Final Thesis Report

2011-2012 AE Senior Thesis

4/5/2012

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Dr. Boothby – Advisor

UPMC Hamot Womens Hospital
201 State Street
Erie, PA 16550

Building Stats

Occupancy Type:	Healthcare/Hospital
Building Size:	163,616 sq. ft.
Number of Stories:	5 (Above Grade)/7(Total)
Construction Dates:	January 2007-January 2011

Project Team

Owner:
UPMC Hamot

Architect:
Rectenwald Architects Inc.

GC/CM:
Perry Construction Group

Structural Engineer:
Atlantic Engineering Services

MEP Engineer:
CJL Engineering

Civil Engineer:
Urban Engineers Inc.

General

*Originally Designed for 4 stories
*Only 2 stories were built
*Instead of adding 2 more stories the deicision was made to strip the building to the shell and build the 5 story building from the existing structure.

Architecture

*Designed to match the adjacent buildings on the UPMC Hamot Campus
*Intended to make the patient feel like they are at home

Structural

*N-S Direction
-49' Braced Frame along N
-43' Moment Frame along B
*E-W Direction
-161' Moment Frame along 1
-173' Moment Frame along 17

Mechanical

*3 AHU Units located in the Mechanical Penthouse
- 1 unit provides conditioned air to floors 1 and 2
- 1 unit provides conditioned air to floors 3 thru 5
- 1 unit provides conditioned air to the OR Suite

*All units equipped with a humidifier and a dehumidifier, along with a variable frequency drive fan

Lighting/Electrical

*2000 KVA Transformer (3 phase, 3 wire)
*6 distribution panels serving 75 electrical panels
*The lighting design was intended to minimize direct light in the babies eyes at all times

Construction

*The construction of UPMC Hamot has a unique history, originally designed as a 4 story building, only 2 stories were built. Later the decision was made to strip those stories down to the shell and build 5 stories, reinforcing beams, columns and footings below (as needed).
*The fit-out phase of the project was conducted from the top floor down, to minimize disruption to the hospital.

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Executive Summary

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

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

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

The structural depth for this thesis was split into three distinct investigations. An investigation on the new building code with a comparison to the previous edition and how it affects the structural weight and performance was done and proved that the loads did increase; although this primarily came from a change in occupancy category. An investigation into the possibility of effectively utilizing braced frames rather than moment frames will be completed. Finally, an investigation into a complete building redesign was done to determine if selective deconstruction of the building was the correct decision to be made by the construction team.

As these elements were completed two breadth studies were undertaken. An architectural breadth was done, which analyzed the impact on the architecture that the braced frame system has on the building. This analysis yielded several concerns, not just on the views that would be potentially ruined by the framing members, but also raised several health concerns for the patients of the hospital. A construction management breadth was also done to analyze the impact of not using the existing structure and grid to build from. This analysis showed that the contractors decision to use selective deconstruction rather than implosion was likely comparable when analyzing cost, this decision and the impacts on the construction schedule and the subsequent impact on the potential revenue from completing the building earlier yielded a drastic improvement in cost-schedule analysis, thus the analysis shows that the building should have been imploded and started again from scratch.

Acknowledgements

Throughout this process I have learned several things. Below I have summarized just a few of these things.

1. Check and Double Check before proceeding
2. If you cannot come within 10% of the computer model solution on pen and paper, then you shouldn't be using the computer
3. Building design and construction requires a team, as did this thesis
4. Regardless of what happened today, the sun will undoubtedly come up tomorrow

The report enclosed and the work completed over this year long project could not have been completed by me. I would like to thank the following people for their loving support, professional support, and mentorship that many of them have taken.

My Lord and Savior Jesus Christ

Without the faith I have in God and Jesus Christ I would not be the man that I am today. As I continue to grow in my faith daily, I intend to use the talents the Lord has blessed me with to glorify his name for many years to come. A special thank you is owed to my rock and my provider.

My Family

Without the love and support of my family there is no doubt in my mind that I would not have exceled within the Architectural Engineering program, because of their constant encouragement I have been able to keep my eye on the prize and continue to work hard day in and day out. For this I am eternally grateful!

Atlantic Engineering Services

I would like to thank Atlantic Engineering Services for providing me with this thesis project and the professional contacts required to receive the information necessary to complete the project. I would like to also thank all of the employees (engineers and drafters) for their mentoring through the several months of internship experience that I was privileged to complete. In particular I would like to thank Gil Taylor for his mentorship throughout my internship and this thesis project. His work and encouragement helped me beyond belief throughout all the stages of this project.

Penn State Faculty

I would like to take this time to thank Dr. Boothby and Dr. Hanagan for their guidance and support throughout the thesis process. Both were instrumental in answering all of my questions pertaining to structural engineering and the thesis process.

Introduction

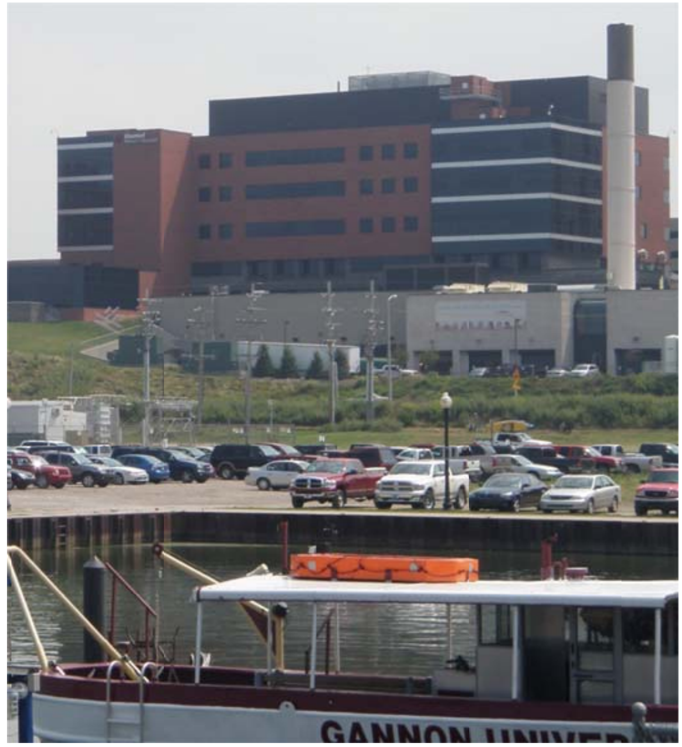


Figure 1: North Façade, Showing 2-D Escarpment

Located on the shoreline of Lake Erie, 201 State Street, which will be referred to as UPMC Hamot Womens 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. Womens Hospital was completed in early January of 2011. This structure has a very unique history; originally the hospital wanted a four story building, but only had the financing for two levels. Thus the structure was designed for four stories, but only the first two were constructed. Then the hospital decided that a five story structure better suited their needs, so the building was stripped down to the shell (structural steel and floor slabs), the current roof slab was then removed, with the columns being truncated 4’-0” above the second story slab. The decision was made to reinforce the columns and beams below this point, as needed, and to build to the desired five stories above.

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

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


Figure 2: Interior Water Wall

The UPMC Hamot Women’s Hospital was designed to match the architectural style of the other buildings on the Hamot Medical Center campus. This includes a brick and glass façade that is intended to allow sufficient amounts of natural light into the building without being uncomfortable to the patients. The interior of the building was constructed to a very luxurious standard. The owner of the building was not primarily concerned about cost, but rather wanted the building to put the patients at ease by making them feel as if they were at home. This is primarily achieved through earth tone colors throughout the interior the water wall located in the lobby and the cabinets in every room to hide the hoses and cables that are typical of a hospital room, 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

Existing Structural System

• Foundation

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

Figure 4: Foundation Excavation during Construction

• Floor Construction

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

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

• Lateral System

The lateral system in the N-S direction consists of a 5 story (6 with mechanical penthouse), 49' long braced frame along column line N, this is the only full height braced frame in the building. The N-S direction also has a full height 42'-8" long moment frame along column line B. In the E-W direction full height moment frames are utilized along column line 1 and 17, which are 161' and 173'-4" long, respectively. The columns are spliced 4'-0" above the second floor, where the existing shell remained and was reinforced below. The columns are also spliced at above the 4th floor, at the same 4'-0" above the slab. 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, found in Appendix L. The column sizes vary from W8 sizes to W14 sizes. The lateral system of the mechanical penthouse is entirely braced frames.

Proposal Objective

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

Through the discoveries of these various Technical Reports and background knowledge of the building history various aspects of this project shall be analyzed. First a comparison of building codes (ASCE 7-05 vs. ASCE 7-10) will be done with special care being taken to analyze how the changes to the wind loading sections of the code affect this and other structures. This will be done for two reasons. Primarily I feel that the new version of the code altered the occupancy category classifications, such that this building would change occupancy categories and thus be subject to a different loading. The other reason being that the student will be designing based on the new code upon graduation, so a more thorough investigation would be beneficial to the educational process. Secondly the existing moment frames will be redesigned as braced frames, with special care being taken to incorporate them with the current architectural theme, or conceal them within the structure as needed. This will be done using the loads determined through the use of ASCE 7-05 to allow for an equivalent comparison to the lateral system that is being utilized in the existing building. Adding braced frames where the current lateral system is located may prove to be difficult, although to move the frames could allow for the structure to be hidden as the architect requested. To conclude the depth portion of this report, an analysis will be done to incorporate concrete shear walls around the vertical circulation elements of the building. This will be done in an attempt completely remove steel lateral frames. Then an analysis will be done to examine how a complete demolition of the existing facility could have affected the structure. Constraints will be imposed to maintain the same building footprint and room areas, etc. This will allow for fewer construction cost variables and a more accurate final assessment. Obviously these alterations will affect other aspects of the building. For example placing braced frames inside of a wall will require a wider wall system and possible relocation of doors. These issues will be dealt with through various breadth topics that will be included within this report.

Structural Depth

- ## ASCE 7-05 vs. ASCE 7-10 Comparisons

Overview

Recently the American Society of Civil Engineers (ASCE) came out with a new copy of the "Minimum Design Loads for Buildings and Other Structures". This standard will eventually become referenced by the International Building Code (IBC) and then that will be adopted by local jurisdictions. Once adopted by the local jurisdiction the code and its contents become law, with some possible local considerations. Since the new design code is ready to be adopted it is imperative for the engineers to use this code in their designs. After learning the old code (ASCE 7-05) throughout my academic career, the importance of learning the differences between ASCE 7-05 and the newer code (ASCE 7-10) is important for my engineering career. Thus a complete comparison between the codes will be attempted, including Live Load, Snow Load, Wind Load, and Earthquake Loads. Dead Loads will be omitted because they will never change; these loads are a property of the material and are not varying from code to code. The major implication to the loads may be effected by the unique change in the occupancy category that occurs between these design codes. ASCE 7-05 would classify this building at an occupancy category of III under the "Buildings or other structures that represent a substantial hazard to human life in the event of failure, including Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities". These specific facilities are located in the main hospital and accessed through a tunnel. ASCE 7-10 changes the way the occupancy categories are listed making occupancy category IV contain all "Buildings or other structures as essential facilities.

Analysis

- **Live Loads**

Live loads were analyzed for both ASCE 7-05 and ASCE 7-10 design standards. This simply consisted of finding the design loads in the live load tables in both ASCE standards. The applicable loads for this project are found below and were found to be unaffected in comparing codes.

Load Type	ASCE 7-05	ASCE 7-10
Live Load	(psf)	(psf)
Lobbies	100	100
Operating Rooms/Labs	60	60
Patient Rooms	40	40
Corridors, above first floor	80	80
First Floor Corridors	100	100
Offices	50	50
Stairs	100	100
Mechanical Space	150	150
Roofs	20	20

Table 1: Design Live Load Comparison Chart

- **Snow Loads**

The city of Erie, PA falls in an area that ASCE requires a case study in order to determine the ground snow load. This case study is often not completed by the engineer for every project, but rather the local jurisdiction mandates the ground snow load for all buildings within that jurisdiction. Thus a phone call was made to Scott Heitzenrater on 8/31/2011. This call led to the allocation of the local amendments to the building code, which mandates a 40 psf ground snow load for both ASCE 7-05 and ASCE 7-10.

Design Parameter	ASCE 7-05	ASCE 7-10
Snow Load		
Ground Snow Load	40 psf	40 psf
Occupancy Category	III	IV
Importance Factor	1.1	1.2
Thermal Factor	1.0	1.0
Exposure Factor	0.8	0.8
Flat Roof Snow Load	24.64 psf	26.88 psf

Table 2: Design Snow Load Comparison Chart

As can be seen in the table above, the flat roof snow load has increased in from ASCE 7-05 to ASCE 7-10. This is due to the change in the occupancy category that this building falls under. This change only occurs due to the unique category that this building falls under, as discussed in the overview. Details for this analysis can be found in Appendix B.

- **Wind Loads**

Wind loads were calculated based on design standards ASCE 7-05 and ASCE 7-10. The wind section of ASCE was completely revamped in the 2010 standard; thus making some things non-comparable. The percent difference was then compared in a third column for several of the relevant parameters. This was done in an attempt to compare the data. Although this needs to be further analyzed with the changes in the load combinations that accompany the new standard.

Design Parameter	ASCE 7-05	ASCE 7-10	% Difference
Wind Load			
Design Wind Speed (mph)	90	120	
Occupancy Category	III	IV	
Importance Factor	1.15	N/A	
Exposure Category	D	D	
Enclosure Classification	Enclosed	Enclosed	
Internal Pressure Coefficient	+/- 0.18	+/- 0.18	
Gust Factor	0.85	0.85	
Cp value, windward/leeward	0.8/-0.5	0.8/-0.5	
p15, N-S Wind (psf)	59.51	92.00	154.6%
p92, N-S Wind (psf)	25.71	39.74	154.6%
Base Shear, N-S Wind (kips)	1040.3	1688.5	162.3%
p15, E-W Wind (psf)	19.20	29.67	154.5%
p92, E-W Wind (psf)	24.58	37.99	154.6%
Base Shear, E-W Wind (kips)	435.9	730.9	167.7%
Load Combination Factor	1.6	1.0	-62.5%

Table 3: Design Wind Load Comparison Chart

The use of the new wind load analysis appears to be similar, with the use of the appropriate load combination factors listed in Chapter 2 of both ASCE standards. After analyzing combinations with base shears and wind pressures the load increases roughly 5% for the worst case scenarios. This increase is not applicable to the change in occupancy category, because the design wind speed of 120 mph would not change between occupancy category III or IV in ASCE 7-10. Thus the debate between practicing engineers and researchers will likely grow as to what warranted this 5% increase. Details for this analysis can be found in Appendix C.

- **Earthquake Loads**

Earthquake loads were calculated based on design standards ASCE 7-05 and ASCE 7-10. The earthquake loads are summarized in the table below.

Design Parameter	ASCE 7-05	ASCE 7-10
Earthquake Load		
R-Value	3	3
Occupancy Category	III	IV
Importance Factor	1.25	1.5
S_{DS}	0.175	0.165
S_{D1}	0.078	0.085
C_S	0.0183	0.024
Building Weight, W (kips)	11,606	11,606
Base Shear, V (kips)	212.4	278.5

Table 4: Design Earthquake Load Comparison Chart

The determination of these loads has yielded an increase in load of approximately 31.1% this is directly related to the change in the occupancy category and the corresponding importance factor. This factor is then used in the calculation of C_S; which directly influences the base shear when maintaining the same building seismic weight. Details for this analysis can be found in Appendix D.

Conclusions

After analyzing the live loads, snow loads, wind loads, and earthquake loads for this structure it is apparent that the major changes in the load calculations is due to the redefinition of the occupancy category. This change only affects this structure based on the lack of an emergency and surgery room. For a typical building the live, snow, and earthquake loads would remain the same. The wind loads would increase by approximately 5%, regardless of the change in occupancy category. This is obviously more conservative although it would likely yield higher steel weights in the lateral frames.

• Moment Frame Analysis and Design

Overview

A steel moment frame design was done to examine building efficiency. The key difference between this design and the design implemented by the design team is the steel grade of the lower levels. When the structure was reused the existing steel grade (A36) was utilized. So in this redesign, the higher steel grade (A992) would replace the older steel. This efficiency will then be compared to that of the existing moment frames, based on several key factors; such as, cost, schedule, architectural impacts, and constructability. Moment frames have many advantages and disadvantages. The major advantage is the open floor plan that this system will provide, but with that advantage comes some key disadvantages, such as, large member sizes to increase stiffness and limit drift, as well as, increased connection costs to resist moment. Below you will find a floor plan showing where the moment frames will be utilized.

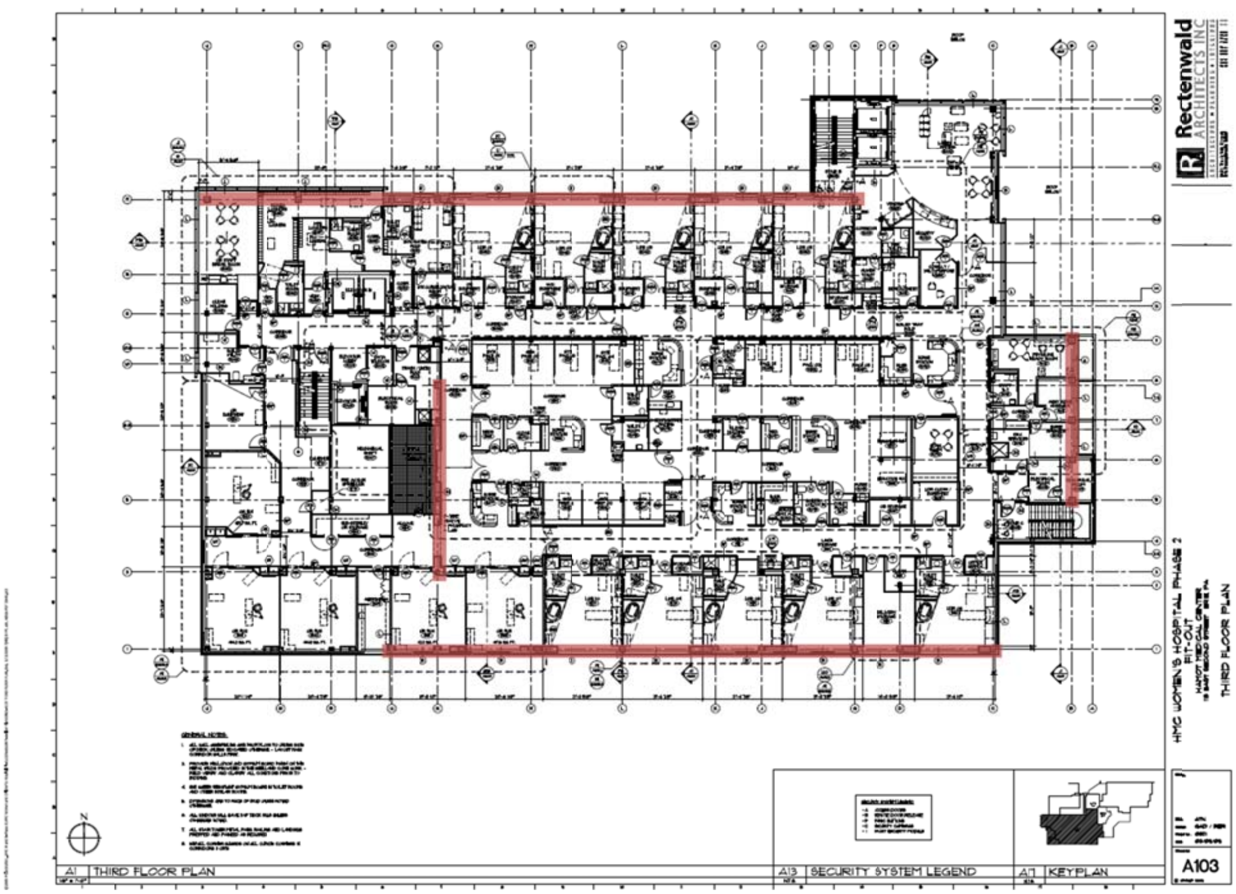


Figure 5: Moment Frame Layout

Member Design

Member designs were primarily completed using the RAM Structural System software package. The program was primarily used to determine lateral loads on the frames, which is a valid method as verified by hand in Technical Report III. The applied loads were identical to those found in the aforementioned report, which is expected because nothing has changed. Thus wind controls base shear and overturning moment in both the x and the y directions. These designs were then checked using a 2-D STAAD model and various hand calculations. For constructability and uniformity, the members were designed using the "worst case" scenarios, such as designing all the lateral beams and columns on a floor level to be one size. This uniformity helps ensure that it will be built in the field the same way that it is designed and detailed by the engineer.

Impact on Foundations

Since the lateral system has not been changed there will likely be minimal impacts on the foundations of the building. The aforementioned footing excavation and reinforcement would likely still need to be completed with this method of construction, but almost all of the affected lateral footings will be located on the perimeter of the building, thus yielding to an easier excavation and reinforcement.

Moment Frame Conclusions

This analysis has shown the feasibility of steel moment frames as a lateral system for the UPMC Hamot Womens Hospital. The moment frames benefit the structure by allowing a nice open environment within the hospital, which was originally desired by the architect. A complete analysis of the impacts to both the architecture and the cost and schedule will be completed within the breadths to come, which will allow a more thorough understanding of the effectiveness of this lateral system.

- ## Braced Frame Analysis and Design

Overview

A steel braced frame design was done to examine building efficiency. This efficiency will be compared to that of the existing moment frames, based on several key factors; such as, cost, schedule, architectural impacts, and constructability. Braced frames have many advantages and disadvantages over the moment frames that currently exist in the UPMC Hamot Womens Hospital. The advantages of the braced frames include increased stiffness (assuming column and beam sizes don't change), which under the same loading will decrease the frame deflection substantially. Since the current moment frames are relatively long in length (in order to minimize deflection), the new braced frames could become much shorter, have smaller member columns and beams, and will potentially have cheaper connections. The connections could be cheaper than the moment frame connections because a shorter frame will require fewer connections. The major disadvantage of the braced frame is that the interior and exterior architecture will almost undoubtedly be changed; this was not desired by the building architect. In order for this to be a feasible system the isolation of the braced frames into areas that minimize architectural disruptions is critical. Below you will find a floor plan showing where the braced frames will be utilized.

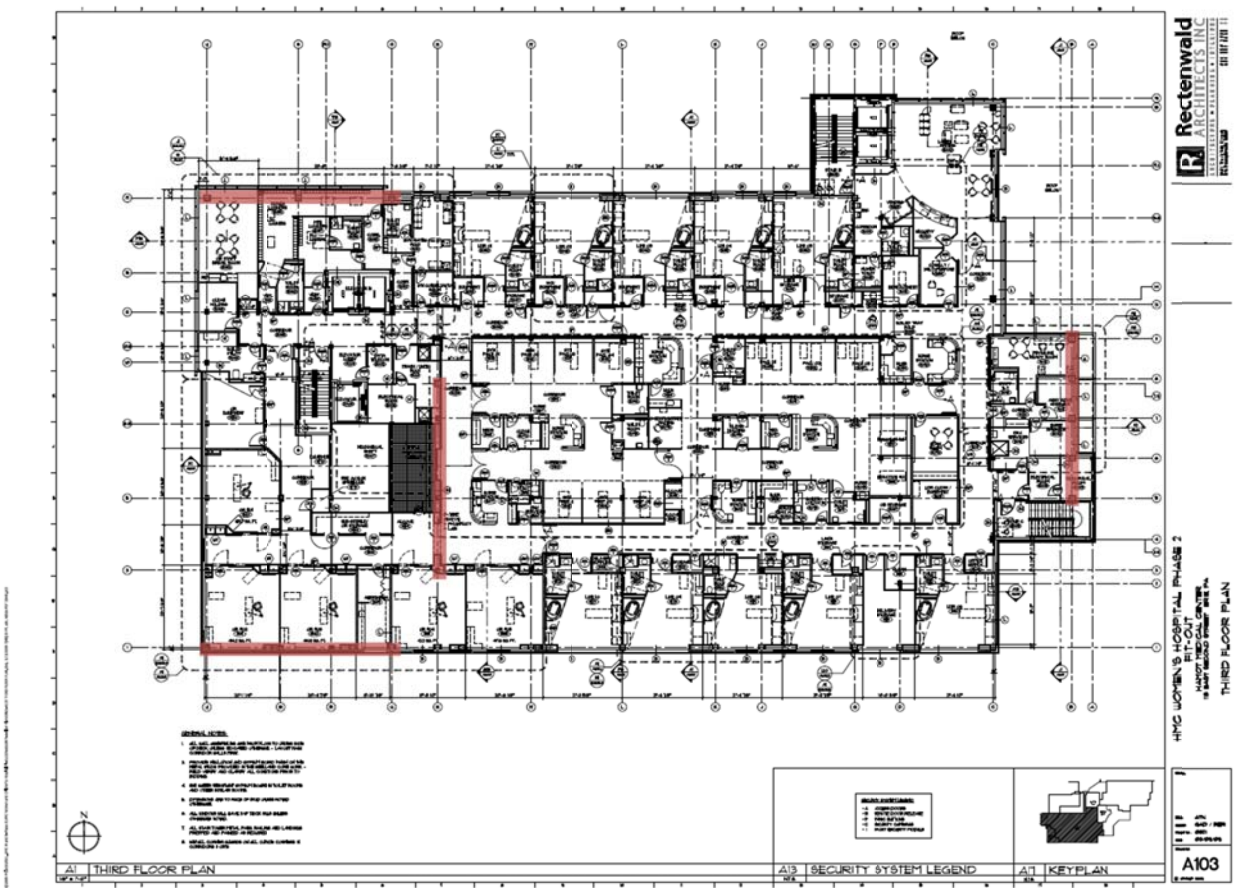


Figure 6: Braced Frame Layout

Member Design

Member designs were primarily completed using the RAM Structural System software package. The program was primarily used to determine lateral loads on the frames, which is a valid method as verified by hand in Technical Report III. The applied loads were identical to those found in the aforementioned report, which is expected because the building wind and seismic factors have not changed. Thus wind controls base shear and overturning moment in both the x and the y directions. These designs were then checked using a 2-D STAAD model and various hand calculations. For constructability and uniformity, the members were designed using the "worst case" scenarios, such as designing all the bracing members on a floor to be one size. This uniformity helps ensure that it will be built in the field the same way that it is designed and detailed by the engineer.

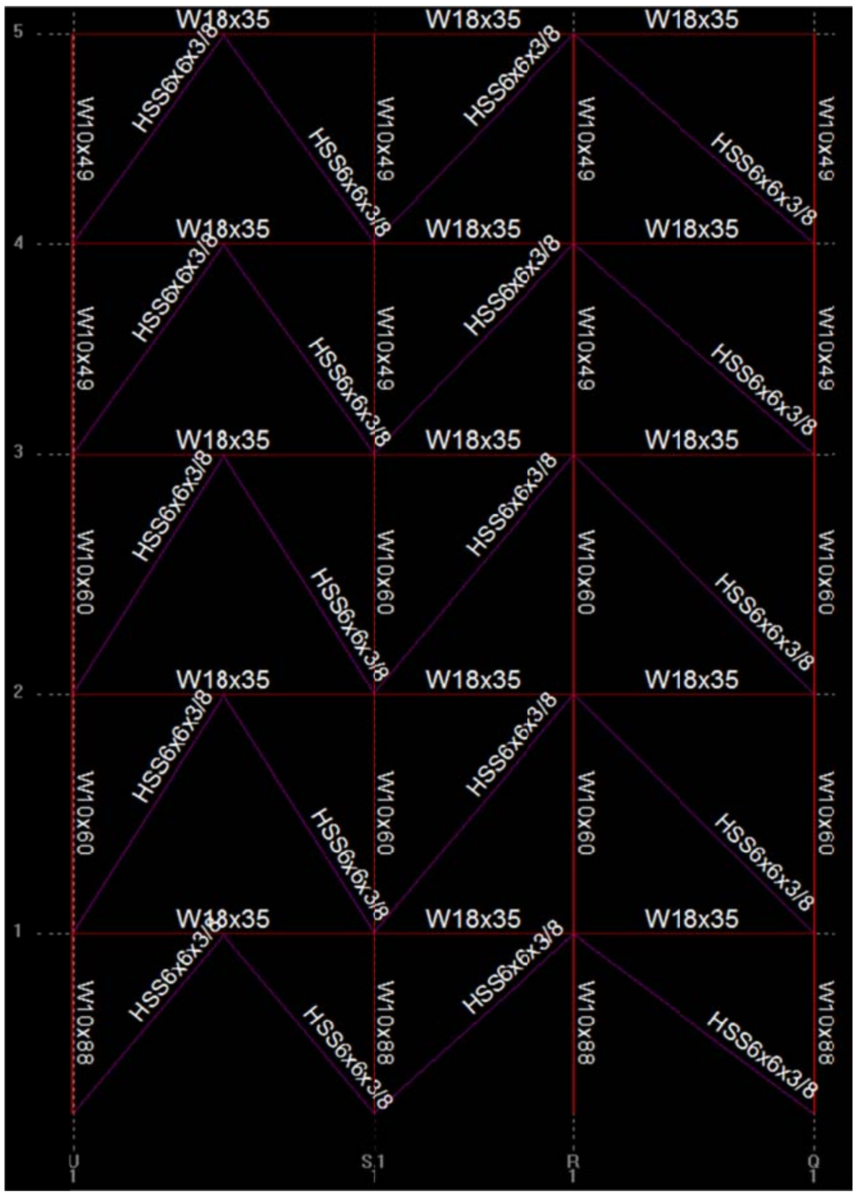


Figure 7: Typical Braced Frame Member Sizes

Detailing

Connection detailing was done as a comparison to the existing moment frames. For a detailed comparison please see the "MAE Course Related Study" section enclosed within this report. The connection highlighted below in Figure 8, was the connection of interest due to maximum tensile force in brace the entering that joint. The calculations have been summarized by the corresponding detail entitled Figure 9, also below. This connection was not "seismically detailed" due to the chosen R-Value of 3, for a "Steel Structure NOT Specifically Detailed for Seismic". This is acceptable per ASCE 7-05 code criterion. The details for the loading can be found in Appendix E and the detailed hand calculations for the connection can be found in Appendix K.

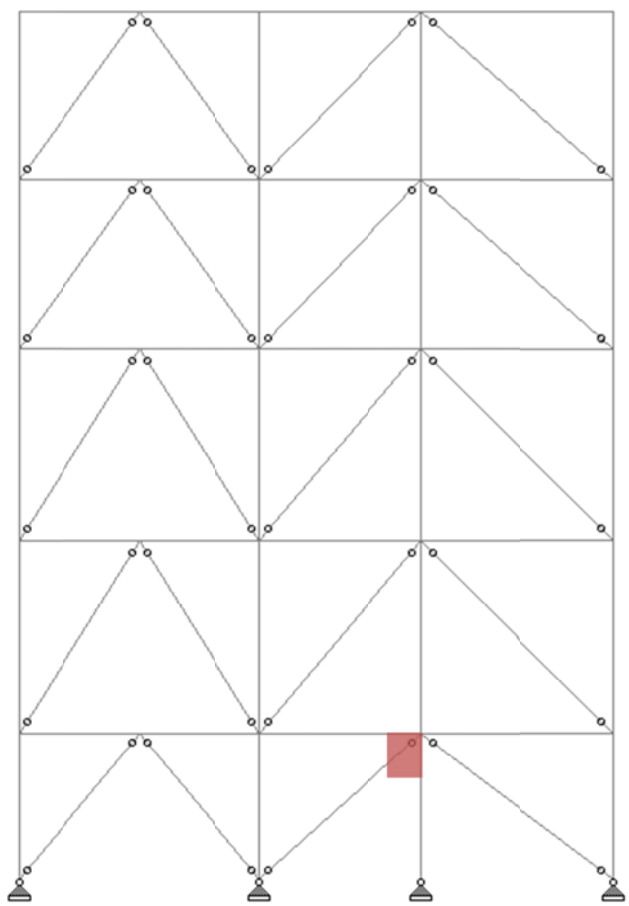


Figure 8: Brace Frame along CL 1

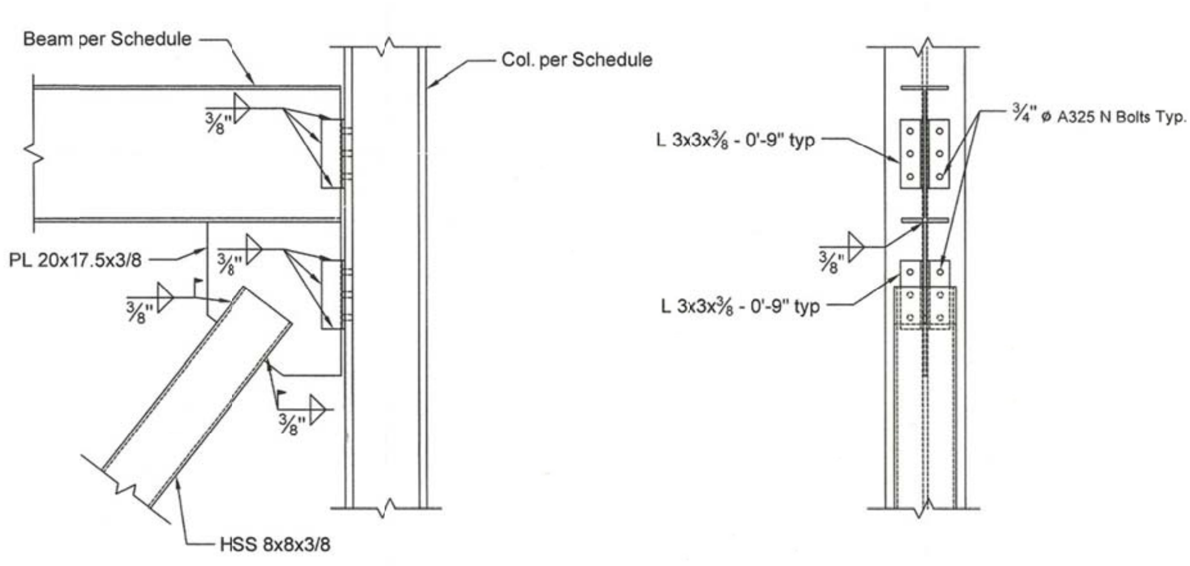


Figure 9: Typical Braced Frame Connection Design

Impact on Foundations

Since the lateral system has been changed there will be an undoubted impact on the foundations of the building. Although the locations of some of the aforementioned spread footing excavation and reinforcement will still likely need to be completed, the locations that this will be required with be minimized and likely not in the same locations. These locations will be moved more to the exterior corners of the building, which will likely aid the excavation and reinforcement that would be required. This was done in an attempt to improve construction cost and schedule, as well as addressing constructability issues.

Braced Frame Conclusions

This analysis has shown the feasibility of steel braced frames as a lateral system for the UPMC Hamot Womens Hospital. The braced frames benefit the structure by decreasing number of connections and thus the welding time and materials associated with the moment connections that were designed and implemented on this project. The reduction in length should also decrease the foundation materials throughout the building. Even the best implementation of the braced frames location cannot completely eliminate braced frames existing within patient rooms, which was the major reason of rejection from the architect. A complete analysis of the impacts to both the architecture and the cost and schedule will be completed within the breadths to come, which will allow a more thorough understanding of the effectiveness of this lateral system.

- ## Concrete Shear Wall Analysis and Design

Overview

A concrete shear wall design was done to examine building efficiency. This efficiency will be compared to that of the existing moment frames, based on several key factors; such as, cost, schedule, architectural impacts, and constructability. Concrete shear walls have many advantages and disadvantages over the moment frames that currently exist in the UPMC Hamot Womens Hospital. The advantages of the concrete shear walls include increased stiffness, which under the same loading will decrease the frame deflection substantially. Since the current moment frames are relatively long in length (in order to minimize deflection), the new shear walls eliminate the large steel lateral members and can hopefully be hidden within the vertical circulation elements (i.e. stairwells, elevator shafts, and mechanical shafts); thus allowing the open floor plan that the architect desires. The major disadvantage of the concrete shear walls is a constructability concern. Typically steel frames can be greatly prefabricated and thus the field installment time is drastically minimized. With concrete shear walls a rebar cage must be constructed, then the forms must be placed around the cage and the concrete poured. Finally the forms cannot be removed for several days, at which point you would move up one floor and start the process again. In order for this to be a feasible system the concrete shear wall must be able to save enough in construction costs to offset the potentially longer construction schedule, while still maintaining the code mandated strength and serviceability requirements. Figure 10 below shows in a floor plan where the concrete shear walls will be utilized.

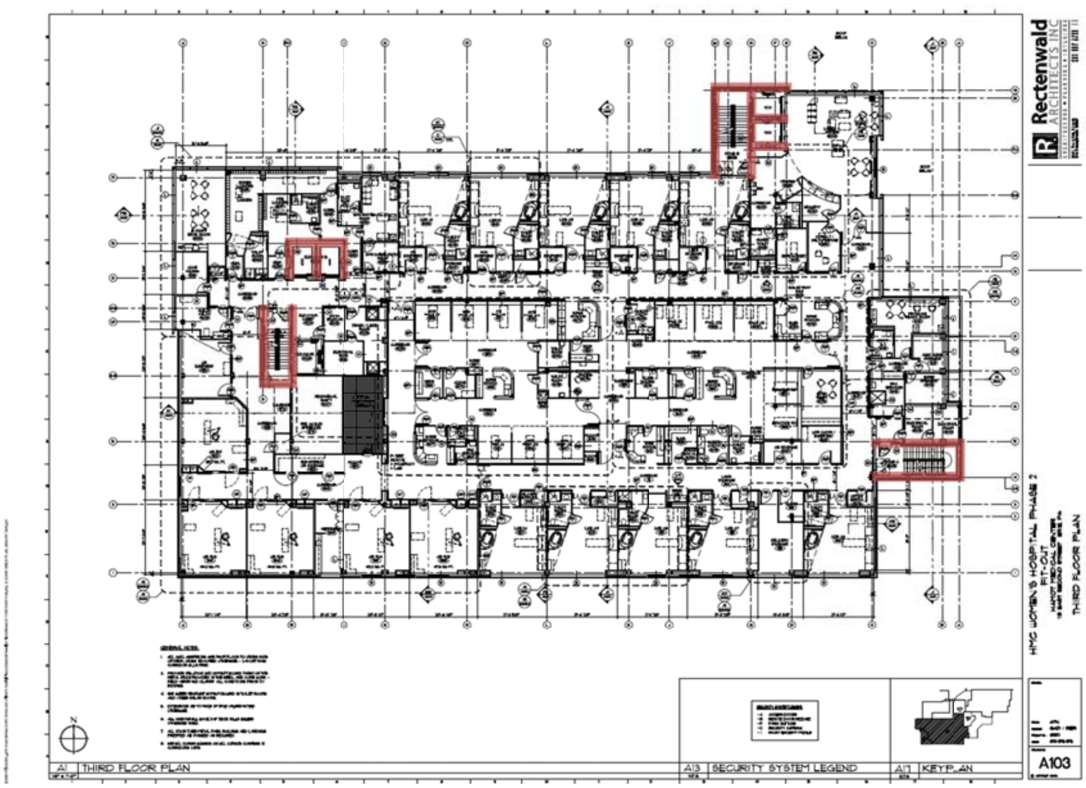


Figure 10: Concrete Shear Wall Layout

Wall Design and Detailing

In order to design the concrete shear walls one must first determine the applicable loads to design for. Modeling the structure in RAM Structural System allows for a complete and thorough load analysis to be completed for both the gravity and lateral systems. This was verified to be an acceptable load calculator with detailed analysis done through Technical Report III, the loads for this analysis were also verified based on expected engineer's judgment (wind loads should remain the same, while earthquake loads increase due to an increase in building weight and change in R-Value). In the steel building wind controlled both the x and y directions of the building for overturning and base shear, although increasing the structural weight increased the earthquake loading, thus wind loads only controls in the x-direction. This is also expected due to the large 2-D escarpment present along the north face of the building. Once the controlling load cases were determined, the center of mass and center of rigidity were determined, the center of mass was assumed to be at the centroid of the floor mass and the center of rigidity was determined based on all the walls having the same rigidity based on length. Both of these assumptions are deemed reasonable and are often done in practice, especially if the wall width is constant throughout the building. Then the loads were applied through the center of rigidity and torsional shear was calculated for each wall. Controlling loads were then derived for each wall based on both load cases. The wall with the most base shear per length was then designed for each direction. This ensures that the wall thickness will be adequate for the assumptions made when calculating the center of rigidity. These calculations can be found in Appendix F.

Finally the controlling walls were designed by hand in each direction. The walls were designed for lap splices at the 4th story, which will allow for the rebar size and number of bars to be reduced according to a much lower load seen on the upper stories. Below you will find the detailed walls; the supporting calculations can also be found in Appendix F. After an appropriate thickness for the walls was determined the anticipated deflection was also computed and found to be well within code limits.

N-S Wall

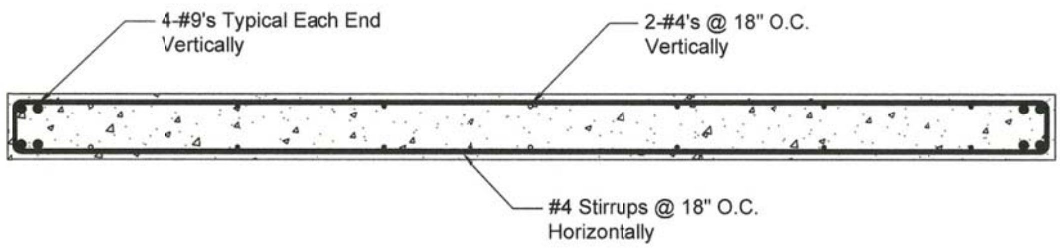


Figure 11: Wall #8 @ 4th Floor

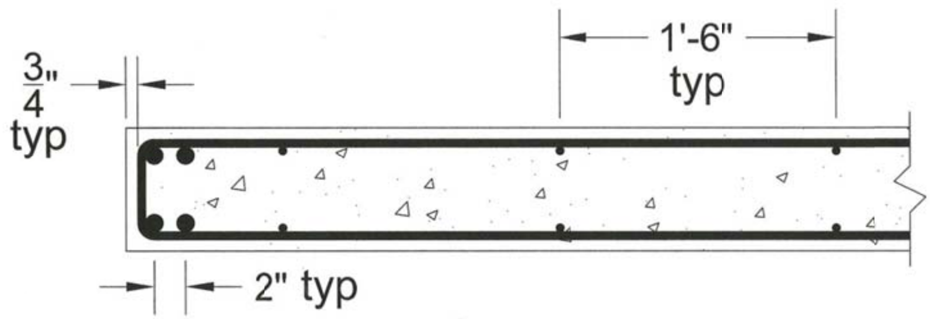


Figure 12: Wall #8 @ 4th Detail

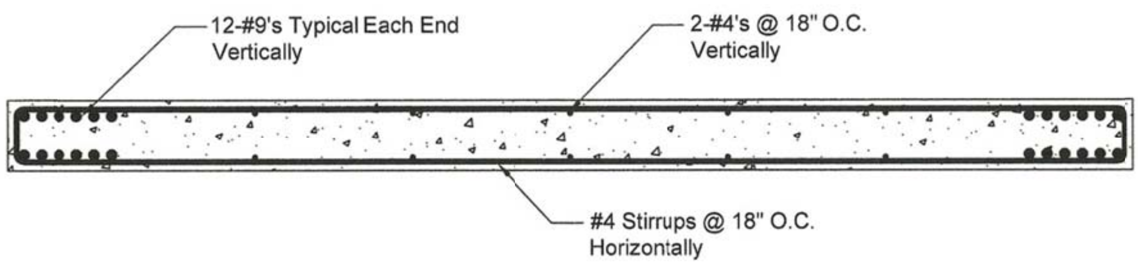


Figure 13: Wall #8 @ Base

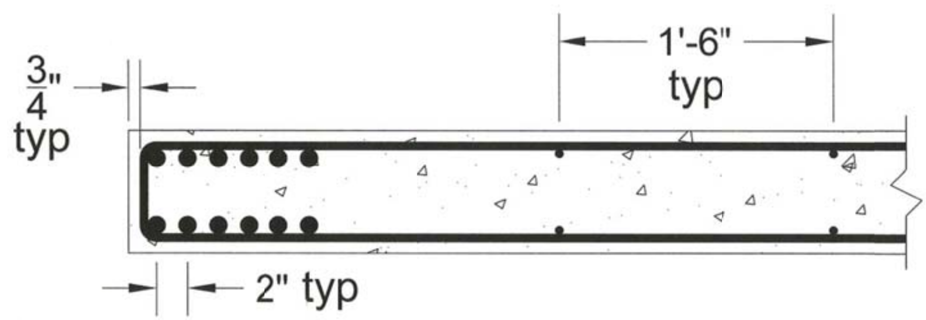


Figure 14: Wall #8 @ Base Detail

E-W Wall

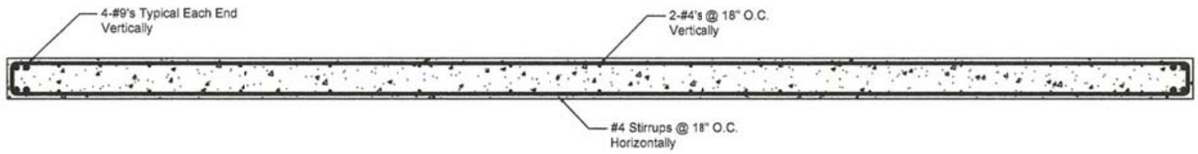


Figure 15: Wall #C @ 4th Floor

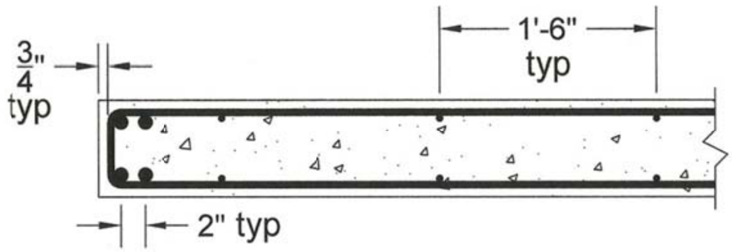


Figure 16: Wall #C @ 4th Detail

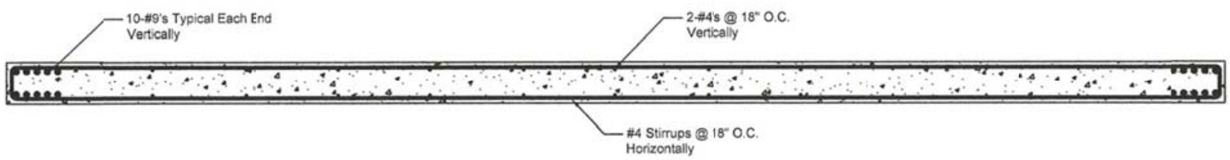


Figure 17: Wall #C @ Base

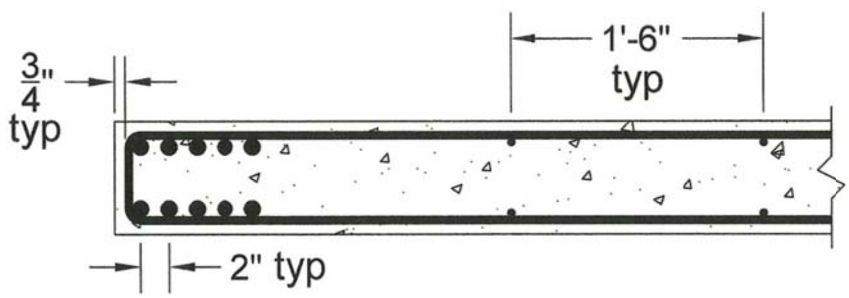


Figure 18: Wall #C @ Base Detail

Impact on Foundations

Since the lateral system has been changed there will be an undoubted impact on the foundations of the building. The columns around the vertical circulation areas (where the walls are proposed) have been removed, so those columns have no need for a foundation, although the walls will undoubtedly need to be supported in some way. This will most likely be done with strip footings along the length of the wall, or with the use of grade beams between spread footings. This will cause the need for more concrete in the foundations, but this could be offset based on the reduction of foundation size in what were formerly the lateral frames, which are now being utilized for gravity framing only.

Shear Wall Conclusions

This analysis has shown the feasibility of shear walls as a lateral system for the UPMC Hamot Womens Hospital. Shear walls as a lateral system benefit the structure by reducing building drifts even though the lateral loads are increased for earthquake design. This also reduces steel weight in the system because the lateral members can be reduced to just carry the gravity loads. The impact of foundations appears to offset itself, and can thus be neglected. Cost and schedule analysis will be completed in the construction management breadth and that will ultimately determine the most feasible system, which will allow a more thorough understanding of the effectiveness of this lateral system.

Breadth Topic I: Architectural Breadth

Overview

The architectural breadth is intended to analyze the impacts of the various lateral systems on the architecture of the overall building. This is particularly important to see how the braced frames compare to the moment frame and/or shear wall systems. One of the most notable features of this building is the beautiful views it provides its patients on the North side of the building. These views overlook the harbor and Lake Erie itself, which feels very relaxing and calming. Protecting these views was of the utmost importance to the architect on this project.

Architectural Impacts

Figures 19 and 20 (found below) are renderings of the 2nd floor employee break room located in the Northwest corner of the building. As you can see the only difference between these images is the braced frame member found in Figure 20. When determining if this is an issue, it is important to focus on the use of the space, and who is most likely to care about the location of this brace. In this instance the room is used by hospital employees only. Thus the issue becomes is the owner willing to accept a brace here, at the expense of their employees comfort? In this instance the owner is not likely to be willing to accept this due to the elegant views that are possible in this area.

Figure 19: 2nd Floor Employee Break Room (Moment Frame)

Figure 20: 2nd Floor Employee Break Room (Braced Frame)

Figures 21 and 22 (found below) are renderings of a 5th floor patient room located in the Northwest corner of the building. As you can see the only difference between these images is the braced frame member found in Figure 22. Again, when determining if this is an issue, it is important to focus on the use of the space, and who is most likely to care about the location of this brace. In this instance the room is primarily used by patients and the patients' families. Thus the issue becomes is the owner willing to accept a brace here, at the expense of their patients comfort? In this instance the owner is highly unlikely to accept this, primarily because the patients are the customers of the hospital and their comfort is of the utmost importance. Not to mention that every horizontal or semi-horizontal surface introduced into a patient room is another surface to collect dust and germs in a hospital environment. So not only is this an issue aesthetically it is also a cleanliness issue.

Figure 21: 5th Floor Typical Patient Room (Moment Frame)

Figure 22: 5th Floor Typical Patient Room (Braced Frame)

Conclusions

After analyzing the potential impacts of the braced frame system as compared to the moment frame or shear wall systems it is pretty apparent that the braced frame system is not as architecturally desired. Not only does it obstruct one of the best views of Lake Erie and its harbor, but it is clearly also a health concern to the patients. With the state of today's medical liabilities it is essential for the architects and engineers to take special care in all buildings, but especially in a hospital setting where germs are constantly a major concern. With this being said I would not recommend the use of the braced frame as a lateral system.

Breadth Topic II: Construction Management Breadth

Overview

The construction management breadth is intended to give a realistic cost and schedule analysis to compare to the existing building structure. This analysis will compare the as-built conditions, where the existing grid system and lateral system was used, with the various systems analyzed in the structural depth. The decision to deconstruct the existing building and reinforce/rebuild to the owners desires, rather than demolish it and begin with a new structure will be compared. This analysis will only analyze the events of Phase 1 of the project, or the structural shell. This does not include any fit-out costs or schedule as well as any building enclosures. It will be assumed that these things will not change and can thus be omitted from analysis. Upon the conclusion of this analysis a comparison based on the cost and schedule implications can be completed to determine the best solution. This analysis was done through the use of the RS Means Building Construction Cost Manuals.

Schedule Analysis

A schedule analysis was completed to compare the construction timelines from the as-built moment frame system and the three lateral system alternatives. For simplicity and uniformity of the analysis all construction schedules are assumed to start on September 1, 2007. Thus making the completion dates the important analysis parameter. It is important to note that the construction teams and tools were assumed to be that which is reasonable for the desired task, i.e. dozens of laborers were not brought in to complete a small aspect of the project very rapidly, and then sent home the following week. Detailed schedules can be found in Appendix H.

Schedule Analysis	
	Phase 1 Completion Date
Existing System	November 2008 +/-
Moment Frames	12/28/07
Braced Frames	1/2/08
Shear Walls	2/08/08

Table 5: Cost Analysis Table

As you can see in Table 5, the construction schedules for all options can be completed prior to the existing structure. The comparison of the new moment frame system with the as-built moment frames shows how much more rapid the implosion and debris removal can occur over the selective deconstruction that was done on the as-built design. This increased schedule may provide a major benefit for the owner due to the increased revenue that can be generated in the extra months of operation. This impact will be analyzed in the cost analysis section.

Cost Analysis

The detailed cost analysis from RS Means was done in several parts. Thus allowing the separate pieces of the analysis to be added and removed as necessary to allow for the most accurate analysis possible.

An analysis of implosion and removal costs was done and will be used with all of the systems. Another analysis that was similar was the floor construction costs. Thus the only variances will come from the change in materials takeoffs for the gravity and lateral systems in the UPMC Hamot Womens' Hospital. As well as the labor and equipment required to install the various lateral systems. All of the new systems in question will be compared to the information provided by the construction manager that performed the work on the UPMC Hamot Womens' Hospital. Appendix I contains the supplemental cost analysis data used to determine the numbers found below in Table 6.

Cost Analysis				
	Demolition/Removal	Gravity/Lateral System	Floor Construction	Total
Existing System	Unknown	Unknown	Unknown	$9,000,000+/-
Moment Frames	$2,345,293	$6,396,503	$1,963,536	$10,705,332
Braced Frames	$2,345,293	$5,884,627	$1,963,536	$10,193,456
Shear Walls	$2,345,293	$5,018,654	$1,963,536	$9,327,483

Table 6: Cost Analysis Table

Existing System: Data supplied by the Contractor
Moment Frames: Demolish the existing structure and completely rebuild with Moment Frames
Braced Frames: Demolish the existing structure and completely rebuild with Braced Frames
Shear Walls: Demolish the existing structure and completely rebuild with Shear Walls

As you can see in Table 6, the demolition costs appear to be what makes the systems cost more than the existing. With a comparison of the Existing System, which was not completely demolished and the moment frame system, which I assumed was completely demolished and rebuilt. You can clearly see that if the system was to remain the same it was best to not completely destroy the building. Although this is the case for when you keep the systems the same, if you demolish the building and switch the system to concrete shear walls the cost of what the contractor decided to do and the analysis that was performed as part of this breadth show very similar cost figures. These numbers have not been adjusted for the impact of the schedule, which will be completed below.

The UPMC Hamot Health Foundation grossed total revenue of $486 million in 2011. This is for the entire hospital campus, or 412 beds. If the number of beds (58) located in the UPMC Hamot Womens Hospital are used to analyze the potential added revenue for a year of operation. Upon analysis it can be assumed that the addition of this hospital added $68.4 million in revenue for the hospital in 2011. Independent research has shown that overall profit margins for hospitals are approximately 4.2%. With this data known it can then be assumed that the UPMC Hamot Womens Hospital made a profit of $2.87 million dollars. With this data a more accurate system comparison can be completed, with a consideration of the additional profit due to the shortened schedule.

Cost and Schedule Analysis				
	Cost	Schedule Adjustment (Mon)	Additional Profit	Comparable Cost
Existing System	$9,000,000+/-	N/A	N/A	$9,000,000+/-
Moment Frames	$10,705,332	10	$2,394,612	$8,310,720
Braced Frames	$10,193,456	10	$2,394,612	$7,798,844
Shear Walls	$9,327,483	9	$2,155,150	$7,172,332

Table 7: Cost and Schedule Analysis Table

Table 7 (above) is not the actual building shell costs that can be expected, but rather a fictional cost that accounts for the increased schedule from imploding the building rather the selective deconstruction. As can be seen the most efficient system, based on cost is the Shear Wall system. Although these numbers suggest that any lateral system would have been 'cheaper' if the building would have been imploded and reconstructed from scratch. Details of these calculations can be found in Appendix I.

Conclusions

After analyzing both the cost and schedule for the UPMC Hamot Womens Hospital, it is quite apparent that the best solution to the question of tearing the entire building down and completely rebuilding or using selective demolition and reinforcing the necessary beams, columns, and foundations was not made correctly. This analysis shows that the most efficient and profitable solution for the owner would have been utilizing the vertical circulation for shear walls after a complete demolition of the existing building.

MAE Course Related Study:

- ## Computer Modeling

Extrapolating the knowledge gained in AE 597A: Computer Modeling of Building Structures, was used to enhance this thesis project. RAM Structural Systems was used to analyze the buildings gravity and lateral systems. This software was not covered within AE 597A, but through various self-study applications and many hours of working within the program a firm grasp of the programs abilities and inabilities was gained. The knowledge from AE 597A was utilized in quickly and efficiently checking the computer outputs and solutions in order to determine relative accuracy of the assumptions made within the model; thus making this coursework applicable and very helpful in analyzing the UPMC Hamot Womens Hospital.

- ## Connection Design

Utilizing the knowledge gained in AE 534: Steel Connections, the following two connections were designed. These connections were chosen in particular because of their repetitive nature throughout the various lateral systems.

- Typical Moment Connection
- Typical Braced Frame Connection

A Flange Welded/Web Bolted moment connection was designed for the typical moment connection. This design was done to ensure that both the beam side limit states and the column side limit states were satisfied. Figure 23 shows the connection that was chosen as the typical connection and Figure 24 shows the detail for this connection. Detailed calculations of this design can be found in Appendix J.

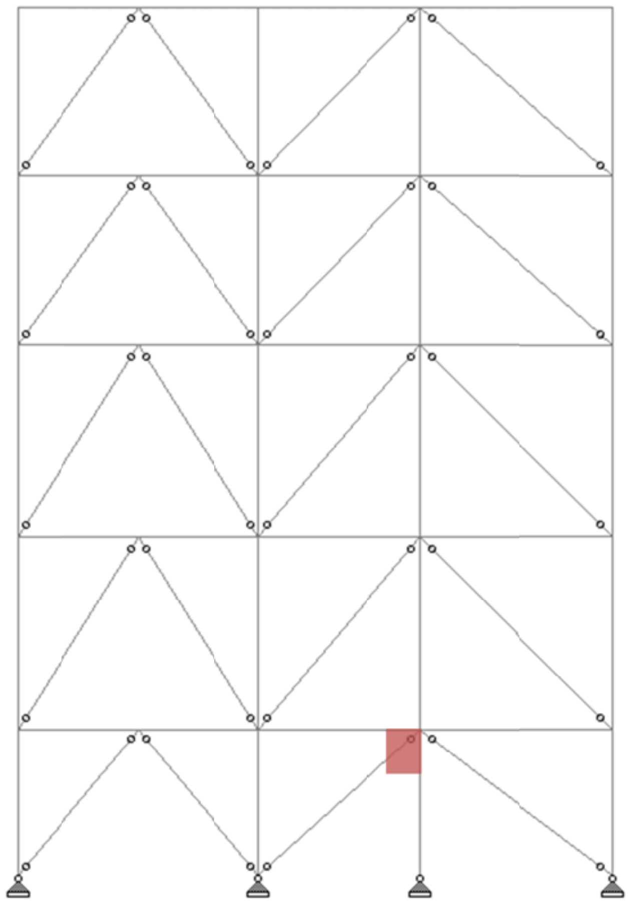


Figure 23: Brace Frame along CL 1

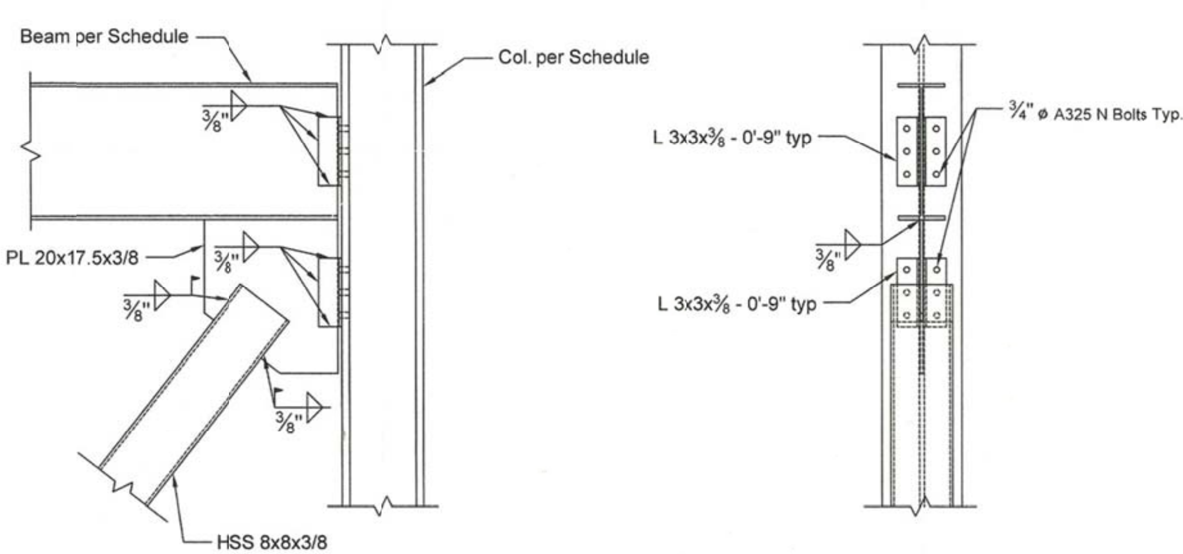


Figure 24: Typical Braced Frame Connection Detail

The braced frame connection was done utilizing knowledge gained from the AISC Design Guide 24 (Hollow Structural Section Connections) and AE 534. The connection to the HSS member is made through a gusset plate that will be shop welded to the beam and then be brought to the field. This beam and plate combination will be bolted to the column and field welded to the HSS member. This design was completed to eliminate moment at the connection. Figure 25 shows the connection that was chosen as the typical connection and Figure 26 shows the detail that was designed for this connection. Detailed calculations of this design can be found in Appendix K.

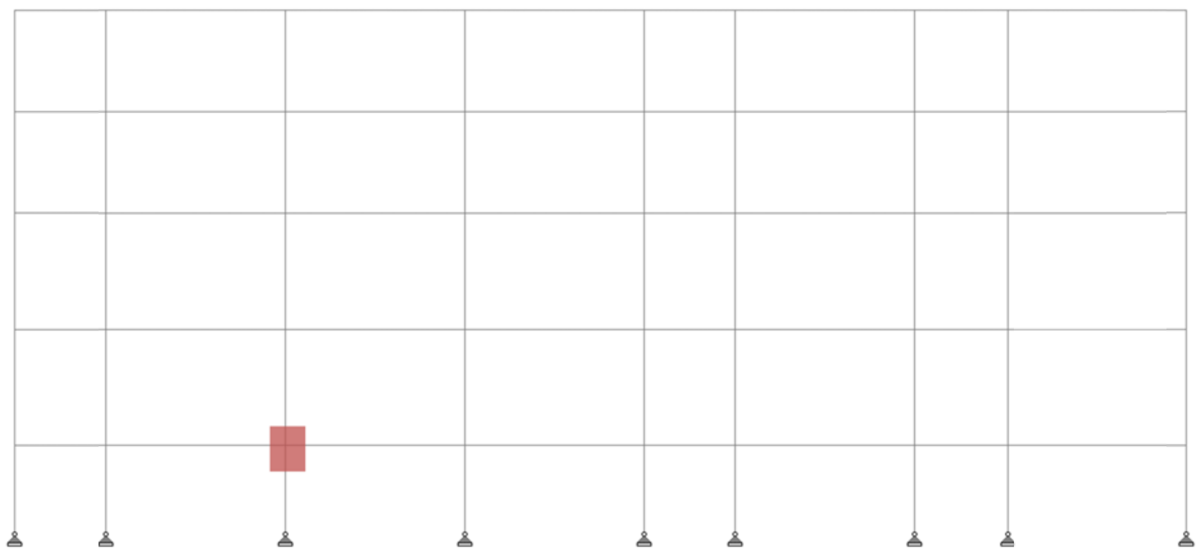


Figure 25: Moment Frame along CL 1

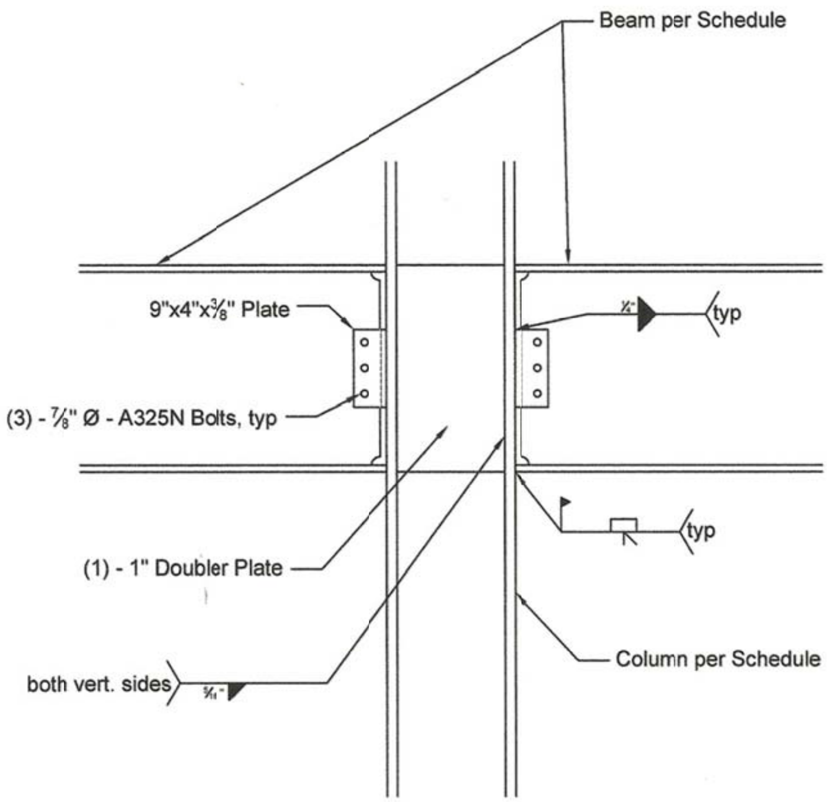


Figure 26: Typical Moment Frame Connection Detail

Conclusion

The structural depth for this thesis was split into three distinct investigations. An investigation on the new building code with a comparison to the previous edition and how it affects the structural weight and performance was done and proved that the loads did increase; although this primarily came from a change in occupancy category. New moment frames, braced frames, and a shear wall system were all successfully designed as lateral systems for the UPMC Hamot Womens Hospital. Any of these systems would meet the code required minimums and thus are acceptable alternatives for this structure. Finally, an investigation into a complete building redesign was done to determine if selective deconstruction of the building was the correct decision to be made by the construction team.

As these elements were completed two breadth studies were undertaken. An architectural breadth was done, which analyzed the impact on the architecture that the braced frame system has on the building. This analysis yielded several concerns, not just on the views that would be potentially ruined by the framing members, but also raised several health concerns for the patients of the hospital. A construction management breadth was also done to analyze the impact of not using the existing structure and grid to build from. This analysis showed that the contractors decision to use selective deconstruction rather than implosion was likely comparable when analyzing cost, this decision and the impacts on the construction schedule and the subsequent impact on the potential revenue from completing the building earlier yielded a drastic improvement in cost-schedule analysis, thus the analysis shows that the building should have been imploded and started again from scratch.

After these various analyses were completed it was determined that the most effective decision would have been to completely demolish the existing building and rebuild the structure from scratch utilizing a concrete shear wall lateral system. This may not have been done for many reasons, such as the construction manager supplying information to the owner that didn't analyze the potential that the schedule had on the cost analysis. Or the construction manager may have supplied the correct information to the owner, but they owner may have elected to reuse the structure in an effort to not completely abandon their initial plan. Regardless of the reasons this project has proven to be a great and realistic learning experience for the student.

Appendix A

Appendix A.1 – ASCE Live Loads Comparison

Live Loads (psf)

	ASCE 7-05	ASCE 7-10
Lobbies	100	100
Hospitals		
Operating Rooms/Labs	60	60
Patient Rooms	40	40
Corridors, above First Floor	80	80
First Floor Corridors	100	100
Offices	50	50
Stairs	100	100
Mechanical	150	150
Roofs	20	20

∴ Live Loads did not change from ASCE 7-05 to ASCE 7-10

Appendix B

Appendix B.1 – ASCE 7-05 Snow Loads

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

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

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

Appendix B.1 – ASCE 7-05 Snow Loads

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Penthouse Roof)

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

N-S Drift

$l_u = 66'-0"$
$h_c = 20'-0"$

$\therefore h_d = 2.8'$

E-W Drift

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

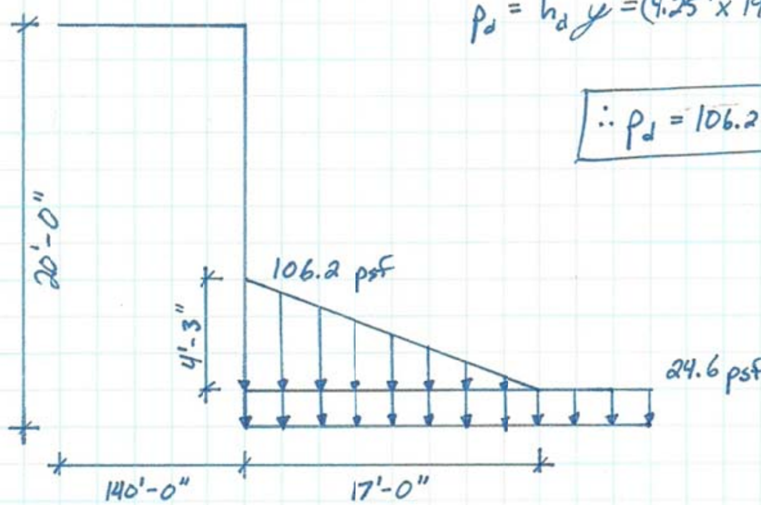
$\therefore h_d = 4.25'$

$\therefore$ 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

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



Appendix B.1 – ASCE 7-05 Snow Loads

Snow Loads (cont)

ASCE 7-05

Drift Snow Load (Stair Pop-Out)

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

N-S Drift

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

$\therefore h_d = 1.75'$

E-W Drift

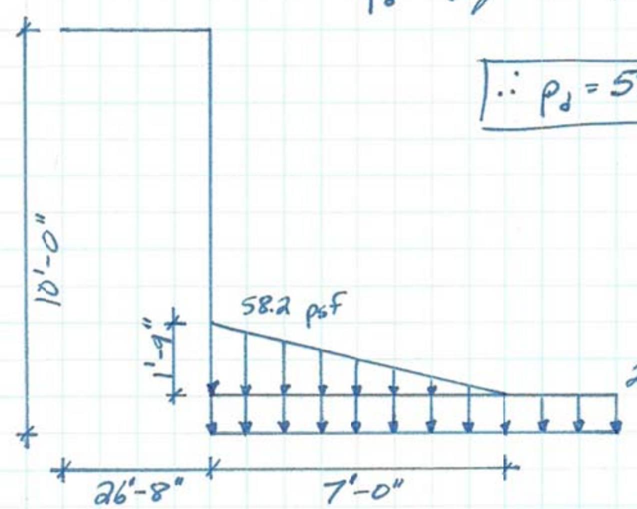
$l_u = 26'-8''$
$h_c = 10'-0''$

$\therefore h_d = 1.75'$

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

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

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



Appendix B.2 – ASCE 7-10 Snow Loads

Snow Loads (cont)

ASCE 7-10

Flat Roof Snow Load

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

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

$p_g = 40$ psf, see note above

$I = 1.2 \rightarrow$ Table 1.5-2
→ Occupancy Category IV ⇒ Essential Facility
↳ Table 1.5-1 (ASCE 7-10) ↳ Engineers Judgement

$C_t = 1.0 \Rightarrow$ Table 7-3 (ASCE 7-10)

$C_e = 0.8 \Rightarrow$ Table 7-2 (ASCE 7-10)
↳ Fully Exposed
↳ Terrain Category D, on lake

$p_f = 0.7(0.8)(1.0)(1.2)(40\text{ psf})$

$\boxed{p_f = 26.88 \text{ psf}}$

* Note: Change in Load due to change in Risk Category between ASCE 7-05 & ASCE 7-10 (Ch 1)

Appendix B.2 – ASCE 7-10 Snow Loads

Snow Loads (cont)

ASCE 7-10

Drift Snow Load (Penthouse Roof)

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

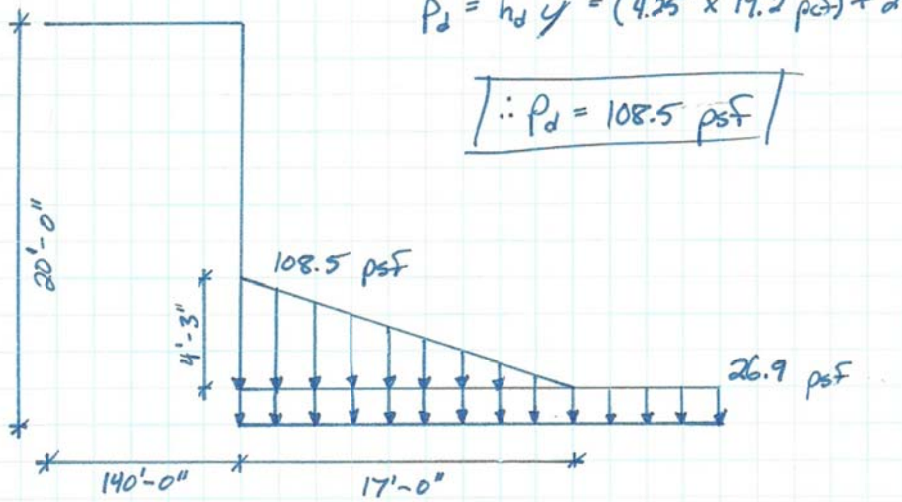
N-S Drift	E-W Drift
$l_u = 60'\text{-}0''$	$l_u = 140'\text{-}0''$
$h_c = 20'\text{-}0''$	$h_c = 20'\text{-}0''$
$\therefore h_d = 2.8'$	$\therefore h_d = 4.25'$

$\therefore$ Use $h_d = 4.25'$

$w = 4\,h_d = 17'\text{-}0''$

$p_d = h_d\,\gamma = (4.25' \times 19.2\text{ pcf}) + 26.9 = 108.5$ psf

$\boxed{\therefore p_d = 108.5 \text{ psf}}$



Appendix B.2 – ASCE 7-10 Snow Loads

Snow Loads (cont)

ASCE 7-10

Drift Snow Load (Stair Pop-Out)

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

N-S Drift

$l_u = 10'\text{-}10''$
$h_c = 10'\text{-}0''$

$\therefore h_d = 1.75'$

E-W Drift

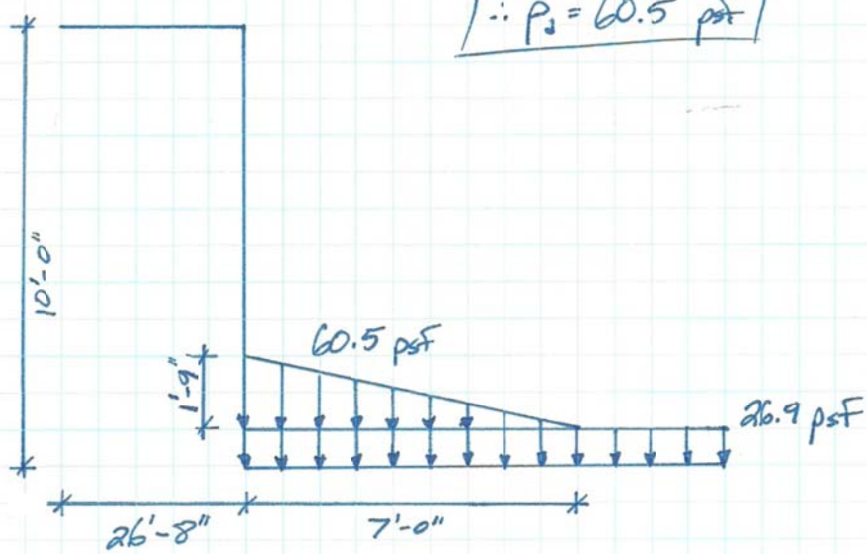
$l_u = 26'\text{-}8''$
$h_c = 10'\text{-}0''$

$\therefore h_d = 1.75'$

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

$p_d = h_d\, \gamma = 1.75'(19.2 \text{ pcf}) + 26.9 = 60.5$ psf

$\boxed{\therefore p_d = 60.5 \text{ psf}}$



Appendix B.2 – ASCE 7-10 Snow Loads

Snow Loads (cont)

ASCE 7-05 vs. ASCE 7-10

ASCE 7-05	ASCE 7-10
$p_f = 24.64$ psf	$p_f = 26.88$ psf
$P_{dPH} = 106.2$ psf	$P_{dPH} = 108.5$ psf
$h_{dPH} = 4.25'$	$h_{dPH} = 4.25'$
$\omega_{PH} = 17'$-$0''$	$\omega_{PH} = 17'$-$0''$
$P_{dSP} = 58.2$ psf	$P_{dSP} = 60.5$ psf
$h_{dSP} = 1.75'$	$h_{dSP} = 1.75'$
$\omega_{SP} = 7'$-$0''$	$\omega_{SP} = 7'$-$0''$

Loads Increase by 9.1% due to change in Risk Category, which changed importance factor

*Note: Change is not typical of ALL structures but does effect healthcare facilities w/o Emergency Rooms.

Change is also "conservative"

Appendix C

Appendix C.1 – ASCE 7-05 Wind Loads

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

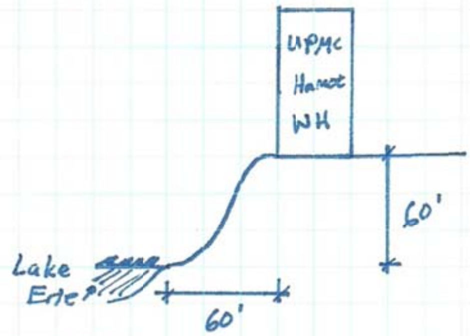
70'-80' = 1.38
60'-70' = 1.34
50'-60' = 1.31
40'-50' = 1.27
30'-40' = 1.22
25'-30' = 1.16
20'-25' = 1.12
15'-20' = 1.08
0'-15' = 1.03

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

Wind Loads (cont)

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

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



$$K_1 = 0.95(1.0)$$

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

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

$$= 1$$

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

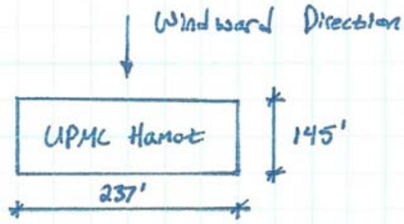
$$K_3 = e^{-\gamma z/L_H} \qquad \gamma = 2.5$$

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

$z = 90 \quad = 0.024$

Appendix C.1 – ASCE 7-05 Wind Loads

Wind Loads (cont)



Plan View

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

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

$L/B = 145/237 = 0.612$

Gust Factor ⇒ Sec 6.5.8

$G = 0.85$

Enclosed Bulding ⇒ Figure 6-5

$GC_{pi} = +/- 0.18$

C_p Values → Figure 6-6

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

Appendix C.1 – ASCE 7-05 Wind Loads

Wind Loads (cont)

q_z Values ⇒ Section 6.5.10

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

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

Windward Wall Pressures ⇒ Sec 6.5.12.4.2

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

Leeward Wall Pressures ⇒ Sec 6.5.12.4.2

$p = -15.55$

Appendix C.1 – ASCE 7-05 Wind Loads

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

$G = 0.85$

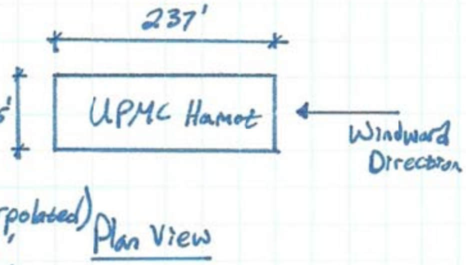
Enclosed Building ⇒ Figure 6-5

$GC_{pi} = \pm 0.18$

Appendix C.1 – ASCE 7-05 Wind Loads

Wind Loads (cont)

C_p Values ⇒ Figure 6-6



$C_p = 0.8$ ⇒ Windward Wall
$C_p = -0.37$ ⇒ Leeward Wall (Interpolated)
$C_p = -0.9$ ⇒ Roof ⇒ 0' to 37'
$C_p = -0.9$ ⇒ Roof ⇒ 39' to 78'
$C_p = -0.5$ ⇒ Roof ⇒ 78' to 156'
$C_p = -0.3$ ⇒ Roof ⇒ > 156'

$L/B = 237/145 = 1.63$

q_z Values ⇒ Section 6.5.10

$q_{z80} = 27.97$
$q_{z70} = 27.16$
$q_{z60} = 26.55$
$q_{z50} = 25.74$
$q_{z40} = 24.73$
$q_{z30} = 23.51$
$q_{z25} = 22.70$
$q_{z20} = 21.87$
$q_{z15} = 20.88$

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

Appendix C.1 – ASCE 7-05 Wind Loads

Wind Loads (cont)

Windward Wall Pressures ⇒ Sec 6.5.12.4.2

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

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

Leeward Wall Pressures ⇒ Sec 6.5.12.4.2

$p = -14.13$

Appendix C.1 – ASCE 7-05 Wind Loads

Justin Kovach — UPMC Hamot Womens Hospital
AE Senior Thesis 2011-2011 — 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} = 49230.8 ft-kips

Justin Kovach — UPMC Hamot Womens Hospital
AE Senior Thesis 2011-2011 — 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 C.2 – ASCE 7-10 Wind Loads

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

<u>ASCE 7-10</u>

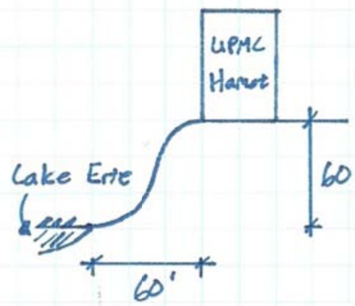
Directional Procedure (Wind from North)

Risk Category = IV ⇒ Table 1.5-1

$V = 120$ mph ⇒ Figure 26.5-1 B

$K_d = 0.85$ → Table 26.6-1

Exposure Category = D



$K_1 = 0.95(1.0) = 0.95$

$K_2 = \left(1 - \frac{|x|}{\mu L_H}\right) = \left(1 - \frac{0}{4(60)}\right) = 1.0$

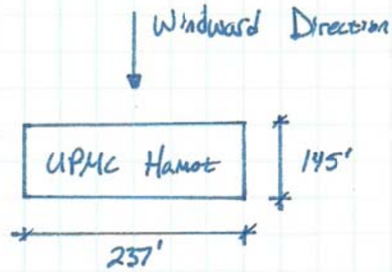
· 2D Escarpment
· $H/L_H = 60/60 = 1.0$

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

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

Appendix C.2 – ASCE 7-10 Wind Loads

Wind Loads (cont)



Plan View

$L/B = 145/237 = 0.612$

$K_{zt} \Rightarrow$ Figure 26.8-1

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

$K_{zt70} = 1.105$
$K_{zt60} = 1.162$
$K_{zt50} = 1.252$
$K_{zt40} = 1.391$
$K_{zt30} = 1.620$
$K_{zt25} = 1.783$
$K_{zt20} = 1.997$
$K_{zt15} = 2.275$
$K_{zt0} = 3.803$

$K_{zt80} = 1.070$
$K_{zt90} = 1.046$

Gust Factor $\Rightarrow$ Sec 26.9.4

$G = 0.85$

Enclosure Classification = Enclosed $\Rightarrow$ Sec 26.10

Internal Pressure Coefficient

$GC_{pi} = +/- 0.18 \rightarrow$ Table 26.11-1

$K_h + K_z \rightarrow$ Table 27.3-1

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 $\Rightarrow$ interpolated value

Appendix C.2 – ASCE 7-10 Wind Loads

Wind Loads (cont)

q_z Values → Sec 27.3.2

$q_{z80} = 47.78$
$q_{z70} = 48.79$
$q_{z60} = 51.39$
$q_{z50} = 55.35$
$q_{z40} = 61.93$
$q_{z30} = 64.81$
$q_{z25} = 70.08$
$q_{z20} = 76.99$
$q_{z15} = 122.74$

$q_{z90} = 46.94$
$q_{z92} = 46.21$

C_p Values → Fig 27.4-1

$C_p = 0.8 \Rightarrow$ Windward Walls
$C_p = -0.5 \Rightarrow$ Leeward Wall

Windward Wall Pressures ⟹ Sec 27.4.1

$p_{80'} = 40.81$
$p_{70} = 41.72$
$p_{60} = 43.48$
$p_{50} = 46.18$
$p_{40} = 50.65$
$p_{30} = 52.61$
$p_{25} = 56.19$
$p_{20} = 60.89$
$p_{15} = 92.00$

$p_{90} = 40.24$
$p_{92} = 39.74$

Leeward Wall Pressure → Sec 27.4.1

$p = -27.96$

Appendix C.2 – ASCE 7-10 Wind Loads

Wind Loads (cont)

Directional Procedure (Wind from East or West)

Risk Category = IV ⇒ Table 1.5-1

$V = 120$ mph ⇒ Figure 26.5-1 B

$K_d = 0.85$ ⇒ Table 26.6-1

Exposure Category = D

$K_{zt} = 1.0$ → Sec 26.8.2

Gust Factor ⇒ Sec 26.9.4

$G = 0.85$

Enclosure Classification = Enclosed ⇒ Sec 26.10

Internal Pressure Coefficient

$GC_{pi} = \pm 0.18$ → Table 26.11-1

K_h + K_z ⇒ Table 27.3-1

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

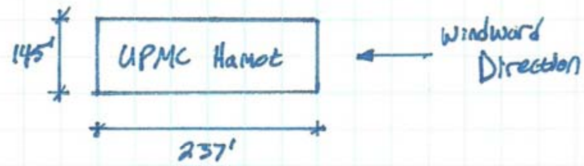
80-90 = 1.40
90-92 = 1.41 ⇒ interpolated value

Appendix C.2 – ASCE 7-10 Wind Loads

Wind Loads (cont)

q_z Values $\Rightarrow$ Sec 27.3.2

$q_{z80} = 43.24$
$q_{z70} = 41.99$
$q_{z60} = 41.05$
$q_{z50} = 39.79$
$q_{z40} = 38.23$
$q_{z30} = 36.35$
$q_{z25} = 35.09$
$q_{z20} = 33.84$
$q_{z15} = 32.27$



$q_{z90} = 43.86$
$q_{z92} = 44.18$

C_p Values $\Rightarrow$ Fig 27.4-1

$C_p = 0.8 \Rightarrow$ Windward Walls
$C_p = -0.5 \Rightarrow$ Leeward Walls

Windward Wall Pressures $\Rightarrow$ Sec 27.4-1

$p_{80} = 37.36$
$p_{70} = 36.28$
$p_{60} = 35.64$
$p_{50} = 34.78$
$p_{40} = 33.72$
$p_{30} = 32.45$
$p_{25} = 31.59$
$p_{20} = 30.74$
$p_{15} = 29.67$

$p_{90} = 37.78$
$p_{92} = 37.97$

Leeward Wall Pressure $\Rightarrow$ Sec 27.4-1

$p = -26.73$

Appendix C.2 – ASCE 7-10 Wind Loads

Justin Kovach
AE Senior Thesis 2011-2011

UPMC Hamot Womens Hospital
Erie, PA

Base Shear and Overturning Moment Calulator

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	40.81 psf	19.3 kips	1373.4 ft-kips
70 ft	60 ft	41.72 psf	98.9 kips	6427.0 ft-kips
60 ft	50 ft	43.48 psf	103.0 kips	5667.6 ft-kips
50 ft	40 ft	46.18 psf	109.4 kips	4925.1 ft-kips
40 ft	30 ft	50.65 psf	120.0 kips	4201.4 ft-kips
30 ft	25 ft	52.61 psf	62.3 kips	1714.4 ft-kips
25 ft	20 ft	56.19 psf	66.6 kips	1498.2 ft-kips
20 ft	15 ft	60.89 psf	72.2 kips	1262.7 ft-kips
15 ft	0 ft	92.00 psf	327.1 kips	2453.0 ft-kips

Stair Pop-Out

h_{top} =	h_{bot} =	p =	V =	M =
82 ft	80 ft	40.24 psf	1.6 kips	130.4 ft-kips
80 ft	72 ft	40.81 psf	6.5 kips	496.2 ft-kips

Mechanical Penthouse

h_{top} =	h_{bot} =	p =	V =	M =
92 ft	90 ft	39.74 psf	12.7 kips	1157.2 ft-kips
90 ft	80 ft	40.24 psf	64.4 kips	5472.6 ft-kips
80 ft	72 ft	40.81 psf	52.2 kips	3970.0 ft-kips

Suction

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	0 ft	27.96 psf	477.1 kips	17175.9 ft-kips
82 ft	72 ft	27.96 psf	5.6 kips	430.6 ft-kips
92 ft	72 ft	27.96 psf	89.5 kips	7336.7 ft-kips

Total

V_{tot} = 1688.5 kips
M_{tot} = 66692.5 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	37.36 psf	10.8 kips	769.2 ft-kips
70 ft	60 ft	36.28 psf	52.6 kips	3419.4 ft-kips
60 ft	50 ft	35.64 psf	51.7 kips	2842.3 ft-kips
50 ft	40 ft	34.78 psf	50.4 kips	2269.4 ft-kips
40 ft	30 ft	33.72 psf	48.9 kips	1711.3 ft-kips
30 ft	25 ft	32.45 psf	23.5 kips	647.0 ft-kips
25 ft	20 ft	31.59 psf	22.9 kips	515.3 ft-kips
20 ft	15 ft	30.74 psf	22.3 kips	390.0 ft-kips
15 ft	0 ft	29.67 psf	64.5 kips	484.0 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	37.99 psf	5.7 kips	518.6 ft-kips
90 ft	80 ft	37.78 psf	28.3 kips	2408.5 ft-kips
80 ft	72 ft	37.36 psf	22.4 kips	1703.6 ft-kips

Suction

h_{top} =	h_{bot} =	p =	V =	M =
72 ft	0 ft	26.73 psf	279.1 kips	10046.2 ft-kips
82 ft	72 ft	26.73 psf	4.0 kips	308.7 ft-kips
92 ft	72 ft	26.73 psf	40.1 kips	3287.8 ft-kips

Total

V_{tot} = 730.9 kips
M_{tot} = 31601.0 ft-kips

Appendix D

Appendix D.1 – ASCE 7-05 Earthquake Loads

EQ Loads

ASCE 7-05

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

$I = 1.25$ ⇒ Table 11.5-1

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

$C_u = 1.7$ ⇒ Table 12.8-1

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

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

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

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

$\therefore C_s = 0.0183$

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

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

Appendix D.1 – ASCE 7-05 Earthquake Loads

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

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

Appendix D.1 – ASCE 7-05 Earthquake Loads

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 D.2 – ASCE 7-10 Earthquake Loads

EQ Loads (cont)

ASCE 7-10

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

$I = 1.5 \Rightarrow$ Table 1.5-2

$\left.\begin{array}{l} S_{DS} = 1.65 \\ S_{D1} = 0.085 \end{array}\right\}$ From USGS

$T_L = 12 \Rightarrow$ Fig 22-12

$T = C_u T_a$

$C_u = 1.7 \Rightarrow$ Table 12.8-1

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

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

$$C_s = \min \left| \begin{array}{lr} S_{DS}/(R/I) = 1.65/(3/1.5) = & 0.825 \\ S_{D1}/(T \cdot R/I) = 0.085/(1.773 \cdot 3/1.5) = & 0.024 \\ S_{D1} \cdot T_L/(T^2 \cdot R/I) = 0.085(12)/(1.773^2 \cdot 3/1.5) = & 0.162 \end{array} \right.$$

$\therefore C_s = 0.024$

$V = C_s W = 0.024(11,606)$

$\boxed{\therefore V = 278.54^{\,k}}$

Appendix D.2 – ASCE 7-10 Earthquake Loads

EQ Loads (cont)

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

$F_{PH} = C_{vPH}\ V = 22.61^k$
$F_{SR} = C_{vSR}\ V = 4.47^k$
$F_{R} = C_{vR}\ V = 79.69^k$
$F_{5} = C_{v5}\ V = 80.92^k$
$F_{4} = C_{v4}\ V = 54.61^k$
$F_{3} = C_{v3}\ V = 28.02^k$
$F_{2} = C_{v2}\ V = 8.21^k$

Appendix E

Appendix E.1 – Braced Frame Load Calcs

Story Forces at UPMC Hamot Womens Hospital (Braced Frames)

X-Direction

	Floor Level							
Load Method	**2nd**	**3rd**	**4th**	**5th**	**Roof**	**Elev Roof**	**Penthouse**	**Base Shear**
W1	0.12	0.03	0.02	0.04	0.58	-0.59	0.00	0.20
W2	0.01	0.00	0.00	0.00	0.34	-0.34	0.00	0.01
W3	0.17	0.04	0.03	0.05	0.53	-0.55	0.00	0.27
W4	51.59	64.51	63.95	61.35	74.87	-22.78	22.30	**315.80**
W5	38.71	48.39	47.97	46.02	56.15	-17.09	16.73	236.88
W6	38.68	48.38	47.95	46.00	56.15	-17.08	16.73	236.82
E1	12.04	35.53	57.72	79.93	129.50	-1.18	0.00	313.54
E2	12.03	35.53	57.71	79.92	129.55	-1.23	0.00	313.51
E3	0.03	0.01	0.01	0.01	-0.07	0.07	0.00	0.06
E4	0.06	0.02	0.02	0.03	-0.15	0.15	0.00	0.13

Y-Direction

	Floor Level							
Load Method	**2nd**	**3rd**	**4th**	**5th**	**Roof**	**Elev Roof**	**Penthouse**	**Base Shear**
W1	159.08	151.34	129.79	117.61	184.42	-57.77	58.79	**743.26**
W2	119.11	113.20	97.14	88.01	138.26	-43.47	44.47	556.72
W3	119.51	113.82	97.55	88.40	138.37	-43.18	44.08	558.55
W4	0.07	0.07	0.05	0.05	0.00	0.04	0.00	0.28
W5	0.08	0.13	0.10	0.09	0.02	0.05	0.00	0.47
W6	0.02	-0.03	-0.02	-0.02	-0.02	0.00	0.00	-0.07
E1	0.04	0.04	0.04	0.05	0.03	0.01	0.00	0.21
E2	0.04	-0.02	-0.02	-0.02	-0.03	-0.01	0.00	-0.06
E3	10.87	33.90	55.25	77.83	126.16	-0.78	0.00	303.23
E4	10.86	33.99	55.35	77.94	126.25	-0.76	0.00	303.63

Load Methods Explained:

W1 = Wind Parallel to Y-Axis according to ASCE 7-05
W2 = Wind Parallel to Y-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
W3 = Wind Parallel to Y-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
W4 = Wind Parallel to X-Axis according to ASCE 7-05
W5 = Wind Parallel to X-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
W6 = Wind Parallel to X-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
E1 = Earthquake Paralled to X-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
E2 = Earthquake Paralled to X-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
E3 = Earthquake Paralled to Y-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
E4 = Earthquake Paralled to Y-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity

Appendix E.1 – Braced Frame Load Calcs

Superstructure Weight at UPMC Hamot Womens Hospital (Braced Frames)

Gravity Beams Weight		
Penthouse	61.8 kips	30.9 tons
Elev Roof	2.9 kips	1.5 tons
Roof	140.3 kips	70.1 tons
5th	136.5 kips	68.3 tons
4th	137.3 kips	68.7 tons
3rd	141.9 kips	71.0 tons
2nd	138.9 kips	69.5 tons
Total	759.7 kips	379.8 tons
Gravity Column Weight		
Total	249.0 kips	124.5 tons
Lateral Frame Weight		
Penthouse		
Beams	2.0 kips	1.0 tons
Columns	5.4 kips	2.7 tons
Braces	7.5 kips	3.8 tons
Penthouse Total	15.0 kips	7.5 tons
Elev Roof		
Beams	0.0 kips	0.0 tons
Columns	3.3 kips	1.6 tons
Braces	0.0 kips	0.0 tons
Elev Roof Total	3.3 kips	1.6 tons
Roof		
Beams	9.4 kips	4.7 tons
Columns	16.2 kips	8.1 tons
Braces	7.9 kips	4.0 tons
Roof Total	33.6 kips	16.8 tons
5th		
Beams	6.8 kips	3.4 tons
Columns	16.2 kips	8.1 tons
Braces	7.9 kips	4.0 tons
5th Total	30.9 kips	15.4 tons
4th		
Beams	6.8 kips	3.4 tons
Columns	25.1 kips	12.6 tons
Braces	10.3 kips	5.1 tons
4th Total	42.1 kips	21.1 tons
3rd		
Beams	6.8 kips	3.4 tons
Columns	25.1 kips	12.6 tons
Braces	10.3 kips	5.1 tons
3rd Total	42.2 kips	21.1 tons
2nd		
Beams	6.8 kips	3.4 tons
Columns	23.0 kips	11.5 tons
Braces	8.6 kips	4.3 tons
2nd Total	38.4 kips	19.2 tons
TOTAL LATERAL FRAME WEIGHT	205.4 kips	102.7 tons
TOTAL SUPERSTRUCTURE WEIGHT	**1214.1 kips**	**607.0 tons**

Appendix E.1 – Braced Frame Load Calcs

Braced Frame Load Calculator

"Diaphragms"

	CMx	CMy	Area
1	105.2	60.2	25310.0
2	187.3	133.7	1228.9
3	223.7	56.0	1422.2

N-S Frames

	Ri	xi	Rixi	di	Ridi	Ridi^2	J
CL B	178.6	24.7	4405.4	-83.0	-14821.56	1230201	3678669
CL N	204.9	180.0	36885.2	72.3	14821.56	1072032	
	383.5		41290.6				

E-W Frames

	Ri	yi	Riyi	di	Ridi	Ridi^2
CL 1	188.0	0.0	0.0	-60.9	-11438.53	696068.2
CL 17	192.3	120.3	23141.0	59.5	11438.53	680367.4
	380.3		23141.0			

Center of Mass

CMx	CMy
114.8	63.2

Center of Rigidity

CRx	CRy
107.7	60.9

Eccentricity

ex	ey
7.1	2.3

Shear Loads

	X-Direction	Y-Direction
Penthouse	25.15	63.31
Elev Roof	8.38	8.37
Roof	51.32	124.47
5th	60.28	115.07
4th	62.41	127.49
3rd	62.86	143.31
2nd	50.70	157.94

Direct Shear Loads (X)

Col Line	Stiffness	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
CL 1	188.0	25.061	31.071	30.849	29.796	25.367	4.142	12.432
CL 17	192.3	25.639	31.789	31.561	30.484	25.953	4.238	12.718
CL B	178.6	0.000	0.000	0.000	0.000	0.000	0.000	0.000
CL N	204.9	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Direct Shear Loads (Y)

Col Line	Stiffness	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
CL 1	188.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
CL 17	192.3	0.000	0.000	0.000	0.000	0.000	0.000	0.000
CL B	178.6	73.545	69.060	59.366	53.582	57.959	3.897	31.808
CL N	204.9	84.395	79.250	68.124	61.488	66.511	4.473	36.502

Appendix E.1 – Braced Frame Load Calcs

Torsional Shear Loads (X-Direction Loads)

Col Line	Stiffness	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
CL 1	188.0	-1.125	-1.394	-1.384	-1.337	-1.138	-0.186	-0.558
CL 17	192.3	1.125	1.394	1.384	1.337	1.138	0.186	0.558
CL B	178.6	-1.457	-1.807	-1.794	-1.732	-1.475	-0.241	-0.723
CL N	204.9	1.457	1.807	1.794	1.732	1.475	0.241	0.723

Torsional Shear Loads (Y-Direction Loads)

Col Line	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
CL 1	188.0	0.0	-1.146	-1.076	-0.925	-0.835	-0.903	-0.061	-0.495
CL 17	192.3	0.0	1.146	1.076	0.925	0.835	0.903	0.061	0.495
CL B	178.6	0.0	-1.146	-1.076	-0.925	-0.835	-0.903	-0.061	-0.495
CL N	204.9	0.0	1.146	1.076	0.925	0.835	0.903	0.061	0.495

Total Shear Load (X)

Col Line	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear
CL 1	188.0	0.0	23.936	29.677	29.465	28.459	24.229	3.956	11.874	151.597
CL 17	192.3	0.0	26.764	33.183	32.945	31.821	27.091	4.424	13.276	169.503
CL B	178.6	0.0	-1.457	-1.807	-1.794	-1.732	-1.475	-0.241	-0.723	9.228
CL N	204.9	0.0	1.457	1.807	1.794	1.732	1.475	0.241	0.723	9.228

Total Shear Load (Y)

Col Line	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear
CL 1	188.0	0.0	-1.146	-1.076	-0.925	-0.835	-0.903	-0.061	-0.495	5.439
CL 17	192.3	0.0	1.146	1.076	0.925	0.835	0.903	0.061	0.495	5.439
CL B	178.6	0.0	72.399	67.985	58.441	52.748	57.057	3.837	31.313	343.779
CL N	204.9	0.0	85.541	80.325	69.049	62.322	67.413	4.533	36.997	406.181

Controlling Shear Loads

Col Line	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear
CL 1	188.0	0.0	23.936	29.677	29.465	28.459	24.229	3.956	11.874	151.597
CL 17	192.3	0.0	26.764	33.183	32.945	31.821	27.091	4.424	13.276	169.503
CL B	178.6	0.0	72.399	67.985	58.441	52.748	57.057	3.837	31.313	343.779
CL N	204.9	0.0	85.541	80.325	69.049	62.322	67.413	4.533	36.997	406.181
		Floor Height	12	28	44	58	72	79.5	92	

Bracing Connection Design

For HSS $6 \times 6 \times 3/8$

ASTM A500 Gr B
$F_y = 46$ ksi
$F_u = 58$ ksi

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

HSS Properties
$A = 7.58\ in^2$
$d = 6\ in$
$t = 0.349\ in$

$P = 46^k$
$P_u = 1.6(46^k) = 73.6^k$

HSS Limit States

Tensile Yielding

$$P_n = F_y A_g = 46(7.58) = 348.7^k$$

$$\phi P_n = 0.9(348.7) = 313.8^k > 73.6^k \quad \therefore OK$$

Tensile Rupture

$$P_n = F_u A_e = 58(4.55) = 263.7^k$$

$$A_e = A_n U = 7.27(0.625) = 4.55\ in^2$$

$$A_n = A_g - 2(t_p + 1/16)t = 7.58 - 2(3/8 + 1/16)0.349 = 7.27\ in^2$$

$$U = 1 - \bar{x}/\ell = 1 - 2.25/6 = 0.625$$

$$\ell = 6''$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{6^2 + 2(6)(6)}{4(6+6)} = 2.25$$

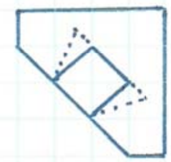
$$\phi P_n = 0.75(263.7^k) = 197.8^k > 73.6^k \quad \therefore OK$$

Appendix E.2 – Braced Frame Design

Plate Limit States

Whitmore Section

$$w = 2[6\tan(30)] + 6$$
$$= 12.93''$$



Tension Yield

$$P_n = F_y A_g = 36(12.93\,t)$$

$$\phi P_n \geq P_u \rightarrow 0.9(36)(12.93\,t) = 73.6^k$$

$$\therefore t_{min} = 0.176'' \rightarrow \text{Use } 3/8'' \text{ PL}$$

Weld Limit States (HSS-to-Gusset)

Base Metal Strength

Try 3/16" weld

$$t_{min} = \frac{D_{eff}}{30.2}\left(\frac{F_{EXX}}{F_y}\right) = \frac{3}{30.2}\left(\frac{70}{36}\right) = 0.193''$$

$$t_p = 0.375 < 2\,t_{min} = 2(0.193) = 0.386$$

$\therefore$ Gusset Base Metal Controls

$$R_n = F_{BM} A_{BM} = 0.6(36)[2(6'')(0.375'')] = 97.2^k$$

$$\phi R_n = 1.0(97.2^k) = 97.2^k > 73.6^k \quad \therefore OK$$

Appendix E.2 – Braced Frame Design

Plate Limit States

Block Shear

$A_{nv} = 2(6)(0.375) = 4.5\,in^2 = A_{gv}$
$A_{nt} = 6(0.375) = 2.25\,in^2$

$R_n = [0.6\,F_u A_{nv} + F_u A_{nt}] \leq [0.6\,F_y A_{gv} + F_u A_{nt}]$
$= 287.1^k \leq 227.7^k$

$\phi R_n = 0.75(227.7^k) = 170.8^k > 73.6^k \quad \therefore OK$

Sizing Gusset Plates

Use General Force Method to ensure no moment in connection

$\alpha = \frac{L_1}{2} + 1/2 \qquad e_c = d_c/2 = 10.8/2 = 5.4''$

$\beta = 10'' \qquad e_b = d/2 = 17.7/2 = 8.85''$

$\tan\theta = 0.7778$

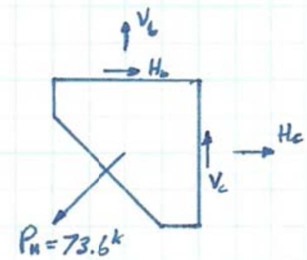
$\alpha - \beta\tan\theta = e_b\tan\theta - e_c$
$L_1/2 + 1/2 - (10)(0.7778) = 8.85(0.7778) - 5.4$

$\therefore L_1 = 17.5''$

$\boxed{\text{Use PL } 20 \times 17.5 \times 3/8''}$

Appendix E.2 – Braced Frame Design

<u>Forces From Gusset Plate</u>



$$r = \sqrt{(\alpha + e_c)^2 + (e_b + \beta)^2}$$
$$= \sqrt{(9.25 + 5.4)^2 + (8.85 + 10)^2}$$
$$= 23.87''$$

Column-to-Gusset Connection

$$H_c = \frac{e_c}{r} P_u = \frac{5.4}{23.87}(73.6^k) = 16.65^k$$

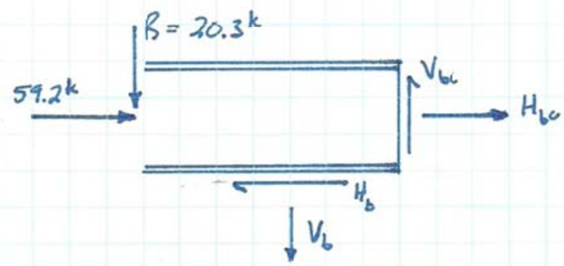
$$V_c = \frac{\beta}{r} P_u = \frac{10}{23.87}(73.6^k) = 30.83^k$$

Beam-to-Gusset Connection

$$H_b = \frac{\alpha}{r} P_u = \frac{9.25}{23.87}(73.6^k) = 28.52^k$$

$$V_b = \frac{e_b}{r} P_u = \frac{8.85}{23.87}(73.6^k) = 27.29^k$$

Beam-to-Column Connection



$$\Sigma F_x = 0 = 59.2 + H_{bc} - H_b$$

$$\therefore H_{bc} = 30.7^k$$

$$\Sigma F_y = 0 = V_{bc} - V_b - 20.3$$

$$\therefore V_{bc} = 47.6^k$$

Appendix E.2 – Braced Frame Design

Beam-to-column Connection

Use 3/4" ϕ A325N bolts

Shear Stress in Bolts

$\phi F_{nt} = 67.5^k$

$\phi F_{nv} = 40.5^k$

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{47.6}{6(0.442)} = 17.95^{ksi}$$

Available Tensile Strength per Bolt

$$F_t' = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{\left(67.5/0.75\right)}{40.5}(17.95)$$

$$F_t' = 77.11^{ksi} < 90^{ksi} \quad \therefore \text{OK}$$

$$\therefore \phi r_{nt} = \phi F_t' A_b = 0.75(77.11)(0.442) = 25.56^k$$

Calculate r_{ut}

$$r_{ut} = \frac{30.7^k}{6} = 5.12^k$$

Does Prying Occur?

$$p =_{min} \left| \begin{array}{l} gage = 3.806" \\ l_a/\#_{bolts} = 9"/3 = 3" \Rightarrow \text{controls} \end{array} \right.$$

$$\phi M_{n1} = \frac{\phi F_u p t^2}{4} = \frac{0.9(58)(3)(3/8)}{4} = 14.68$$

$$b = g - t/2 = 1.75 - \frac{3/8}{2} = 1.56"$$

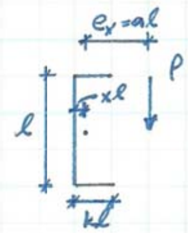
$$b' = b - d_b/2 = 1.56 - \frac{0.75}{2} = 1.19"$$

$$r_{ut} b' = 5.12^k (1.19") = 6.08 < \phi M_{n1} \quad \therefore \text{Prying does NOT Occur}$$

Appendix E.2 – Braced Frame Design

Weld Design (Angle)

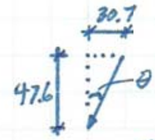
Table 8-8 ⇒ Use $\theta = 30°$



$l = 9''$
$kl = 2.5''$
$\therefore k = 0.2778$

	0.2	0.278	0.3
0.25	2.62	3.04	3.10
0.298		2.86	
0.3	2.45	2.85	2.91

$C = 2.86$



$\theta = 32.8°$

From Table 8-8
$k = 0.2 \Rightarrow x = 0.029$
$k = 0.3 \Rightarrow x = 0.056$
$\therefore k = 0.2778 \Rightarrow x = 0.035$

$$D_{min} = \frac{P_u}{\phi C C_1 l} = \frac{56.64}{0.75(2.86)(1.0)(9)} = 2.93/_{16ths}$$

$xl = 0.035(9) = 0.315''$

$al = 3'' - 0.315'' = 2.685''$
$\therefore a = 0.298''$

$$\boxed{\therefore \text{Use } 3/16'' \text{ Weld Both Sides}}$$

Angle Shear Yield

$$\phi R_n = \phi(0.6F_y)A_{gv} = 1.0(0.6)(36)(3/8)(9) = 72.9^k > 47.6^k \quad \therefore \underline{OK}$$

Angle Shear Rupture

$$\phi R_n = \phi(0.6F_u)A_{nv} = 0.75(0.6)(58)(3/8)(9 - 2(7/8)) = 71.0^k > 47.6^k \quad \therefore \underline{OK}$$

Bearing/Tear-Out (Angle)

$$\phi R_n = \phi[1.2 L_c F_u t] \le \phi[2.4 d_b F_u t] = 0.75[1.2(1.5 - \tfrac{7/8}{2})(58)(3/8)] \le [2.4(0.75)(58)(3/8)]$$
$$= 0.75[2(27.23) + 4(39.15)]$$

$$\phi R_n = 159^k > 47.6^k \quad \therefore \underline{OK}$$

Appendix E.2 – Braced Frame Design

Angle Block Shear

$$\phi R_n = \phi[0.6F_u A_{nv} + U_{bs} F_u A_{nt}] \le \phi[0.6F_y A_{gv} + U_{bs} F_u A_{nt}]$$
$$= 0.75[0.6(58)(1.992) + 1.0(58)(0.3047)] \le 0.75[0.6(36)(2.8125) + 1.0(58)(0.3047)]$$
$$= 58.8^k > 47.6^k \qquad \therefore OK$$

$$A_{nv} = 3/8\,(7.5 - 2.5(7/8)) = 1.992\ in^2$$
$$A_{gv} = 3/8\,(7.5'') = 2.8125\ in^2$$
$$A_{nt} = 3/8\,(1.25 - 0.5(7/8)) = 0.3047\ in^2$$

Gusset-to-Beam Connection

$$\phi R_n = 1.392\ D\ l(2) = 28.52^k \Rightarrow D = \frac{28.52}{1.392(17.5)(2)}$$

$$\therefore D_{min} = 0.585/16^{th}$$

$$D_{min} = 3/16'' \Rightarrow \text{Table J2.4}$$

$$\boxed{\therefore \text{Use } D = 3/16''}$$

Gusset – to – Column Connection

Shear Stress in Bolts

Use 3/4" ϕ A325 N Bolts

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{30.8^k}{6(0.442)} = 11.61^{ksi}$$

$\phi F_{nt} = 67.5^k$

$\phi F_{nv} = 40.5^k$

Available Tensile Strength per Bolt

$$F_t' = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{(67.5/0.75)}{40.5}(11.61) = 91.2 > 90 \quad \therefore NG$$

Try 4 bolts

Shear Stress in Bolts

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{30.8}{4(0.442)} = 17.42 \text{ ksi}$$

Available Tensile Strength per Bolt

$$F_t' = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{(67.5/0.75)}{40.5}(17.42) = 78.3 < 90 \quad \therefore OK$$

$$\therefore \phi r_{nt} = \phi F_t' A_b = 0.75(78.3)(0.442) = 25.95^k > 16.65^k \quad \therefore OK$$

Calculate r_{ut}

$$r_{ut} = \frac{16.65}{4} = 4.16^k$$

Does Prying Occur?

$$p = \min \begin{cases} \text{gage} = 3.806'' \\ L/\#_{bolts} = 6''/2 = 3'' \Rightarrow \text{Controls} \end{cases}$$

$$b = g - t/2 = 1.75 - \tfrac{3/8}{2} = 1.56''$$

$$\phi M_{n1} = \frac{\phi F_u p t^2}{4} = \frac{0.9(58)(3)(3/8)^2}{4} = 14.68$$

$$b' = b - d_b/2 = 1.56 - \tfrac{0.75}{2} = 1.19$$

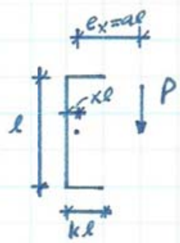
$$r_{ut} b' = 4.16(1.19) = 4.95 < \phi M_{n1}$$

$\therefore$ Prying Does NOT Occur

Appendix E.2 – Braced Frame Design

Weld Design (Angle)

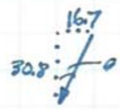
Table 8-8 ⇒ Use $\theta = 30°$



$l = 9''$
$kl = 2.5''$
$\therefore k = 0.2778$

	0.2	0.278	0.3
0.25	2.62	3.04	3.10
0.298		2.86	
0.3	2.45	2.85	2.91

$C = 2.86$



$\theta = 28.5°$

From Table 8-8
$k = 0.2 \Rightarrow x = 0.029$
$k = 0.3 \Rightarrow x = 0.036$

$\therefore k = 0.2778 \Rightarrow x = 0.035$

$xl = 0.035(9) = 0.315$

$al = 3'' - 0.315 = 2.685$
$\therefore a = 0.298$

$$D_{min} = \frac{P_u}{\phi C C_1 l} = \frac{36.04}{0.75(2.86)(1.0)(9)} = 1.81/16^{th}$$

To Satisfy min weld sizes

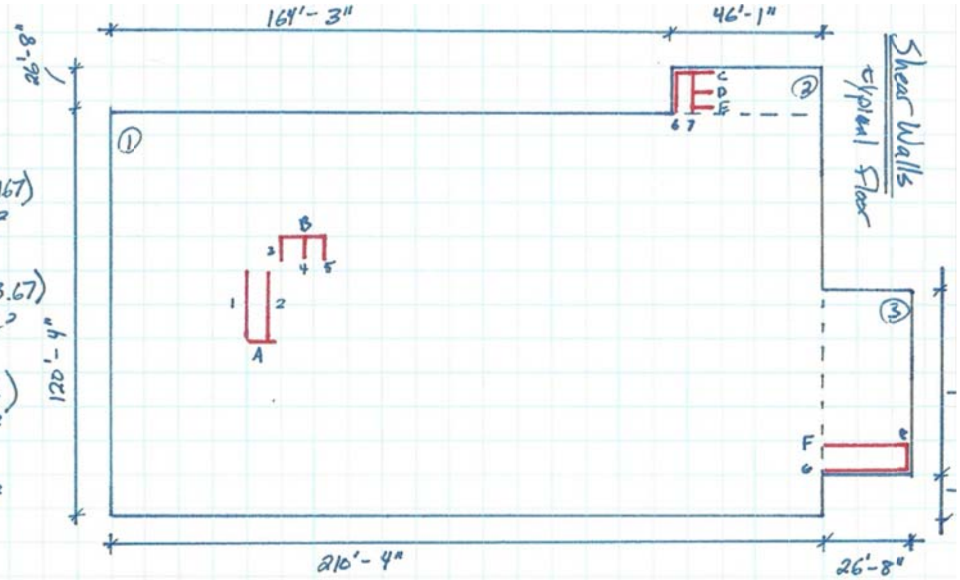
$\boxed{\text{Use } 3/16'' \text{ Weld Both Sides}}$

Angle Shear Yield, Shear Rupture, Bearing/Tearout, & Block Shear

OK by previous work

Appendix F

Appendix F.1 – Shear Wall CM v. CR



Center of Mass

① $CM = (105.167, 60.167)$
$A = 25{,}310 \text{ ft}^2$

② $CM = (187.292, 133.67)$
$A = 1{,}228.9 \text{ ft}^2$

③ $CM = (223.67, 56)$
$A = 1{,}422.2 \text{ ft}^2$

$\Sigma A = 27{,}961.1 \text{ ft}^2$

$$R_x = \frac{\Sigma M_i r_{ix}}{\Sigma A}$$

$$= \frac{25{,}310(105.167) + 1228.9(187.292) + 1422.2(223.67)}{27{,}961.1}$$

$R_x = 114.8 \text{ ft}$

$$R_y = \frac{\Sigma M_i r_{iy}}{\Sigma A}$$

$$= \frac{25{,}310(60.167) + 1228.9(133.67) + 1422.2(56)}{27{,}961.1}$$

$R_y = 63.2 \text{ ft}$

$\boxed{CM = 114.8, 63.2}$

Appendix F.1 – Shear Wall CM v. CR

Center of Rigidity

Wall Stiffness is proportional to length because all walls have the same thickness

X-Walls

	Length	X_i
1	23.67	24.67
2	23.67	33.33
3	10.55	32.00
4	10.55	46.67
5	10.55	49.33
6	26.67	164.25
7	26.67	173.33
8	10.67	180.00
	143.00	

Y-Walls

	Length	y_i
A	8.67	57.00
B	17.33	100.38
C	19.71	147.00
D	10.63	138.00
E	10.63	129.00
F	26.67	40.00
G	26.67	29.33
	120.31	

$$CR_x = \frac{\Sigma R_x X_i}{\Sigma R_x} = \frac{13{,}583.8}{143} = 95.0 \text{ ft}$$

$$CR_y = \frac{\Sigma R_y Y_i}{\Sigma R_y} = \frac{9818.4}{120.31} = 81.6 \text{ ft}$$

$$\boxed{CR = 95.0,\ 81.6}$$

Eccentricity

$$e_x = 114.8' - 95' = 19.8'$$

$$e_y = 81.6' - 63.2' = 18.4'$$

Appendix F.2 – Shear Wall Load Calcs

Superstructure Weight at UPMC Hamot Womens Hospital (Shear Walls)

Gravity Beams Weight		
Penthouse	58.4 kips	29.2 tons
Elev Roof	1.0 kips	0.5 tons
Roof	135.0 kips	67.5 tons
5th	128.9 kips	64.5 tons
4th	129.8 kips	64.9 tons
3rd	134.4 kips	67.2 tons
2nd	132.6 kips	66.3 tons
Total	720.1 kips	360.0 tons
Gravity Column Weight		
Total	266.5 kips	133.3 tons
TOTAL STEEL WEIGHT	**986.6 kips**	**493.3 tons**

Shear Wall Weight

Wall	Length	Width	Height	Volume (ft^3)	Weight (k)	Volume (yd^3)
1	23.7	0.667	92	1451.8	218	53.8
2	23.7	0.667	92	1451.8	218	53.8
3	10.6	0.667	92	647.1	97	24.0
4	10.6	0.667	92	647.1	97	24.0
5	10.6	0.667	92	647.1	97	24.0
6	26.7	0.667	79.5	1413.5	212	52.4
7	26.7	0.667	79.5	1413.5	212	52.4
8	10.7	0.667	92	654.4	98	24.2
A	8.7	0.667	92	531.8	80	19.7
B	17.3	0.667	92	1062.9	159	39.4
C	19.7	0.667	79.5	1044.6	157	38.7
D	10.6	0.667	79.5	563.4	85	20.9
E	10.6	0.667	79.5	563.4	85	20.9
F	26.7	0.667	92	1635.8	245	60.6
G	26.7	0.667	92	1635.8	245	60.6
					2305	569.0

Appendix F.2 – Shear Wall Load Calcs

Story Forces at UPMC Hamot Womens Hospital (Shear Walls)

X-Direction

	Floor Level							
Load Method	**2nd**	**3rd**	**4th**	**5th**	**Roof**	**Elev Roof**	**Penthouse**	**Base Shear**
W1	0.14	0.03	-0.04	-0.08	-0.14	0.00	0.00	-0.09
W2	0.07	0.02	-0.01	-0.03	-0.03	0.00	0.03	0.05
W3	0.14	0.03	-0.04	-0.09	-0.18	0.00	-0.02	-0.16
W4	48.34	60.67	60.58	58.65	50.00	7.62	24.05	309.91
W5	36.27	45.50	45.43	43.98	37.48	5.71	18.03	232.40
W6	36.25	45.50	45.44	44.00	37.52	5.71	18.04	232.46
E1	18.10	50.15	77.51	102.98	139.10	0.00	39.64	427.48
E2	18.08	50.14	77.51	103.00	139.14	0.00	39.65	**427.52**
E3	0.10	0.03	-0.03	-0.06	-0.09	0.00	0.00	-0.05
E4	0.14	0.04	-0.04	-0.08	-0.16	0.00	-0.01	-0.11

Y-Direction

	Floor Level							
Load Method	**2nd**	**3rd**	**4th**	**5th**	**Roof**	**Elev Roof**	**Penthouse**	**Base Shear**
W1	152.99	144.50	124.86	113.05	122.64	7.73	66.25	**732.02**
W2	114.83	108.42	93.64	84.75	91.90	5.79	49.68	549.01
W3	114.66	108.34	93.65	84.82	92.07	5.79	49.69	549.02
W4	0.05	0.01	-0.02	-0.03	-0.05	0.00	0.00	-0.04
W5	0.02	0.00	-0.01	-0.01	-0.01	0.00	0.00	-0.01
W6	0.06	0.02	-0.01	-0.03	-0.06	0.00	0.00	-0.02
E1	0.09	0.02	-0.03	-0.05	-0.09	0.00	-0.01	-0.07
E2	0.12	0.04	-0.03	-0.07	-0.13	0.00	-0.01	-0.08
E3	21.12	56.37	85.70	112.81	151.30	0.00	42.79	470.09
E4	21.06	56.34	85.70	112.83	151.37	0.00	42.80	470.10

Load Methods Explained:

W1 = Wind Parallel to Y-Axis according to ASCE 7-05
W2 = Wind Parallel to Y-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
W3 = Wind Parallel to Y-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
W4 = Wind Parallel to X-Axis according to ASCE 7-05
W5 = Wind Parallel to X-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
W6 = Wind Parallel to X-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
E1 = Earthquake Paralled to X-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
E2 = Earthquake Paralled to X-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity
E3 = Earthquake Paralled to Y-Axis according to ASCE 7-05 w/ Positive (+) Eccentricity
E4 = Earthquake Paralled to Y-Axis according to ASCE 7-05 w/ Negative (-) Eccentricity

Appendix F.2 – Shear Wall Load Calcs

Shear Wall Load Calculator

"Diaphragms"

	CMx	CMy	Area
1	105.2	60.2	25310.0
2	187.3	133.7	1228.9
3	223.7	56.0	1422.2

N-S Walls

	Length	xi	Rixi	di	Ridi	Ridi^2	J
1	23.7	24.7	583.9	-70.3	-1664.516	117051.6	943122.6
2	23.7	33.3	788.9	-61.7	-1459.533	89997.36	
3	10.6	32.0	337.6	-63.0	-664.5628	41861.97	
4	10.6	40.7	429.1	-54.3	-573.0943	31131.48	
5	10.6	49.3	520.4	-45.7	-481.7313	21996.69	
6	26.7	164.3	4380.5	69.3	1847.118	127928.2	
7	26.7	173.3	4622.7	78.3	2089.281	163670.7	
8	10.7	180.0	1920.6	85.0	907.0382	77105.74	
	143.0		13583.8				

E-W Walls

	Length	yi	Riyi	di	Ridi	Ridi^2
A	8.7	57.0	494.2	-24.6	-213.3606	5250.604
B	17.3	100.4	1739.6	18.8	325.3003	6106.191
C	19.7	147.0	2897.4	65.4	1288.855	84279.46
D	10.6	138.0	1466.9	56.4	599.4356	33802.74
E	10.6	129.0	1371.3	47.4	503.7656	23873.93
F	26.7	40.0	1066.8	-41.6	-1109.714	46174.15
G	26.7	29.3	782.2	-52.3	-1394.283	72891.79
	120.3		9818.4			

Center of Mass

CMx	CMy
114.8	63.2

Center of Rigidity

CRx	CRy
95.0	81.6

Eccentricity

ex	ey
19.8	18.4

Shear Loads	X-Direction	Y-Direction
Penthouse	39.44	66.30
Elev Roof	0.00	7.73
Roof	137.84	121.17
5th	102.06	112.04
4th	76.88	124.14
3rd	50.23	144.49
2nd	19.22	154.14

Appendix F.2 – Shear Wall Load Calcs

Direct Shear Loads (X)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
1	23.7	0.0	25.514	23.917	20.548	18.545	20.057	1.280	10.974
2	23.7	0.0	25.514	23.917	20.548	18.545	20.057	1.280	10.974
3	10.6	0.0	11.372	10.660	9.159	8.266	8.939	0.570	4.891
4	10.6	0.0	11.372	10.660	9.159	8.266	8.939	0.570	4.891
5	10.6	0.0	11.372	10.660	9.159	8.266	8.939	0.570	4.891
6	26.7	0.0	28.748	26.948	23.153	20.896	22.599	1.442	12.365
7	26.7	0.0	28.748	26.948	23.153	20.896	22.599	1.442	12.365
8	10.7	0.0	11.501	10.781	9.263	8.360	9.041	0.577	4.947
A	0.0	8.7	0.000	0.000	0.000	0.000	0.000	0.000	0.000
B	0.0	17.3	0.000	0.000	0.000	0.000	0.000	0.000	0.000
C	0.0	19.7	0.000	0.000	0.000	0.000	0.000	0.000	0.000
D	0.0	10.6	0.000	0.000	0.000	0.000	0.000	0.000	0.000
E	0.0	10.6	0.000	0.000	0.000	0.000	0.000	0.000	0.000
F	0.0	26.7	0.000	0.000	0.000	0.000	0.000	0.000	0.000
G	0.0	26.7	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Direct Shear Loads (Y)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
1	120.3	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	0.0	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	0.0	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	CMx	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	114.8	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6	0.0	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	0.0	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	CRx	0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
A	0.0	19.8	1.385	3.620	5.540	7.355	9.933	0.000	2.842
B	0.0	0.0	2.769	7.235	11.074	14.701	19.855	0.000	5.681
C	0.0	0.0	3.149	8.229	12.595	16.720	22.582	0.000	6.461
D	0.0	0.0	1.698	4.438	6.793	9.018	12.179	0.000	3.485
E	0.0	0.0	1.698	4.438	6.793	9.018	12.179	0.000	3.485
F	0.0	0.0	4.261	11.135	17.043	22.624	30.556	0.000	8.743
G	0.0	0.0	4.261	11.135	17.043	22.624	30.556	0.000	8.743

Torsional Shear Loads (X-Direction Loads)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
1	23.7	0.0	-0.672	-1.756	-2.688	-3.569	-4.820	0.000	-1.379
2	23.7	0.0	-0.589	-1.540	-2.357	-3.129	-4.226	0.000	-1.209
3	10.6	0.0	-0.268	-0.701	-1.073	-1.425	-1.924	0.000	-0.551
4	10.6	0.0	-0.231	-0.605	-0.926	-1.229	-1.659	0.000	-0.475
5	10.6	0.0	-0.195	-0.508	-0.778	-1.033	-1.395	0.000	-0.399
6	26.7	0.0	0.746	1.949	2.983	3.960	5.349	0.000	1.530
7	26.7	0.0	0.844	2.205	3.374	4.479	6.050	0.000	1.731
8	10.7	0.0	0.366	0.957	1.465	1.945	2.626	0.000	0.751
A	0.0	8.7	-0.086	-0.225	-0.345	-0.457	-0.618	0.000	-0.177
B	0.0	17.3	0.131	0.343	0.525	0.697	0.942	0.000	0.270
C	0.0	19.7	0.520	1.360	2.082	2.763	3.732	0.000	1.068
D	0.0	10.6	0.242	0.633	0.968	1.285	1.736	0.000	0.497
E	0.0	10.6	0.203	0.532	0.814	1.080	1.459	0.000	0.417
F	0.0	26.7	-0.448	-1.171	-1.792	-2.379	-3.213	0.000	-0.919
G	0.0	26.7	-0.563	-1.471	-2.252	-2.989	-4.037	0.000	-1.155

Appendix F.2 – Shear Wall Load Calcs

Torsional Shear Loads (Y-Direction Loads)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse
1	23.7	0.0	-5.012	-4.698	-4.036	-3.643	-3.940	-0.251	-2.156
2	23.7	0.0	-4.395	-4.120	-3.539	-3.194	-3.455	-0.220	-1.890
3	10.6	0.0	-2.001	-1.876	-1.612	-1.455	-1.573	-0.100	-0.861
4	10.6	0.0	-1.726	-1.618	-1.390	-1.254	-1.357	-0.087	-0.742
5	10.6	0.0	-1.451	-1.360	-1.168	-1.054	-1.140	-0.073	-0.624
6	25.7	0.0	5.562	5.214	4.479	4.043	4.372	0.279	2.392
7	25.7	0.0	6.291	5.897	5.067	4.573	4.945	0.315	2.706
8	10.7	0.0	2.731	2.560	2.200	1.985	2.147	0.137	1.175
A	0.0	8.7	-0.642	-0.602	-0.517	-0.467	-0.505	-0.032	-0.276
B	0.0	17.3	0.980	0.918	0.789	0.712	0.770	0.049	0.421
C	0.0	19.7	3.881	3.638	3.126	2.821	3.051	0.195	1.669
D	0.0	10.6	1.805	1.692	1.454	1.312	1.419	0.091	0.776
E	0.0	10.6	1.517	1.422	1.222	1.103	1.192	0.076	0.652
F	0.0	26.7	-3.341	-3.132	-2.691	-2.429	-2.627	-0.168	-1.437
G	0.0	26.7	-4.198	-3.935	-3.381	-3.052	-3.300	-0.211	-1.806

Total Shear Load (X)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear
1	23.7	0.0	24.842	22.160	17.860	14.977	15.237	1.280	9.595	105.950
2	23.7	0.0	24.925	22.377	18.191	15.416	15.830	1.280	9.765	107.783
3	10.6	0.0	11.104	9.959	8.085	6.841	7.015	0.570	4.341	47.915
4	10.6	0.0	11.140	10.055	8.233	7.037	7.280	0.570	4.417	48.733
5	10.6	0.0	11.177	10.152	8.381	7.233	7.545	0.570	4.492	49.550
6	25.7	0.0	29.493	28.897	26.136	24.856	27.947	1.442	13.896	152.666
7	25.7	0.0	29.591	29.152	26.527	25.375	28.648	1.442	14.096	154.832
8	10.7	0.0	11.867	11.738	10.728	10.305	11.668	0.577	5.698	62.581
A	0.0	8.7	-0.086	-0.225	-0.345	-0.457	-0.618	0.000	-0.177	1.908
B	0.0	17.3	0.131	0.343	0.525	0.697	0.942	0.000	0.270	2.909
C	0.0	19.7	0.520	1.360	2.082	2.763	3.732	0.000	1.068	11.525
D	0.0	10.6	0.242	0.633	0.968	1.285	1.736	0.000	0.497	5.360
E	0.0	10.6	0.203	0.532	0.814	1.080	1.459	0.000	0.417	4.505
F	0.0	26.7	-0.448	-1.171	-1.792	-2.379	-3.213	0.000	-0.919	9.923
G	0.0	26.7	-0.563	-1.471	-2.252	-2.989	-4.037	0.000	-1.155	12.468

Total Shear Load (Y)

Wall	Length (x)	Length (y)	2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear
1	23.7	0.0	-5.012	-4.698	-4.036	-3.643	-3.940	-0.251	-2.156	23.737
2	23.7	0.0	-4.395	-4.120	-3.539	-3.194	-3.455	-0.220	-1.890	20.814
3	10.6	0.0	-2.001	-1.876	-1.612	-1.455	-1.573	-0.100	-0.861	9.477
4	10.6	0.0	-1.726	-1.618	-1.390	-1.254	-1.357	-0.087	-0.742	8.173
5	10.6	0.0	-1.451	-1.360	-1.168	-1.054	-1.140	-0.073	-0.624	6.870
6	26.7	0.0	5.562	5.214	4.479	4.043	4.372	0.279	2.392	26.341
7	26.7	0.0	6.291	5.897	5.067	4.573	4.945	0.315	2.706	29.794
8	10.7	0.0	2.731	2.560	2.200	1.985	2.147	0.137	1.175	12.935
A	0.0	8.7	0.743	3.018	5.023	6.888	9.428	-0.032	2.566	27.633
B	0.0	17.3	3.748	8.154	11.863	15.413	20.625	0.049	6.102	65.954
C	0.0	19.7	7.030	11.867	15.721	19.541	25.633	0.195	8.131	88.116
D	0.0	10.6	3.503	6.130	8.246	10.329	13.598	0.091	4.261	46.158
E	0.0	10.6	3.215	5.860	8.014	10.120	13.371	0.076	4.137	44.794
F	0.0	26.7	0.919	8.003	14.351	20.196	27.929	-0.168	7.306	78.536
G	0.0	26.7	0.062	7.199	13.661	19.573	27.256	-0.211	6.937	74.478

Appendix F.2 – Shear Wall Load Calcs

Controlling Shear Loads

Wall	Length (x)	Length (y)		2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse	Base Shear	Moment at Base	Moment at 4th	Moment at Roof
1	23.7	0.0		24.842	22.160	17.860	14.977	15.237	1.280	9.595	105.950	4654.6	1142.3	201.5
2	23.7	0.0		24.925	22.377	18.191	15.416	15.830	1.280	9.765	107.783	4760.1	1173.2	204.9
3	10.6	0.0		11.104	9.959	8.085	6.841	7.015	0.570	4.341	47.915	2114.4	520.8	91.1
4	10.6	0.0		11.140	10.055	8.233	7.037	7.280	0.570	4.417	48.733	2161.5	534.6	92.6
5	10.6	0.0		11.177	10.152	8.381	7.233	7.545	0.570	4.492	49.550	2208.5	548.4	94.1
6	26.7	0.0		29.493	28.897	26.136	24.856	27.947	1.442	13.895	152.666	7159.9	1848.7	288.7
7	26.7	0.0		29.591	29.152	26.527	25.375	28.648	1.442	14.095	154.832	7284.4	1885.2	292.7
8	10.7	0.0		11.867	11.738	10.728	10.305	11.668	0.577	5.698	62.581	2950.9	765.0	118.3
A	0.0	8.7		0.743	3.018	5.023	6.888	9.428	-0.032	2.565	27.633	1626.2	482.4	51.1
B	0.0	17.3		3.748	8.154	11.863	15.413	20.625	0.049	6.102	65.954	3739.5	1087.9	122.4
C	0.0	19.7		7.030	11.867	15.721	19.541	25.633	0.195	8.131	88.116	4850.7	1388.5	164.1
D	0.0	10.6		3.503	6.130	8.246	10.329	13.598	0.091	4.261	46.158	2553.9	733.1	85.9
E	0.0	10.6		3.215	5.860	8.014	10.120	13.371	0.076	4.137	44.794	2491.7	717.4	83.3
F	0.0	26.7		0.919	8.003	14.351	20.196	27.929	-0.168	7.305	78.536	4707.6	1409.5	144.9
G	0.0	26.7		0.062	7.199	13.661	19.573	27.256	-0.211	6.937	74.478	4522.5	1362.7	137.2
			Floor Height	12	28	44	58	72	79.5	92				

Deflection Checks

Wall

Wall										
8	Wall Thickness	8 in	f'c =	4000 psi	E =	3604997 psi	I =	1399412 in^4		
	Wall Length	128.04 in								
		2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse		
	Deflection Contribution	0.0258	0.1303	0.2751	0.4314	0.7044	0.0409	0.5065	Total Deflection	2.114 in

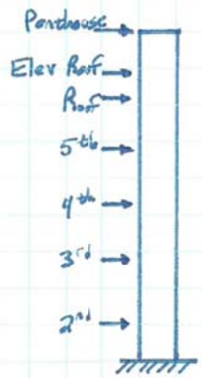
Wall										
C	Wall Thickness	8 in	f'c =	4000 psi	E =	3604997 psi	I =	8820889 in^4		
	Wall Length	236.52 in								
		2nd	3rd	4th	5th	Roof	Elev Roof	Penthouse		
	Deflection Contribution	0.0024	0.0209	0.0639	0.1298	0.2455	0.0022	0.1147	Total Deflection	0.579 in

Appendix F.3 – Shear Wall Designs

Shear Wall Design

Design Shear Wall #8 ⇒ Greatest Base Shear per Length

Assume 8" Wall for Full Height + Check Deflection



$\Delta_{top} = 2.114''$ ⇒ Earthquake Loads Control this Direction

$$\delta_x = \frac{C_d \Delta_{top}}{I} = \frac{4(2.114)}{1.25} = 6.848''$$

$$\Delta_{max} = 0.015\,h = 0.015(92 \times 12) = 16.56''$$

$\delta_x < \Delta_{max}$ $\quad \therefore$ OK

Wall Design @ Base

$$V_u = 1.6\,V_L = 1.6(62.581) = 100.13^k$$

$$\phi V_{n,max} = \phi 10 \sqrt{f'_c}\, h\, d = 0.75(10)\sqrt{4000}\; 8(102.4)/1000 = 388.6^k > V_u \quad \therefore OK$$

$$h = 8''$$

$$d = 0.8\,l_w = 0.8(128.04'') = 102.4''$$

$$V_c = 2\sqrt{f'_c}\, h\, d = 2\sqrt{4000}\; 8(102.4)/1000 = 103.6^k$$

$$V_u \geq \phi(V_c + V_s) \rightarrow 100.13 = 0.75(103.6 + V_s)$$

$$\therefore V_s \geq 29.9^k$$

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

$$V_s = \frac{A_v F_y d}{s} \Rightarrow \frac{A_v}{s} = \frac{V_s}{f_y d} = \frac{29.9^k}{60(102.4)} = 0.00487 \text{ in}$$

Try 2-#4's $\quad s = \frac{A_v}{0.00487} = \frac{2(0.2)}{0.00487} = 82.2'' \rightarrow \therefore \text{Use } S_{max} = 18''$

$$\frac{A_v}{s} = \frac{2(0.2)}{18} = 0.022 \text{ in} > 0.00478 \quad \therefore OK$$

$\boxed{\therefore \text{Use 2-\#4's @ 18'' O.C.}}$

Vertical Reinforcement

$$\rho_\ell = \frac{A_v}{sh} \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_t - 0.0025)$$

$$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{18(8)} = 0.00277$$

$$= 0.0025 + 0.5\left(2.5 - \frac{12}{10.7}\right)(0.00277 - 0.0025) = 0.0025$$

$$\frac{A_v}{s} = \rho_\ell h = 0.0025(8) = 0.02 \text{ in}$$

Try 2-#4's @ 18" O.C.

$$\frac{A_v}{s} = \frac{2(0.2)}{18} = 0.0222 \text{ in} \geq 0.02 \quad \therefore OK$$

$\boxed{\therefore \text{Use 2-\#4's @ 18'' O.C}}$

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

Flexural Design

$$M_u = 1.6(2950.9) = 4721.4 \text{ ft-k}$$

$$M_n = A_s f_y (d - a/2) = A_s f_y \, jd$$

Let $jd = 0.9d = 0.9(102.4) = 92.16''$

$$M_u = \phi M_n = \phi A_s f_y \, jd = 0.9(A_s)(60)(92.16) = 4721.6(12)$$

$$\therefore A_s \geq 11.39 \text{ in}^2$$

$$0.85(f'_c)ab = A_s f_y$$

$$0.85(4)(a)(8) = 11.39(60)$$

$$\therefore a = 25.11''$$

$$jd = d - a/2 = 102.4 - 25.11/2 = 89.84''$$

$$M_u = \phi M_n = \phi A_s f_y \, jd \Rightarrow 0.9 A_s (60)(89.84) = 4721.6(12)$$

$$\therefore A_s = 11.67 \text{ in}^2$$

$$\boxed{\therefore \text{Use } 12\text{-}\#9\text{'} \text{ @ Each End}} \Rightarrow A_s = 12 \text{ in}^2$$

$$0.85 f'_c ab = A_s f_y \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{12(60)}{0.85(4)(8)}$$

$$\therefore a = 26.47''$$

$$c = a/\beta_1 = \frac{26.47}{0.85} = 31.14$$

$$\varepsilon_t = \varepsilon_u \frac{d - c}{c} = 0.003\left(\frac{121.6875 - 31.14}{31.14}\right) = 0.0087 > 0.005$$

$\therefore$ OK $\Rightarrow$ Tension Controlled Section

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

Wall Design @ 4th Floor

$$V_u = 1.6\ V_y = 1.6(28.248) = 45.2^k$$

$$\phi V_{n,max} = \phi\, 10\sqrt{f'_c}\ h\ d = 0.75(10)\sqrt{4000}\ 8(102.4)/_{1000} = 388.6^k > V_u \quad \therefore OK$$

$h = 8''$

$d = 0.8\, l_w = 0.8(128'') = 102.4''$

$$V_c = 2\sqrt{f'_c}\ h\ d = 2\sqrt{4000}\ 8(102.4)/_{1000} = 103.6^k$$

$$V_u \geq \phi(V_c + V_s) \Rightarrow 45.2^k = 0.75(103.6 + V_s)$$

$\therefore V_s = 0.0 \Rightarrow \therefore$ Use Temp + Shrinkage Reinf.

$$\rho_t = 0.0025 = \frac{A_s}{sh} = \frac{0.2(2)}{18(8)} = 0.00278 > 0.0025 \quad \therefore OK$$

$\boxed{\text{Use } 2\text{-}\#4\text{'s @ } 18''\text{ O.C.}} \Rightarrow$ Horizontally

Vertical Reinforcement

$$\rho_\ell = \frac{A_v}{sh} \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025)$$
$$= 0.0025 + 0.5\left(2.5 - \frac{12}{10.7}\right)(0.00278 - 0.0025)$$

$$\rho_\ell = 0.00269 = \frac{A_v}{sh} \rightarrow \frac{A_v}{s} = \rho_\ell(h) = 0.00269(8) = 0.0215$$

$\boxed{\text{Use } 2\text{-}\#4\text{'s @ } 18''\text{ O.C.}} \rightarrow$ Vertically

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

Flexural Design

$M_u = 1.6(765.0) = 1224 \text{ ft-k}$

$M_n = A_s F_y (d - a/2) = A_s F_y \, jd$

Let $jd = 0.9d = 0.9(102.4) = 92.16''$

$M_u = \phi M_n = \phi A_s F_y \, jd = 0.9 A_s (60)(92.16) = 1224(12)$

$\therefore A_s \geq 2.95 \text{ in}^2$

$0.85 f'_c \, ab = A_s F_y \Rightarrow 0.85(4)a(8) = 2.95(60)$

$\therefore a = 6.51''$

$jd = d - a/2 = 102.4 - 6.51/2 = 99.14''$

$M_u = \phi M_n = \phi A_s F_y \, jd \Rightarrow 0.9 A_s (60)(99.14) = 1224(12)$

$\therefore A_s = 2.74 \text{ in}^2$

$\boxed{\therefore \text{Use } 4\text{-}\#9\text{'s @ Each End}} \rightarrow A_s = 4 \text{ in}^2$

$0.85 f'_c \, ab = A_s F_y \rightarrow a = \frac{A_s F_y}{0.85 f'_c b} = \frac{4(60)}{0.85(4)(8)} = 8.82''$

$c = a/\beta_1 = \frac{8.82''}{0.85} = 10.38''$

$\varepsilon_t = \varepsilon_u \frac{d-c}{c} = 0.003 \left(\frac{125.5875 - 10.38}{10.38} \right) = 0.033 > 0.005$

$\therefore$ OK $\Rightarrow$ Tension Controlled Section

Appendix F.3 – Shear Wall Designs

Shear Wall Design

Design Shear Wall #C → Greatest Base shear per Length

Assume 8" Wall for Full Height + Check Deflection

$\Delta_{top} = 0.579"$ → Wind Loads Control this Direction

$\Delta_{max} = h/600 = 1.84"$

$\Delta_{top} < \Delta_{max}$ $\therefore$ OK

Wall Design @ Base

$V_u = 1.6 V_b = 1.6(88.116) = 140.99^k$

$\phi V_{n,max} = \phi 10 \sqrt{f'_c}\ hd = 0.75(10)\sqrt{4000}\ (8)(189.2)/1000 = 718^k > V_u$ $\therefore$ OK

$h = 8"$
$d = 0.8\ l_w = 0.8(236.5") = 189.2"$

$V_c = 2\sqrt{f'_c}\ hd = 2\sqrt{4000}\ (8)(189.2)/1000 = 191.46^k$

$V_u \geq \phi(V_c + V_s) \Rightarrow 140.99 = 0.75(191.46 + V_s)$

$\therefore V_s = 0.0^k \Rightarrow \therefore$ Use Temp + Shrinkage Reinf.

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

$$\rho_t = 0.0025 = \frac{A_s}{sh} = \frac{0.2(2)}{18(8)} = 0.00278 > 0.0025 \quad \therefore OK$$

$\boxed{\text{Use 2-\#4's @ 18" o.c.}}$ ⇒ Horizontally

Vertical Reinforcement

$$\rho_\ell = \frac{A_v}{sh} \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_t - 0.0025)$$
$$= 0.0025 + 0.5\left(2.5 - 12/10.7\right)(0.00278 - 0.0025)$$

$$\rho_\ell = 0.00269 = \frac{A_v}{sh} \Rightarrow \frac{A_v}{s} = \rho_\ell h = 0.00269(8) = 0.0215$$

$\boxed{\therefore \text{Use 2-\#4's @ 18" o.c.}}$ ⇒ Vertically

Flexural Design

$$M_u = 1.6(4850.7) = 7761.1^{k\text{-}ft}$$

$$M_n = A_s f_y (d - a/2) = A_s f_y \, jd$$

Let $jd = 0.9d = 0.9(189.2) = 170.3"$

$$M_u = \phi M_n = \phi A_s f_y jd \Rightarrow 0.9(A_s)(60)(170.3) = 7761.1(12)$$

$$\therefore A_s = 10.13 \text{ in}^2$$

$$0.85 f'_c a b = A_s f_y \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{10.13(60)}{0.85(4)(8)}$$

$$\therefore a = 22.34"$$

$$jd = d - a/2 = 189.2" - 22.34/2 = 178.0"$$

Appendix F.3 – Shear Wall Designs

Shear Wall Design (cont)

$M_u = \phi M_n = \phi A_s f_y jd \Rightarrow 0.9 A_s(60)(178.0) = 7761.1\ (12)$

$\therefore A_s = 9.69\ in^2$

$\boxed{\therefore \text{Use } 10 - \#9\text{'s @ Each End}} \Rightarrow A_s = 10\ in^2$

$0.85\ f'_c ab = A_s f_y \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{10(60)}{0.85(4)(8)} = 22.06''$

$c = a/\beta_1 = \frac{22.06}{0.85} = 25.95''$

$\varepsilon_t = \varepsilon_u \frac{d-c}{c} = 0.003\left(\frac{230.6875 - 25.95}{25.95}\right) = 0.0237 > 0.005$

$\therefore$ OK $\Rightarrow$ Tension Controlled Section

Appendix F.3 – Shear Wall Designs

Shear Wall Designs (cont.)

Wall Design @ 4th Floor

$V_u = 1.6 V_4 = 1.6(53.5) = 85.6^k$

$\phi V_{n,max} = 78^k \Rightarrow$ From Previous $\therefore$ OK

$V_c = 2\sqrt{f'_c}\,hd = 2\sqrt{4000}\,(8)(189.2)/1000 = 191.46^k$

$V_u \geq \phi(V_c + V_s) \rightarrow 85.6 = 0.75(191.46 + V_s)$

$\therefore V_s = 0.0^k \Rightarrow \therefore$ Use Temp + Shrinkage Reinf.

By Inspection $\boxed{\text{Use } 2\text{-}\#4\text{'s @ } 18\text{" O.C.}}$ $\Rightarrow$ Horizontally + Vertically

Flexural Design

$M_u = 1.6(1388.5) = 2221.6$ ft-k

$M_n = A_s f_y (d - a/2) = A_s f_y\, jd$

Let $jd = 0.9d = 0.9(189.2) = 170.3"$

$M_u = \phi M_n = \phi A_s f_y\, jd = 0.9 A_s (60)(170.3) = 2221.6(12)$

$\therefore A_s = 2.90$ in^2

$0.85 f'_c\, ab = A_s f_y \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.9(60)}{0.85(4)(8)}$

$\therefore a = 6.39"$

$jd = d - a/2 = 189.2 - 6.39/2 = 186.0"$

Appendix F.3 – Shear Wall Designs

Shear Wall Designs (cont)

$$M_u = \phi M_n = \phi A_s f_y \, jd = 0.9(A_s)(60)(186) = 2221.6(12)$$

$$\therefore A_s = 2.65 \text{ in}^2 \Rightarrow \text{Use } 4\text{-}\#9 \Rightarrow A_s = 4.0 \text{ in}^2$$

$$0.85 f'_c ab = A_s f_y \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.0(60)}{0.85(4)(8)}$$

$$\therefore a = 8.82''$$

$$c = a/\beta_1 = 8.82/0.85 = 10.38''$$

$$\varepsilon_t = \varepsilon_u \frac{d-c}{c} = 0.003\left(\frac{233.6875 - 10.38}{10.38}\right) = 0.0645 > 0.005$$

$\therefore$ OK $\Rightarrow$ Tension Controlled Section

Appendix G

Appendix G.1 – RS Means Data and Analysis (General)

Historical Cost Index

Jan	2012	100
July	2011	96.3
	2010	95.2
	2009	93.4
	→ 2008	93.6
	2007	87.9
	2006	84.0
	2005	78.6
	2004	74.5
	2003	68.5
	2002	66.8

Relate current prices to previous prices

$$\frac{\text{Index Old}}{\text{Index 2012}} \times 2012 \text{ Cost} = \text{Old year Cost}$$

	Mat	Inst	Total
Location Factors	94.6	93.6	94.1

Size Factor

$$\frac{\text{Proposed Area}}{\text{Typical Area}}$$

Hospital Typical Area = 55,000 SF

RS Means Building Construction Cost Data – 2012

Library Code TH435. B64

Appendix G.2 – RS Means Data and Analysis (Structural Steel)

Hospitals, Steel Bearing, 3 to 6 stories

Crew	E-6
Daily Output	14.40
Labor-Hours	8.889
Unit	Ton
2012 Bare Costs	
Material	2,550
Labor	435
Equipment	124
Total	3,109
Total Incl O&P	3,700

Crew E-6 Details

Appendix G.3 – RS Means Data and Analysis (Concrete)

Structural Concrete, Normal Weight, Ready Mix, delivered, Includes Local Aggregate, Sand, Portland Cement, and water, 4000 psi

Units	CY
2012 Bare Costs	
Material	103
Labor	—
Equipment	—
Total	103
Total Incl O+P	113

Placing Concrete, Includes labor & equiptment to place, strike off & consolidate
Walls, 8" thick, pumped

Unit	CY
Crew	C-20
Daily Output	100
Labor Hours	0.64
2012 Bare Costs	
Material	—
Labor	24
Equipment	7.70
Total	31.70
Total Incl O+P	45.50

Appendix G.3 – RS Means Data and Analysis (Concrete)

Placing Concrete, Includes Labor & Equipment to Place, Strike off, and consolidate, Elevated Slabs, less than 6" thick, pumped

Crew	C-20
Daily Output	140
Labor Hours	0.457
Unit	CY
2012 Bare Costs	
Material	—
Labor	17.25
Equipment	5.50
Total	22.75
Total Incl O&P	32.50

Floor Decking, Non-Cellular composite decking, galvanized, 2" deep, 20 GA

Crew	E-4
Daily Output	3600
Labor Hours	0.009
Unit	SF
2012 Bare Costs	
Material	1.83
Labor	0.44
Equipment	0.03
Total	2.30
Total Incl O&P	2.84

Appendix G.3 – RS Means Data and Analysis (Concrete)

Forms In Place, Walls, Job-Built Plywood, Over 8' to 16' high, 3use

Crew	C-2
Daily Output	375
Labor-Hours	0.128
Unit	SFCA
2012 Bare Costs	
Material	0.73
Labor	5.50
Equipment	—
Total	6.23
Total Incl O+P	9.25

Appendix G.4 – RS Means Data and Analysis (Demolition)

Explosive/Implosive Demolition, Large Projects, No Disposal Fee based on Building Volume, Steel Building

Crew	B-5B
Daily Output	16,900
Labor Hours	0.003
Unit	C.F.
2012 Bare Costs	
Materials	—
Labor	0.12
Equipment	0.15
Total	0.27
Total Incl O+P	0.34

Disposal of Material, Minimum

Crew	B-3
Daily Output	445
Labor-Hours	0.108
Unit	CY
2012 Bare Costs	
Material	—
Labor	4.04
Equipment	5.10
Total	9.14
Total Incl O+P	11.75

Appendix G.5 – RS Means Data and Analysis (Crews)

E-6

3 Structural Steel Foreman (Outside)
9 Structural Steel Workers
1 Equip Operator (crane)
1 Welder
1 Equip Operator (oiler)
1 Equip Operator (light)
1 Lattice Boom Crane, 90 ton
1 Welder, Gas Engine, 300 amp
1 Air Compressor, 160 CFM
2 Impact Wrenches

128 L.H, Daily Total

Incl Subs O+P

$12,849.34/day

C-20

1 Labor Foreman (outside)
5 Laborers
1 Cement Finisher
1 Equipment Operator (med)
2 Gas Engine Vibrators
1 Concrete Pump (small)

64 L.H., Daily Total

Incl Subs O+P

$4524.64/day

Appendix G.5 – RS Means Data and Analysis (Crews)

B-5B

1 Powderman
2 Equip Oper (medium)
3 Truck Drivers (heavy)
1 F.E. Loader, W.M., 2.5 C.Y.
3 Dump Trucks, 12 C.Y., 400 H.P.
1 Air Compressor, 365 CFM

Incl Subs O+P

48 LH, Daily Totals $5671.52

B-3

1 Labor Foreman (outside)
2 Laborers
1 Equip Oper (medium)
2 Truck Drivers (heavy)
1 Crawler Loader, 3 C.Y.
2 Dump Trucks, 12 C.Y., 400 HP

Incl Subs O+P

48 LH, Daily Totals $5242.32

C-2

1 Carpenter Foreman (outside)
4 Carpenters
1 Laborer

Incl Subs O+P

48 LH, Daily Totals $3170.80

Appendix G.5 – RS Means Data and Analysis (Crews)

C-20

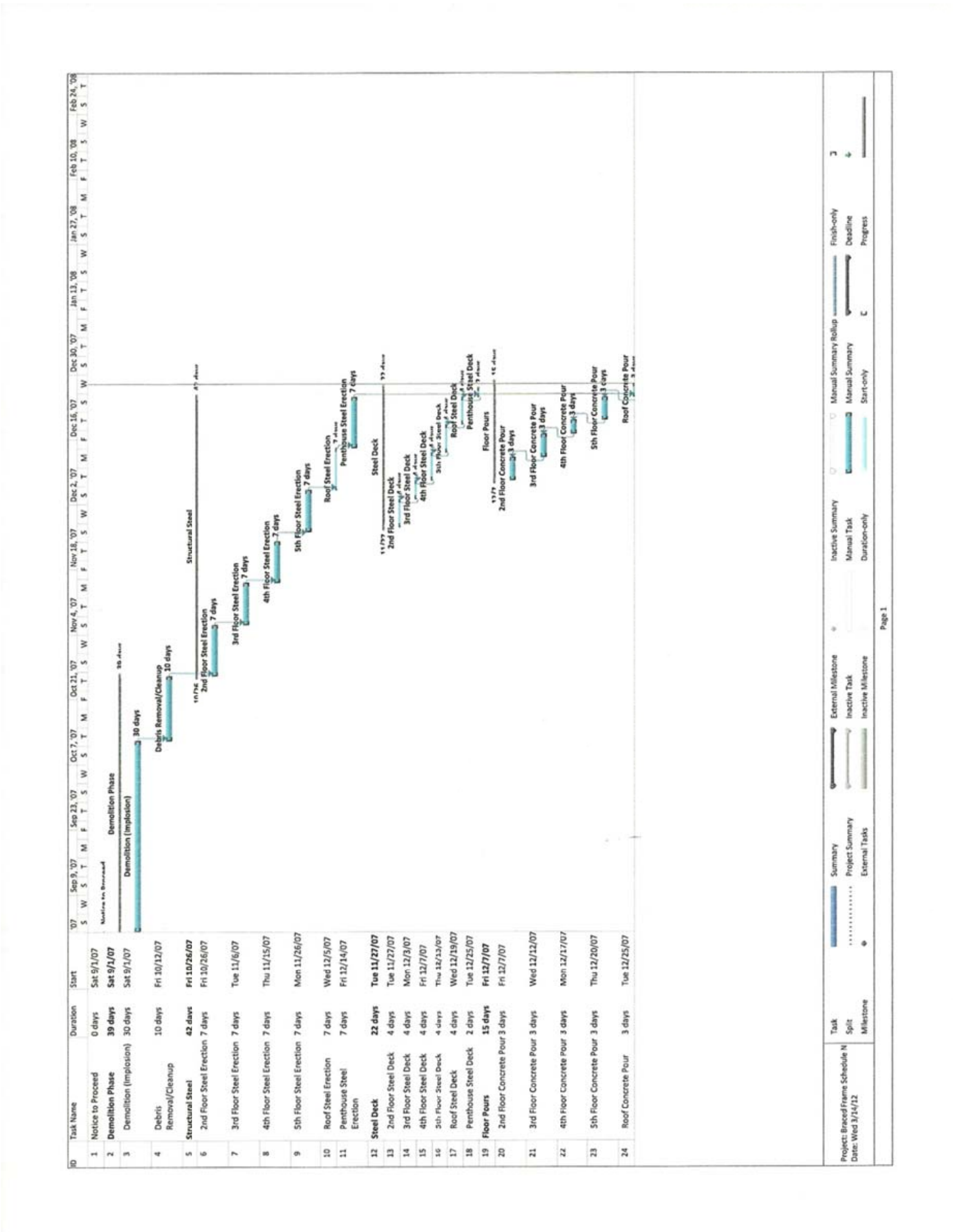
		Total Incl Subs O+P (Daily)
1	Labor Foreman (outside)	
5	Laborers	
1	Cement Finisher	
1	Equip. Oper. (med)	
2	Gas Engine Vibrators	$60.28
1	Concrete Pump (small)	
64	LH, Daily Totals	$4,524.64

E-4

		Total Incl Subs O+P
1	Structural Steel Foreman (outside)	
3	Structural Steel Workers	
1	Welder, gas engine, 300 amp	
32	LH, Daily Totals	$2986.16

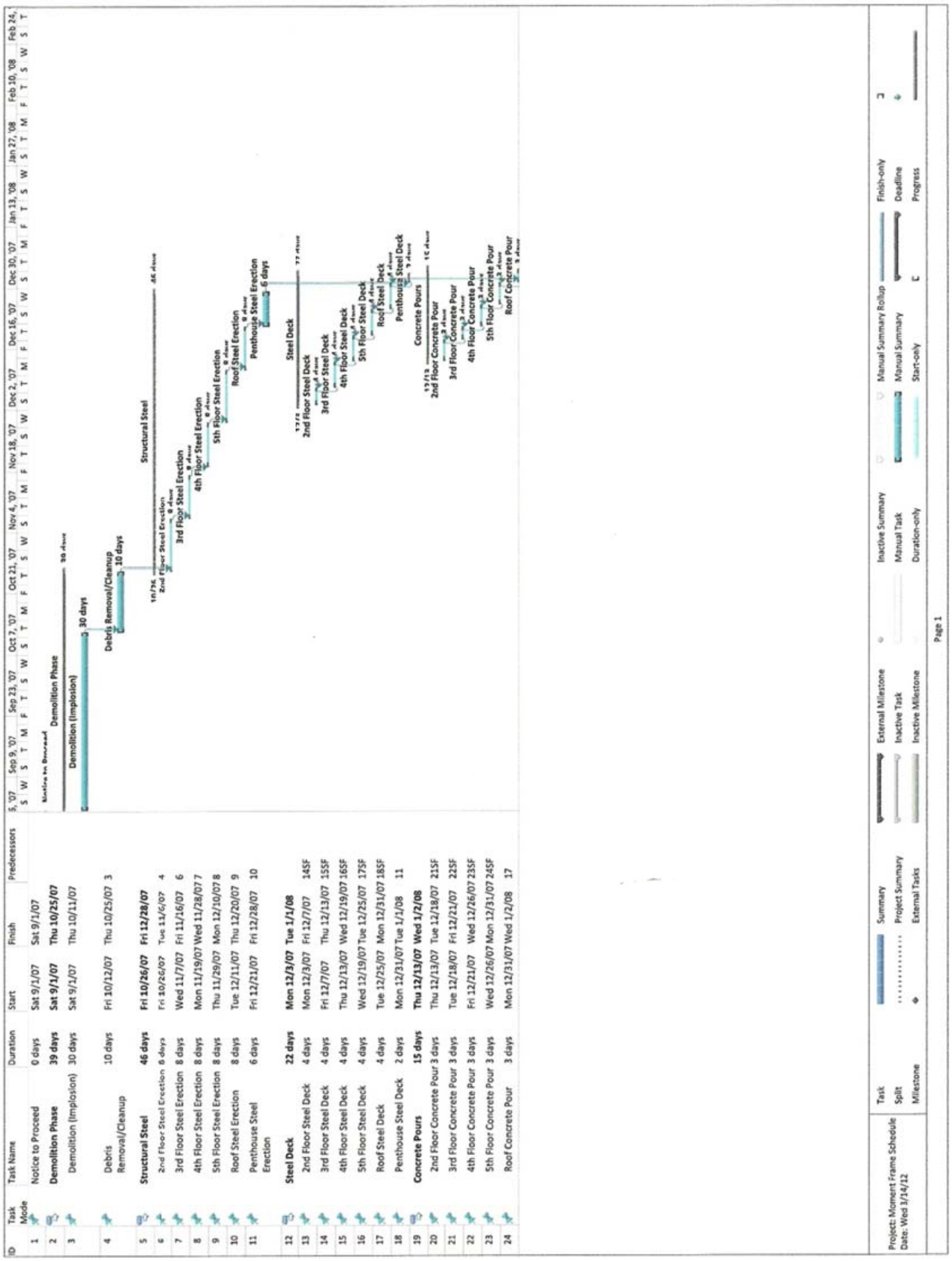
Appendix H

Appendix H.1 – Schedule Details (Moment Frame)



ID	Task Name	Duration	Start
1	Notice to Proceed	0 days	Sat 9/1/07
2	**Demolition Phase**	**39 days**	**Sat 9/1/07**
3	Demolition (Implosion)	30 days	Sat 9/1/07
4	Debris Removal/Cleanup	10 days	Fri 10/12/07
5	**Structural Steel**	**42 days**	**Fri 10/26/07**
6	2nd Floor Steel Erection	7 days	Fri 10/26/07
7	3rd Floor Steel Erection	7 days	Tue 11/6/07
8	4th Floor Steel Erection	7 days	Thu 11/15/07
9	5th Floor Steel Erection	7 days	Mon 11/26/07
10	Roof Steel Erection	7 days	Wed 12/5/07
11	Penthouse Steel Erection	7 days	Fri 12/14/07
12	**Steel Deck**	**22 days**	**Tue 11/27/07**
13	2nd Floor Steel Deck	4 days	Tue 11/27/07
14	3rd Floor Steel Deck	4 days	Mon 12/3/07
15	4th Floor Steel Deck	4 days	Fri 12/7/07
16	5th Floor Steel Deck	4 days	Thu 12/13/07
17	Roof Steel Deck	4 days	Wed 12/19/07
18	Penthouse Steel Deck	2 days	Tue 12/25/07
19	**Floor Pours**	**15 days**	**Fri 12/7/07**
20	2nd Floor Concrete Pour	3 days	Fri 12/7/07
21	3rd Floor Concrete Pour	3 days	Wed 12/12/07
22	4th Floor Concrete Pour	3 days	Mon 12/17/07
23	5th Floor Concrete Pour	3 days	Thu 12/20/07
24	Roof Concrete Pour	3 days	Tue 12/25/07

Appendix H.2 – Schedule Details (Braced Frame)

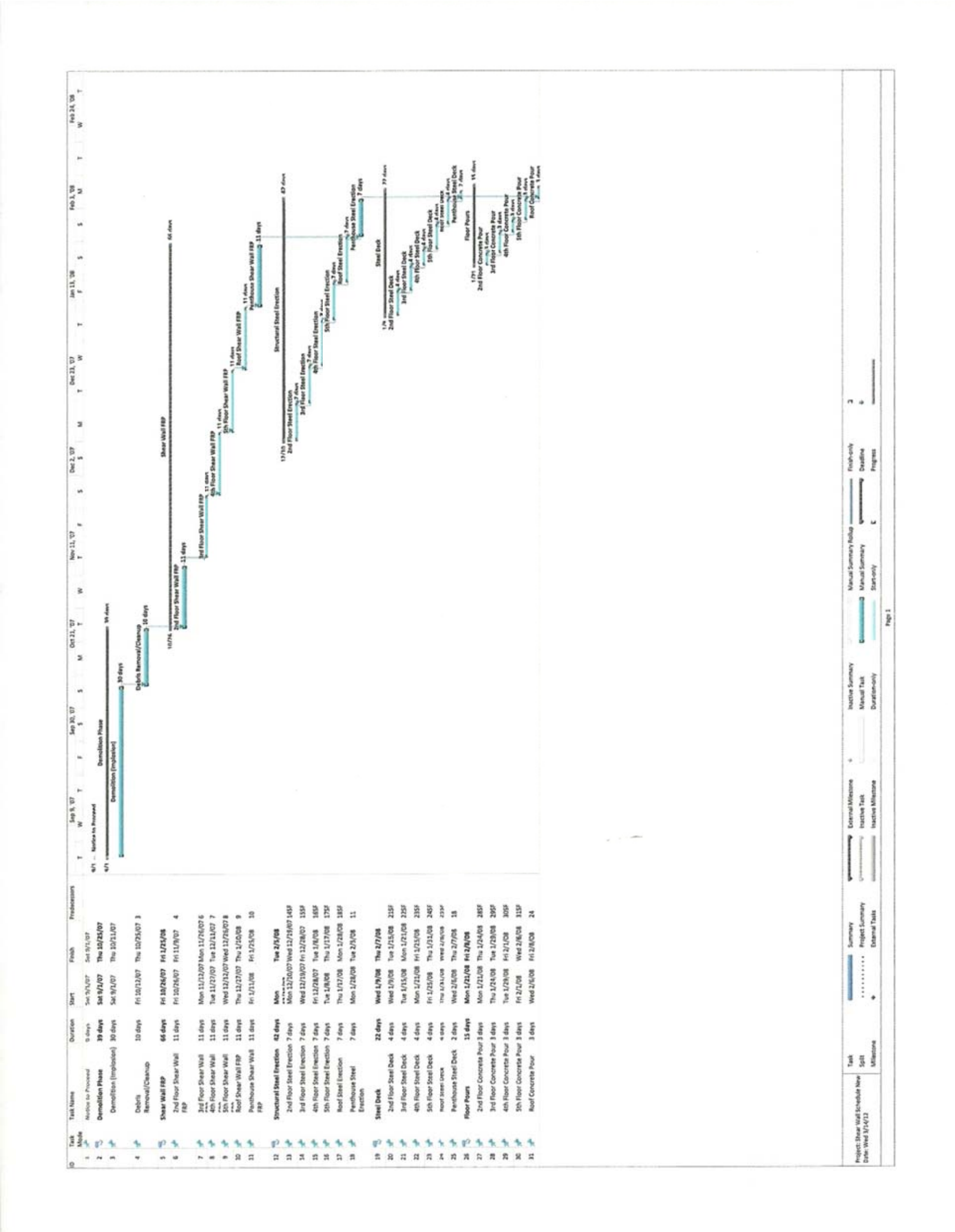


ID	Task Name	Duration	Start	Finish	Predecessors
1	Notice to Proceed	0 days	Sat 9/1/07	Sat 9/1/07	
2	**Demolition Phase**	**39 days**	**Sat 9/1/07**	**Thu 10/25/07**	
3	Demolition (Implosion)	30 days	Sat 9/1/07	Thu 10/11/07	
4	Debris Removal/Cleanup	10 days	Fri 10/12/07	Thu 10/25/07	3
5	**Structural Steel**	**46 days**	**Fri 10/26/07**	**Fri 12/28/07**	
6	2nd Floor Steel Erection	6 days	Fri 10/26/07	Tue 11/6/07	4
7	3rd Floor Steel Erection	8 days	Wed 11/7/07	Fri 11/16/07	6
8	4th Floor Steel Erection	8 days	Mon 11/19/07	Wed 11/28/07	7
9	5th Floor Steel Erection	8 days	Thu 11/29/07	Mon 12/10/07	8
10	Roof Steel Erection	8 days	Tue 12/11/07	Thu 12/20/07	9
11	Penthouse Steel Erection	6 days	Fri 12/21/07	Fri 12/28/07	10
12	**Steel Deck**	**22 days**	**Mon 12/3/07**	**Tue 1/1/08**	
13	2nd Floor Steel Deck	4 days	Mon 12/3/07	Fri 12/7/07	14SF
14	3rd Floor Steel Deck	4 days	Fri 12/7/07	Thu 12/13/07	15SF
15	4th Floor Steel Deck	4 days	Thu 12/13/07	Wed 12/19/07	16SF
16	5th Floor Steel Deck	4 days	Wed 12/19/07	Tue 12/25/07	17SF
17	Roof Steel Deck	4 days	Tue 12/25/07	Mon 12/31/07	18SF
18	Penthouse Steel Deck	2 days	Mon 12/31/07	Tue 1/1/08	11
19	**Concrete Pours**	**15 days**	**Thu 12/13/07**	**Wed 1/2/08**	
20	2nd Floor Concrete Pour	3 days	Thu 12/13/07	Tue 12/18/07	21SF
21	3rd Floor Concrete Pour	3 days	Tue 12/18/07	Fri 12/21/07	22SF
22	4th Floor Concrete Pour	3 days	Fri 12/21/07	Wed 12/26/07	23SF
23	5th Floor Concrete Pour	3 days	Wed 12/26/07	Mon 12/31/07	24SF
24	Roof Concrete Pour	3 days	Mon 12/31/07	Wed 1/2/08	17

Project: Moment Frame Schedule
Date: Wed 3/14/12

Page 1

Appendix H.3 – Schedule Details (Shear Walls)



Appendix I

Appendix I.1 – Cost and Schedule Calculations (Moment Frame System)

Moment Frame (Existing) System Costs

DESCRIPTION: Braced Frame System Cost Analysis

Input			
Cost Factors			
Historical Cost Index		93.6	(July 2008)
Location Factor		94.1	
Size Factor		2.97	
Actual Size	163616 sf		
Typical Size	55000 sf		
Steel Weight		659.8	tons
Hospitals, Steel Bearing, 3 to 6 Stories			
Crew	E-6		
Daily Output		14.40	
Labor-Hours		8.889	
Unit	Tons		
2012 Bare Costs			
Material	$	2,550.00	
Labor	$	435.00	
Equipment	$	124.00	
Total	$	3,109.00	
Total Including Overhead and Profit	$	3,700.00	
Analysis			
Steel Costs			
Total Including Overhead and Profit	$	2,441,260	
Steel Schedule			
Total Days to Complete		45.8	days
Total 2012 Costs	$	2,441,260	(Completely Un-Adjusted)
Adjusted 2008 Cost	**$**	**6,396,503**	

Appendix I.2 – Cost and Schedule Calculations (Braced Frame System)

Braced Frame System Costs

DESCRIPTION: Braced Frame System Cost Analysis

Input		
Cost Factors		
Historical Cost Index		93.6 (July 2008)
Location Factor		94.1
Size Factor		2.97
Actual Size	163616 sf	
Typical Size	55000 sf	
Steel Weight		607.0 tons
Hospitals, Steel Bearing, 3 to 6 Stories		
Crew	E-6	
Daily Output		14.40
Labor-Hours		8.889
Unit	Tons	
2012 Bare Costs		
Material	$	2,550.00
Labor	$	435.00
Equipment	$	124.00
Total	$	3,109.00
Total Including Overhead and Profit	$	3,700.00
Analysis		
Steel Costs		
Total Including Overhead and Profit	$	2,245,900
Steel Schedule		
Total Days to Complete		42.2 days
Total 2012 Costs	$	2,245,900 (Completely Un-Adjusted)
Adjusted 2008 Cost	**$**	**5,884,627**

Appendix I.3 – Cost and Schedule Calculations (Shear Wall System)

Shear Wall System Costs

DESCRIPTION: Shear Wall System Cost Analysis

Input		
Cost Factors		
Historical Cost Index		93.6 (July 2008)
Location Factor		94.1
Size Factor		2.97
Actual Size	163616 sf	
Typical Size	55000 sf	
Steel Weight		493.3 tons
Shear Walls		569.0 yds^3
Square Footage of Contact Area		47848.0 ft^2
Hospitals, Steel Bearing, 3 to 6 Stories		
Crew	E-6	
Daily Output		14.40
Labor-Hours		8.889
Unit	Tons	
2012 Bare Costs		
Material	$	2,550.00
Labor	$	435.00
Equipment	$	124.00
Total	$	3,109.00
Total Including Overhead and Profit	$	3,700.00
Structural Concrete, Normal Weight, Ready Mix, Delivered, Includes Local Aggregate, Sand, Portland Cement, & Water, 4000 psi		
Crew	N/A	
Daily Output	N/A	
Labor-Hours	N/A	
Unit	C.Y.	
2012 Bare Costs		
Material	$	103.00
Labor	$	-
Equipment	$	-
Total	$	103.00
Total Including Overhead and Profit	$	113.00
Forms in Place, Walls, Job-Built Plywood, Over 8' to 16' high, 3 use		
Crew	C-2	
Daily Output		375.00
Labor-Hours		0.128
Unit	SFCA	
2012 Bare Costs		
Material	$	0.73
Labor	$	5.50
Equipment	$	-
Total	$	6.23
Total Including Overhead and Profit	$	9.25
Placing Concrete, Includes Labor & Equipment to place, strike off, and consolidate, Walls, 8" thick, pumped		
Crew	C-20	
Daily Output		100.00
Labor-Hours		0.64
Unit	C.Y.	
2012 Bare Costs		
Material	$	-
Labor	$	24.00
Equipment	$	7.70
Total	$	31.70
Total Including Overhead and Profit	$	45.50
Analysis		
Steel Costs		
Total Including Overhead and Profit	$	1,825,210
Steel Schedule		
Total Days to Complete		34.3 days
Shear Wall Costs		
Total Including Overhead and Profit	$	90,187
Shear Wall Schedule		
Total Days to Form Concrete		127.6 days
Total Days to Place Concrete		5.7 days
Total 2012 Costs	$	1,915,397 (Completely Un-Adjusted)
Adjusted 2008 Cost	**$**	**5,018,654**

Appendix I.4 – Cost and Schedule Calculations (Demolition Costs)

Demoltion Costs

DESCRIPTION: Demolition Cost Analysis

Input		
Cost Factors		
Historical Cost Index		93.6 (July 2008)
Location Factor		94.1
Size Factor		2.97
Actual Size	163616 sf	
Typical Size	55000 sf	
Structural Volume		2474280 ft^3
Material to Dispose of		4582.0 yds^3 (Assumed 5%)
Explosive/Implosive Demolition, Large Projects, No Disposal Fee based on Building Volume, Steel Building		
Crew	B-5B	
Daily Output		16900
Labor-Hours		0.003
Unit	C.F.	
2012 Bare Costs		
Material	$	-
Labor	$	0.12
Equipment	$	0.15
Total	$	0.27
Total Including Overhead and Profit	$	0.34
Disposal of Material, Minimum		
Crew	B-3	
Daily Output		445
Labor-Hours		0.108
Unit	C.Y.	
2012 Bare Costs		
Material	$	-
Labor	$	4.04
Equipment	$	5.10
Total	$	9.14
Total Including Overhead and Profit	$	11.75
Analysis		
Demolition Costs		
Total Including Overhead and Profit	$	841,255
Demolition Schedule		
Total Days to Complete		146.4 days
Disposal Costs		
Total Including Overhead and Profit	$	53,839
Disposal Schedule		
Total Days to Complete		10.3 days
Total 2012 Costs	$	895,094 (Completely Un-Adjusted)
Adjusted 2008 Cost	**$**	**2,345,293**

Appendix I.5 – Cost and Schedule Calculations (Floor Slab Costs)

Floor Slab Costs

DESCRIPTION: Floor Slab Cost Analysis

Input

Cost Factors		
Historical Cost Index		93.6 (July 2008)
Location Factor		94.1
Size Factor		2.97
Actual Size	163616 sf	
Typical Size	55000 sf	
Floor Square Footage		163616 ft^2
Concrete Cubic Yards		2519.7 yd^3

Placing Concrete, Includes Labor and Equipment to Place, Strike-Off, and Consolidate, Elevated Slabs, Less than 6" thick, Pumped

Crew	C-20	
Daily Output		140
Labor-Hours		0.457
Unit	CY	
2012 Bare Costs		
Material	$	-
Labor	$	17.25
Equipment	$	5.50
Total	$	22.75
Total Including Overhead and Profit	$	32.50

Structural Concrete, Normal Weight, Ready Mix, Delivered, Includes Local Aggregate, Sand, Portland Cement, & Water, 4000 psi

Crew	N/A	
Daily Output	N/A	
Labor-Hours	N/A	
Unit	C.Y.	
2012 Bare Costs		
Material	$	103.00
Labor	$	-
Equipment	$	-
Total	$	103.00
Total Including Overhead and Profit	$	113.00

Floor Decking, Non-Cellular Composite Decking, Galvanized, 2" Deep, 20 GA

Crew	E-4	
Daily Output		3600.00
Labor-Hours		0.009
Unit	S.F.	
2012 Bare Costs		
Material	$	1.83
Labor	$	0.44
Equipment	$	0.03
Total	$	2.30
Total Including Overhead and Profit	$	2.84

Analysis

Floor Decking Costs		
Total Including Overhead and Profit	$	464,669
Floor Decking Schedule		
Total Days to Complete		45.4
Floor Concrete Costs		
Total Including Overhead and Profit	$	284,725
Floor Concrete Schedule		
Total Days to Place Concrete		18.0
Total 2012 Costs	$	749,394 (Completely Un-Adjusted)
Adjusted 2008 Cost	**$**	**1,963,536**

Appendix I.6 – Cost and Schedule Calculations

Total Revenue	\$486 Million
Total Beds	412
Total Beds in Womens Hospital	58
Assumed Revenue From Womens Hospital	\$68.4 Million
Approximate overall profit Margin	4.2%
Estimated Womens Hospital Profit (2011)	\$2.87 Million

Appendix J

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

Column Above

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

01/18/12 14:38:45

STEEL COLUMN INFORMATION:

Column Number: 30 **Frame Number: 0**

Level Top: New 3rd Floor
Bot: 2nd Floor

Column Line (93.33,0.00)

Fy (ksi) = 50.00
Column Size = W14X132
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 0.00
Length (ft) = 16.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	12.05	12.05
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

Member Force Output: At Centerline of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
Sn	NegRoofLiveLoad	RAMUSER
W1	WIND_Y	Wind_IBC06_1_Y
W2	WIND_Y	Wind_IBC06_2_Y+E
W3	WIND_Y	Wind_IBC06_2_Y-E
W4	WIND_X	Wind_IBC06_1_X
W5	WIND_X	Wind_IBC06_2_X+E
W6	WIND_X	Wind_IBC06_2_X-E
E1	EQ	EQ_IBC06_X_+E_Drft
E2	EQ	EQ_IBC06_X_-E_Drft
E3	EQ	EQ_IBC06_Y_+E_Drft
E4	EQ	EQ_IBC06_Y_-E_Drft
E5	EQ_MEMB	EQ_IBC06_X_+E_F
E6	EQ_MEMB	EQ_IBC06_X_-E_F
E7	EQ_MEMB	EQ_IBC06_Y_+E_F
E8	EQ_MEMB	EQ_IBC06_Y_-E_F

MEMBER FORCE MAXIMA AND MINIMA

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

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	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	kips	kip-ft	kip-ft	kips	kips	kip-ft
Max @ T:	122.68	11.75	0.07	17.59	0.15	0.01
LC:	Lp	W3	E3	W4	W3	W2
Max @ B:	122.68	129.71	1.39	17.59	0.15	0.01
LC:	Lp	W4	E4	W4	W3	W2
Maximum:	122.68	129.71	1.39	17.59	0.15	0.01
LC:	Lp	W4	E4	W4	W3	W2
@ (ft):	0.00	16.00	16.00	0.00	0.00	0.00
Min @ T:	-0.04	-151.75	-1.40	-1.63	-0.21	-0.00
LC:	W3	W4	W1	W3	W2	W5
Min @ B:	-0.04	-14.36	-4.11	-1.63	-0.21	-0.00
LC:	W3	W3	W2	W3	W2	W5
Minimum:	-0.04	-151.75	-4.11	-1.63	-0.21	-0.00
LC:	W3	W4	W2	W3	W2	W5
@ (ft):	0.00	0.00	16.00	0.00	0.00	0.00

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

Column Below

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

01/18/12 14:38:45

STEEL COLUMN INFORMATION:

Column Number: 37	**Frame Number: 0**
Level Top: 2nd Floor	Column Line (93.33,0.00)
Bot: Base	
Fy (ksi) = 36.00	Column Size = W14X159
Elastic Modulus (ksi) = 29000.00	
Orientation (deg) = 0.00	Length (ft) = 12.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Pin
	Minor Axis:	Fix	Pin
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	12.05	0.00
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)
Member Force Output:		At Centerline of Joint	

P-Delta: Yes Scale Factor: 1.00
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
Sn	NegRoofLiveLoad	RAMUSER
W1	WIND_Y	Wind_IBC06_1_Y
W2	WIND_Y	Wind_IBC06_2_Y+E
W3	WIND_Y	Wind_IBC06_2_Y-E
W4	WIND_X	Wind_IBC06_1_X
W5	WIND_X	Wind_IBC06_2_X+E
W6	WIND_X	Wind_IBC06_2_X-E
E1	EQ	EQ_IBC06_X_+E_Drft
E2	EQ	EQ_IBC06_X_-E_Drft
E3	EQ	EQ_IBC06_Y_+E_Drft
E4	EQ	EQ_IBC06_Y_-E_Drft
E5	EQ_MEMB	EQ_IBC06_X_+E_F
E6	EQ_MEMB	EQ_IBC06_X_-E_F
E7	EQ_MEMB	EQ_IBC06_Y_+E_F
E8	EQ_MEMB	EQ_IBC06_Y_-E_F

MEMBER FORCE MAXIMA AND MINIMA

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

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	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ T:	151.56	0.51	1.39	19.38	0.34	0.05
LC:	Lp	Lp	E4	W4	W2	W2
Max @ B:	151.56	-0.00	0.00	19.38	0.34	0.05
LC:	Lp	D	D	W4	W2	W2
Maximum:	151.56	0.51	1.39	19.38	0.34	0.05
LC:	Lp	Lp	E4	W4	W2	W2
@ (ft):	0.00	0.00	0.00	0.00	0.00	0.00
Min @ T:	-0.66	-232.56	-4.11	-0.04	-0.12	-0.01
LC:	W4	W4	W2	Lp	E4	W5
Min @ B:	-0.66	-0.00	0.00	-0.04	-0.12	-0.01
LC:	W4	D	D	Lp	E4	W5
Minimum:	-0.66	-232.56	-4.11	-0.04	-0.12	-0.01
LC:	W4	W4	W2	Lp	E4	W5
@ (ft):	0.00	0.00	0.00	0.00	0.00	0.00

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

Left Beam

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

01/18/12 14:38:45

STEEL BEAM INFORMATION:

Beam Number: 141 | **Frame Number: 0**
Level: 2nd Floor | I-End (68.67,0.00) | J-End (93.33,0.00)
Fy (ksi) = 36.00 | Beam Size = W24X84
Length (ft) = 24.67
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)
Member Force Output:		At Centerline of Joint	

P-Delta: Yes Scale Factor: 1.00
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
Sn	NegRoofLiveLoad	RAMUSER
W1	WIND_Y	Wind_IBC06_1_Y
W2	WIND_Y	Wind_IBC06_2_Y+E
W3	WIND_Y	Wind_IBC06_2_Y-E
W4	WIND_X	Wind_IBC06_1_X
W5	WIND_X	Wind_IBC06_2_X+E
W6	WIND_X	Wind_IBC06_2_X-E
E1	EQ	EQ_IBC06_X_+E_Drft
E2	EQ	EQ_IBC06_X_-E_Drft
E3	EQ	EQ_IBC06_Y_+E_Drft
E4	EQ	EQ_IBC06_Y_-E_Drft
E5	EQ_MEMB	EQ_IBC06_X_+E_F
E6	EQ_MEMB	EQ_IBC06_X_-E_F
E7	EQ_MEMB	EQ_IBC06_Y_+E_F
E8	EQ_MEMB	EQ_IBC06_Y_-E_F

MEMBER FORCE MAXIMA AND MINIMA

	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	kips	kip-ft	kip-ft	kips	kips	kip-ft
Max @ i:	0.00	163.80	0.00	13.56	0.00	0.00
LC:	W5	W4	W2	Lp	W3	W5
Max @ j:	0.00	5.24	0.00	0.42	0.00	0.00
LC:	W5	W3	W3	W3	W3	W5

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

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01/18/12 14:38:45

	P	Mmajor	Mminor	Vmajor	Vminor	Tors
Maximum:	0.00	163.80	0.00	13.56	0.00	0.00
LC:	W5	W4	W2	Lp	W3	W5
@ (ft):	0.00	0.00	0.00	0.00	0.00	0.00
Min @ i:	-0.00	-51.96	-0.00	-13.81	-0.00	-0.01
LC:	W4	Lp	Ln	W4	W1	W2
Min @ j:	-0.00	-176.74	-0.00	-13.95	-0.00	-0.01
LC:	W4	W4	W1	Lp	W1	W2
Minimum:	-0.00	-176.74	-0.00	-13.95	-0.00	-0.01
LC:	W4	W4	W1	Lp	W1	W2
@ (ft):	0.00	24.67	24.33	24.67	0.00	0.00

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

Bright Beam

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

01/18/12 14:38:45

STEEL BEAM INFORMATION:

Beam Number: 178 **Frame Number: 0**
Level: 2nd Floor I-End (93.33,0.00) J-End (118.00,0.00)
Fy (ksi) = 36.00 Beam Size = W24X84
Length (ft) = 24.67
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)
Member Force Output:		At Centerline of Joint	

P-Delta: Yes Scale Factor: 1.00
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Ln	NegLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
Sn	NegRoofLiveLoad	RAMUSER
W1	WIND_Y	Wind_IBC06_1_Y
W2	WIND_Y	Wind_IBC06_2_Y+E
W3	WIND_Y	Wind_IBC06_2_Y-E
W4	WIND_X	Wind_IBC06_1_X
W5	WIND_X	Wind_IBC06_2_X+E
W6	WIND_X	Wind_IBC06_2_X-E
E1	EQ	EQ_IBC06_X_+E_Drft
E2	EQ	EQ_IBC06_X_-E_Drft
E3	EQ	EQ_IBC06_Y_+E_Drft
E4	EQ	EQ_IBC06_Y_-E_Drft
E5	EQ_MEMB	EQ_IBC06_X_+E_F
E6	EQ_MEMB	EQ_IBC06_X_-E_F
E7	EQ_MEMB	EQ_IBC06_Y_+E_F
E8	EQ_MEMB	EQ_IBC06_Y_-E_F

MEMBER FORCE MAXIMA AND MINIMA

	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	kips	kip-ft	kip-ft	kips	kips	kip-ft
Max @ i:	0.00	185.54	0.00	13.86	0.00	0.00
LC:	W6	W4	W3	Lp	W2	W5
Max @ j:	0.00	5.17	0.00	0.42	0.00	0.00
LC:	W6	W3	W1	W3	W2	W5

Appendix J.1 – Moment Frame Connection Loads

Member Force Envelope

RAM Frame v14.03.02.00
DataBase: NEW BUILDING DESIGN
Building Code: IBC

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	P	Mmajor	Mminor	Vmajor	Vminor	Tors
Maximum:	0.00	185.54	0.00	13.86	0.00	0.00
LC:	W6	W4	W1	Lp	W2	W5
@ (ft):	0.00	0.00	24.67	0.00	0.00	0.00
Min @ i:	-0.00	-58.12	-0.00	-15.03	-0.00	-0.01
LC:	E5	Lp	W2	W4	E3	W2
Min @ j:	-0.00	-185.30	-0.00	-15.03	-0.00	-0.01
LC:	E5	W4	W3	W4	E3	W2
Minimum:	-0.00	-185.30	-0.00	-15.03	-0.00	-0.01
LC:	E5	W4	W2	W4	E3	W2
@ (ft):	0.00	24.67	0.00	0.00	0.00	0.00

Appendix J.2 – Moment Frame Connection Calculations

loment Connection Calculator per AISC 14th Edition (ASD)
Checks Moment Connection Strength versus Applied Load

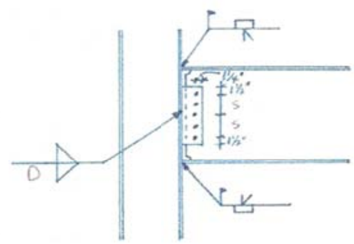
Description:

GLOBAL STATUS	
PASS	
Plate Limit States	PASS
Weld/Bolt Limit States	PASS

INPUT DATA

Load Data

Beam End Shear Reaction	R_a =	15.03	kips
Beam End Moment Reaction	M_a =	185.54	ft-kips



Beam Data — W24x84

Depth	d =	24.1	in
Flange Width	b_f =	9.02	in
Flange Thickness	t_f =	0.77	in
Web Thickness	t_w =	0.47	in
Yield Strength	F_y =	36	ksi
Ultimate Strength	F_u =	58	ksi

Column Data — W14x159

Depth	d =	15	in
Flange Width	b_f =	15.6	in
Flange Thickness	t_f =	1.19	in
Yield Strength	F_y =	36	ksi
Ultimate Strength	F_u =	58	ksi

Plate Data

Thickness	t_p =	0.375	in
Depth	d_p =	9	in
Width	w_p =	4	in
Yield Strength	F_y =	36	ksi
Ultimate Strength	F_u =	58	ksi
Horizontal Edge Distance	L_h =	1.25	in
Vertical Edge Distance	L_v =	1.5	in

Bolt Data
Plate Bolts

# of Bolts	$\#_{bolts}$ =	3	
Bolt Diameter	d_b =	0.875	in
Thread Condition		N Bolts	
Bolt Type		A325	
Bolt Spacing (Center-to-Center)	S =	3	in
Available Shear Strength	r_n/omega =	16.2	kips

Weld Data
Plate Welds

Diameter of Weld	D =	0.25	in

ANALYSIS

Plate Limit States

Shear Yield — PASS

Gross Area	A_g =	3.375	in^2
Nominal Strength	r_n/omega =	48.6	kips

r_n/omega = 48.6 kips > R_a = 15.0 kips

Shear Rupture — PASS

Net Area	A_n =	2.250	in^2
Nominal Strength	r_n/omega =	39.15	kips

r_n/omega = 39.2 kips > R_a = 15.0 kips

Block Shear — PASS

Net Tension Area	A_{nt} =	0.38	in^2
Net Shear Area	A_{nv} =	1.88	in^2
Gross Shear Area	A_{gv} =	2.81	in^2
Block Shear Strength	r_n/omega =	41.3	kips

r_n/omega = 41.3 kips > V_a = 15.0 kips

Bearing/Tearout

Plate Bearing	r_n/omega =	22.84	kips
Plate Tearout Exterior	r_n/omega =	13.05	kips
Plate Tearout Interior	r_n/omega =	26.10	kips

Bolt Limit States — PASS
Bolt Shear

Available Bolt Shear Strength	r_n/omega =	16.2	kips

r_n/omega = 45.5 kips > R_a = 15.0 kips

Weld Limit States — PASS
Plate Weld

Available Weld Strength	r_n/omega =	33.41	kips

r_n/omega = 33.4 kips > R_a = 15.0 kips

Shear Rupture of Base Metal — PASS

Available Base Metal Strength	r_n/omega =	29.39	kips

r_n/omega = 29.4 kips > R_a = 15.0 kips

Appendix J.2 – Moment Frame Connection Calculations

Column Side Limit State Calculator per AISC 14th Edition (ASD)
Checks Column Side Limit State in Moment Connections for the Allowable Load versus Applied Load

Description:

INPUT DATA

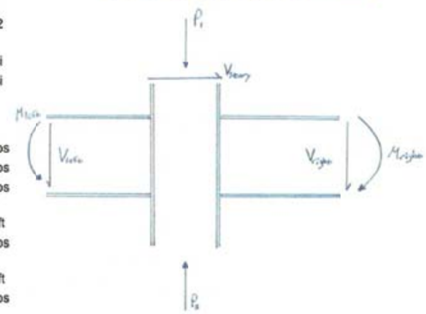
Beam Data

	Left Beam	W24x84		Right Beam	W24x84	
Beam Depth	$d =$	24.1	in	$d =$	24.1	in
Web Thickness	$t_w =$	0.5	in	$t_w =$	0.5	in
Flange Width	$b_f =$	9.0	in	$b_f =$	9.0	in
Flange Thickness	$t_f =$	0.8	in	$t_f =$	0.8	in
Moment Capacity	$M_n =$	933	k-ft	$M_n =$	933	k-ft
Yield Strength	$F_y =$	36	ksi	$F_y =$	36	ksi
Ultimate Strength	$F_u =$	58	ksi	$F_u =$	58	ksi

Column Data

		W14x159	
Column Depth	$d =$	15.0	in
Web Thickness	$t_w =$	0.7	in
Flange Width	$b_f =$	15.6	in
Flange Thickness	$t_f =$	1.2	in
Distance from Outside Edge of Flange to Fillet Radius	$k_{des} =$	1.8	in
Column Area	$A =$	46.7	in2
	$h/t_w =$	15.3	
Yield Strength	$F_y =$	36	ksi
Ultimate Strength	$F_u =$	58	ksi

GLOBAL STATUS
OK
Doubler Plate Required



Load Data

Column Loads			
Axial Load Above Level of Interest	$P_1 =$	129.71	kips
Axial Load Below Level of Interest	$P_2 =$	157.13	kips
Story Shear	$V_{story} =$	17.59	kips
Left Beam Loads			
Moment	$M_{left} =$	-176.74	k-ft
Shear	$V_{left} =$	13.56	kips
Right Beam Loads			
Moment	$M_{right} =$	185.54	k-ft
Shear	$V_{right} =$	13.86	kips

ANALYSIS

Left Beam

Load Derivation

Moment Force	$M_a =$	-176.74	k-ft
Tension Force	$T_a =$	-90.9	kips
Compression Force	$C_a =$	-90.9	kips

Check						Status		
Local Flange Bending (Tension Force) — Nominal Strength	R_n/omega =	190.8 kips	>	$T_r =$	90.9 kips	OK	R_n/omega =	190.8 kips
Local Web Yielding (Tension or Compression) — Nominal Strength	R_n/omega =	173.8 kips	>	$P_r =$	90.9 kips	OK	R_n/omega =	173.8 kips
Local Web Crippling (Compression Force) — Nominal Strength	R_n/omega =	308.6 kips	>	$C_r =$	90.9 kips	OK	R_n/omega =	308.6 kips
Local Web Buckling (Compression Force both Flanges) — Nominal Strength	R_n/omega =	533.7 kips	>	$C_r =$	90.9 kips	OK	R_n/omega =	533.7 kips

Right

Load Derivation

Moment Force	$M_a =$	185.5	k-ft
Tension Force	$T_a =$	95.4	kips
Compression Force	$C_a =$	95.4	kips

Check						Status		
Local Flange Bending (Tension Force) — Nominal Strength	R_n/omega =	190.8 kips	>	$T_r =$	95.4 kips	OK	R_n/omega =	190.8 kips
Local Web Yielding (Tension or Compression) — Nominal Strength	R_n/omega =	173.8 kips	>	$P_r =$	95.4 kips	OK	R_n/omega =	173.8 kips
Local Web Crippling (Compression Force) — Nominal Strength	R_n/omega =	308.6 kips	>	$C_r =$	95.4 kips	OK	R_n/omega =	308.6 kips
Local Web Buckling (Compression Force both Flanges) — Nominal Strength	R_n/omega =	533.7 kips	>	$C_r =$	95.4 kips	OK	R_n/omega =	533.7 kips

Panel Zone Shear — Doubler Plate Required

Required Axial Strength	$P_r =$	143.42	kips
	$P_c =$	1008.72	kips
Nominal Strength	R_n/omega =	144.8	kips

R_n/omega = 144.8 kips < $V_r =$ 198.0 kips

Appendix J.2 – Moment Frame Connection Calculations

Doubler Plate Design

$$t_{p,req} = \frac{V_{story} - R_n/\Omega}{0.6 F_y d_c/\Omega} = \frac{199.4 - 144.8}{0.6(36)(15)/1.67} = 0.96''$$

$\boxed{\therefore \text{ Use } 1'' \text{ doubler plate on one side}}$

Check Plate Buckling

$$h/t_p \le 1.10\sqrt{\frac{kE}{F_y}}$$

$$\frac{15 - 2(1.2)}{1''} \le 1.1\sqrt{\frac{5(29000)}{36}}$$

$$12.6 \le 69.8 \qquad \therefore OK$$

Weld Design

Short side requires no welds based on mechanics

Long Side

$$V_{weld} = \frac{V_{story} - R_n/\Omega}{2} = \frac{199.4 - 144.8}{2} = 27.3^k$$

$$D_{req} = \frac{V_{weld}}{0.928\, d_b} = \frac{27.3}{0.928(24.1)} = 1.22 \Rightarrow \therefore \text{ needs } 1/8'' \text{ weld each side}$$

Min Weld size is 5/16"

$\boxed{\therefore \text{ Use } 5/16'' \text{ Fillet welds}}$

Appendix K

Appendix K.1 –Braced Frame Connection Calculations

Bracing Connection Design

For HSS $6 \times 6 \times 3/8$

ASTM A500 Gr B
$F_y = 46$ ksi
$F_u = 58$ ksi

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

HSS Properties
$A = 7.58 \text{ in}^2$
$d = 6 \text{ in}$
$t = 0.349 \text{ in}$

$P = 46^k$
$P_u = 1.6(46^k) = 73.6^k$

HSS Limit States

Tensile Yielding

$P_n = F_y A_g = 46(7.58) = 348.7^k$

$\phi P_n = 0.9(348.7) = 313.8^k > 73.6^k \quad \therefore \text{OK}$

Tensile Rupture

$P_n = F_u A_e = 58(4.55) = 263.7^k$

$A_e = A_n U = 7.27(0.625) = 4.55 \text{ in}^2$

$A_n = A_g - 2(t_p + 1/16)\,t = 7.58 - 2(3/8 + 1/16)0.349 = 7.27 \text{ in}^2$

$U = 1 - \bar{x}/\ell = 1 - 2.25/6 = 0.625$

$\ell = 6''$

$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{6^2 + 2(6)(6)}{4(6+6)} = 2.25$

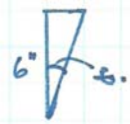
$\phi P_n = 0.75(263.7^k) = 197.8^k > 73.6^k \quad \therefore \text{OK}$

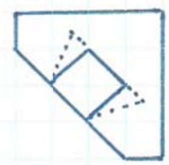
Appendix K.1 –Braced Frame Connection Calculations

Plate Limit States

Whitmore Section

$w = 2[6\tan(30)] + 6$
$= 12.93''$





Tension Yield

$P_n = F_y A_g = 36(12.93\,t)$

$\phi P_n \geq P_u \rightarrow 0.9(36)(12.93\,t) = 73.6^k$

$\therefore t_{min} = 0.176'' \rightarrow$ Use 3/8" PL

Weld Limit States (HSS-to-Gusset)

Base Metal Strength

Try 3/16" weld

$t_{min} = \frac{D_{eff}}{30.2}\left(\frac{F_{EXX}}{F_y}\right) = \frac{3}{30.2}\left(\frac{70}{36}\right) = 0.193''$

$t_p = 0.375 < 2t_{min} = 2(0.193) = 0.386$

$\therefore$ Gusset Base Metal Controls

$R_n = F_{BM} A_{BM} = 0.6(36)[2(6'')(0.375'')] = 97.2^k$

$\phi R_n = 1.0(97.2^k) = 97.2^k > 73.6^k$ $\therefore$ OK

Appendix K.1 –Braced Frame Connection Calculations

Plate Limit States

Block Shear

$$A_{nv} = 2(6)(0.375) = 4.5\,in^2 = A_{gv}$$
$$A_{nt} = 6(0.375) = 2.25\,in^2$$

$$R_n = [0.6\,F_u A_{nv} + F_u A_{nt}] \leq [0.6\,F_y A_{gv} + F_u A_{nt}]$$
$$= 287.1^k \leq 227.7^k$$

$$\phi R_n = 0.75(227.7^k) = 170.8^k > 73.6^k \therefore \underline{OK}$$

Sizing Gusset Plates

Use General Force Method to ensure no moment in connection

$$\alpha = \frac{L_1}{2} + 1/2 \qquad e_c = d_c/2 = 10.8/2 = 5.4''$$

$$\beta = 10'' \qquad e_b = d/2 = 17.7/2 = 8.85''$$

$$\tan\theta = 0.7778$$

$$\alpha - \beta\tan\theta = e_b\tan\theta - e_c$$
$$L_1/2 + 1/2 - (10)(0.7778) = 8.85(0.7778) - 5.4$$

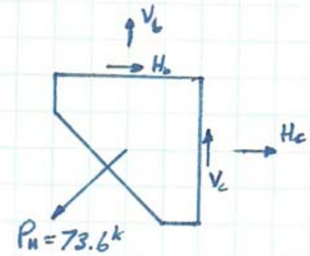
$$\therefore L_1 = 17.5''$$

$$\boxed{\text{Use } PL\ 20 \times 17.5 \times 3/8''}$$

Appendix K.1 –Braced Frame Connection Calculations

Forces From Gusset Plate

$$r = \sqrt{(\alpha + e_c)^2 + (e_b + \beta)^2}$$
$$= \sqrt{(9.25 + 5.4)^2 + (8.85 + 10)^2}$$
$$= 23.87''$$



Column-to-Gusset Connection

$$H_c = \frac{e_c}{r} P_u = \frac{5.4}{23.87}(73.6^k) = 16.65^k$$

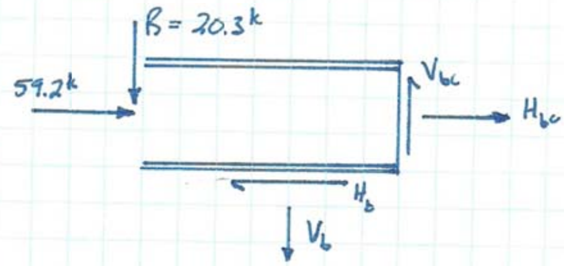
$$V_c = \frac{\beta}{r} P_u = \frac{10}{23.87}(73.6^k) = 30.83^k$$

Beam-to-Gusset Connection

$$H_b = \frac{\alpha}{r} P_u = \frac{9.25}{23.87}(73.6^k) = 28.52^k$$

$$V_b = \frac{e_b}{r} P_u = \frac{8.85}{23.87}(73.6^k) = 27.29^k$$

Beam-to-Column Connection



$$\Sigma F_x = 0 = 59.2 + H_{bc} - H_b$$

$$\therefore H_{bc} = 30.7^k$$

$$\Sigma F_y = 0 = V_{bc} - V_b - 20.3$$

$$\therefore V_{bc} = 47.6^k$$

Appendix K.1 –Braced Frame Connection Calculations

Beam-to-column Connection

Use 3/4" ϕ A325N bolts

$\phi F_{nt} = 67.5^k$
$\phi F_{nv} = 40.5^k$

Shear Stress in Bolts

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{47.6}{6(0.442)} = 17.95^{ksi}$$

Available Tensile Strength per Bolt

$$F_t' = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{(67.5/0.75)}{40.5}(17.95)$$

$$F_t' = 77.11^{ksi} < 90^{ksi} \quad \therefore \underline{OK}$$

$$\therefore \phi r_{nt} = \phi F_t' A_b = 0.75(77.11)(0.442) = 25.56^k$$

Calculate r_{ut}

$$r_{ut} = \frac{30.7^k}{6} = 5.12^k$$

Does Prying Occur?

$$p = \min \left| \begin{array}{l} gage = 3.806" \\ l_n/\#_{bolts} = 9"/3 = 3" \Rightarrow \text{controls} \end{array} \right.$$

$$\phi M_n = \frac{\phi F_u p t^2}{4} = \frac{0.9(58)(3)(3/8)}{4} = 14.68$$

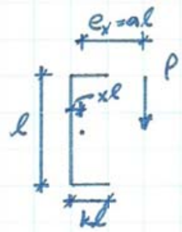
$$b = g - t/2 = 1.75 - \frac{3/8}{2} = 1.56"$$

$$b' = b - d_b/2 = 1.56 - \frac{0.75}{2} = 1.19"$$

$$r_{ut} b' = 5.12^k(1.19") = 6.08 < \phi M_n \qquad \therefore \underline{\text{Prying does NOT Occur}}$$

Appendix K.1 –Braced Frame Connection Calculations

Weld Design (Angle)

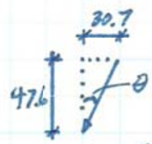


$\ell = 9''$
$k\ell = 2.5''$
$\therefore k = 0.2778$

Table 8-8 ⇒ Use $\theta = 30°$

	0.2	0.278	0.3
0.25	2.62	3.04	3.10
0.298		2.86	
0.3	2.45	2.85	2.91

$C = 2.86$



$\theta = 32.8°$

From Table 8-8
$k = 0.2 \Rightarrow x = 0.029$
$k = 0.3 \Rightarrow x = 0.056$
$\therefore k = 0.2778 \Rightarrow x = 0.035$

$$D_{min} = \frac{P_u}{\phi C C_1 \ell} = \frac{56.64}{0.75(2.86)(1.0)(9)} = 2.93/_{16ths}$$

$x\ell = 0.035(9) = 0.315''$

$a\ell = 3'' - 0.315'' = 2.685''$
$\therefore a = 0.298''$

$\boxed{\therefore \text{ Use } 3/16'' \text{ Weld Both Sides}}$

Angle Shear Yield

$$\phi R_n = \phi(0.6F_y)A_{gv} = 1.0(0.6)(36)(3/8)(9) = 72.9^k > 47.6^k \quad \therefore \underline{OK}$$

Angle Shear Rupture

$$\phi R_n = \phi(0.6F_u)A_{nv} = 0.75(0.6)(58)(3/8)(9 - 2(7/8)) = 71.0^k > 47.6^k \quad \therefore \underline{OK}$$

Bearing/Tear-Out (Angle)

$$\phi R_n = \phi[1.2 L_c F_u t] \le \phi[2.4 d_b F_u t] = 0.75[1.2(1.5 - \tfrac{3/4}{2})(58)(3/8)] \le [2.4(0.75)(58)(3/8)]$$
$$= 0.75[2(27.23) + 4(39.15)]$$

$$\phi R_n = 159^k > 47.6^k \quad \therefore \underline{OK}$$

Appendix K.1 –Braced Frame Connection Calculations

Angle Block Shear

$$\phi R_n = \phi[0.6F_u A_{nv} + U_{bs} F_u A_{nt}] \le \phi[0.6F_y A_{gv} + U_{bs} F_u A_{nt}]$$
$$= 0.75[0.6(58)(1.992) + 1.0(58)(0.3047)] \le 0.75[0.6(36)(2.8125) + 1.0(58)(0.3047)]$$
$$= 58.8^k > 47.6^k \quad \therefore \underline{OK}$$

$$A_{nv} = 3/8\,(7.5 - 2.5(7/8)) = 1.992\ in^2$$
$$A_{gv} = 3/8\,(7.5") = 2.8125\ in^2$$
$$A_{nt} = 3/8\,(1.25 - 0.5(7/8)) = 0.3047\ in^2$$

<u>Gusset-to-Beam Connection</u>

$$\phi R_n = 1.392\ D\ \ell(2) = 28.52^k \Rightarrow D = \frac{28.52}{1.392(17.5)(2)}$$

$$\therefore D_{min} = 0.585/16^{th}$$

$$D_{min} = 3/16" \Rightarrow \text{Table J2.4}$$

$$\boxed{\therefore \text{Use } D = 3/16"}$$

Appendix K.1 –Braced Frame Connection Calculations

Gusset - to- Column Connection

Shear Stress in Bolts — Use 3/4" ϕ A325 N Bolts

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{30.8^k}{6(0.442)} = 11.61^{ksi}$$

$\phi F_{nt} = 67.5^k$

$\phi F_{nv} = 40.5^k$

Available Tensile Strength per Bolt

$$F_t' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{(67.5/0.75)}{40.5}(11.61) = 91.2 > 90 \quad \therefore NG$$

Try 4 bolts

Shear Stress in Bolts

$$f_v = \frac{V_u}{\#_{bolts} A_b} = \frac{30.8}{4(0.442)} = 17.42 \text{ ksi}$$

Available Tensile Strength per Bolt

$$F_t' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3\left(\frac{67.5}{0.75}\right) - \frac{(67.5/0.75)}{40.5}(17.42) = 78.3 < 90 \quad \therefore \underline{OK}$$

$$\therefore \phi r_{nt} = \phi F_t' A_b = 0.75(78.3)(0.442) = 25.95^k > 16.65^k \quad \therefore \underline{OK}$$

Calculate r_{ut}

$$r_{ut} = \frac{16.65}{4} = 4.16^k$$

Does Prying Occur?

$$p = \min \begin{cases} \text{gage} = 3.806'' \\ 6/\#_{bolts} = 6''/2 = 3'' \Rightarrow \text{Controls} \end{cases}$$

$$b = g - t/2 = 1.75 - \frac{3/8}{2} = 1.56''$$

$$\phi M_n = \frac{\phi F_u p t^2}{4} = \frac{0.9(58)(3)(3/8)^2}{4} = 14.68$$

$$b' = b - d_b/2 = 1.56 - \frac{0.75}{2} = 1.19$$

$$r_{ut} b' = 4.16(1.19) = 4.95 < \phi M_n$$

$\underline{\therefore \text{Prying Does NOT Occur}}$

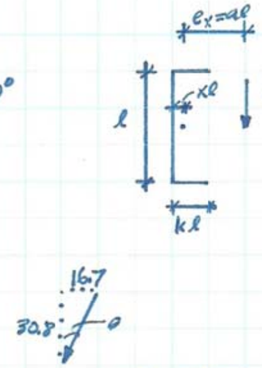
Appendix K.1 –Braced Frame Connection Calculations

Weld Design (Angle)

Table 8-8 ⇒ Use $\theta = 30°$

	0.2	0.278	0.3
0.25	2.62	3.04	3.10
0.298		2.86	
0.3	2.45	2.85	2.91

$C = 2.86$



$\theta = 28.5°$

$l = 9"$
$kl = 2.5"$
$\therefore k = 0.2778$

From Table 8-8
$k = 0.2 \Rightarrow x = 0.029$
$k = 0.3 \Rightarrow x = 0.036$

$\therefore k = 0.2778 \Rightarrow x = 0.035$

$xl = 0.035(9) = 0.315$

$al = 3" - 0.315 = 2.685$
$\therefore a = 0.298$

$$D_{min} = \frac{P_u}{\phi C C_1 l} = \frac{35.04}{0.75(2.86)(1.0)(9)} = 1.81/16^{th}$$

To Satisfy min weld sizes

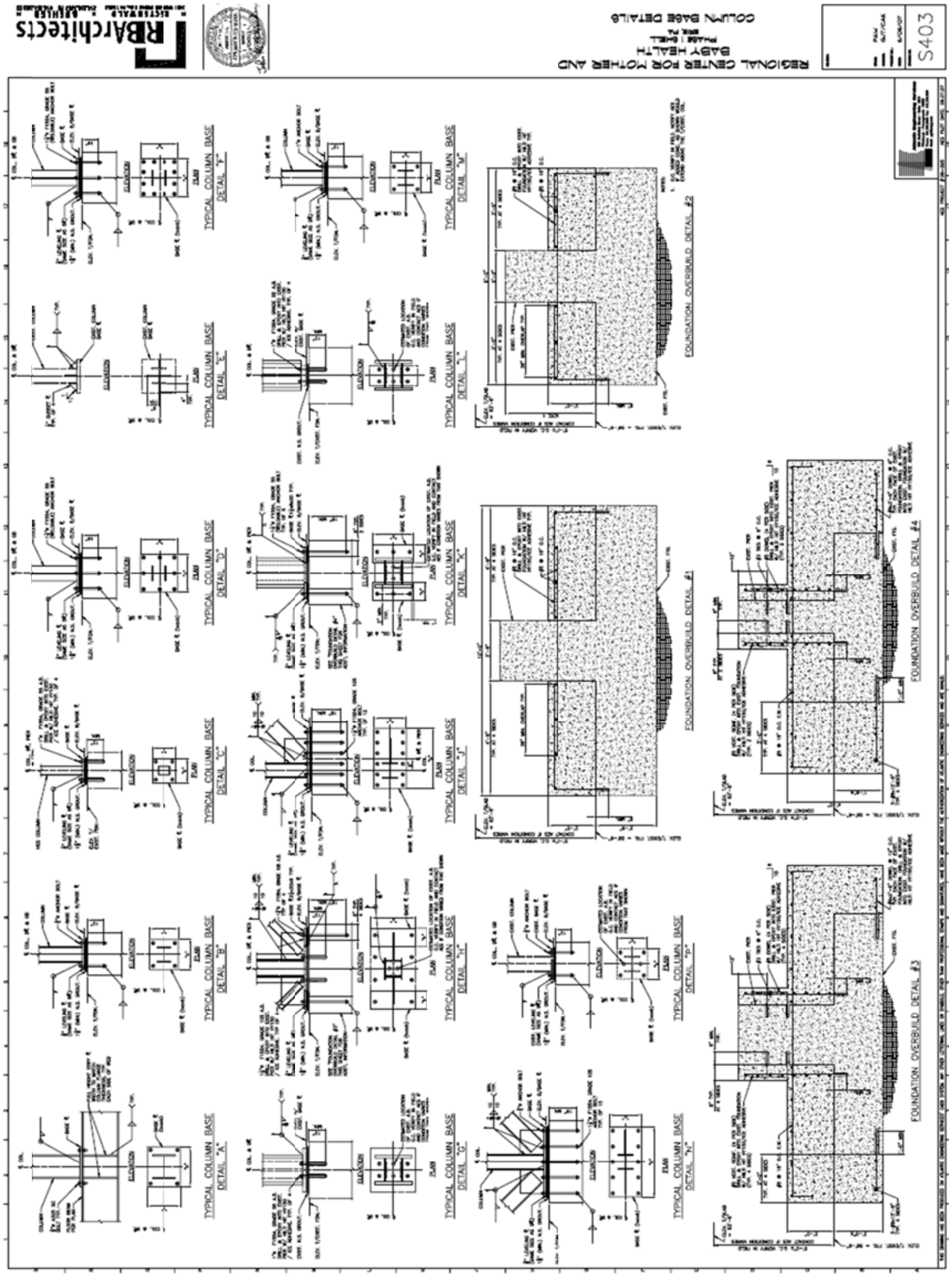
$\boxed{\text{Use } 3/16" \text{ Weld Both Sides}}$

Angle Shear Yield, Shear Rupture, Bearing/Tearout, + Block Shear

OK by previous work

Appendix L

Appendix L.1 – Sheet S403



Appendix L.1 – Sheet S400