 1.17" > 1.067"$ ∴ 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)

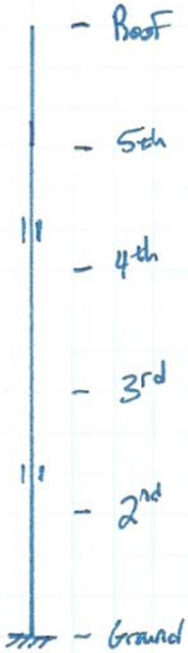
Use W16x67 $\Rightarrow I_x = 954\ in^4 > 875\ in^4$ $\therefore$ OK

Comes From a self imposed depth limit of $\approx 16"$

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

Hollow Core Concrete on Steel Beams

Column Design




Influence Area

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

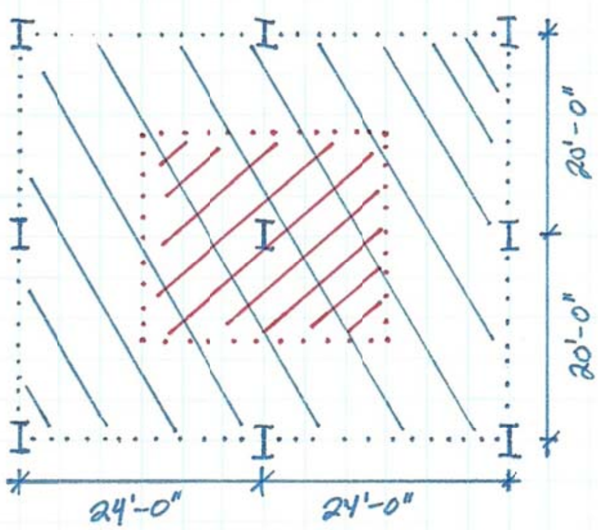
$\therefore A_i = 1920\ ft^2$

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 (cont)

Load Below 5th

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

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

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

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

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

Load Below 3rd

$P_S = 11.83^k$
$P_D = 480[20 + 3(95)] = 146.4^k$
$P_L = 0.448(3)(80)(480) = 51.57^k$

$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 3(480)}} = 0.448$

$P_{u3} = 1.2(146.4) + 1.6(51.57) + 0.5(11.83)^k = 264.1^k$

Load Below 2nd

$P_s = 11.83^k$
$P_D = 480[20 + 4(95)] = 192^k$
$P_L = 0.421(4)(80)(480) = 64.69^k$

$LL_{red} = 0.25 + \frac{15}{\sqrt{4 \times 4(480)}} = 0.421$

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

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

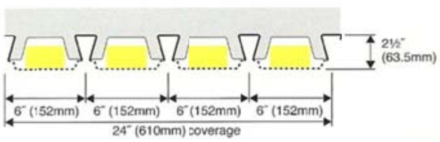
$\phi P_n = 366^k \geq 339.8^k$

# Appendix H: Long Span Deck Calculations

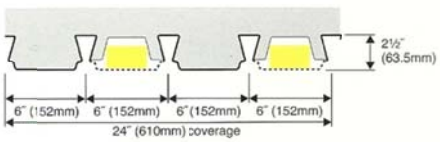
H.1 Long Span Deck Calculations



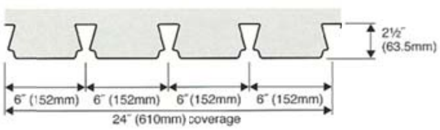
### Toris CA




### 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 | 075 | 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 |

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

11

**Toris CA* & C Composite Floor Deck Systems**

| Concrete | Slab Depth and Weight | Design Thickness (in.) | Maximum Clear Span Without Shoring (ft.-in.) Simple Span | Double Span | Triple Span | Uniform Service Load Slab Capacity, PSF/Spans (C-C of Supports) — Simple Span Condition (see Note 2) 6'0" | 8'0" | 10'0" | 12'0" | 14'0" | 15'0" | 16'0" | 17'0" | 18'0" | 19'0" | 20'0" | Continuous Span Condition Negative Moment Steel Reinforcing REQUIRED (see Note 5) 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.

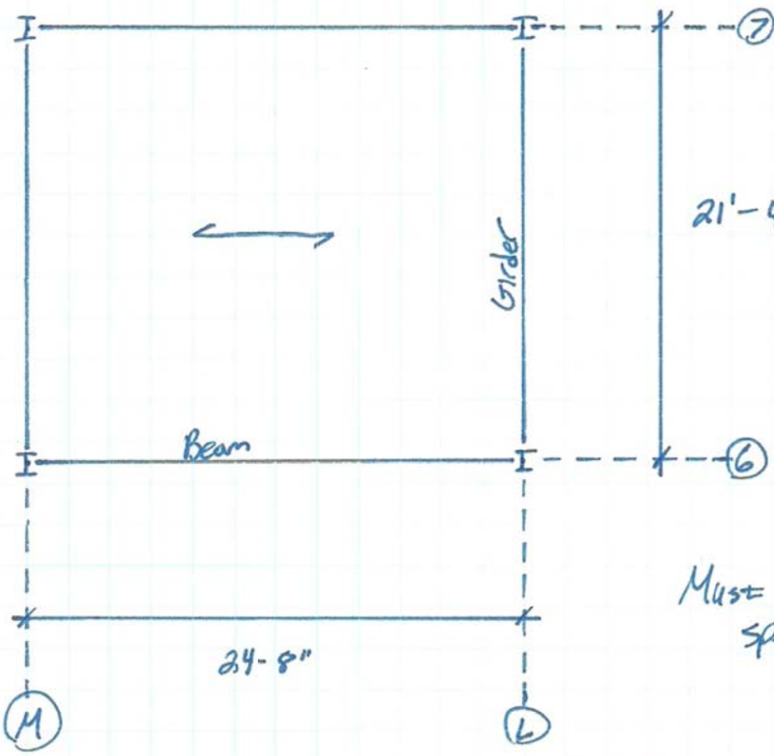
DECK DESIGN AS A FORM:

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.

EPIC METALS CORPORATION

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

Long Span Deck (EPIC Deck) on Steel Beams




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

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

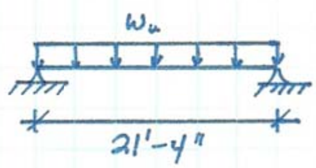
In theory the beams carry no load because the deck runs parallel to the beam, therefore many would wonder why they are needed. They serve the primary job of bracing the columns in their weak axis. So the decision was made to use W10x12 beams. Although W8x10's would work for gravity loads, I decided to 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 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

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

$$w_\ell = 1.92 \text{ klf}$$




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

$$\therefore V_u = 68.1^k$$

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

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

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 $\therefore$ OK

Shear Check

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

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

Live Load Deflection

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

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

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

Total Load Deflection

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

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

$\Delta_{TL} = 1.290'' > 1.067''$ $\therefore$ 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)

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

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

Table 3-3 (Steel Manual)

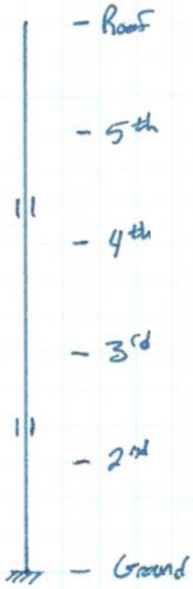
Use W16 x 77 $\Rightarrow I_x = 1110 \text{ in}^4 > 961.2 \text{ in}^4$ $\therefore$ OK

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

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

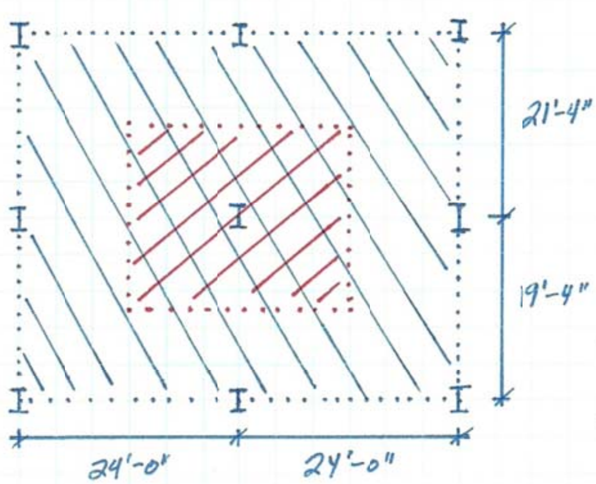
$$\therefore A_t = 488 \text{ ft}^2$$

Influence Area

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

$$\therefore A_i = 1952 \text{ ft}^2$$

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




= Influence Area

= Tributary Area

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

Long Span Deck (EPIC Deck) on Steel Beams

Column Design (con't)

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: Lateral System Analysis and Calculations

I.1 – Stiffness Calculations and Wind Force Distribution

Justin Kovach
AE Senior Thesis 2011-2011

UPMC Hamot Womens Hospital
Erie, PA

Description: Distributes the Lateral Wind Forces to the Applicable Frame

N-S Direction

| Frame | Load (kips) | Δ (inches) | k | $k_{relative}$ |
|---|---|---|---|---|
| B | 100 | 1.499 | 66.7 | 0.217 |
| N | 100 | 0.416 | 240.4 | 0.783 |

| Story | Story Forces | Frame B Forces | Frame N Forces |
|---|---|---|---|
| Roof | 161.64 | 18.23 | 65.69 |
| 5th | 143.57 | 27.93 | 100.64 |
| 4th | 163.80 | 29.27 | 105.48 |
| 3rd | 188.95 | 30.21 | 108.86 |
| 2nd | 225.21 | 25.22 | 90.88 |

E-W Direction

| Frame | Load (kips) | Δ (inches) | k | $k_{relative}$ |
|---|---|---|---|---|
| 1 | 100 | 3.209 | 31.2 | 0.536 |
| 17 | 100 | 3.705 | 27.0 | 0.464 |

| Story | Story Forces | Frame 1 Forces | Frame 17 Forces |
|---|---|---|---|
| Roof | 83.92 | 44.97 | 38.95 |
| 5th | 128.57 | 68.90 | 59.67 |
| 4th | 134.75 | 72.21 | 62.54 |
| 3rd | 139.08 | 74.53 | 64.55 |
| 2nd | 116.11 | 62.22 | 53.89 |

I.2 – Stiffness Calculations and Earthquake Force Distribution

Justin Kovach
AE Senior Thesis 2011-2011

UPMC Hamot Womens Hospital
Erie, PA

Description: Distributes the Lateral EQ Forces to the Applicable Frame

N-S Direction

| Frame | Load (kips) | Δ (inches) | k | $k_{relative}$ |
|---|---|---|---|---|
| B | 100 | 1.499 | 66.7 | 0.217 |
| N | 100 | 0.416 | 240.4 | 0.783 |

| Story | Story Forces | Frame B Forces | Frame N Forces |
|---|---|---|---|
| Roof | 60.77 | 13.20 | 47.57 |
| 5th | 61.71 | 13.41 | 48.30 |
| 4th | 41.64 | 9.05 | 32.59 |
| 3rd | 21.36 | 4.64 | 16.72 |
| 2nd | 6.26 | 1.36 | 4.90 |

E-W Direction

| Frame | Load (kips) | Δ (inches) | k | $k_{relative}$ |
|---|---|---|---|---|
| 1 | 100 | 3.209 | 31.2 | 0.536 |
| 17 | 100 | 3.705 | 27.0 | 0.464 |

| Story | Story Forces | Frame 1 Forces | Frame 17 Forces |
|---|---|---|---|
| Roof | 60.77 | 32.56 | 28.21 |
| 5th | 61.71 | 33.07 | 28.64 |
| 4th | 41.64 | 22.31 | 19.33 |
| 3rd | 21.36 | 11.45 | 9.91 |
| 2nd | 6.26 | 3.35 | 2.91 |

I.3 – Beam Spot Check

Beam

$P_x = 62.38^k$
$M_x = 3.16'^k$
$V_x = 13.88^k$
Envelope (ASD Forces)

W24x62 Beam

Table 3-2 ⇒ Available Shear Strength

$V_n/\Omega_v = 204^k$

Table 3-19 ⇒ Available Flexural Strength, Composite W-Shapes

$M_p/\Omega_b = 382'^k$ ⇒ Conservative

Spec Section E7

Unstiffened Slender Elements ⇒ ∴ $Q_a = 1.0$

$b/t = \frac{7.04}{0.59} = 11.93 \leq 0.56\sqrt{E/F_y} = 0.56\sqrt{\frac{29{,}000}{50}} = 13.49$

∴ $Q_s = 1.0 \Rightarrow Q = Q_s Q_a = 1.0$

$kL_x = 0 \Rightarrow$ Fully Braced $\qquad k_z L = 10'-8"$

$F_e = \left[\frac{\pi^2 E C_w}{(k_z L)^2} + GJ\right]\frac{1}{I_x + I_y} = \left[\frac{\pi^2 (29000)(4606.4)}{(12 \times 10.67)^2} + 11{,}200(1.71)\right] + \frac{1}{1550 + 34.5}$

$C_w = \frac{I_y h_o^2}{4} = \frac{34.5(23.7 - 0.59)^2}{4} = 4606.4$

$\frac{P_r}{P_c} = 0.091 < 0.2$
∴ ↳ Interaction

∴ $F_e = 62.87 \Rightarrow P_n/\Omega_c = F_{cr}A_g = 62.87(18.2) = 1144.3^k/1.67 = 685.2$

$\frac{P_{rx}}{2P_{cx}} + \frac{V_r}{V_c} + \left(\frac{M_{rx}}{M_{cx}} + \cancel{\frac{M_{ry}}{M_{cy}}}\right) \leq 1.0 \Rightarrow \frac{62.38}{2(685.2)} + \frac{13.88}{204} + \frac{3.16}{382} = 0.122 \leq 1.0$

∴ Beam is OK

I.4 – Column Spot Check

Column

$$\left.\begin{array}{l} P_x = 98.51^k \\ M_x = 9.12'^k \\ V_x = 0.70^k \end{array}\right\} \text{Envelope (ASD Forces)}$$

W10x60 ⇒ Column N7

Table 4-1 → Available Strength in Axial Compression

Assume $k = 1.0$ ⇒ Use $kL = 16'$

$P_n/\Omega_c = 352^k$ @ $16' > 98.51^k$ ∴ OK

$P_r/P_c = \frac{98.51}{352} = 0.280 > 0.2$

$$\therefore \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \cancel{\frac{M_{ry}}{M_{cy}}}\right) \leq 1.0$$

Table 3-10 → Available Moment vs. Unbraced Length

$M_n/\Omega_b = 169'^k$ @ $16' > 9.12'^k$

Interaction

$$\frac{98.51}{352} + \frac{8}{9}\left(\frac{9.12}{169}\right) = 0.328 \leq 1.0 \qquad \therefore OK$$

$\boxed{\text{Column is OK}}$

I.5 – Brace Spot Check

Brace

$P_X = 112.98^k$ ⇒ Envelope → Axial Force only in Brace (ASD Force)

HSS 8x8x3/8"

Assume $k = 1.0$ ⟶ Use $kL = 19.23' \cong 20'$

Table 4-4 ⇒ Available Strength in Axial Compression

$P_n/\Omega_c = 191^k$ @ $20' > 112.98^k$ $\therefore$ OK

Table 5-5 ⇒ Available Strength in Axial Tension

$P_n/\Omega_T = 286^k > 112.98^k$ → Tension Yielding $\therefore$ OK

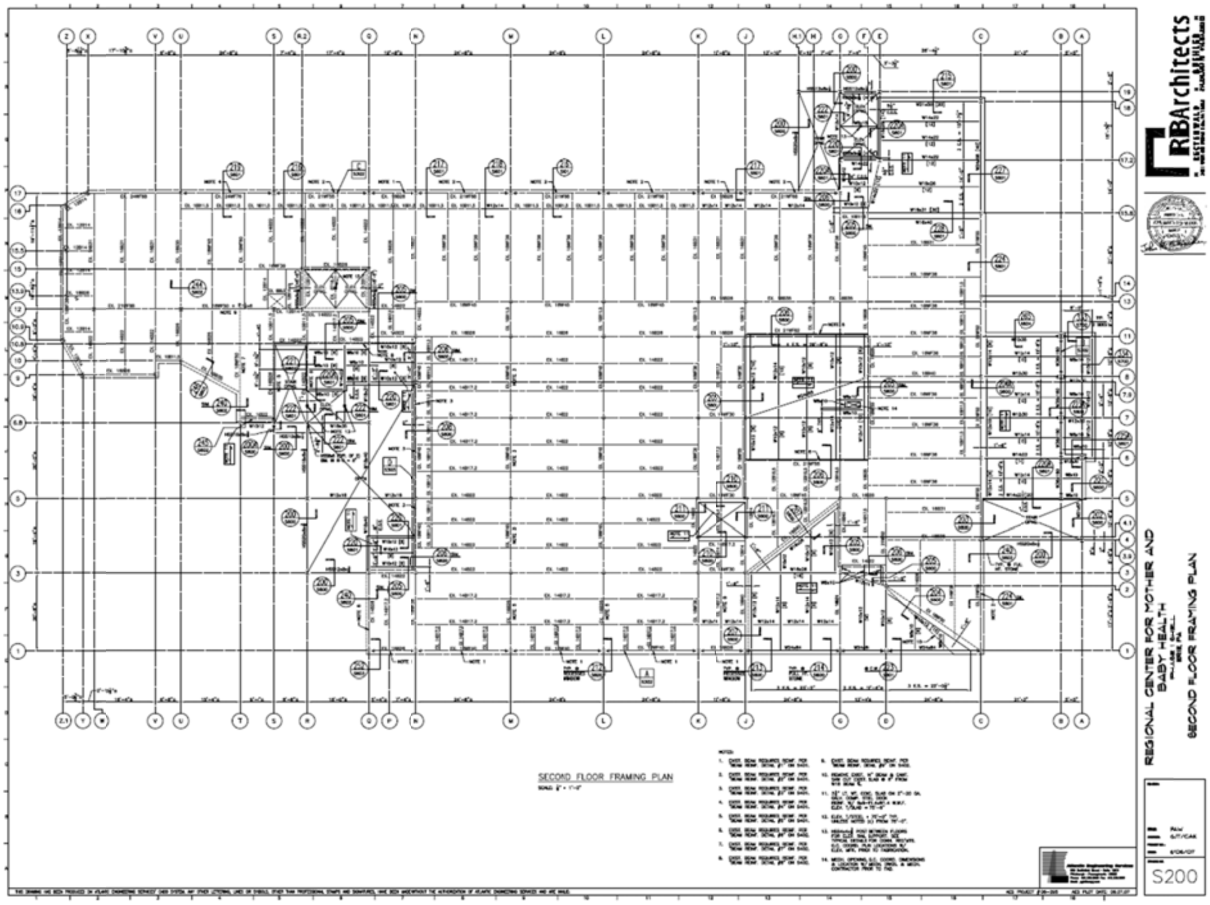
$P_n/\Omega_T = 226^k > 112.98^k$ → Tension Rupture $\therefore$ OK

$\boxed{\therefore \text{Brace is OK}}$

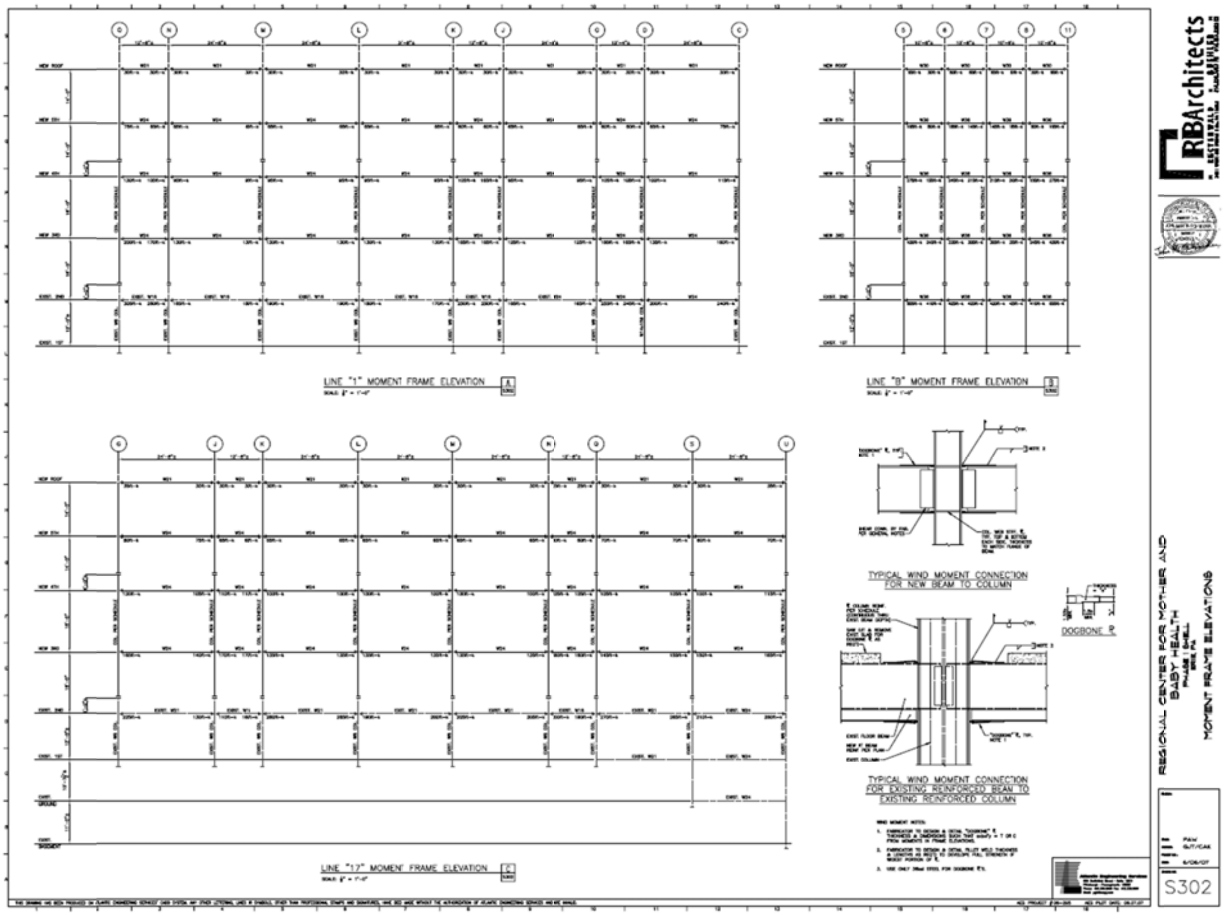
Drawings Indicate the brace has a capacity of $160^k$ T or C, which may be the controlling limit state of the connection or that number is conservative for compression strength

## Appendix J: Relevant Building Plans

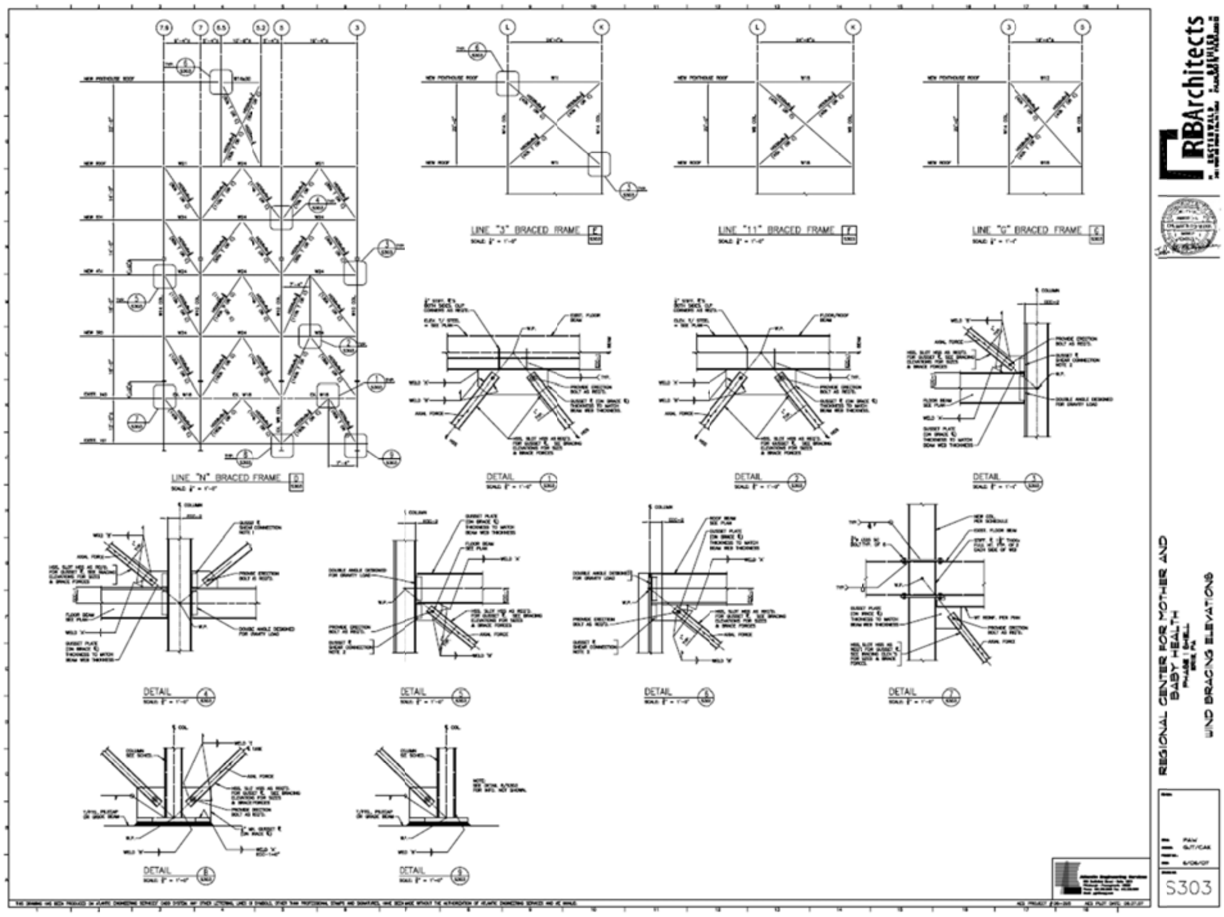
J.1 – S200 - Second Floor Structural Plan



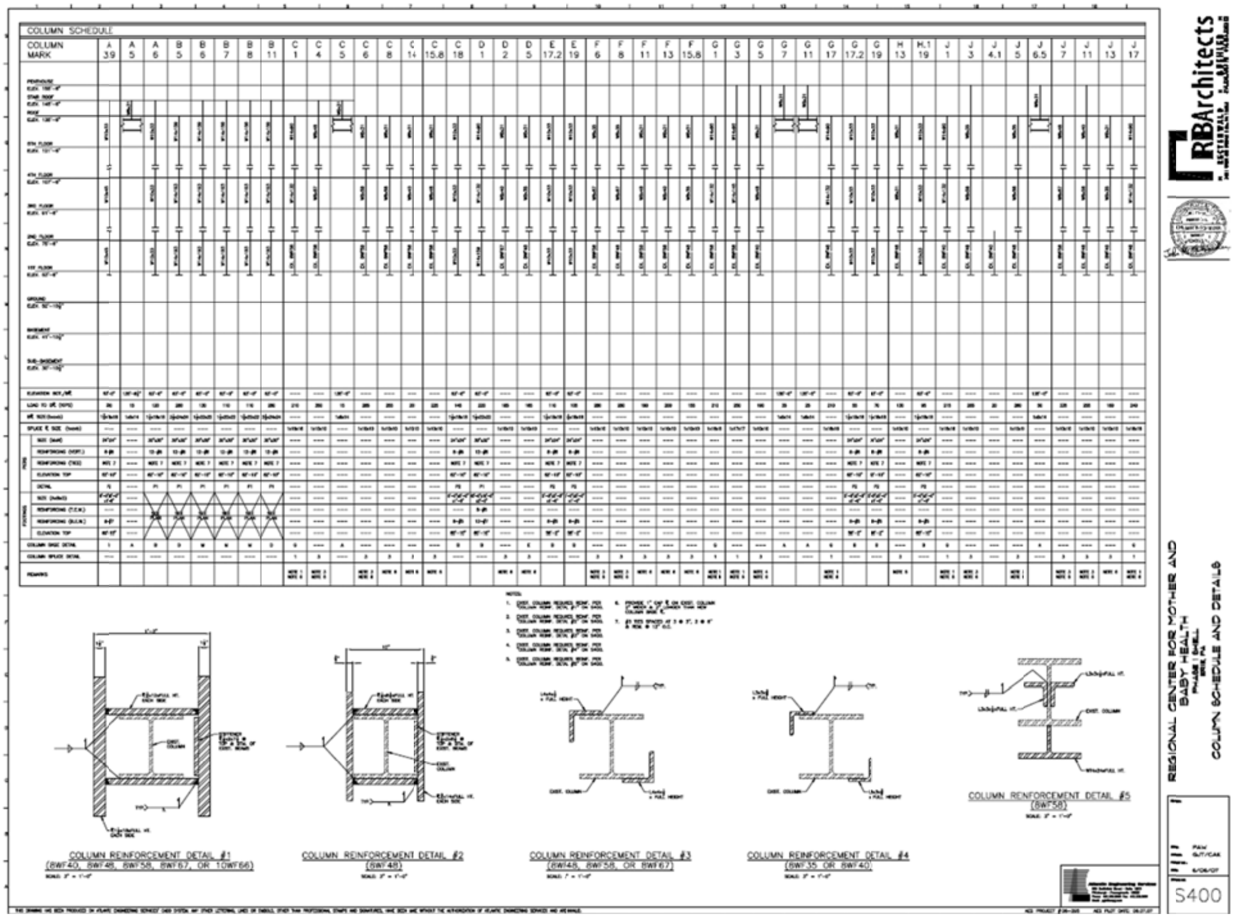
J.2 – S302 - Moment Frame Elevations



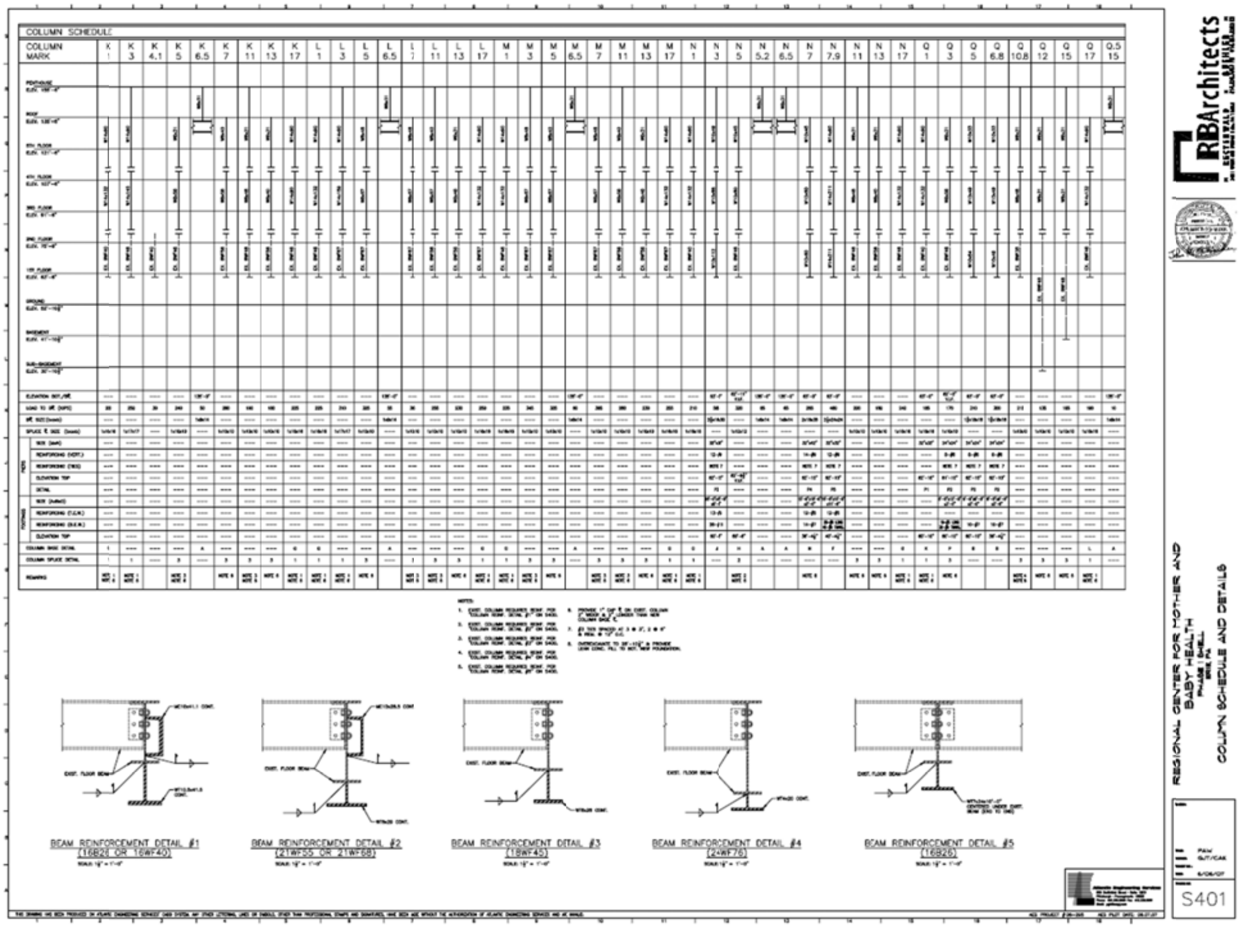
J.3 – S303 - Braced Frame Elevations



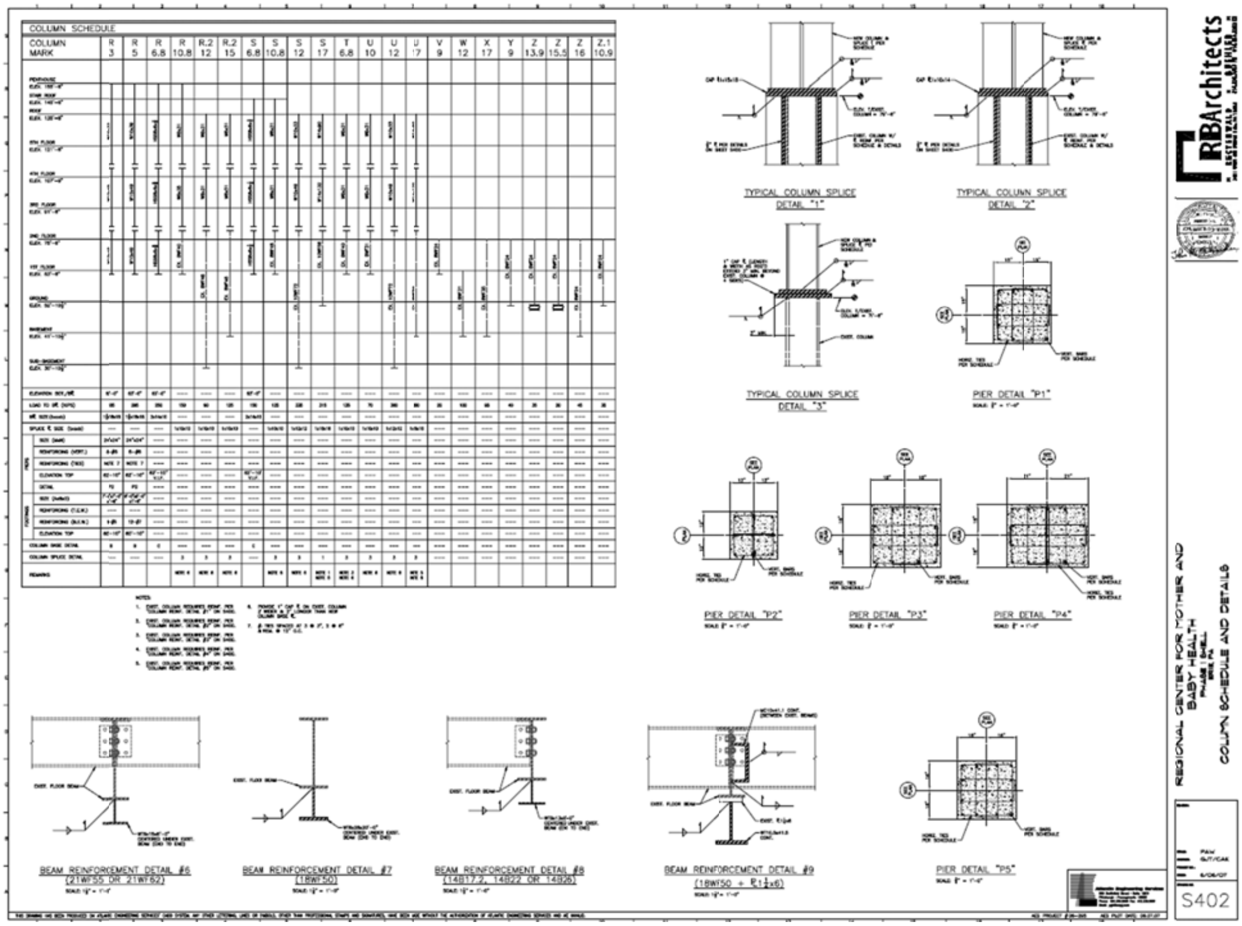
J.4 – S400 - Column Schedule and Typical Column Reinforcements



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



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



J.7 – S403 - Foundation Overbuilds