St. Vincent Mercy Medical Center
Heart Pavilion

Toledo, Ohio

Final Thesis Report

Optimization of the Foundation & Lateral Systems

Kristen Marie Lechner | Advisor: Dr. Linda Hanagan | Structural Option | April 7, 2009

ST. VINCENT MERCY MEDICAL CENTER HEART PAVILION

TOLEDO, OHIO

PROJECT TEAM

OWNER: ST. VINCENT MERCY MEDICAL CENTER
ARCHITECT: MARTELL ASSOCIATES
STRUCTURAL: RUBY + ASSOCIATES
CIVIL: MANNIK & SMITH, INC.
MEP: JDRM ENGINEERING, INC.
CONSTRUCTION: THE LATHROP COMPANY

BUILDING STATISTICS

SIZE: 144,000 SQ. FT.
HEIGHT: 4 LEVELS, 57'-5" T/STEEL
CONSTRUCTION DATES: SUMMER 2005 - SPRING 2007
COST: $45 MILLION
DELIVERY METHOD: DESIGN BUILD

ARCHITECTURE

FIRST AND ONLY FACILITY IN TOLEDO FOR TREATMENT OF VASCULAR DISEASE

BRICK FACADE ACCENTED WITH STONE BANDS COMPLIMENTS A CURVED ALUMINUM AND SPANDREL GLASS CURTAIN WALL

FIRESTONE ROOF SYSTEM

PEDESTRIAN BRIDGE ALLOWS EFFICIENT EGRESS TO ADJACENT PARKING GARAGE

STRUCTURAL

STRUCTURAL STEEL FRAMING WITH 2" COMPOSITE STEEL DECK & 4-1/2" CONCRETE SLAB

TYPICAL BAY SIZE IS 25' x 30'

LATERAL LOAD RESISTING SYSTEM UTILIZES STEEL MOMENT FRAMES

FOUNDATION COMPRISED OF DRILLED CAISSONS AND SPREAD FOOTINGS

MEP

FORCED AIR SYSTEM UTILIZINGING CHILLED WATER, HEATED WATER, AND STEAM CONDITIONS ALL SPACES

EACH OPERATING ROOM SUPPLIED BY DEDICATED AIR HANDLING UNIT WITH HUMIDIFIER

POWER SUPPLIED BY ADJACENT HOSPITAL VIA 15KV UNDERGROUND CONCRETE DUCT BANK

PRIMARY LIGHTING PROVIDED BY LINEAR FLOURESCENT AND COMPACT FLUORESCENT LAMPS

RING OF SIX LAMP SURGICAL TROFFERS FOCUS LIGHT ON SURGICAL TABLES

TABLE OF CONTENTS

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LPCiminelli, Inc.:	Arnold Cubins
The Pennsylvania State University:	Dr. Linda Hanagan
	Dr. Andres Lepage
	Dr. Louis Geschwindner
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I would also like to say thank you to my fellow peers. We spent many hours together in thesis lab and I couldn't have done it without your support.

Lastly, I would like to thank my family, especially my mom and dad, for their endless encouragement over the past five years.

EXECUTIVE SUMMARY

St. Vincent Mercy Medical Center Heart Pavilion is a four story hospital that provides diagnostics, surgery, and patient care. It was constructed for St. Vincent's Mercy Medical Center Campus, established in 1855, in downtown Toledo, Ohio.

The facility is approximately 144,000 square feet and reaches a height of 57'6" above grade with a typical floor to floor height of approximately 14 feet. A typical interior bay is 30 feet by 35 feet and is comprised of composite steel with a concrete slab on deck. Non-seismic steel moment frames are utilized to resist lateral forces at every column in both directions.

The current site of St. Vincent Mercy Medical Center Heart Pavilion was chosen by the owner because it was already owned by Mercy Health Partners and it is adjacent to the main hospital. For these reasons, the Heart Pavilion was kept on the existing site. Upon investigation of the soil classification within the site, it was determined that the soil was classified as Seismic Site Class E. This significantly impacted the base shear value, leading to a seismically controlled building even when torsion effects were considered.

This final thesis report evaluates the efficiency of redesigning the foundation and lateral systems utilizing Geopiers and special steel moment frames. Improvements in soil conditions were achieved through the use of the Geopier System by providing vertical reinforcement to the soil. In addition, the construction time was reduced by approximately 50% as Geopier elements can be installed at a rate of 30 per day. This allowed steel erection to begin approximately 10 weeks earlier than originally scheduled.

The SMF's also prove to be more economical even though the fabrication time for the reduced beam section is twice that of the existing beams. The duration of detailing for the SMF system is 17 days as opposed to the 56 days required to detail the existing system. Due to this considerable reduction in installation time, the SMF system is more cost efficient even when special inspections are considered.

The façade breadth study focuses on improvements in occupant comfort with respect to heat transfer through the wall system. By implementing the brick façade on the third floor of the Heart Pavilion, heat transfer through the wall is reduced by approximately 30% of that transferred by the existing curtain wall system. Heat loss within patient rooms on this floor is reduced, thus improving occupant comfort.

The goals of this thesis were to create an efficient foundation and lateral system for the Heart Pavilion. Based on the results discussed, these goals are clearly met. From a feasibility standpoint, each proposed study impacts the structure in a positive manner. It is the recommendation of the author to implement all changes proposed within this thesis report.

BUILDING OVERVIEW

Function

St. Vincent Mercy Medical Center Heart Pavilion is a four story hospital that provides diagnostics, surgery, and patient care. It was constructed for St. Vincent's Mercy Medical Center Campus, established in 1855, in downtown Toledo, Ohio.

St. Vincent's Heart Pavilion is one of the seven hospitals that comprise Mercy Health Partners. As Toledo's first and only facility for the treatment of vascular disease, St. Vincent's Heart Pavilion has become a staple within the community. St. Vincent's Mercy Medical Center Campus is now able to take a leadership role in providing education to its students as well as saving lives through the treatment of vascular disease.

Architecture

Architectural considerations are centered around the patients' needs. With bed-side check in, patients are able to be taken directly to their room in every effort to make their stay comfortable. The facility is set up in such a manner that patients do not have to change rooms during the duration of their stay and accommodations for visitors staying the night are available in every room. Patient rooms are private and spacious with large windows that provide a great view to the outside, creating a positive mood within the space.

Modernization is emphasized through the façade of St. Vincent Mercy Medical Center Heart Pavilion. As one approaches the building from the North, a beautiful curtain wall composed of curved aluminum and spandrel glass is seen, thus adding great verticality to the building. As the eye gazes along the façade, stone bands and brick veneer promote horizontal progression to an attractive vertical component of stairs wrapped in stone veneer and spandrel glass. The eye is then led to the pedestrian bridge, connecting the Heart Pavilion to a parking garage, which shows off its structure through exposed chevron bracing.

Construction Management

The construction of St. Vincent Mercy Medical Center Heart Pavilion started in the late summer of 2005 and was completed in the spring of 2007. The general contractor was The Lathrop Company (now Turner Construction) and was delivered as design-build.

From the time of design, the construction schedule was a priority. Matching the height of the deck, the structural engineers placed all girders 2" higher than the beams on a typical floor and 1 ½" higher on the roof. This design saved money due to the fact that the infill beams no longer required coping. Steel was erected much quicker as a result of saved fabrication time. In addition to these benefits, the deck connection to the girder automatically provides a pour-stop, making placement of the concrete easier. Please reference Figure 1 below to view a site plan for the Heart Pavilion.

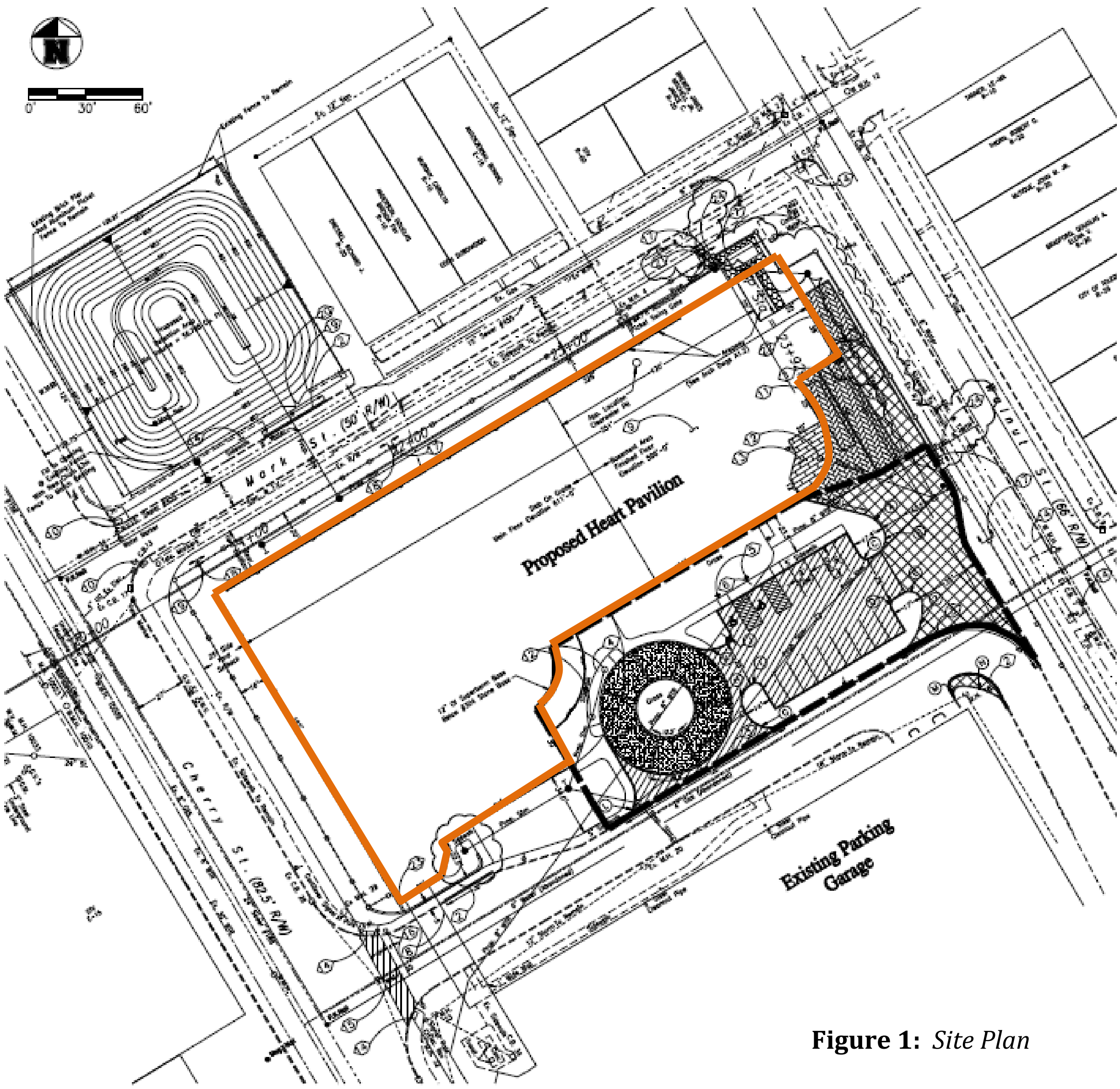


Figure 1: *Site Plan*

Mechanical System

A forced air system is utilized for this facility, employing chilled water, heating water, and steam to condition all spaces. Three medium pressure rooftop air handlers supply a total of 134,000 CFM to the main hospital. In order to obtain acceptable humidity levels for a hospital space, each rooftop unit is equipped with a humidifier.

As a high level of zoning is required by hospital guidelines, air is distributed via a medium pressure duct system to a series of VAV (Variable Air Volume) and CAV (Constant Air Volume) terminal units. Each terminal unit has a reheat coil to maintain space temperature. A hot water ceiling radiant panel system supplements the heating of the space.

Each operating room is supplied by a dedicated air handling unit with a humidifier. Air is supplied to the space with a Price HORD air distribution system, as shown in Figure 2. Heat is supplied by three 210 GPM boilers. Two of these boilers are primarily used, while the additional boiler is used for backup. Steam for humidification, domestic water, and sterilization is provided by two (one primary and one backup) 2940 MBH 60 psi steam boilers. Chilled water is supplied by two 375 ton chillers with a 750 ton cooling tower on the roof.

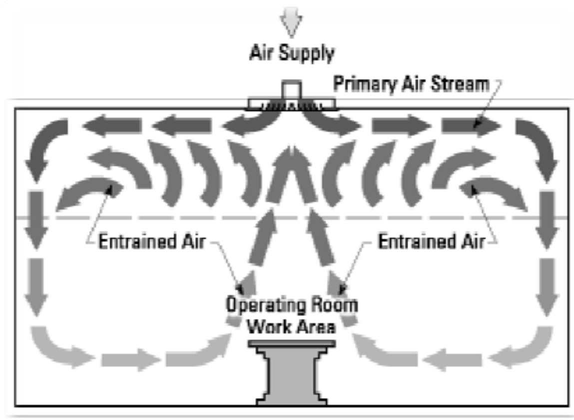


Figure 2: *Detail of Price HORD Air System photo courtesy of: www.price-hvac.com*

Lighting & Electrical Systems

The main hospital across Cherry Street supplies normal and emergency power to St. Vincent Mercy Medical Center Heart Pavilion via a 15kV underground concrete duct bank. The 15kV normal feeder is actually two feeds, each of which is capable of supplying all necessary power to the hospital under normal operating conditions and feeds a double ended substation. Dual feeds are utilized for two reasons: they provide redundancy in the electrical system in the event of a failure of one feeder, and they provide a way for periodic maintenance of the breakers on either end of the feeders. The emergency feeder feeds a single ended substation. Each substation is equipped with a transformer that steps the voltage down from 12,470V to 480Y/277V. From there the power is distributed to the chillers and automatic transfer switches.

For the use of lights and receptacles throughout the building, the voltage is further stepped down to 208Y/120V by large central transformers located in the basement. Also located in the basement are two uninterruptible power systems (UPS). These systems are very important as they provide power to operating rooms and telecommunication rooms during the event of an outage as the emergency generators are brought online.

Various types of light fixtures were used throughout the building. Linear fluorescent and compact fluorescent lamps are the primary lamps seen throughout the facility. The corridors utilize 2'x2' direct/indirect fixtures and 24" diameter acrylic bowl fixtures. Wall sconces are placed on either side of the caregiver stations which are located between the patient rooms. Compact fluorescent downlights and a 2'x4' multi-function fixture are placed over patients' beds. The multi-function fixture provides exam, ambient, and reading light that can be controlled either by wall switches or the pillow speaker.

EXISTING STRUCTURAL DESCRIPTION

Floor System

St. Vincent Mercy Medical Center Heart Pavilion's typical floor system is made up of composite steel framing and normal weight concrete, creating a total floor thickness of 6½". Composite action is created by the use of 2" 20 gauge steel deck with 5½" long, ¾" diameter shear studs evenly spaced over the length of each beam. Even though a composite system is used, the girders are actually non-composite. In order to avoid coping of the infill beams, the girders are placed 2" higher than the beams on a typical floor and 1½" higher on the roof as seen in Figure 3. This system saved money and fabrication time which resulted in faster steel erection. Please reference Figures 4 & 5 to view a typical interior bay and floor framing plan of the Heart Pavilion.

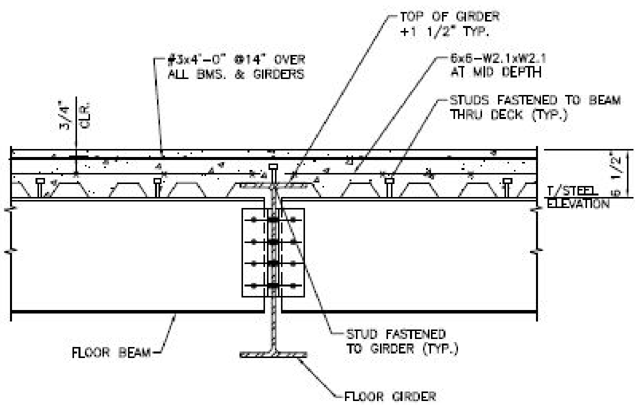


Figure 3: *Detail of Existing Composite Steel Floor System*

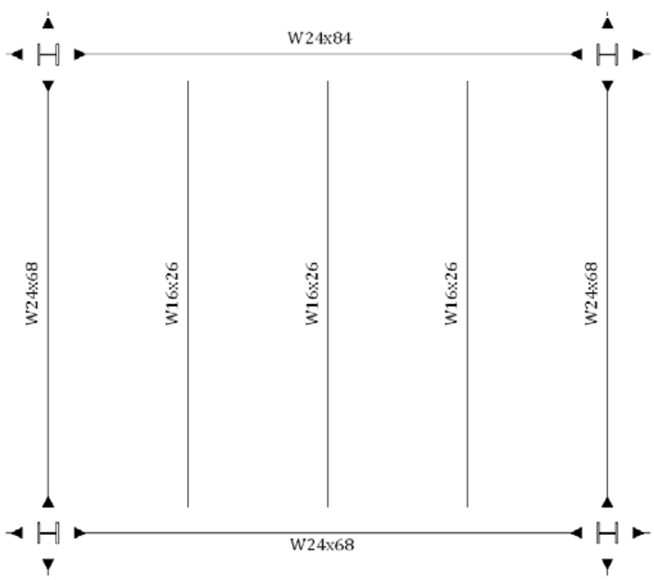


Figure 4: *Typical Interior Bay*

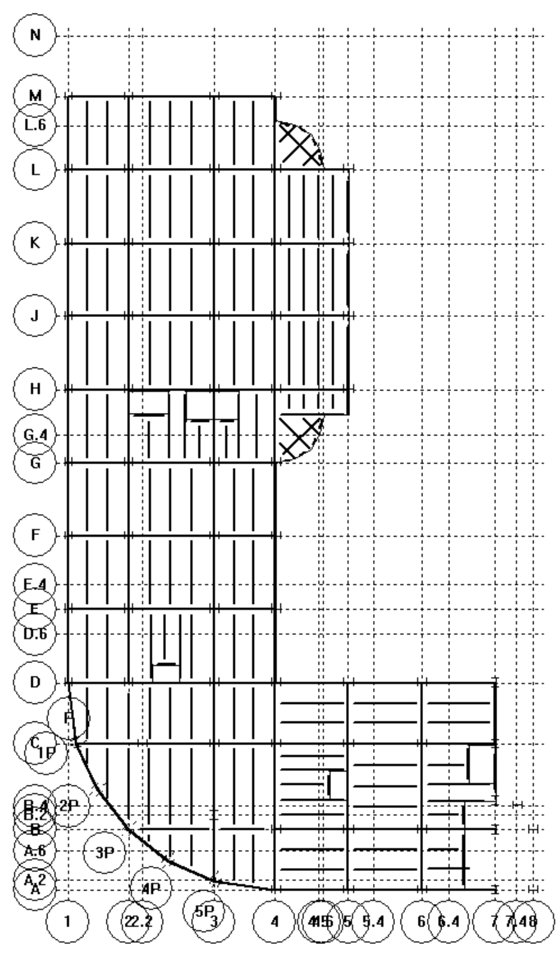


Figure 5: *Typical Floor Framing Plan*

Roof System

The roof system is comprised W14x22 beams framing into W18X40 girders. This system was used as opposed to joists because surgery space was originally located directly below the main roof. The mechanical and medical equipment required for these spaces is easier to hang from wide flange beams.

The roof envelope is surrounded by a parapet wall. The roof system is made up of two systems: 1 ½" x 22 GA. galvanized steel roof deck providing enclosure to the building, and 6 ½" normal weight concrete on 1 ½" 20 GA. galvanized metal deck providing support for the mechanical equipment located in the two penthouses. Both systems are topped with a single ply membrane, ¾" plywood, and a minimum of 3" rigid insulation.

Columns

The columns used in St. Vincent Mercy Medical Center Heart Pavilion range from W10x119's to W12x210's, depending on their location within the building. While these sizes may seem large based purely on gravity, each column must resist induced moment since all columns are part of a moment frame. Pipe columns are used to support the roof for the main entrance lobby and the emergency vestibule canopy. All of the main building columns are spliced at the 2nd-3rd floor. Base plates range in thickness from 1" to 2 ¼" depending on which columns they are supporting. Each base plate utilizes a standard 4 bolt connection using either ¾" A325 or 1 ¼" A325 bolts.

Lateral System

At the time of design, braced frames were thought to be architecturally incompatible with this floor plan. As a result, non-seismic steel moment frames were used for the lateral load resisting system. Classified as Seismic Site Class E soil, the number of moment frames required to resist seismic loading are shown in Figure 6.

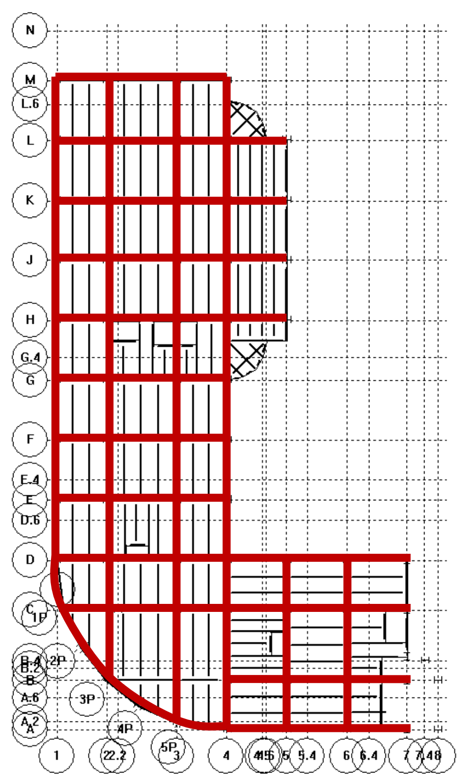


Figure 6: *Plan View of Lateral System*

The moment frames are connected in two different fashions as seen in Figures 7 and 8. The beam to column web moment connection is comprised of flange plates that are fillet welded to the column web and flange. The beam flanges are full-penetration welded to these plates. The beam to column flange moment connection utilizes double angles connecting the beam to the column flange, where the column flange is then full penetration welded to the beam flange.

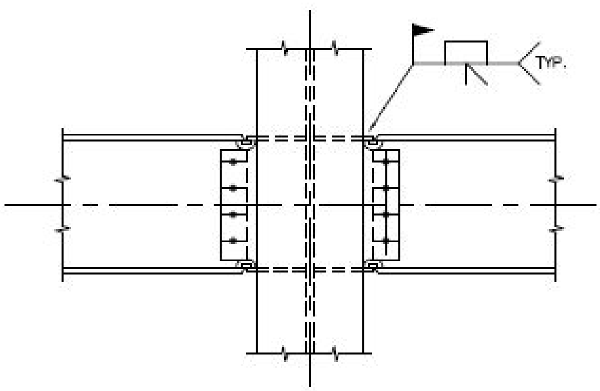


Figure 7: *Beam to Column Web Connection*

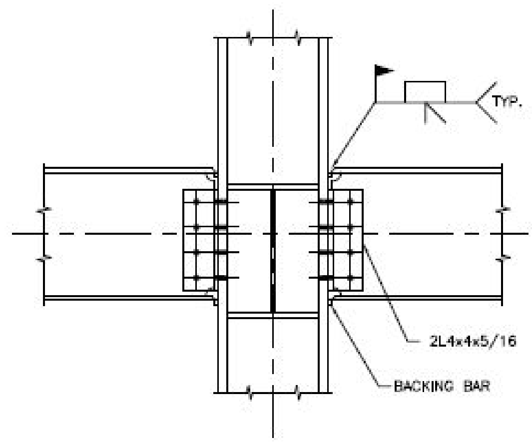


Figure 8: *Beam to Column Flange Connection*

Foundation System

The foundation system is made up of 80 drilled caissons and 6 spread footings that support the entrance lobby. The caisson caps are a uniform size of 4'x4'x3' thick. Between caissons are grade beams, varying in depth from 2' to 4' depending on the location, which transfer façade and wall load to the foundation system. Please reference Figure 9 to view a typical caisson detail. The ground (main) floor rests on a 6" concrete slab reinforced with W/4x4-W4.0x4.0 welded wire fabric.

The use of a deep foundation system with structural steel framing may seem odd at first glance, however the site soils were reported to be very soft in nature. When the geotechnical investigation was done, it was found that the first 12 feet below grade were poorly graded sand with silt, silty sand, silty clay, and lean clays. Very stiff lean clays were encountered to a depth of 80 feet below existing grade. Below depths of 80 to 85 feet, very dense silty and sandy soils and heavily consolidated clayey soils were encountered in all borings to depths of 89 to 100 feet, where the drilling process was terminated. As a result, drilled caissons with belled bases were recommended for the foundation system based upon the soft clays found 12 to 40 feet below grade.

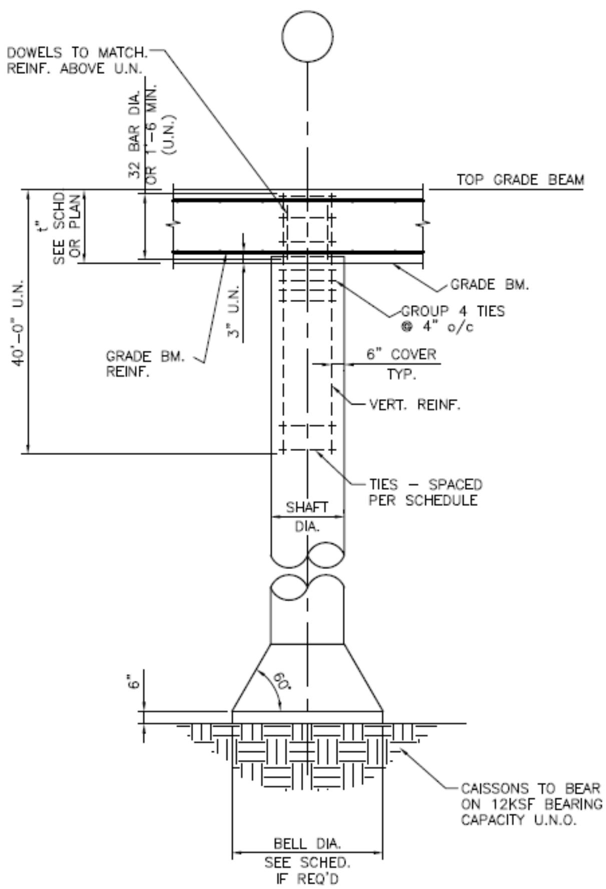
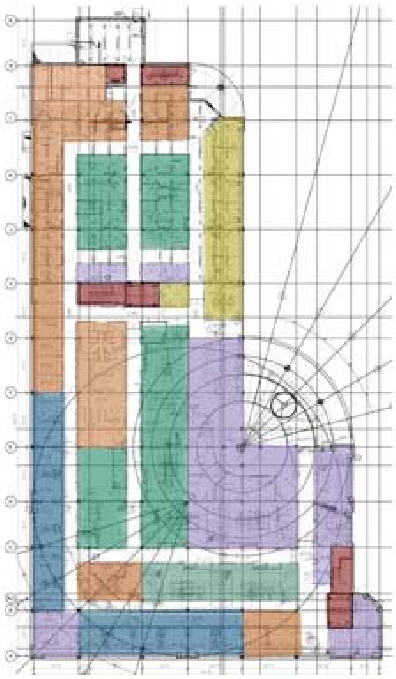


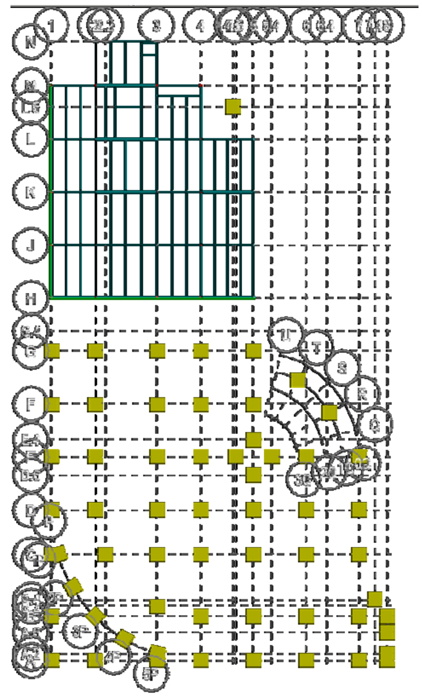
Figure 9: *Caisson Detail at Interior Grade Beam*

FLOOR PLANS & BUILDING PHOTOS

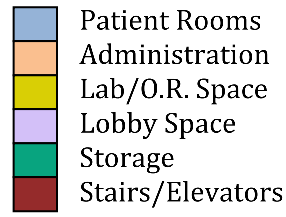
The following figures are provided for a side by side reference of architectural function and floor framing for each floor within the Heart Pavilion.

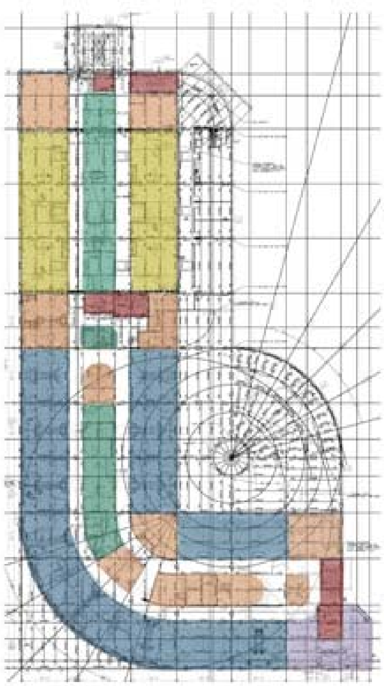


Main Floor Architectural Plan

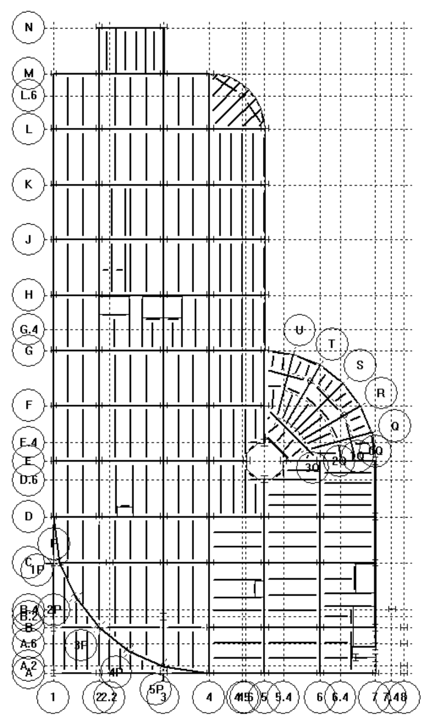


Main Floor Framing Plan

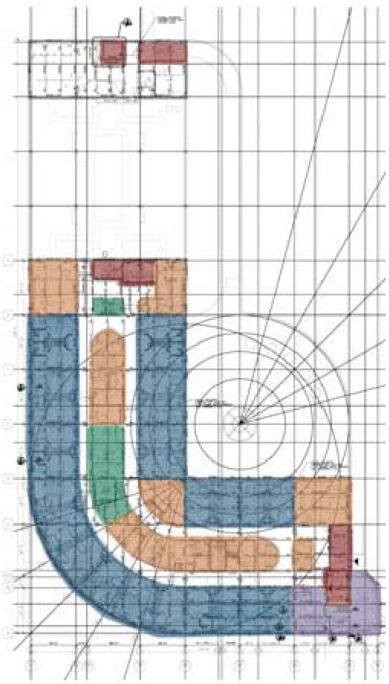




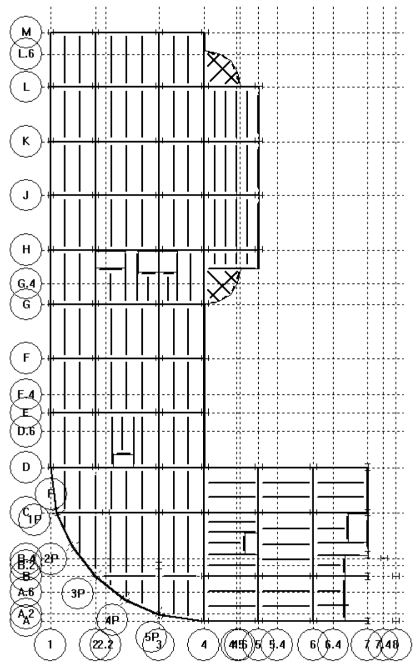
First Floor Architectural Plan



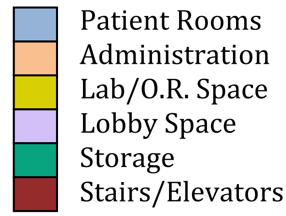
First Floor Framing Plan

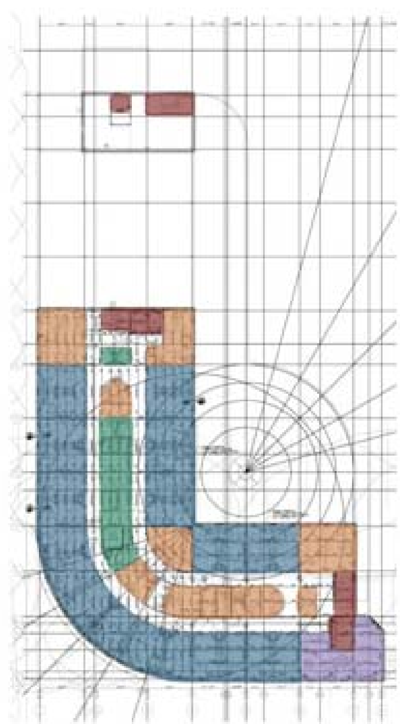


Second Floor Architectural Plan

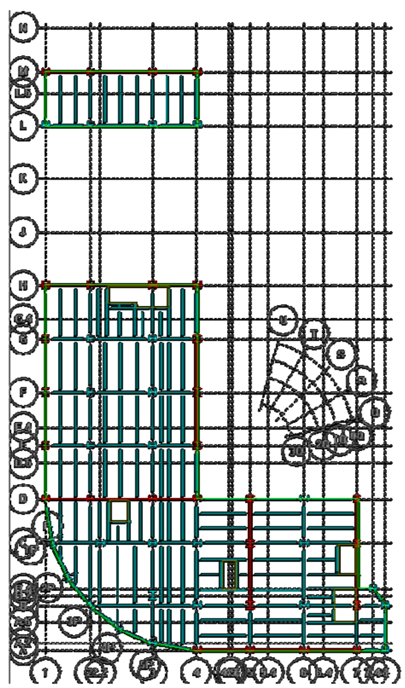


Second Floor Framing Plan





Third Floor Architectural Plan



Third Floor Framing Plan

CODE REFERENCES & MATERIAL PROPERTIES

The following table shows the code references used by the design engineer and those used throughout the duration of this thesis study.

Codes used for this thesis	Codes used by the engineer of record
2006 IBC as adopted by the State of Ohio	2002 IBC as adopted by the State of Ohio
Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers	Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), American Society of Civil Engineers
Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings—LRFD, Thirteenth Edition, American Institute of Steel Construction	Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings—LRFD, Third Edition, American Institute of Steel Construction
The Building Code Requirements for Structural Concrete (ACI 318-08), American Concrete Institute	The Building Code Requirements for Structural Concrete (ACI 318-02), American Concrete Institute

Multiple materials were used for the construction of St. Vincent Mercy Medical Center Heart Pavilion. The details of these materials are listed in the following table.

Concrete	Strength	Density
Foundations	3000 psi	150 pcf
Walls	3000 psi	150 pcf
Slabs	3500 psi	150 pcf
Grade Beams	4000 psi	150 pcf

Reinforcing Steel	ASTM	Metal Deck & Shear Studs	Size
Reinforcing Bar	A-615	Composite Floor	2” 20. GA.
Tie Wire	A-82	Roof Deck	1 ½” 22 GA.
Welded Wire Fabric	A-185	Shear Studs	¾” x 5 ½”

Structural Steel	ASTM	F_u (ksi)	F_y (ksi)
Wide Flange	A992	65	50
Angle, Plate, Channel	A36	65	50
Square/Rectangle (HSS)	A500, Grade B	58	46
Round (HSS)	A500, Grade B	58	42
Connection Bolts	A325		
Anchor Bolts	A307 or A36		

STRUCTURAL DEPTH

Existing Structural System Check

Loading Conditions

Loading conditions are a very important consideration for the design of any structure. The dead load conditions assumed by the engineer of record at the time of design and live load conditions obtained from ASCE 7-02 are provided for reference below. The dead and live load values listed in Figure 10 are the values used for the duration of this thesis project.

Applicable Loads			
Dead Loads		**Live Loads**	
Concrete	150 pcf	1st Floor Corridors	100 psf
Steel	490 pcf	Lobbies	100 psf
Partitions	20 psf	Loading Dock	100 psf
M.E.P.	10 psf	Penthouse Floor	100 psf
Windows & Framing	10 psf	Corridors above 1st Floor	80 psf
Finishes & Misc.	5 psf	Patient Rooms	60 psf
Roof	20 psf	Operating Rooms	60 psf
		Bridge Floor	60 psf
		Roof	20 psf

Figure 10: *Applicable Design Loads*

Vibration Criteria

In the early design concepts for the Heart Pavilion, the surgery suite was located on the third floor as shown in Figure 11. However, this space was later moved to the East side of the building on the first floor as shown in Figure 12. The beams supporting this space were not designed for vibration criteria since it was not the original surgery space. Please reference Appendix B for detailed calculations using AISC Design Guide 11 for the vibration criteria check on this space.

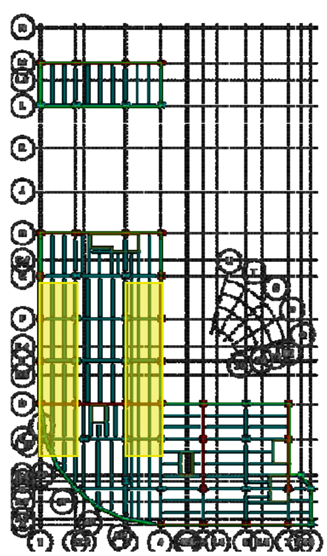


Figure 11: *Original Surgery Suite on 3rd Floor*

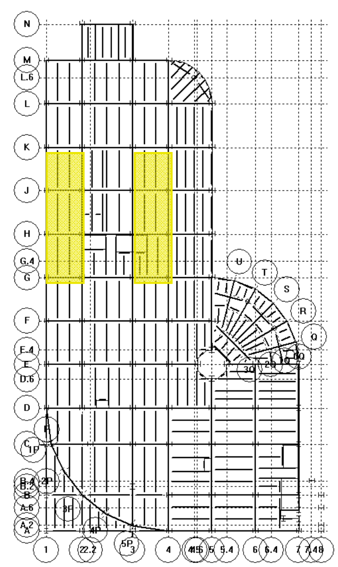


Figure 12: *Relocated Surgery Suite on 1st Floor*

Wind Loads

Wind loads were analyzed using the analytical procedure of ASCE 7-05 §6.5. The assumptions listed below were used to determine gust effect factors, wind pressures, and story shears. The following tables show calculated story forces for wind acting in the North-South direction and the East-West direction. As expected, wind forces were not found to control the structural design of the lateral system once torsion was considered in the RAM Model, since this facility sits on Seismic Site Class E soil. Please refer to Appendix A for more information regarding wind analysis.

Basic Wind Speed	90 mph
Exposure Category	B
Importance Factor	1.15
Internal Pressure Coefficient	±0.18
Directionality Factor	0.85
Topography Factor	1.0

Figures 13 & 14 below show the calculated wind pressures and story forces for wind acting in the North-South direction and the East-West direction, respectively. As expected, wind forces were not found to control the structural design of the lateral system once torsion was considered in the RAM Model, since this facility sits on Seismic Site Class E soil. Please refer to Appendix A for more information regarding wind analysis.

Floor Height (ft)	Level	Total Height (ft)	K_z	q_z	Wind Pressures (psf)					
					N-S Wind-ward	N-S Lee-ward	N-S Side Wall	E-W Wind-ward	E-W Lee-ward	E-W Side Wall
14.50	Roof	57.50	0.84	17.09	14.10	-9.97	-12.54	14.31	-7.54	-12.91
14.00	3	43.00	0.78	15.74	13.23	-9.97	-12.54	13.42	-7.54	-12.91
14.00	2	29.00	0.69	14.06	12.15	-9.97	-12.54	12.32	-7.54	-12.91
15.00	1	15.00	0.57	11.65	10.59	-9.97	-12.54	10.73	-7.54	-12.91

Figure 13: *Distribution of Windward and Leeward Pressures*

Level	Wind Design					
	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	42	28	0	0	2437	1580
3	82	53	42	28	3536	2287
2	78	50	125	81	2254	1450
1	76	48	202	131	1137	726
Total	**278**	**179**	**278**	**179**	**9364**	**6043**

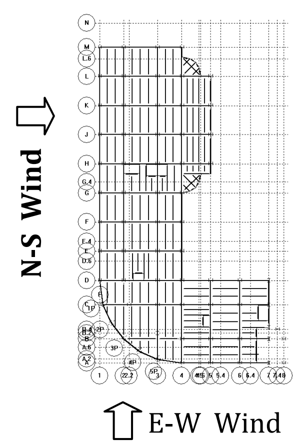


Figure 14: *Total Base Shear from Windward and Leeward Pressures*

Seismic Loads

The Heart Pavilion is a hospital in which surgery is performed, therefore it is categorized as occupancy category IV and uses an importance factor of 1.5 as shown in the table below.

Occupancy Category	IV
Importance Factor (I)	1.5
Seismic Design Category	C

The following values describe the site's response to earthquake ground motion.

Mapped Spectral Response Accelerations	S_S=0.170 S_1=0.056

The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters were determined according to ASCE 7-05 § 11.4.3.

Site Class	E
Site Class Factors	F_a=2.5 F_v=3.5
$S_{MS}=F_a(S_a)$	0.425
$S_{M1}=F_v(S_1)$	0.196

The following design spectral acceleration parameters were determined per ASCE 7-05 § 11.4.4.

$S_{DS}=2/3(S_{MS})$	0.283
$S_{D1}=2/3(S_{M1})$	0.131

The main lateral force resisting system for this facility is non-seismic steel moment frames. The base shear value was determined in accordance with Chapter 12 of ASCE 7-05. The following design values and limitations were used for the existing design.

Response Modification Factor (R)	3 (non-seismic steel moment frames)
Deflection Amplification Factor (C_d)	3
Over Strength Factor (Ω_0)	3
Building Height Limitation	Not Limited
Span to Depth Ratio	35'/30' = 1.167
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid

The Seismic Response Coefficient was determined per ASCE 7-05 § 12.8.1.1.

C_t	0.028
C_s	0.092

After calculating all of these seismic coefficients, the story forces were then calculated based on the weight of each floor. Once this was done, the base shear and overturning moment were determined as seen in Figure 15 below.

Base Shear and Overturning Moment Distribution

Story	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.5	1132	100432	0.219	241	241	13817
3	43	2824	181955	0.396	436	677	29103
2	29	2751	114571	0.250	275	951	27591
1	15	3100	62203	0.135	149	1100	16507
Main	0	2236	0	0.000	0	1100	0
Total	57.5	12043	459162	1.000	1100		87017
Base Shear =	**1100**	**k**					

Figure 15: *Base Shear and Overturning Moment Distribution*

RAM Modeling of the Existing Lateral System

The Heart Pavilion was modeled in RAM Structural System in order to verify lateral forces calculated by hand. The following assumptions were made during the modeling process:

- A rigid diaphragm was assigned to every floor within the model.
- Dead and live loads were assigned to the diaphragm with respect to what function the spaces served.
- All columns were assumed to be pinned at the base because this more conservatively predicts the actual behavior.
- All beams and columns within the moment frames were assigned fixed at each end (except the columns at the base).
- The total number of load combinations generated within RAM was 321.
- A 5% eccentricity was applied to account for accidental torsion of seismic loading.
- P-Delta effects were automatically taken into account within the model.

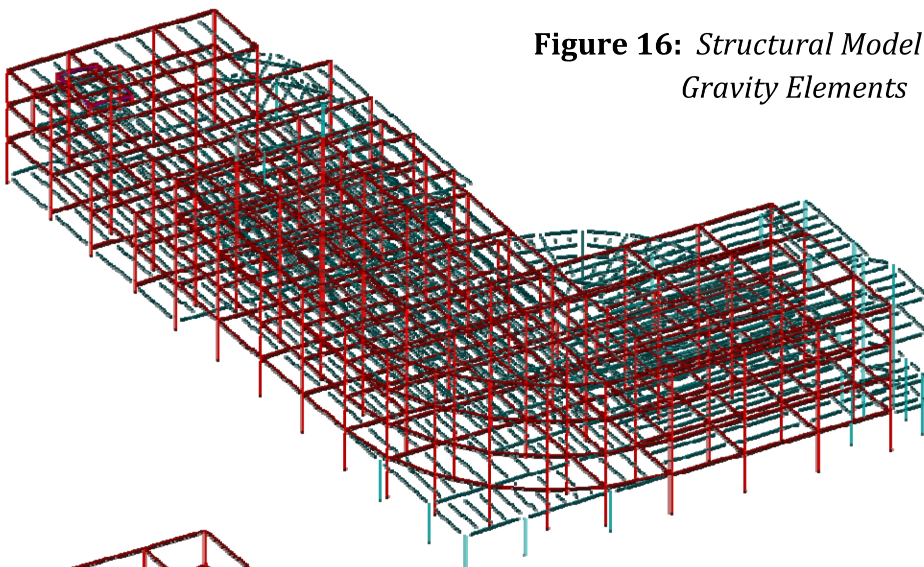


Figure 16: *Structural Model Including Gravity Elements*

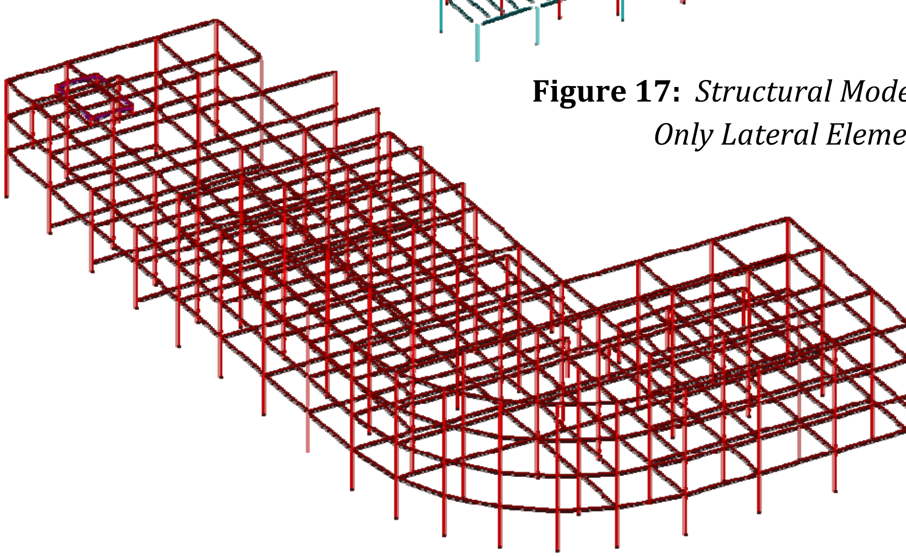


Figure 17: *Structural Model Displaying Only Lateral Elements*

Torsion Effects

Inherent Torsion

Per ASCE 7-05 §12.8.4.1, diaphragms that are not flexible must consider inherent torsional moment at each level. When the resultant shear force of lateral loads acts at an eccentricity, the resultant force will try to twist the building around its center of rigidity. This concept is known as torsion. Depending on the building footprint, torsion effects can have a significant impact on the controlling load case used for structural design.

Accidental Torsion

Per ASCE 7-05 §12.8.4.2, diaphragms that are not flexible must also consider accidental torsional moment for seismic loading. This is caused by assumed displacement of the center of mass away from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces.

Controlling Load Case

After taking torsion effects from lateral loads into account as well as the load combination factors for both wind and seismic, it was concluded that seismic loading controls the structural design of St. Vincent Mercy Medical Center Heart Pavilion. This was expected as the base shear for seismic loads was approximately 1100 kips as opposed to a base shear of 278 kips for wind in the North-South direction. However, a torsion analysis was necessary because greater torsion forces were generated by wind loading. This result was expected because the eccentricity for torsion from wind is measured from the center of pressure to the center of rigidity whereas the eccentricity for torsion from seismic is measured from the center of mass to the center of rigidity. Based upon this conclusion, the controlling LRFD load combination for this structure is 1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live) since the structural design is ultimately controlled by seismic loading.

Seismic Design Forces

Upon reaching the conclusion that this is a seismically controlled site, a comparison of the hand calculated story forces and shears and the RAM output was prepared. Please reference Figure 18 below to view the percent difference between these values.

	Seismic Design					
	Story Loads (k)			Story Shears (k)		
Story	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	241	231.85	3.9	241	235.41	2.4
3	436	395.18	10.3	677	647.49	4.6
2	275	259.65	5.9	951	916.40	3.8
1	149	157.31	5.3	1100	1112.48	1.1
Total Base Shear (k)	**1100**	**1044**	**5.4**			
Overturning Moment (ft-k)	**87,017**	**84,617**	**2.8**			

Figure 18: *Story Forces for Seismic Design*

Serviceability

Drift is an important serviceability requirement that can cause several problems within a building if the limitations are not met. Seismic drift is addressed in ASCE 7-05 and is limited based on the occupancy category of the building. St. Vincent Mercy Medical Center is classified as occupancy category IV and normally would be limited to an allowable story drift of 0.010 h_{sx}. However, since the facility is only 4 stories, the allowable story drift is limited to 0.015h_{sx} per ASCE 7-05 Table 12.12-1. Story drift ratios for seismic loading were determined by RAM Frame as summarized in Figure 19.

Seismic Drift					
Story	Story Height (ft)	Actual Drift Ratio		Allowable Drift Ratio	
Roof	57.5	0.0021	<	0.0075	OK
3	43	0.0036	<	0.0075	OK
2	29	0.0046	<	0.0075	OK
1	15	0.0047	<	0.0075	OK

Figure 19: *Actual Seismic Drift Ratio vs. Code Limitations*

Existing Design Check Summary

The following table provides a summary of findings upon completion of the analysis of the existing lateral system.

Check	Comment	Status
Modal Period	ASCE 7-05 Approximate period= 1.22 s RAM model period= 1.427 s Since the RAM model period is higher than the approximate period and the structure is proportionally related to the inverse of the stiffness, it can be concluded that the structure is not overdesigned	OK
Torsion	Inherent and accidental torsion were both taken into account in the RAM Model	OK
Redundancy	Structure is assigned to SDC C, therefore value for ρ is allowed to be taken as 1.0 per ASCE 7-05 § 12.3.4.1	OK
Member Spot Checks	Member sizes meet strength requirements; however vibration criteria for O.R. spaces are not met. Refer to Appendix B for detailed calculations.	NG
Story Drift	Drift requirements are met in both orthogonal directions	OK

Existing Lateral System & Serviceability Problem Statement

The nature of the site for St. Vincent Mercy Medical Center Heart Pavilion had a significant impact on the structural design of the building. Based on field and laboratory test data within the geotechnical report, it was determined that the soil located 12 to 40 feet below existing grade has an un-drained shear strength of less than 500 psf. For this reason, the soil is classified as very soft to soft lean clay and is characterized by the Ohio Building Code as Seismic Site Class E, “Soft Soil Profile”. This significantly impacted the base shear value, leading to a seismically controlled building even when torsion effects were considered.

In order to avoid seismic detailing, non-seismic steel moment frames were placed at every column line in both directions. While avoiding seismic detailing is typically the most cost effective choice, it may prove to be a valid solution for this poor soil site. By using a higher response modification coefficient, seismic loads are lowered and the base shear value is decreased. As a result, fewer steel moment frames would be required to resist the seismic forces which may have a significant impact on construction cost and time.

At the time of design, the third floor was designed for vibration criteria as the operating rooms were originally located on this floor. However, the O.R. spaces were later moved to the first floor on the East side of the building. Since it was late in the design process, the structure supporting these floors was not redesigned to meet vibration criteria.

Drilled caissons with belled bases were used for the foundation system based upon the soft clays found 12 to 40 feet below grade. Deep foundation systems are widely used; however, there are other solutions that may be explored to actually improve soil conditions. A Geopier Intermediate Foundation System is a soil replacement method that actually provides vertical reinforcement for the soil through the method in which they are placed. A beveled tamper is used to ram well graded aggregate into the drilled cavity where the poor soil was removed. A bottom bulb is formed at the bottom of the cavity from the beveled tamper ramming the aggregate down. This ramming process prestresses and prestrains the aggregate at the bottom of the cavity, causing lateral pressure to build up in the surrounding soil. The aggregate is rammed into the cavity in thin lifts of approximately one foot to form the full Geopier element. This foundation system may prove to be very efficient and cost effective for the Heart Pavilion.

M.A.E. Acknowledgement

Structural computer modeling will be used intensely throughout the duration of this thesis project. The Heart Pavilion will be modeled in RAM Structural System in order to predict the reaction of the redesigned structure under gravity and lateral loads. In addition, basic connection design principles will be utilized to carry out more advanced calculations for the design of seismically detailed connections.

Surgery Space Redesign

The beams supporting the operating rooms in the Heart Pavilion were redesigned to accommodate vibration criteria per AISC Design Guide 11 Chapter 6. According to Table 6.1, Vibration Criteria for Sensitive Equipment, operating rooms are required to meet a vibrational velocity limit of 8,000 μ in/sec. The final design of the beams for the surgery space is shown in Figure 20. Please reference Appendix C for detailed calculations on the design of these beams for this critical space.

The following assumptions were taken into account for the redesign of these beams:

- Assumed weight of a person was 185 pounds
- Assumed walking velocity was 100 steps per minute (considered "fast walking" per AISC Design Guide 11)

In order to design these beams for vibration, the following steps were taken:

- Determine y_{bar}, effective slab width, and transformed moment of inertia for beams and girders
- Find the mid-span flexibilities of the beams and girders (Δ_{oj} & Δ_{gP})
- Determine the effective number of tee-beams (N_{eff})
- Determine mid-bay flexibility (Δ_P)
- Establish footfall to weight ratio (F_m/W) from Table 6.2
- Determine the footfall impulse parameter (F_m)
- Find the corresponding pulse rise frequency (f_o)
- Establish the vibrational frequency of the floor slab (f_n)
- Determine pulse rise frequency to floor slab frequency ratio (f_o/f_n)
- Find maximum displacement of the floor (X_{max})
- Find the constant corresponding to walker weight and walking speed (U_v)
- Determine the maximum vibrational velocity of the floor (V) and compare to allowable vibrational velocity as determined in Table 6.1

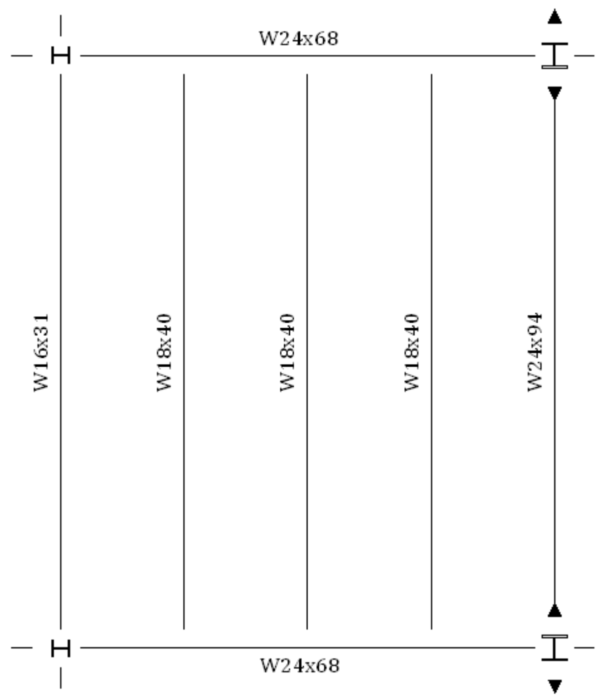


Figure 20: *Typical Interior Bay for O.R. Spaces*

Lateral Force Resisting System Redesign

Introduction

The current site of St. Vincent Mercy Medical Center Heart Pavilion was chosen by the owner because it was already owned by Mercy Health Partners and it is adjacent to the main hospital. This structural depth will focus on the effects of using a higher response modification coefficient to reduce seismic loads, thus reducing the number of lateral resisting elements required to resist lateral loads within this poor soil site. Ideally, this solution would optimize the lateral system while still meeting the demands of the seismically controlled site.

The reduced beam section was not a very commonly used detail prior to the 1994 Northridge earthquake and the 1995 Kobe earthquake. However, after these two disasters, it was observed that the welded connections within steel moment frames were experiencing premature brittle fracture. As a result, the reduced beam section (RBS) became more commonly used for seismically detailed connections.

The RBS configuration is a weakening method. It essentially forces yielding to occur in the beam, away from the connection. This is done by reducing the plastic moment capacity of the beam at a short distance from the column face. The design concept is to concentrate damage at certain points that will not affect the gravity load carrying capacity of the structure. The reduced flange portion of the beam can be thought of as a "structural fuse" that dissipates energy by going through plastic deformation. By weakening this portion of the beam, it is forced to yield before anything else. This allows the rest of the beam, the columns, and connections to remain elastic. Figure 21 below illustrates this concept.

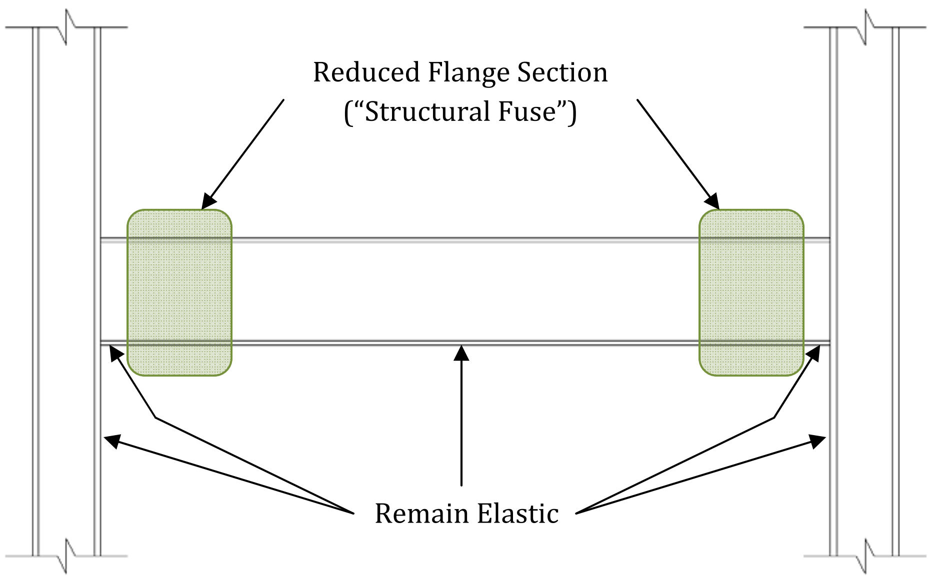


Figure 21: *Design concept of RBS connections*

A higher response modification coefficient is permitted to use for an SMF system because of the energy dissipation capacity of the structure. Since SMF's are very flexible and can dissipate energy very efficiently, seismic forces can be reduced by using a higher R value. As seen in the table below, the special moment frame system will reduce approximately 38% of the base shear value.

	Non-Seismic Steel Moment Frames (Existing System)	**Seismically Detailed Steel Moment Frames (New System)**
Response Modification Coefficient (R)	3	8
Approximate Period (C_uT_a)	1.22	1.22
Seismic Response Coefficient (C_S)	0.092	0.034

Structural Depth Design Goals

The goals of this structural depth are listed as follows:

- Reducing the number of steel moment frames required by using a higher response modification coefficient.
- Reducing the base shear value in efforts to reduce the tonnage of steel used for the lateral system.
- Reduce cost and construction time by using fewer frames of a more complex system.
- Improve soil conditions by redesigning the foundation system with Geopiers.

SMF (RBS) Design Codes

The codes used to design the SMF system are listed as follows:

- American Institute of Steel Construction, Seismic Design Manual
- American Institute of Steel Construction, Steel Construction Manual 13th Edition
- American Institute of Steel Construction, Specification for Structural Steel Buildings (AISC 360-05)
- American Institute of Steel Construction, Seismic Provisions for Structural Steel Buildings (AISC 341-05)
- American Institute of Steel Construction, Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (AISC 358-05)
- Federal Emergency Management Agency, Recommended Seismic Design Criteria for New Steel Moment Frame Buildings (FEMA-350)

SMF (RBS) Design Limitations

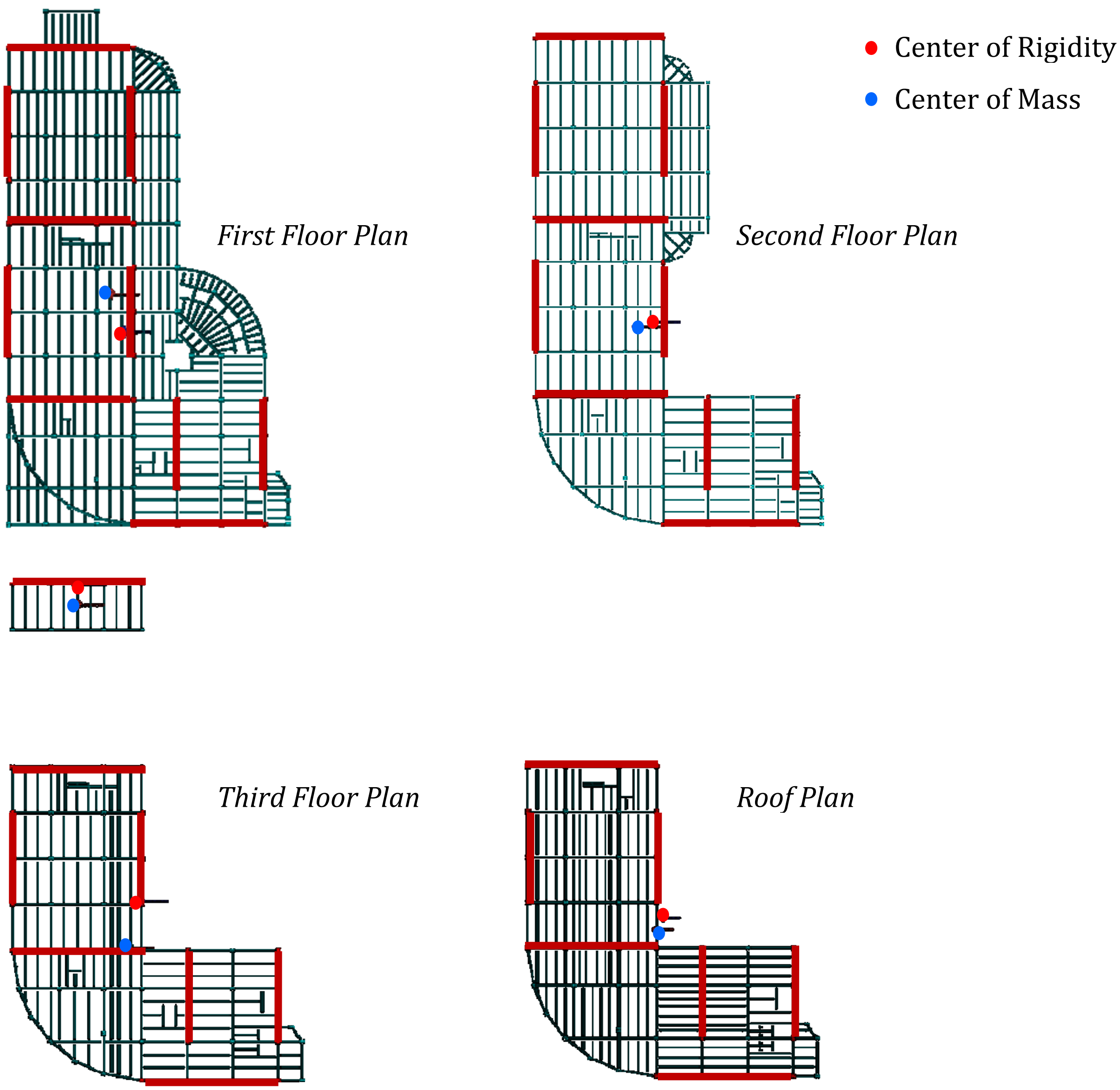
- Use only standard wide-flange sections
- Use in only flexurally controlled beam spans (L/d ratio > 5)
- Do not reduce the flange by more than 50%
- Ensure that the left and right sides of the beam and the top and bottom flanges are symmetrically reduced

SMF Design Considerations

The following considerations were taken into account during the preliminary layout of the SMF system.

- Minimize the number of SMF's placed on the interior of the building to ultimately reduce interior member sizes.
- Keep the layout of the SMFs as symmetrical as possible to reduce torsion effects from lateral forces.
- Orient the SMFs in such a way to keep the center of rigidity and center of mass as close as possible to reduce torsion within the system.

After taking these ideas into account, the following SMF layout was designed:



SMF Design Process

Seismic forces in the X and Y direction were recalculated using the response modification coefficient corresponding to special steel moment frames. A summary of seismic values are provided in the table below.

Occupancy Category	IV
Importance Factor (I)	1.5
Seismic Design Category	C
Mapped Spectral Response Accelerations	S_s=0.170 S_1=0.056
Site Class	E
Site Class Factors	F_a=2.5 F_v=3.5
$S_{MS}=F_a(S_a)$	0.425
$S_{M1}=F_v(S_1)$	0.196
$S_{DS}=2/3(S_{MS})$	0.283
$S_{D1}=2/3(S_{M1})$	0.131
Response Modification Factor (R)	8 (special steel moment frames)
Deflection Amplification Factor (C_d)	5.5
Over Strength Factor (Ω_0)	3
Building Height Limitation	Not Limited
Diaphragm Type	Concrete filled metal deck
Diaphragm Flexibility	Rigid
C_t	0.028
C_s	0.034

After calculating all of these seismic coefficients, the story forces were then calculated based on the weight of each floor. Once this was done, the base shear and overturning moment were determined as seen in Figure 22 below.

Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$h_x^kW_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.5	1093	97320	0.174	118	118	6790
3	43	2917	188250	0.337	228	347	14900
2	29	4074	169941	0.304	206	553	16029
1	15	5136	103204	0.185	125	678	10169
Main	0	6593	0	0.000	0	678	0
Total	57.4	19812	558715	1.000	678		47888
Base Shear =	**678**	**k**					

Figure 22: *Base Shear and Overturning Moment Distribution*

RAM Modeling of the New Lateral System

The Heart Pavilion was modeled in RAM Structural System in order to verify lateral forces calculated by hand as well as ensure that the new lateral system was able to withstand lateral forces while applied in 321 combinations. The same modeling assumptions were made for the new lateral system model as were made for the existing system's model. Note that a 5% eccentricity was applied within the model to account for accidental torsion of seismic loading.

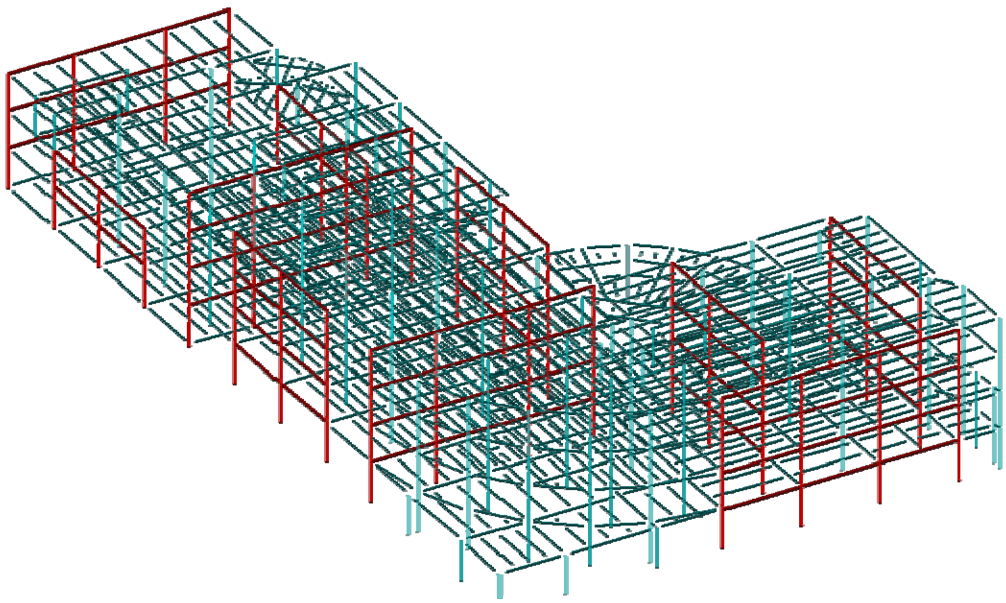


Figure 23: *Structural Model Including Gravity Elements*

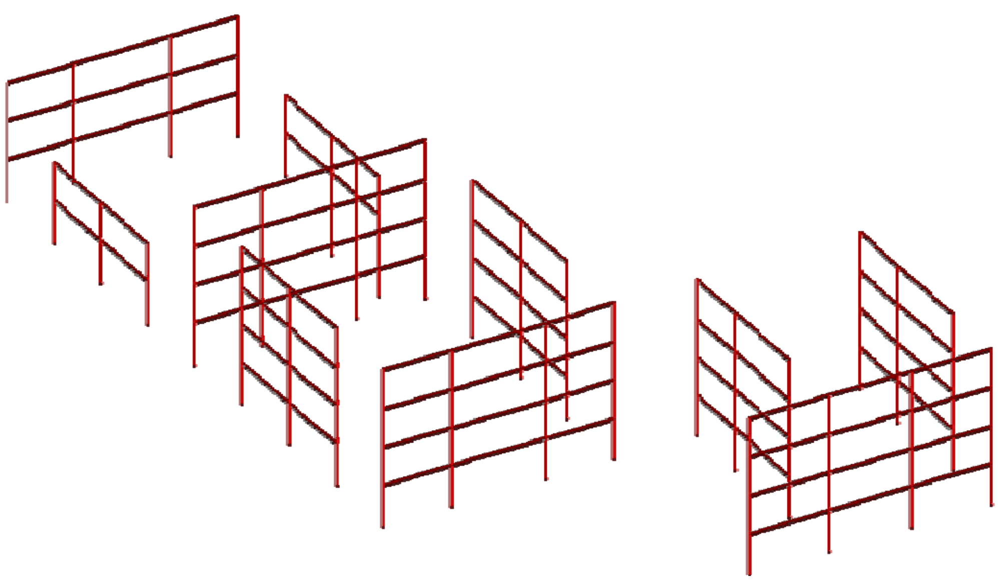


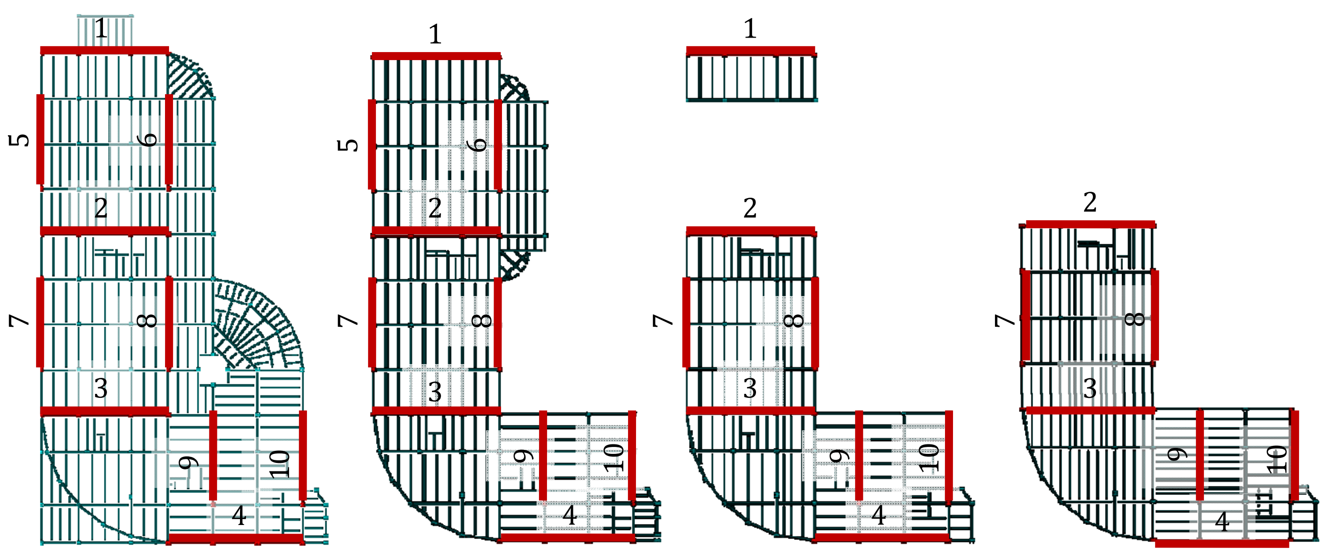
Figure 24: *Structural Model Displaying Only Lateral Elements*

Relative Stiffness & Distribution Factors

Relative stiffness was computed using SAP 2000 for each frame using the concept that stiffness is load divided by deflection. A one kip load was applied, the deflection was measured, and the inverse was taken, thus producing the relative stiffness of that frame. This procedure was carried out for each moment frame within the building and Figure 25 below was provided based upon this data.

	Frame	Stiffness (k/in)				Distribution Factors			
		Roof	3rd	2nd	1st	Roof	3rd	2nd	1st
X-Direction	1	-	25.6	25.6	25.6	-	0.224	0.224	0.224
	2	26.9	26.9	26.9	26.9	0.303	0.235	0.235	0.235
	3	31.2	31.2	31.2	31.2	0.352	0.273	0.273	0.273
	4	30.6	30.6	30.6	30.6	0.345	0.268	0.268	0.268
						1.000	**1.000**	**1.000**	**1.000**
Y-Direction	5	-	-	29.2	29.2	-	-	0.209	0.209
	6	-	-	30.8	30.8	-	-	0.220	0.220
	7	20.1	20.1	20.1	20.1	0.252	0.252	0.144	0.144
	8	20.1	20.1	20.1	20.1	0.252	0.252	0.144	0.144
	9	20.9	20.9	20.9	20.9	0.262	0.262	0.149	0.149
	10	18.7	18.7	18.7	18.7	0.234	0.234	0.134	0.134
						1.000	**1.000**	**1.000**	**1.000**

Figure 25: *Frame Stiffness and Distribution Factors*



First Floor Plan *Second Floor Plan* *Third Floor Plan* *Roof Plan*

Center of Rigidity & Center of Mass

The center of rigidity was calculated by hand in order to verify the values obtained from RAM Frame. This was done for all floors by multiplying frame stiffness by the distance the frame is from the origin and dividing by the sum of all stiffness times the distance from the origin. Figure 26 was provided based upon this data. The equations used are as follows:

$K_{iy}*d_{iy}/\Sigma K_{iy}$ (for frames 1-4) $K_{ix}*d_{ix}/\Sigma K_{ix}$ (for frames 5-10)

The intersection of column lines A and 1 are taken as x=0.00 and y=0.00, respectively.

	Hand Calculations		**Ram Output**		**% Difference**	
	COR (ft)		COR (ft)		% Diff. $_x$	% Diff. $_y$
Floor	x_R	y_R	x_R	y_R	x_R	y_R
Roof	92.50	92.07	89.65	103.88	3.18	11.37
3	92.50	92.07	84.53	104.76	9.43	12.11
2	71.50	144.24	79.30	132.40	9.84	8.94
1	71.50	144.24	78.19	131.38	8.56	9.79

Figure 26: *Center of Rigidity*

The RAM output for the center of rigidity is very close to what was calculated by hand. The variation is a result of ignoring the entrance canopies, pedestrian bridge, and openings in the floors for the hand calculations. Therefore, it was concluded that the values obtained from RAM Frame were satisfactory to use within this report.

Torsion Effects

A total building torsion analysis was done for lateral forces acting along the two major axes of the building. Torsional moment due to seismic loading is caused by the eccentricity measured from the center of mass to the center of rigidity. This torsional moment was found in accordance with ASCE 7-05 §12.8.4.1. Accidental torsional moment was also accounted for within the RAM model. Figures 27 & 28 provide a summary of the torsional moment acting on each story for the North-South and East-West direction, respectively.

Story	**Torsional Moment Due to Seismic Loading**				
	North-South Torsional Moment				
	COM (ft)	COR (ft)	e_x (ft)	Story Force (k)	Torsional Moment (ft-k)
Roof	84.05	89.65	5.60	114	638
3	74.79	84.53	9.74	271	2640
2	67.77	79.30	11.53	208	2398
1	69.20	78.19	8.99	130	1169

Figure 27: *North- South Torsional Moment*

Story	Torsional Moment Due to Seismic Loading				
	East-West Torsional Moment				
	COM (ft)	COR (ft)	e_y (ft)	Story Force (k)	Torsional Moment (ft-k)
Roof	97.00	103.88	6.88	114	784
3	102.42	104.76	2.34	271	634
2	132.04	132.40	0.36	208	75
1	157.03	131.38	25.65	130	3335

Figure 28: *East-West Torsional Moment*

Since the Heart Pavilion is categorized within SDC C, consideration of the torsional amplification factor was required per ASCE 7-05 §12.8.4.2. These amplification factors were found based on the total displacement of each story in the x, y, and z-directions. For detailed calculations regarding the amplification factors for each story, please reference Appendix C. Figures 29 & 30 show a summary of the amplification factors for each story in the North-South and East-West direction. Since all of these values were less than 1.0, the amplification factor was taken as 1.0.

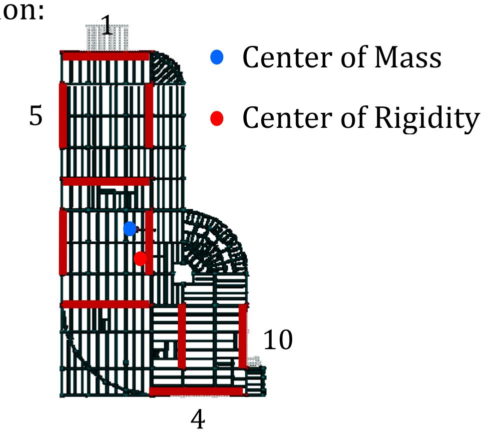
Story	Amplification Factor, A_o				
	North-South Direction				
	δ_X (in)	δ_Z (in)	δ_{AVG} (in)	δ_{MAX} (in)	A_X
Roof	5.08	0.363	5.08	5.44	0.796
3	4.26	0.362	4.26	4.62	0.817
2	3.15	0.302	3.15	3.45	0.833
1	1.56	0.110	1.56	1.67	0.796

Figure 29: *North-South Amplification Factor*

Story	Amplification Factor, A_o				
	East-West Direction				
	δ_X (in)	δ_Z (in)	δ_{AVG} (in)	δ_{MAX} (in)	A_X
Roof	4.66	0.030	4.66	4.69	0.703
3	3.90	0.114	3.90	4.01	0.734
2	2.86	0.120	2.86	2.98	0.754
1	1.42	0.122	1.42	1.54	0.817

Figure 30: *East-West Amplification Factor*

In order to spot check frame story force values obtained from RAM Frame, story forces for all ten SMF's were calculated by hand. To obtain the direct force on each story, the distribution factor of the frame was multiplied by the total story force. Next, the torsional force on each frame was calculated using the following equation:



torsional force = $F_i = M_y(k_i x_i)/I_p + M_x(k_i y_i)/I_p$

where M_y = torsional moment in the y-direction
M_x = torsional moment in the x-direction
k_i = frame stiffness
x_i = distance of frame from x-axis
y_i = distance of frame from y-axis
$I_p = I_x + I_y$

The total forces for these frames are calculated by adding the direct force and the torsional force. These forces were then multiplied by a factor of 1.0 because this is the LRFD load factor for seismic loading.

By looking at the orientation of the SMF's with respect to the center of rigidity and center of mass, it would be expected that SMF 1, 4, 5, and 10 would take the most torsional force. This conclusion is confirmed by the figures provided below.

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
	--	114	--	--	--
Frame 1	3	271	60.7	**3.079**	63.8
	2	208	46.6	**2.254**	48.8
	1	130	29.1	**1.675**	30.8

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
	Roof	114	34.6	1.162	35.7
Frame 2	3	271	63.8	1.550	65.3
	2	208	49.0	0.902	49.9
	1	130	30.6	1.045	31.6

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 3	Roof	114	40.1	0.650	40.7
	3	271	73.9	0.486	74.4
	2	208	56.7	0.688	57.4
	1	130	35.5	1.023	36.5

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 4	Roof	114	39.3	**1.276**	40.6
	3	271	72.6	**1.849**	74.4
	2	208	55.7	**1.864**	57.6
	1	130	34.8	**1.893**	36.7

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 5	Roof	114	--	--	--
	3	271	--	--	--
	2	208	43.5	**1.797**	45.3
	1	130	27.2	**2.312**	29.5

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 6	Roof	114	--	--	--
	3	271	--	--	--
	2	208	45.8	1.858	47.6
	1	130	28.6	1.044	29.7

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 7	Roof	114	28.7	0.827	29.6
	3	271	68.3	0.637	69.0
	2	208	29.9	0.138	30.1
	1	130	18.7	1.056	19.8

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 8	Roof	114	28.7	0.256	29.0
	3	271	68.3	0.424	68.7
	2	208	29.9	0.117	30.0
	1	130	18.7	0.147	18.9

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 9	Roof	114	29.8	0.412	30.2
	3	271	70.9	0.541	71.4
	2	208	31.1	0.674	31.7
	1	130	19.4	0.805	20.2

	Story	Force (k)	Direct Force (k)	Torsional Force (k)	Total Factored Force on Each Story (k)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 10	Roof	114	26.7	**0.743**	27.4
	3	271	63.5	**0.698**	64.2
	2	208	27.8	**0.692**	28.5
	1	130	17.4	**1.597**	19.0

Member Contributions

In order to optimize the member sizes chosen for the special steel moment frame system, the method of virtual work was used to calculate the contribution of each member within the frame.

The following list represents a summary in the steps used in the virtual work method, as presented in Structural Analysis using Virtual Work by F. Thompson and G. G. Haywood:

- Forces applied at nodes, or at the ends of members, are considered to contribute to external virtual work
- Forces acting in members themselves are considered to contribute to internal virtual work
- Σ external virtual work = Σ internal virtual work (where only applied forces are considered)
- A complete system of forces is represented by a capital letter; the actual system is represented by a letter only; the virtual system is represented by a letter with the subscript "i"
- A complete pattern of displacements is represented by a lowercase letter, where actual displacements and virtual displacements are distinguished as above
- A virtual work operation is defined as the product: (P_i*p) or (m_i*M)

The following procedure was followed to analyze the real work and virtual work done by each moment frame to determine each member's contribution within that frame:

$$\sum 1^k(\Delta_i) = \underbrace{\sum \int (M_i m_i)/(EI_i)dx}_{\text{Beam Contribution}} + \underbrace{\sum (F_i f_i L_i)/(AE)}_{\text{Column Contribution}}$$

where $M = M_i - (2M_i x)/L_i$ *and* $m = m_i - (2m_i x)/L_i$

$$1^k(\Delta_i) = 1/(EI_iL_i^2)\int_0^{L_i}[(M_iL_i - 2M_ix)(m_iL_i - 2m_ix)dx] + (F_if_iL_i)/(AE)$$

$$= 1/(EI_iL_i^2)\int_0^{L_i}[(M_im_iL_i^2 - 4M_im_iL_ix + 4M_im_ix^2)dx] + (F_if_iL_i)/(AE)$$

$$= 1/(EI_iL_i^2)\,[(M_im_iL_i^3) - (4M_im_iL_i(L_i^2))/2 + (4M_im_iL_i^3)/3] + (F_if_iL_i)/(AE)$$

$$= (M_im_i)/(EI_iL_i^2)\,[(L_i^3) - (4L_i^3)/2 + (4L_i^3)/3] + (F_if_iL_i)/(AE)$$

$$= (M_im_iL_i^3)/(EI_iL_i^2)\,[1 - 4/2 + 4/3] + (F_if_iL_i)/(AE)$$

$$= (M_im_iL_i^3)/(EI_iL_i^2)\,[1 - 4/2 + 4/3] + (F_if_iL_i)/(AE)$$

$$= 1/3(M_im_iL_i^3)/(EI_iL_i^2) + (F_if_iL_i)/(AE)$$

Frame Modeling in SAP 2000

To determine the force in each column and the moment in each beam within a particular SMF, each frame was modeled using SAP 2000. The frame story shears calculated by hand were applied to the corresponding stories as seen in Figure 31 with results for frame 3 shown in Figures 32 & 33. A one kip load case was also created to provide the frame contributions for the virtual case as seen in Figures 34, 35, and 36 on the following page.

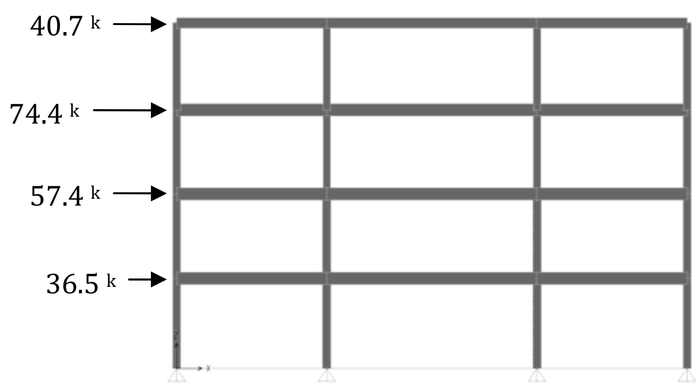


Figure 31: *Frame 3 Story Shears*

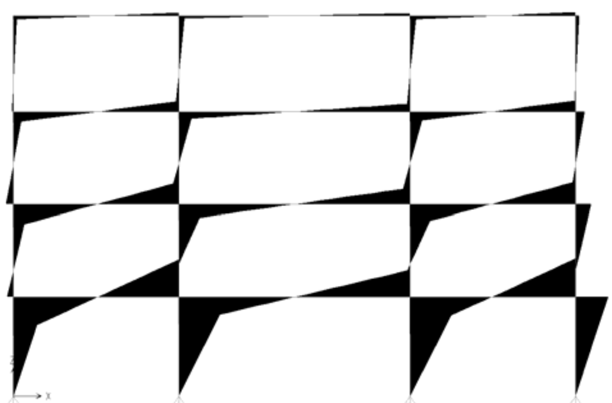


Figure 32: *Moment Diagram (k-ft)*
(Scaled at 1/125)

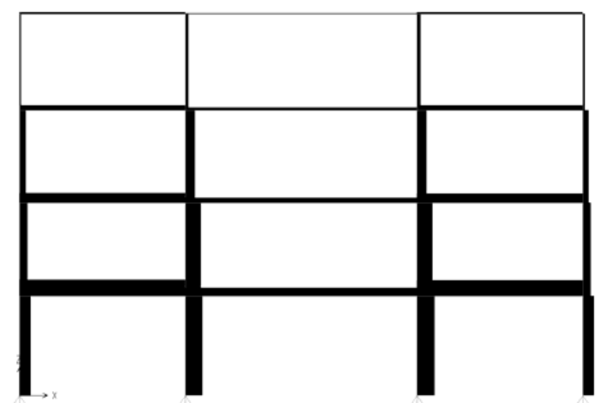


Figure 33: *Shear Diagram (k)*
(Scaled at ½)

Figure 34: *1^k Load Case*

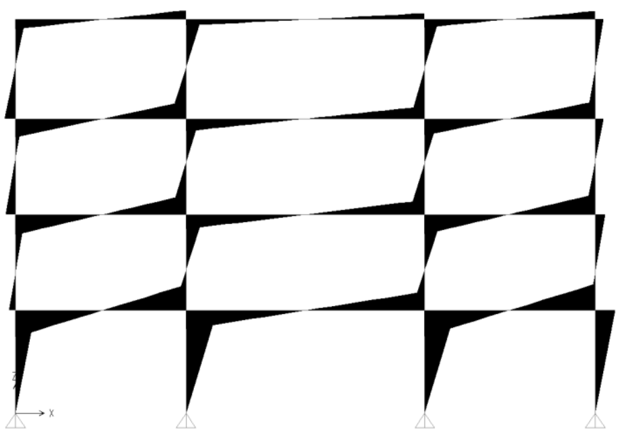


Figure 35: *Moment Diagram (k-ft)*
(Scaled at 1)

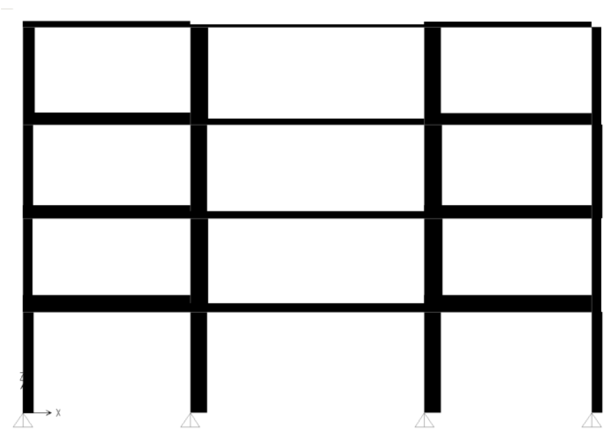


Figure 36: *Shear Diagram (k)*
(Scaled at 100)

After obtaining the force in each column and moment in each beam within an SMF from the SAP 2000 models, each member's contribution was able to be determined based on the following equation, as previously computed:

$$1^k(\Delta_i) = 1/3(M_i m_i L_i^3)/(EI_i L_i^2) + (F_i f_i L_i)/(AE)$$

Figure 37 on the following page was provided to show the contribution of each member within SMF 3. It was determined that the beams within the SMF's contribute approximately 40% while the columns contribute approximately 60%.

Frame 3

	Member	E (ksi)	I_x (in^4)	M_i (ft-k)	m_i (ft-k)	L_i (ft)	Δ_i=1/3 $(M_i m_i L_i^3)/(EI_i L_i^2)$ + $(F_i f_i L_i)/(AE)$	Member Contribution
L. Beams	1	29000	2700	741	3.88	25	0.00031	8.66%
	2	29000	2700	435	2.96	25	0.00014	3.88%
	3	29000	1830	213	2.69	25	0.00009	2.55%
	Roof	29000	843	68	1.42	25	0.00003	0.93%
Middle Beams	1	29000	2700	518	2.75	35	0.00021	6.01%
	2	29000	2700	309	2.13	35	0.00010	2.78%
	3	29000	1830	151	1.82	35	0.00006	1.71%
	Roof	29000	843	51	0.92	35	0.00002	0.63%
Rt. Beams	1	29000	2700	734	3.89	25	0.00030	8.60%
	2	29000	2700	426	2.98	25	0.00014	3.83%
	3	29000	1830	210	2.51	25	0.00008	2.34%
	Roof	29000	843	70	1.23	25	0.00003	0.83%

	Member	E (ksi)	A (in^2)	F_i (k)	f_i (k)	L_i (ft)	Δ_i=1/3 $(M_i m_i L_i^3)/(EI_i L_i^2)$ + $(F_i f_i L_i)/(AE)$	Member Contribution
Col D-1	1	29000	56.8	12	0.03	15	0.00031	8.75%
	2	29000	56.8	7	-0.01	14	0.00014	3.86%
	3	29000	42.7	17	-0.03	14	0.00008	2.39%
	Roof	29000	42.7	7	0.21	14.5	0.00005	1.42%
Col D-2	1	29000	75.6	7	-0.03	15	0.00021	5.97%
	2	29000	75.6	21	0.02	14	0.00010	2.85%
	3	29000	51.8	22	-0.01	14	0.00006	1.65%
	Roof	29000	51.8	13	0.32	14.5	0.00006	1.80%
Col D-3	1	29000	75.6	7	-0.03	15	0.00021	5.97%
	2	29000	75.6	22	0.02	14	0.00010	2.86%
	3	29000	51.8	21	0.02	14	0.00006	1.82%
	Roof	29000	51.8	14	0.29	14.5	0.00006	1.72%
Col D-4	1	29000	56.8	11	0.03	15	0.00031	8.69%
	2	29000	56.8	8	-0.03	14	0.00013	3.77%
	3	29000	42.7	15	0.03	14	0.00009	2.48%
	Roof	29000	42.7	7	0.17	14.5	0.00004	1.23%
				∑=	1.00		∑ Δ_i= 0.00538	100%

Beams	42.8%
Col D-1	16.4%
Col D-2	12.3%
Col D-3	12.4%
Col D-4	16.2%

Figure 37: *Member Contribution of SMF 3*

Seismic Design Forces

Once the member sizes were finalized based on member contribution analysis, hand calculated story forces and shears were compared to the values given by the RAM model. Please reference Figure 38 below for a summary of the percent difference in hand calculations and the computer model.

	Seismic Design					
	Story Loads (k)			Story Shears (k)		
Story	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	118	114.74	2.92	118	114.74	2.92
3	228	271.76	15.95	347	386.50	10.35
2	206	207.98	0.85	553	594.48	7.03
1	125	130.33	3.91	678	724.81	6.47
Total Base Shear (k)	**678**	**724.81**	**6.47**			
Overturning Moment (ft-k)	**47,888**	**51,329**	**6.70**			

Figure 38: *Story Forces for Seismic Design*

Once the final design of the lateral system was complete, RAM Frame was used to ensure the interaction equation for each member was less than 1.00. The interaction equation for a member is calculated by taking the ratio of load to capacity for the controlling load case. The contributions of both axial load and moment are taken into account in the interaction equation. Figure 39 below shows a color coding of each lateral member's interaction equation.

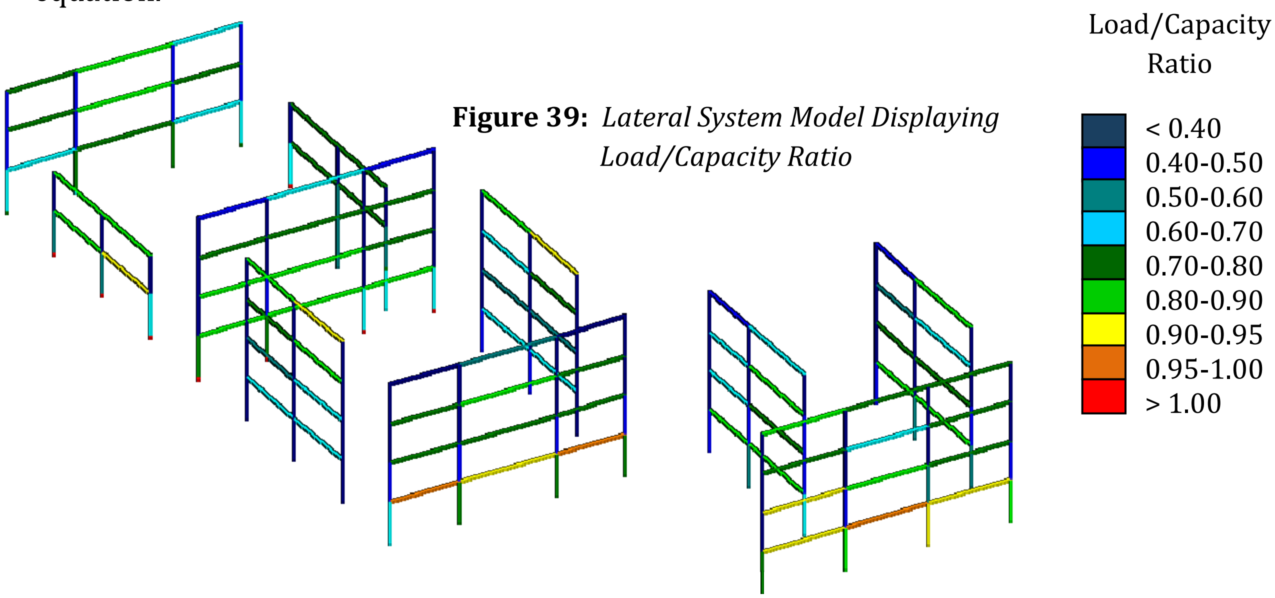


Figure 39: *Lateral System Model Displaying Load/Capacity Ratio*

Redundancy

Figure 40 represents the contribution of each frame's resistance to seismic forces for both orthogonal axes of the building.

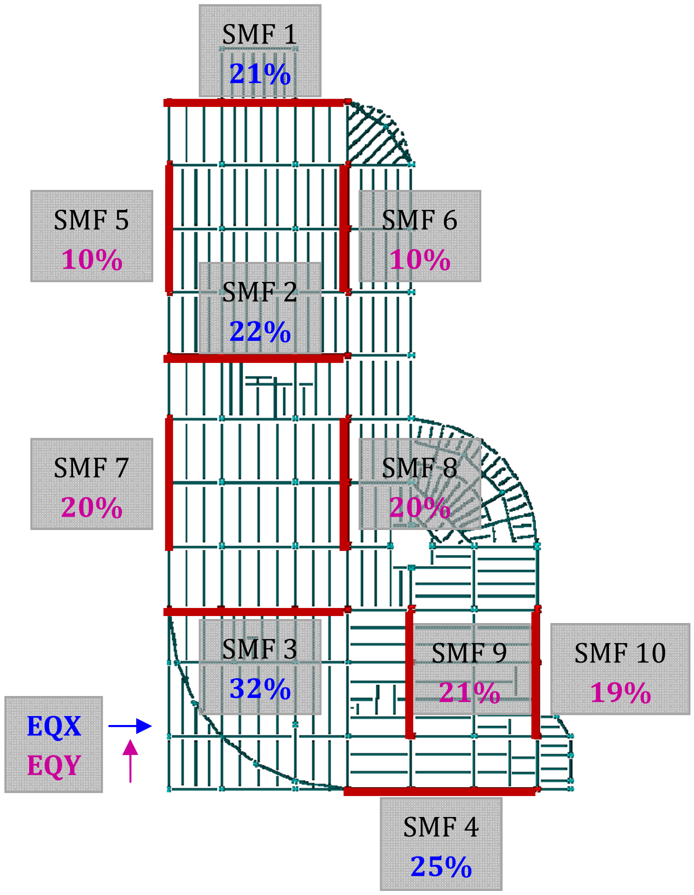


Figure 40: *Distribution of Lateral Forces among SMF's*

Even though this structure is categorized within SDC C, checks were made in accordance with ASCE 7-05 §12.3.4.2 to ensure that a redundancy factor of 1.0 is acceptable for design. Since none of the SMF's are resisting more than 33% of the base shear, the redundancy factor is not required to be taken as 1.3. Therefore, the use of 1.0 for the redundancy factor is justified.

Seismic Drift

Drift is an important serviceability requirement that can cause several problems within a building if the limitations are not met. Seismic drift is addressed in ASCE 7-05 and is limited based on the occupancy category of the building. St. Vincent Mercy Medical Center is classified as occupancy category IV, therefore the allowable story drift is limited to $0.015h_{sx}$ since the Heart Pavilion is only four stories.

As seen in Figure 41, story drift values were obtained from RAM Frame and compared to the allowable values obtained from the following equations.

$$\delta_x = C_d\delta_{xe}/I \qquad (12.8\text{-}15)$$

$$\Delta_a = 0.015h_{sx} \qquad (\text{Table } 12.12\text{-}1)$$

Story	Total Drift (in)	Story Drift (in)	Amplified Drift (in)	Reduction $(C_uT_a)/T_x$		Allowable Story Drift (in)	
Roof	5.08	0.82	3.01	1.27	<	2.61	OK
3	4.26	1.12	4.09	1.73	<	2.52	OK
2	3.15	1.59	5.83	2.47	<	2.52	OK
1	1.56	1.56	5.70	2.42	<	2.70	OK

Figure 41: *Seismic Story Drift*

A soft story can also cause serviceability issues and should be addressed during the structural design. A soft story is defined as being 70% as stiff as the floor immediately above it, or less than 80% as stiff as the average stiffness of the three floor above it. Figure 42 below checks the soft story status of the newly designed lateral system for St. Vincent Mercy Medical Center Heart Pavilion.

Story	Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
Roof	0.00471	0.00330	0.00377	--	No
3	0.00664	0.00465	0.00531	--	No
2	0.00946	0.00663	0.00757	--	No
1	0.00864	0.00605	0.00691	0.00694	No

Figure 42: *Soft Story Check*

Horizontal & Vertical Irregularities

After the design of the SMF system, it was necessary to check if there were any horizontal or vertical irregularities within the newly designed lateral system. The following table summarizes the irregularity status per ASCE 7-05 §12.3.2.1.

	Irregularity Type	Comment	Status
Horizontal Irregularities	Torsional	Upon completion of the RAM Model, irregularity does not exist. Please reference Appendix C for detailed calculations	OK
	Re-entrant Corner	This irregularity does not apply to SDC C	OK
	Diaphragm Discontinuity	By looking at the floor plans, irregularity does not exist	OK
	Out-of-Plane Offsets	By looking at the floor plans, irregularity does not exist	OK
	Non Parallel System	All lateral force resisting frames are parallel to the orthogonal grid	OK
Vertical Irregularities	Stiffness-Soft Story	Upon completion of story drift check, irregularity does not exist (Reference Seismic Drift Section)	OK
	Weight Mass	Roof Wt./Adjacent Story Wt.= = 44psf/108 psf < 150% Reference Appendix A for story weights	OK
	Vertical Geometric	All SMF's are uniform throughout the entire height of the building	OK
	In-Plane Discontinuity of Vertical Lateral Force Resisting Element	By looking at the floor plans, irregularity does not exist	OK
	Discontinuity in Lateral Strength	Member sizes are increased going down the building, therefore there is higher strength at the lower floors	OK

RBS Connection Design

The chance of web local buckling and lateral torsional buckling within RBS beams is increased due to reducing the stiffness of the flange. For this reason, the web local buckling criterion has been modified as shown below:

Change to 418

$$h/t_w \leq 520/\sqrt{F_y}[1-(1.54P_u)/(\Phi P_y)]$$

The four basic design concerns for an RBS connection are:

- Determining the moment at the plastic hinge of the beam
- Determining the moment at the column face
- Ensuring the "strong column-weak beam" criterion is met
- Ensuring the panel zone strength of the column is adequate

The "strong column-weak beam" criterion can clearly be met based on member selection. However, if the panel zone strength of the beam to column connection is not adequate, it can produce a much more detailed connection. A panel zone is a flexible component whose deformation can contribute to the overall frame displacement. Please reference Figure 43 to view a diagram of internal forces acting on a panel zone.

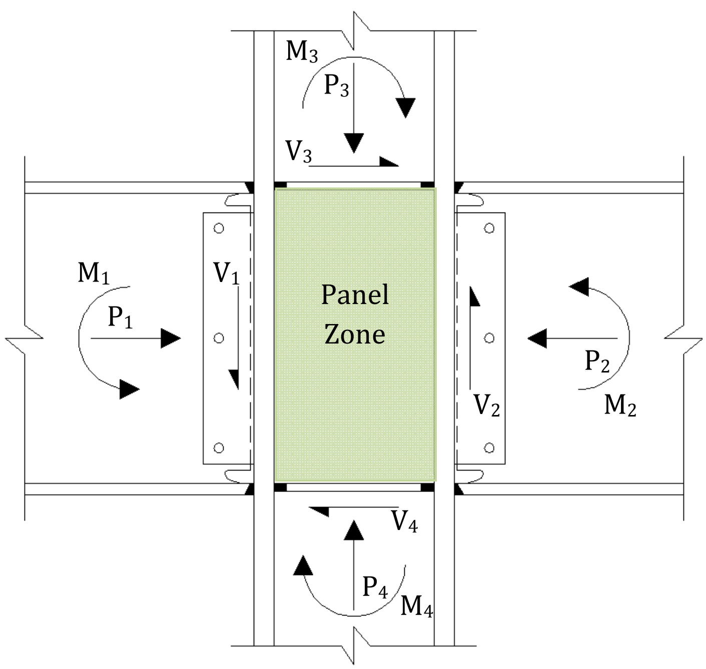


Figure 43: *Internal Forces Acting on a Panel Zone*

The beam moments can be replaced with flange forces, as seen in Figure 44, if the following assumptions are made:

- the flanges resist 100% of the moment
- the distance between the centroid of the flanges equals 95% of the beam depth

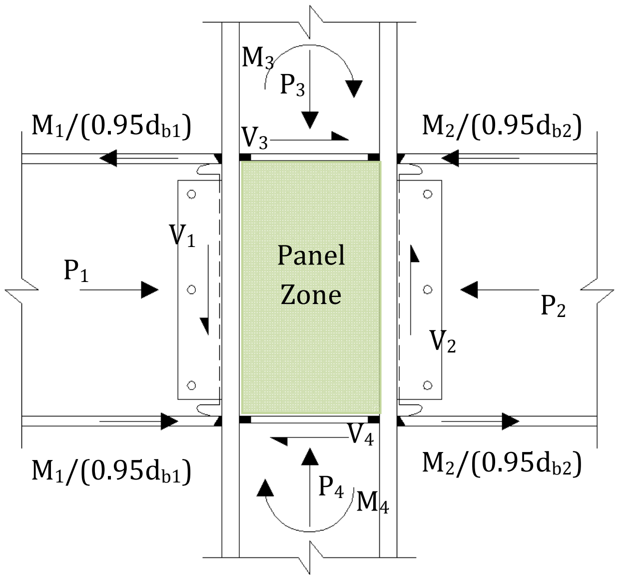


Figure 44: *Internal Forces Acting on a Panel Zone*

The required shear strength of the panel zone is then defined as follows:

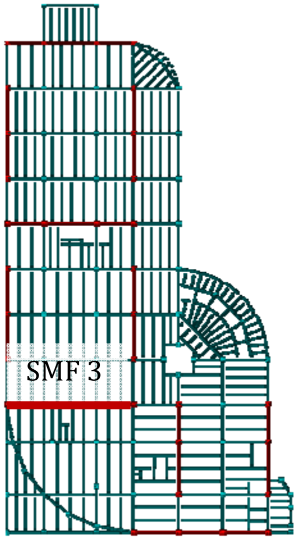
$$V_{pz} = M_1/(0.95d_{b1}) + M_2/(0.95d_{b2}) - V_3$$

Note that since V_3 (shear in the column) reduces the shear within the panel zone and it is very small in magnitude with respect to the first two terms, it is typically ignored.

Columns within a special moment frame are designed to avoid axial yielding, buckling, and flexural yielding. The "strong column-weak beam" criterion ensures that:

- Energy-dissipation capacity is improved
- Plastic hinge is formed within the beam
- Seismic resistance of the frame is increased
- Soft story formation is prevented

Typical Frame for Design



Frame 3 was chosen for the typical RBS connection design because it resists the largest portion of the base shear in the x-direction. Therefore, it was assumed that this frame represents the most critical loading case. Three different alternatives for the interior beam to column connection were designed for comparative purposes. The three alternatives for the interior connection and column splice design are discussed in the following section.

A typical reduced beam section connection was designed and detailed for the conditions shown in Figure 45 per requirements stated in the AISC Seismic Design Manual. Please reference Appendix D for detailed calculations on the design of the RBS connections.

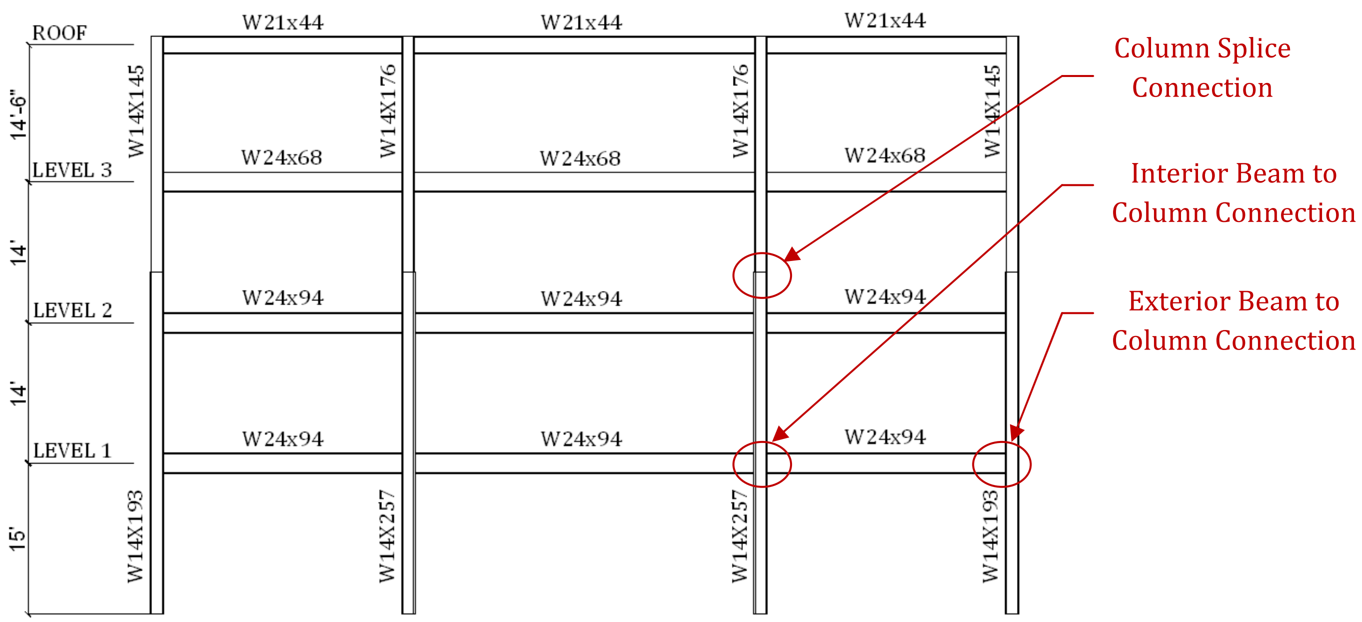


Figure 45: *Elevation of SMF 3*

Interior Beam to Column RBS Connection

Alternative I

Figure 46 represents the first alternative for the SMF beam to column connection. It utilizes a W24x94 beam and W14x211 column with a 1" web doubler plate and a pair of 1"x5" full depth transverse stiffeners. Please reference Appendix D for detailed calculations on the connection design for this configuration.

In order to accommodate a smaller column size, a lot of stiffening is required. Seismic shear buckling requirements of the panel zone within this particular configuration were met. However, a web doubler plate was needed as the panel zone shear was too large to be resisted without reinforcement. In addition, full depth transverse stiffeners were needed to resist panel zone web shear and tensile/compressive flange forces.

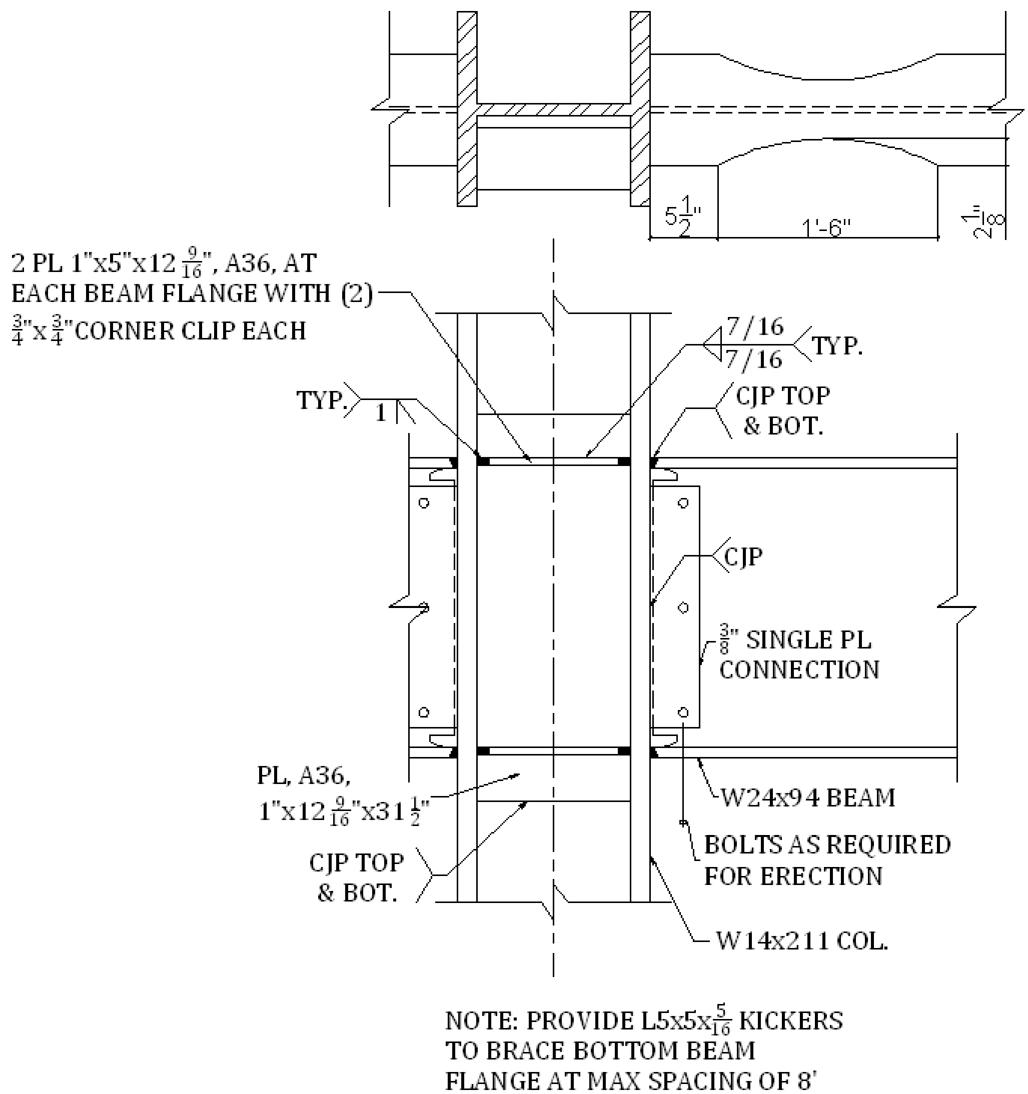


Figure 46: *RBS Connection Design- Alternative I*

Alternative II

The second viable configuration for the SMF beam to column connection is a W24x94 beam and W14x233 column. By utilizing this column size, the panel zone web shear and tensile/compressive forces on the flange were able to be resisted. Therefore, the need for the pair of full depth transverse stiffeners was eliminated. However, the shear strength of the panel zone was still inadequate. Therefore, the 1" web doubler plate is still required for this connection as shown in Figure 47.

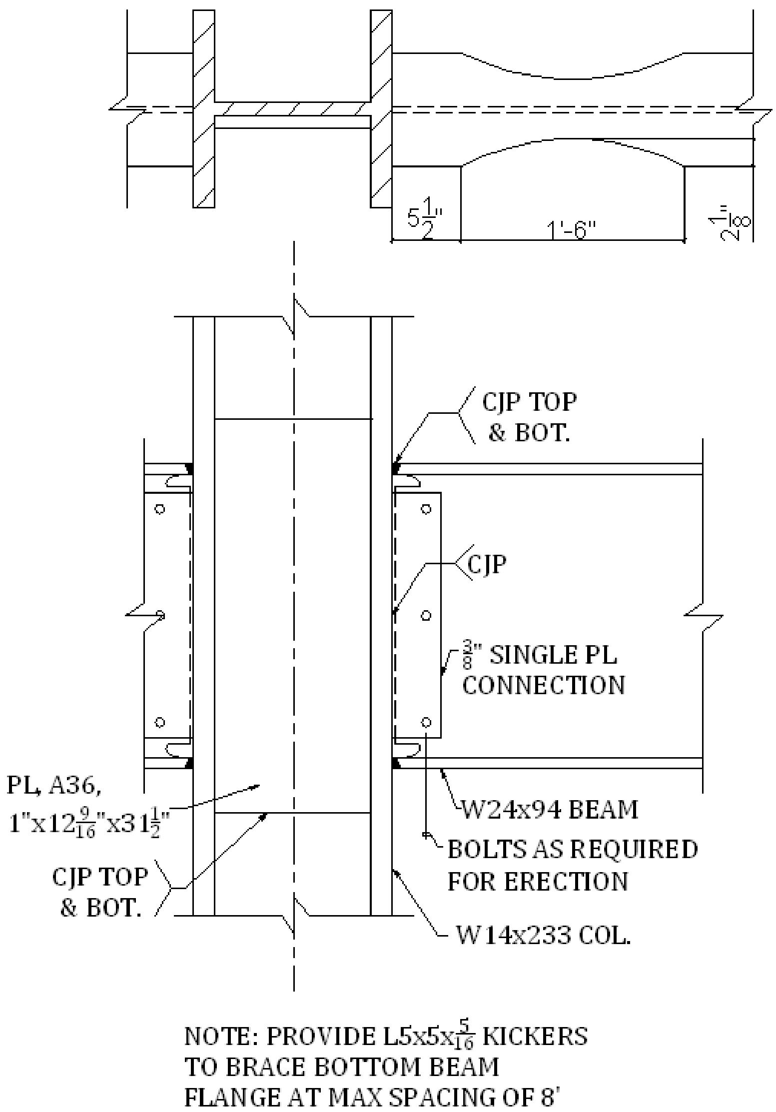


Figure 47: *RBS Connection Design- Alternative II*

Alternative III

In order to obtain a "clean column" configuration, the column size was increased to a W14x257. By utilizing a larger column size, the panel zone is strong enough to resist all forces acting on it. As seen in Figure 48, there is much less detail incorporated into the connection. Please see Appendix D for detailed calculations on the design of this connection.

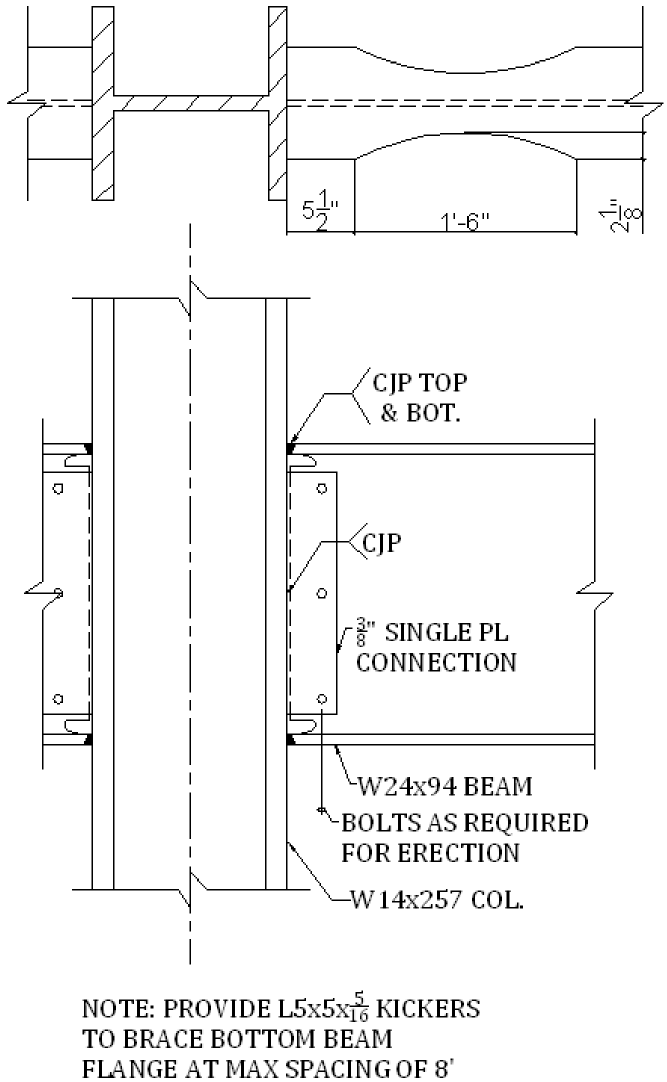


Figure 48: *RBS Connection Design- Alternative III*

RBS Connection Selection Based on Economy

The figures below were prepared to draw conclusions on which connection configuration is the most economical detail for the newly designed lateral system. First, the three configurations were compared with respect to equivalent weight of steel.

Basic Design Rules for Economy

Item	Equivalent wt. of steel (lbs)
1 Pair of Groove Welded Stiffeners	300
1 Groove Welded Doubler Plate	300

Connection Configuration	Stiffening Requirement	Equivalent Wt. of Steel (lbs)	Total Column Wt. (lbs)	Total Column Wt. + Add'l Wt. for Stiffening (lbs)
Alternative I	1 Pair of groove welded stiffeners & web doubler plate	600	6963	7563
Alternative II	1 web doubler plate	300	7689	7989
Alternative III	No stiffening required	-	8481	8481

After speaking with the steel fabricator for the Heart Pavilion, the comparison was taken one step further by comparing the fabrication time and cost of each alternative as shown below.

Connection Configuration	Stiffening Requirement	Beam End Prep (hrs)	Flange Reduction (hrs)	Stiffening Req. + Single PL (hrs)	Total Fabrication (hrs)
Alternative I	1Pair of groove welded stiffeners & web doubler plate	2.4	1.5	3.8	7.7
Alternative II	1 web doubler plate	2.4	1.5	2.3	6.2
Alternative III	No stiffening required	2.4	1.5	0.9	4.8

Connection Configuration	Stiffening Requirement	Total Fabrication (hrs)	Cost ($/Fab. hr.)	Total (per connection)
Alternative I	1Pair of groove welded stiffeners & web doubler plate	7.7	45.00	$347
Alternative II	1 web doubler plate	6.2	45.00	$279
Alternative III	No stiffening required	4.8	45.00	$216

Upon completion of the comparison with respect to fabrication cost, it was concluded that the best connection solution is alternative III. Using the W14x257 column eliminates the need for any stiffeners or doubler plates and requires approximately 60% of the fabrication time of alternative I.

Exterior Beam to Column RBS Connection

The connection for the end bay of the SMF was also designed in accordance with the Seismic Design Manual. A W14x193 column size was used in order to avoid stiffening of the column, based on conclusions drawn with respect to fabrication time. The final design is shown in Figure 49.

The design approach for this configuration was the same approach followed for the design of the interior connections. However, since there is only one beam framing into the column, the amount of moment that the column is required to resist changes. For this reason, detailed calculations are available only upon request. Please refer to Appendix D for detailed calculations of the interior beam to column RBS connection design.

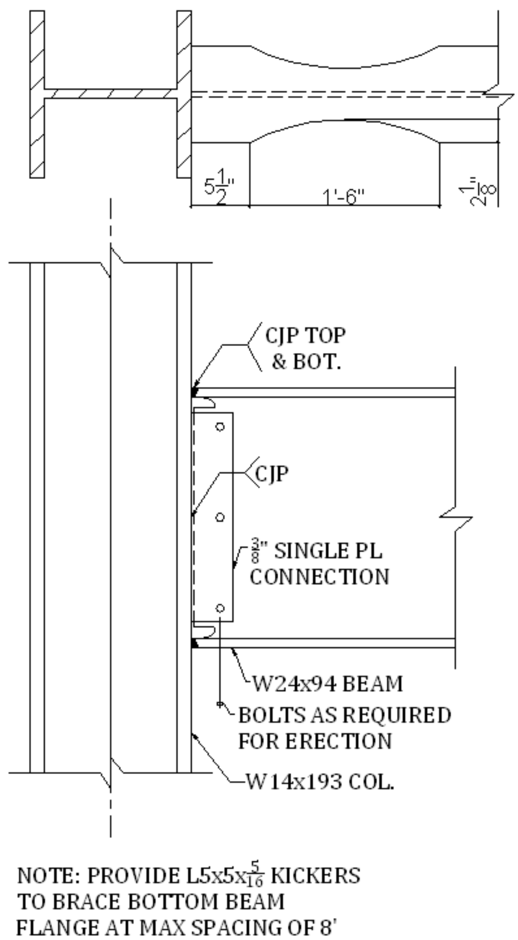


Figure 49: *RBS Connection Design-Exterior Column*

Typical Column Splice

Column splices were designed in accordance with AISC 341-05 §8.4. This connection was designed based on the required axial, flexural, and shear strength of the splice. The maximum length of the weld access holes were determined to ensure that the shear strength of the web splice is developed through shear yielding. Please refer to Figure 50 to view a detail of a typical column splice for the newly designed lateral system.

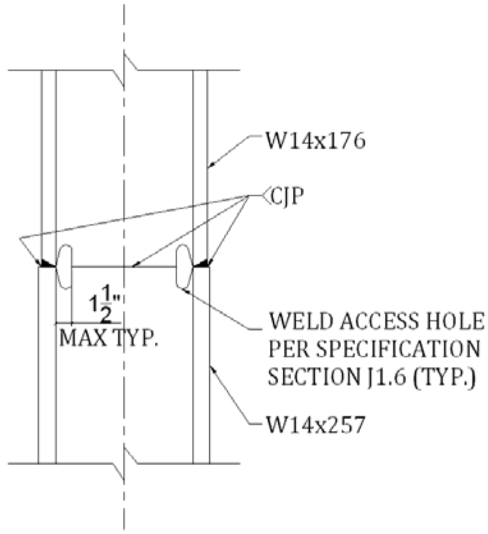


Figure 50: *Typical SMF Column Splice*

Diaphragm & Collector Elements

Diaphragms and collector elements were beyond the scope of this report, but were addressed briefly. Figure 51 shows a plan of the first floor with all SMF's shown. Collector elements are not typically used with a moment frame system as lateral forces will not tend to concentrate into the frames. Collector elements become more critical when utilizing braced frames. In addition, the SMF's for the Heart Pavilion are oriented in such a way that the diaphragm will distribute lateral loads uniformly.

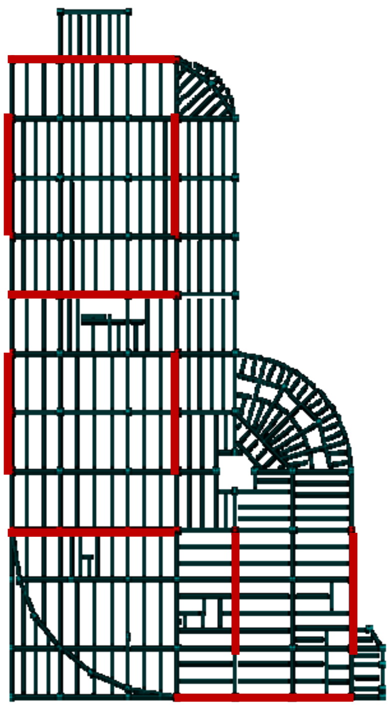


Figure 51: *First Floor Plan Displaying SMF Layout*

SMF Design Conclusion

The number of steel moment frames required to resist lateral forces was significantly reduced by utilizing a seismically detailed system. The following figures provide a comparative summary of the existing and redesigned lateral system with respect to tonnage, and density.

The ratio of the tonnage of steel used is approximately 41% of the existing system. The reduction in base shear using the SMF system is 38%. Therefore, it can be concluded that the reduction in base shear is proportional to the reduction of steel used.

Lateral System	Tonnage of Steel	Density of Steel (psf)
Existing System	610	7.98
Redesigned System	248	3.22

Lateral + Gravity System	Tonnage of Steel	Density of Steel (psf)
Existing System	894	11.69
Redesigned System	678	8.80

After speaking with the steel fabricator for the Heart Pavilion, the typical billing rate per fabrication hour was obtained. As seen in the comparison above, the seismically detailed connections take twice the amount of time to fabricate. However, since there are fewer of them, the overall system is more cost efficient.

Lateral System	# of MF's	# of Connections	Fabrication Time (hrs)	Cost ($/Fab. hr.)	Total
Existing System	19	636	2.4 Ea.	45.00	$68,688
Redesigned System	10	170	4.8 Ea.	45.00	$36,720

Please refer to the Construction Management Breadth Study Section of this report on page 67 for a more detailed comparison of the cost and construction time for these two systems.

Geopier Design

Introduction

Geopiers are an intermediate foundation system that is used in poor soil regions to provide vertical reinforcement to the soil. It was developed by Dr. Nathaniel Fox in 1984. The basic concept is that the poor soil is replaced with stronger and stiffer materials, such as graded aggregate or granular materials.

The properties of Geopiers are gained by the construction process. This process is carried out as follows:

- Removal of a volume of the poor soil by drilling a hole or excavating with a backhoe
- Construction of the bottom bulb by prestraining and prestressing the soil
- Forming the Geopier shaft by compacting thin lifts of well-graded aggregate using a "ramming" action

As the beveled tamper rams the aggregate into the cavity to form the Geopier shaft, lateral stress is built up within the surrounding soil. This buildup over-consolidates the soil, resulting in a stiffened Geopier element and soil mass. Please reference Figures 52 & 53 to view a diagram of the construction process.

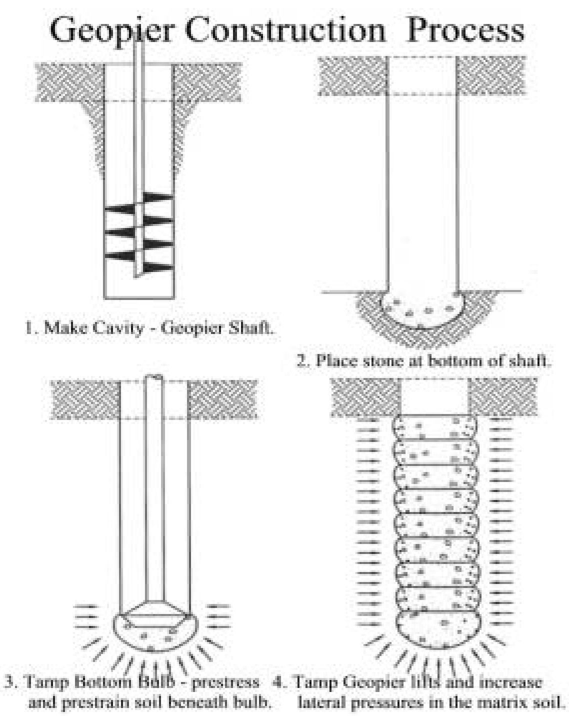


Figure 52: *Detail of Ramming Process*
Photo courtesy of www.farrellinc.com

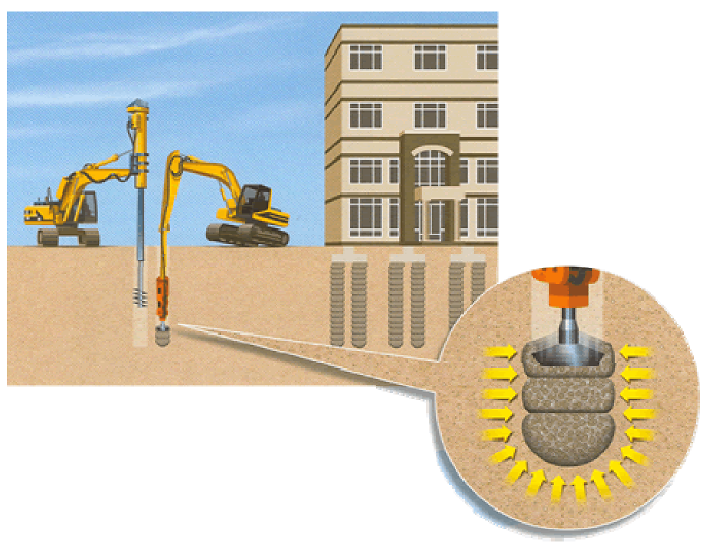


Figure 53: *Geopier Installation*
Photo courtesy of www.farrellinc.com

For a long time, shallow foundations or deep foundations were the only popular choices widely used in building projects. Now, it is no longer a choice of an inch or a mile, there is an intermediate choice. Geopier Intermediate Foundations can be used in poor soil regions to improve the soil or in high-quality soil regions to accommodate unusually high compressive loads. This system has three great advantages over deep foundations:

- More cost efficient
- Quicker installation
- Can be installed in poor weather conditions to facilitate schedule

The more that soil is confined laterally, the stronger it is and the less compressible it is. For this reason, settlement is minimized by using a Geopier system. In addition to this benefit, bearing capacity of the soil is actually increased. Bearing capacity does not come from the soil directly underneath the foundation; rather, it comes from the soil alongside the foundation that keeps the soil underneath from squeezing out. Therefore, if the soil alongside the foundation is made stronger, bearing capacity is ultimately increased. As the Geopier elements settle, the aggregate within the shaft begins to bulge out creating even more lateral pressure within the surrounding soil.

Geopier Design Steps

The main design steps for a Geopier Intermediate Foundation System are as follows:

Geopier Footing Capacity

- Establish allowable bearing capacity of the soil from the geotechnical report
- Determine soil properties to a depth five times deeper than the spread footing width
- Determine soil consistency to a depth five feet deeper than the cavity depth to find the stiffness modulus of the Geopier element and the allowable footing bearing pressure
- Estimate soil capacity, allowable footing bearing pressure, and Geopier stiffness modulus using the Standard Penetration Resistance (N-values)

Settlement Analysis

- Determine upper and lower zone of the Geopier element (reference Figure 54)
- Estimate the elastic modulus of the soil by the N-values and the cone penetration results
- Measure compressibility characteristics of cohesive soils from consolidation testing

The upper zone is defined as the depth of the Geopier element underneath the footing. The settlement contribution of this zone is primarily a function of the stiffness modulus of the Geopier and the concentrated stress on the Geopier. The lower zone is defined as the depth of soil below the Geopier that is influenced by the loaded area of the footing.

As seen in Figure 55, the Geopier element acts as a stress sink. This means that the actual stress on the Geopier element is much greater than that on the soil.

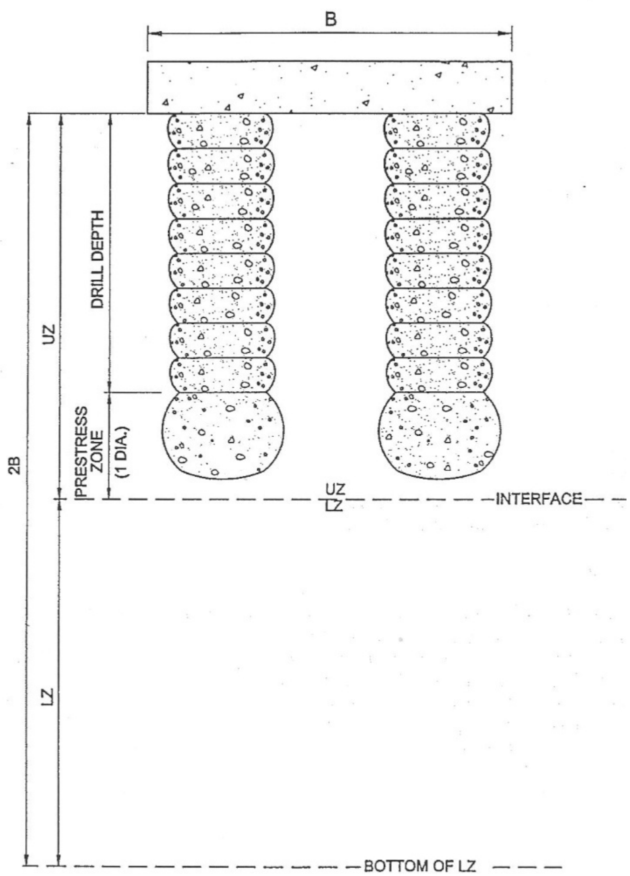


Figure 54: *Upper and Lower Zone of Geopier Element*

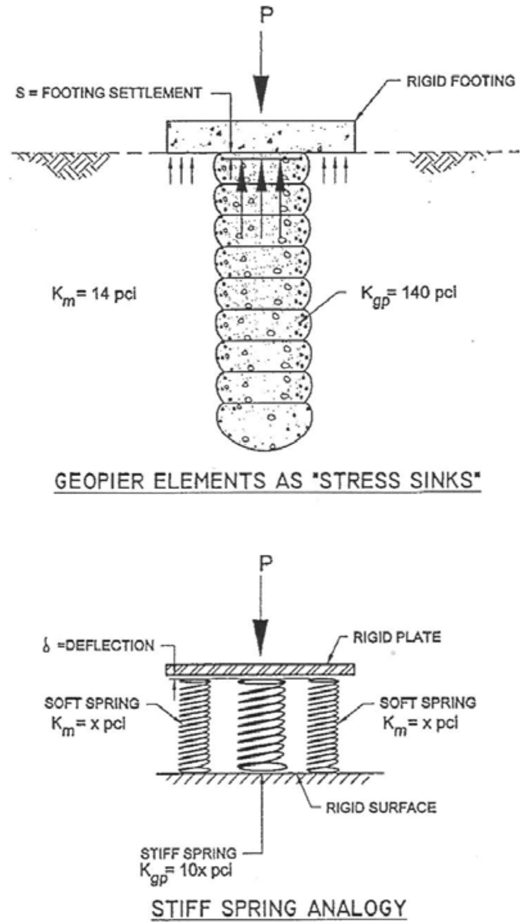


Figure 55: *Stiff Spring Analogy of Geopier Element*

Uplift Capacity

- Establish the effective friction angle of the soil from the geotechnical report
- Determine the coefficient of passive earth pressure, K_p, to estimate the normal stress on the Geopier element
- Uplift capacity is typically taken as the successful uplift load test values divided by a safety factor of 1.5

Lateral Load Resistance

- Find the stiffness ratio of the Geopier element to the soil to calculate stress concentration due to dead load on the Geopiers
- Perform lateral sliding analysis

Slope Stability

- Determine undrained shear strength of the soil from the geotechnical report
- Perform slope stability analysis

Strength History

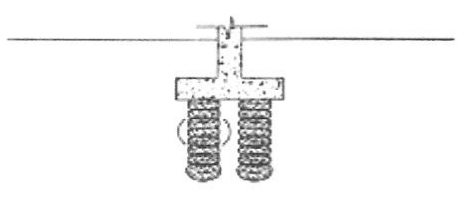
- Clarify if soils were previously exposed to prestressing as this will decrease settlement potential of the Geopier elements

Groundwater

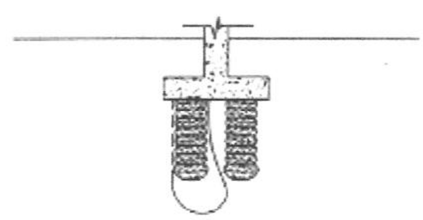
- Locate the water table as this may require the use of special aggregate

Failure Modes

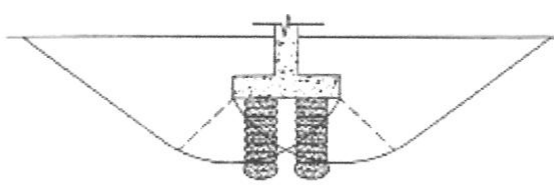
The main failure modes that are considered in the design of a Geopier Intermediate Foundation System are shown in Figure 56 (all images courtesy of GeoStructures, Inc.):



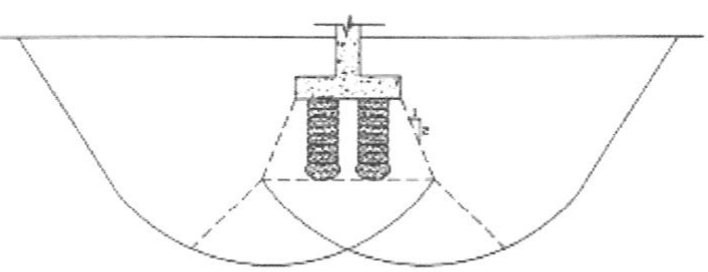
Bulging failure of individual Geopier Elements



Shearing below the tips of Geopier Elements



Shearing within the Geopier-reinforced soil matrix



Shearing below the bottom of the Geopier-reinforced soil matrix

Figure 56: *Failure Modes of a Geopier Element*

Aging of Geopier Element and Surrounding Soil

The capacity and stiffness modulus of a Geopier element is the lowest immediately after it is placed. This is due to pore pressure buildup during the installation process. As this pore pressure dissipates near the perimeter of the Geopier, effective stresses increase. As a result, effective lateral stress is increased over time, creating a stiffer and stronger Geopier element over time. The only anomaly is the rare situation of clay soils that exhibit negative pore water pressure.

Final Geopier Intermediate Foundation System Design

After using the RAM Model to determine the axial force in the columns due to gravity and lateral loads, the following design was provided by Geostructures, Inc.

Footing Width (ft)	Footing Length (ft)	Geopier Diameter (in)	Geopier Length (ft)	No. of Geopiers under Ftg.	Quantity
3.5	3.5	30	8	1	19
4.0	4.0	30	10	1	3
5.0	5.0	30	8	2	5
5.5	5.5	30	10	2	1
6.0	6.0	30	14	2	1
6.5	6.5	30	14	3	9
7.0	7.0	30	14	3	3
7.5	7.5	30	9	4	2
7.5	7.5	30	14	4	4
7.5	7.5	30	16	4	1
8.0	8.0	30	10	4	11
8.0	8.0	30	15	5	5
8.5	8.5	30	10	5	3
8.5	8.5	30	11	5	2
9.0	9.0	30	11	5	2
9.0	9.0	30	11	6	2
9.0	9.0	30	12	5	2
9.0	9.0	30	15	6	2
9.5	9.5	30	12	6	1
10.0	10.0	30	13	7	4

Please refer to Appendix E to view the newly designed foundation plan and the calculations from GeoStructures, Inc. For an enlarged plan and section view of the Geopier elements for SMF 3, please reference Figures 57 & 58 on the following page.

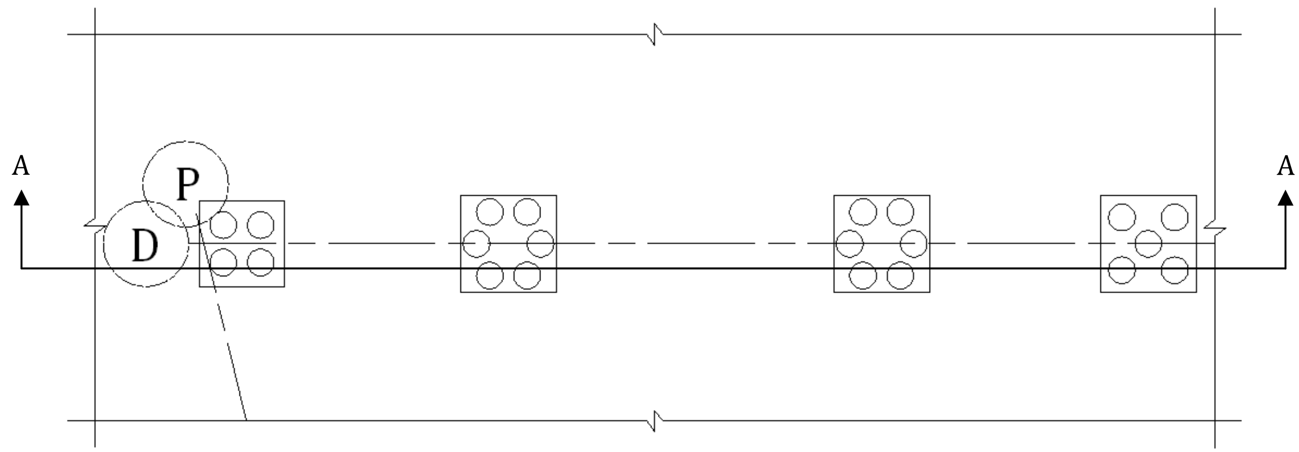


Figure 57: *Enlarged Geopier Foundation Plan for SMF 3*

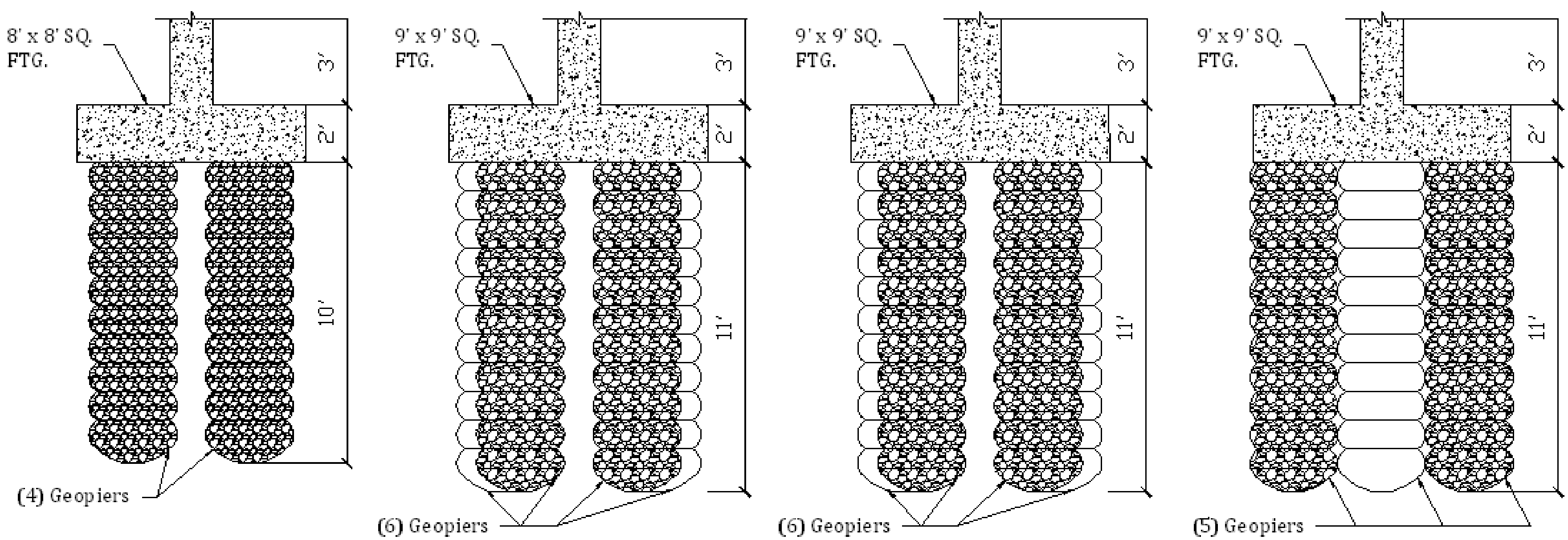


Figure 58: *Enlarged View of Section A-A*

BREADTH STUDY I: FAÇADE STUDY

In the original design concepts, a surgery suit and conference spaces were located on the third floor. To provide natural day lighting to these spaces, a curtain wall was utilized along this entire level as seen in Figures 59 & 61. However, the surgery space was later moved to a lower floor and patient rooms were added in its place. Since the functionality of the third floor became just like the floors below it, this breadth study will focus on changing the curtain wall system to the brick façade used on the lower levels as shown in Figures 60 & 62. These two wall systems will then be compared with respect to the thermal gradient, cost, and construction time.

Figure 59: *Existing Façade-View from Entrance*

Figure 60: *Redesigned Façade-View from Entrance*

Figure 61: *Existing Façade-View from Main Street*

Figure 62: *Redesigned Façade-View from Main Street*

Thermal Gradient Comparison

The thermal gradient for both wall systems was determined by establishing the thermal resistance (R-value) for each material within the wall. The R-values for the brick veneer system were determined in accordance with the 2001 ASHRAE Handbook — Fundamentals. The curtain wall system is a Kawneer 1600 Wall System and the R- value was determined from the product specifications. Once the R values were known, the temperature difference between materials was determined by the following equation:

$$T_x = T_{outdoor} + (T_{indoor} - T_{outdoor})(\sum R_{o\text{-}x} / \sum R_{o\text{-}i})$$

The following assumptions were made for these calculations:

- The outdoor air temperature ($T_{outdoor}$) was taken as 0˚F
- The indoor air temperature (T_{indoor}) was taken as 70˚F
- The relative humidity was taken as 50%

The thermal gradients for the brick veneer and the curtain wall system are shown in Figures 63 & 64. Please refer to Appendix F for detailed calculations showing how these values were determined.

Between Material	$\sum R_{o-x}$ (˚F ft² h/BTU)	Temperature (˚F)
0-1	0.17	0
1-2 *(brick)*	0.28	0.85
2-3 *(air space)*	1.54	4.67
3-4 *(vegetable board)*	2.86	8.67
4-5 *(batting insulation)*	21.86	66.24
5-i	22.54	68.30
	23.10	**70**
U (BTU/˚F ft² h)	**0.0433**	

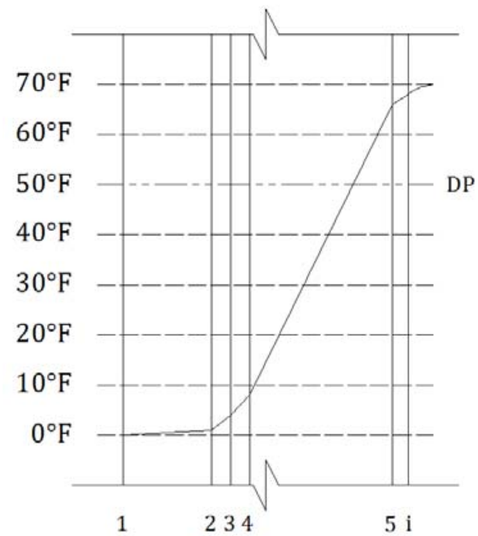


Figure 63: *Thermal Gradient for the Brick Veneer System*

Between Material	$\sum R_{o-x}$ (˚F ft² h/BTU)	Temperature (˚F)
0-1	0.17	0
1-2 *(glass panel)*	2.80	27.64
2-3 *(air space)*	3.95	40.00
3-i *(glass panel)*	6.58	64.96
	7.09	**70**
U (BTU/˚F ft² h)	**0.141**	

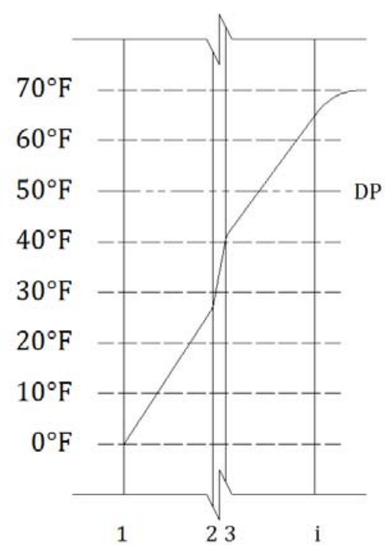


Figure 64: *Thermal Gradient for the Kawneer 1600 Wall System*

Cost & Construction Time Comparison

A rough estimate of cost and construction time of the brick veneer system and the Kawneer 1600 Curtain Wall system were prepared using RS Means. The estimate for the brick veneer system is solely base on square footage and does not include cost for scaffolding, grout, or horizontal reinforcement. For a more in depth estimate, scaffolding would need to be considered as it would take time to set up. However, for the purposes of this breadth topic, a simple square footage analysis is sufficient. This estimate is summarized in the table below.

Wall System	S.F.	Crew Size	Material	Labor	Total	Daily Output	Time of Construction (days)
Brick Veneer	7,178	3 Brick Layers	$470.25 ($/M.S.F.)	$758.25 ($/M.S.F.)	$8,820	1900 S.F.	4
Curtain Wall	7,178	2 Glaziers	30.49 ($/S.F.)	6.94 ($/S.F.)	$268,673	195 S.F.	37

Conclusion

The thermal gradient of the brick veneer wall is very gradual due to the batting insulation used within the system. Upon comparison of the heat transfer values (U-values), the curtain wall system transfers approximately 30.7% more BTU/hr than the brick veneer system. Therefore, it can be concluded that utilizing the brick veneer system on the third floor would minimize heat loss within patient rooms. Ultimately, occupant comfort would be improved by utilizing this system.

In addition, the brick veneer wall is more cost efficient with respect to construction time. As previously stated, this cost estimate was only based on square footage and scaffolding would need to be considered for a more accurate comparison. However, it can still be concluded that the installation of the brick veneer system is more efficient than that of the curtain wall system. It is recommended that the brick veneer wall system be implemented for the façade of the third floor.

BREADTH STUDY II: CONSTRUCTION MANAGEMENT

Utilizing the Geopier System will impact when the first sequence of steel erection can begin. Since this system is intermediate, it does not require as much excavation as a deep foundation system. Ideally, this will save significant time on the construction schedule which will ultimately allow the Heart Pavilion to open sooner.

Implementing seismic detailing of the lateral system will affect the construction schedule of the Heart Pavilion. Welders with a higher level of qualification would be required to install the seismically detailed connections. In addition, highly qualified inspectors must be available for regular visits during construction to ensure that the seismic detailing is being constructed properly.

For these reasons, a cost and schedule analysis was prepared for both the existing foundation and lateral systems, and the redesigned foundation and lateral systems.

Construction Schedule of the Existing Structural System

The existing structure of the Heart Pavilion was scheduled to start on August 1, 2008. However, due extensive labor required to install the deep foundation system, the steel erection was delayed until August 24, 2008.

The steel erection plan is divided up into six sequences for the building. Since this facility is only four stories high, one sequence was completed from the main floor to the roof before moving on to the next sequence. This sequencing plan is shown in Figure 65.

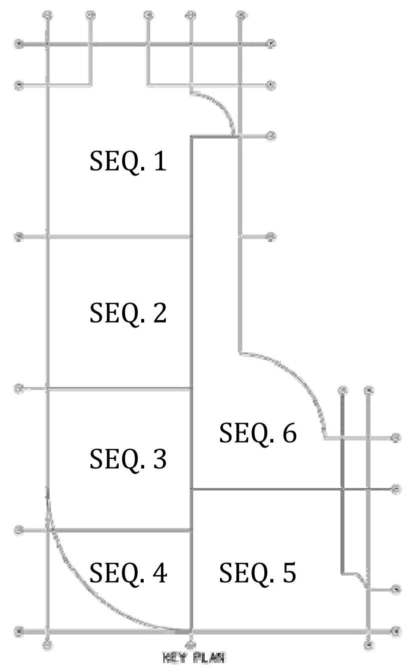


Figure 65: *Sequencing Plan*

After talking with the steel fabricator and erector for the Heart Pavilion, a mock schedule for the foundations and steel structure was created for the existing system. A summary of the construction time is provided in Figure 66 on the following page. Please refer to Appendix G to view a more detailed construction schedule.

Construction Schedule of the Redesigned Structural System

The redesigned structural system follows the same sequencing plan as shown above.

A lot of construction time was saved by utilizing Geopiers and the SMF system. This result makes sense as the Geopier elements can be installed at an approximate rate of 30 per day.

The Geopier elements are considered an intermediate foundation system; therefore, they do not require nearly the amount of excavation of a drilled caisson system.

In addition, a lot of field labor was saved by using a SMF system. This system requires a total of 170 moment connections as opposed to the 636 required by the existing lateral system. As a result, the duration of detailing on site was significantly reduced. Please see Appendix G for a more detailed construction schedule of the redesigned structural system.

Please refer to Figure 66 to view a side by side comparison of the construction time for the existing and redesigned system.

Component	**Existing System (days)**	**Redesigned System (days)**	**Savings (days)**
Foundations	92	44	+ 48
Structural Steel	119	88	+ 31
Connections	56	17	+ 39
Total			**+ 118**

Figure 66: *Construction Time Comparison*

Cost Estimate of the Existing Systems

Foundations					
Foundation Component	Amount	Material Cost	Labor Cost	Equipment Cost	Total
Footings	17 C.Y.	100/C.Y.	12.70/C.Y.	0.41/C.Y.	$1,923
Caissons	1385 C.Y.	100/C.Y.	8.70/C.Y.	3.24/C.Y.	$155,037
Grade Beams	478 C.Y.	106/C.Y.	10.15/C.Y.	0.33/C.Y.	$55,677
					$371,344

Gravity System						
Item	Pounds of Steel	Tonnage of Steel	Material ($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	547,063	274	2,250	375	130	$754,870
Columns	20,042	10.0	2,250	375	130	$27,550
						$782,420

Lateral System						
Item	Pounds of Steel	Tonnage of Steel	Material ($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	690,027	345	2,250	375	130	$950,475
Columns	530,154	265	2,250	375	130	$730,075
						$1,680,550

Connection Fabrication				
# of MF's	# of Connections	Fabrication Time (hrs)	Cost ($/Fab. hr)	Total
19	636	2.4 Ea.	45.00	**$68,688**

Connection Installation			
Installation Time (days)	Installation Time (hrs)	Cost ($/Labor hr)	Total
56	448	$67.20	**$30,106**

Cost Estimate of the Redesigned Systems

Foundations				
Foundation Component	Mobilization ($)	Modulus Test ($)	Material & Installation ($)	Total
Geopiers	25,000	15,000	150,000	**$190,000**

The cost of the Geopier Foundation System was provided by GeoStructures, Inc.

Gravity System						
Item	Pounds of Steel	Tonnage of Steel	Material ($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	763,333	382	2,250	375	130	$1,052,410
Columns	96,508	48.3	2,250	375	130	$133,067
						$1,185,477

Lateral System						
Item	Pounds of Steel	Tonnage of Steel	Material ($/ton)	Labor ($/ton)	Equipment ($/ton)	Total
Beams	179,495	89.7	2,250	375	130	$247,124
Columns	317,449	159	2,250	375	130	$438,045
						$685,169

Connection Fabrication				
# of MF's	# of Connections	Fabrication Time (hrs)	Cost ($/Fab. hr)	Total
19	170	4.8 Ea.	45.00	**$36,720**

Connection Installation			
Installation Time (days)	Installation Time (hrs)	Cost ($/Labor hr)	Total
17	136	$67.20	**$9,140**

Seismically detailed connections require inspection after they are installed. For this reason, a highly qualified inspector will carry out ultrasonic testing on the RBS connection welds after each sequence of detailing is complete. Ultrasonic testing allows the inspector to determine if subsurface defects exist within the weld that cannot be seen by any other inspection method. A high level of qualification is required to carry out this testing to ensure that pulse-echo patterns are interpreted correctly.

Since there are only four sequences within the newly designed lateral system that contain SMF's, the inspector would only be required to be on site for four days. After speaking with the steel erector for the Heart Pavilion, the labor rate of a highly qualified inspector was obtained and the total cost for this service is provided in the table below.

Connection Inspection			
Inspection Time (days)	Inspection Time (hrs)	Cost ($/hr)	Total
4	32	$115	**$3,680**

Cost Comparison

After completing the cost estimate of the separate systems, the following table was put together to draw conclusions on the overall efficiency of the redesigned systems.

Component	Existing System	Redesigned System	Savings
Foundations	$371,344	$190,000	+ $181,344
Structural Steel	$2,462,970	$1,870,646	+ $592,324
MF Connections	$98,795	$49,540	+ $49,255
Total	**$2,933,000**	**$2,110,000**	**+ $823,000**

Conclusion

The use of the Geopier Intermediate Foundation System significantly impacts the cost and construction time of the Heart Pavilion. Since Geopier elements can be installed at a rate of 30 per day, the construction time was reduced by approximately 50%. In addition, the Geopier Intermediate Foundation System is more cost efficient since less excavation is required.

The SMF's also prove to be more economical even though the fabrication time for the reduced beam section is twice that of the existing beams. Due to the considerable reduction in installation time, the SMF system is more cost efficient even when special inspections are considered.

Please reference Appendix G to view a detailed construction schedule for these systems.

CONCLUSIONS & RECOMMENDATIONS

The main focus of this final thesis report is to optimize the foundation and lateral systems for the Heart Pavilion. Classified as Seismic Site Class E soil, it was necessary at the time of design to utilize a deep foundation system to support the structure and 19 non-seismic steel moment frames to resist lateral forces. While this design exhibits no problems structurally, both systems are an area of possible optimization.

The lateral analysis proves that the use of a special steel moment frame system considerably improves the efficiency of the lateral system. By using a higher response modification coefficient, seismic loads were lowered and the base shear value was decreased by approximately 38%. As a result, only 4 three-bay SMF's and 6 two-bay SMF's were required to resist seismic forces. The ratio of the tonnage of steel used for the SMF system is approximately 41% of the existing system, which ultimately reduced erection time. Due to this significant reduction, the SMF's prove to be more economical even though special inspections are required and the fabrication time for the reduced beam sections is twice that of the existing beams.

Improvements in soil conditions were achieved through the use of the Geopier Intermediate Foundation System. Vertical reinforcement is provided to the soil due to the over consolidation of the soil from the Geopier placement. In addition, the construction time was reduced by 50% as Geopier elements can be installed at a rate of 30 per day. By implementing the Geopier Foundation System and the SMF System, steel erection could begin approximately 10 weeks earlier than originally scheduled.

The façade breadth study focuses on improvements in occupant comfort with respect to heat transfer through the wall system. By implementing the brick façade on the third floor of the Heart Pavilion, heat transfer through the wall is reduced by approximately 30% of that transferred by the existing curtain wall system. Heat loss within patient rooms on this floor would be reduced, thus improving occupant comfort.

The goals of this thesis were to create an efficient foundation and lateral system for the Heart Pavilion. Based on the results discussed, these goals are clearly met. From a feasibility standpoint, each proposed study impacts the structure in a positive manner. It is the recommendation of the author to implement all changes proposed within this thesis report.

All design values used and procedures carried out were done in accordance with applicable codes. Please refer to the appendices for further review of detailed notes, figures, or tables regarding this matter. Questions should be directed to Kristen M. Lechner via email: kml5016@psu.edu.

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APPENDIX A: *Wind and Seismic Supplementary Material*

Existing Moment Connection
Photo courtesy of Ruby + Associates

Wind Supplementary Material

(Hand calculations available upon request)

Building Information	
Number of Floors	4
Building Height (ft)	57.4
N-S Building Length (ft)	245
E-W Building Length (ft)	175
L/B in N-S Direction	1.40
L/B in E-W Direction	0.71

Building Location Factors	
Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1.15
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0

Variables to Obtain Gust Factor

Variable	Wind Direction	
	N-S	E-W
n_1 (Hz)	0.869	0.869
Stiffness	Flexible	Flexible
B	245	175
L	175	245
h	57.4	57.4
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.16	4.16
z_{BAR}	34	34
ε_{BAR}	0.333	0.333
L_{BAR}	320	320
b_{BAR}	0.45	0.45
α_{BAR}	0.25	0.25
Iz_{BAR}	0.298	0.298
Lz_{BAR}	325	325
Q	0.790	0.814
Vz_{BAR}	60.0	60.0
N_1	4.70	4.7
n_h	3.82	3.82
n_B	22.32	11.66
n_L	39.03	74.71
R_h	0.227	0.227
R_B	0.059	0.082
R_L	0.025	0.013
R_n	0.0528	0.0528
R	0.0879	0.1028
G_f	0.806	0.822

Floor Height (ft)	Level	Total Height (ft)	K_z	q_z	Wind Pressures (psf)					
					N-S Wind-ward	N-S Lee-ward	N-S Side Wall	E-W Wind-ward	E-W Lee-ward	E-W Side Wall
14.50	Roof	57.40	0.84	17.09	14.10	-9.97	-12.54	14.31	-7.54	-12.91
14.00	3	43.00	0.78	15.74	13.23	-9.97	-12.54	13.42	-7.54	-12.91
14.00	2	29.00	0.69	14.06	12.15	-9.97	-12.54	12.32	-7.54	-12.91
15.00	1	15.00	0.57	11.65	10.59	-9.97	-12.54	10.73	-7.54	-12.91

Distribution of Windward and Leeward Pressures

Level	Wind Design					
	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	42	28	0	0	2437	1580
3	82	53	42	28	3536	2287
2	78	50	125	81	2254	1450
1	76	48	202	131	1137	726
Total	**278**	**179**	**278**	**179**	**9364**	**6043**

Total Base Shear from Windward and Leeward Pressures

Story	Story Height (ft)	Total Drift (in)	Story Drift (in)	Allowable Story Drift (in) Δwind=H/400			Total Drift (in)	Allowable Total Drift (in) Δwind=H/400		
Roof	57.5	1.24	0.120	<	0.435	OK	1.240	<	1.725	OK
3	43	1.12	0.310	<	0.420	OK	1.120	<	1.290	OK
2	29	0.81	0.370	<	0.420	OK	0.810	<	0.870	OK
1	15	0.44	0.440	<	0.450	OK	0.440	<	0.450	OK

Wind Drift in X Direction

Story	Story Height (ft)	Total Drift (in)	Story Drift (in)	Allowable Story Drift (in) Δwind=H/400			Total Drift (in)	Allowable Total Drift (in) Δwind=H/400		
Roof	57.5	0.870	0.130	<	0.435	OK	0.870	<	1.725	OK
3	43	0.740	0.190	<	0.420	OK	0.740	<	1.290	OK
2	29	0.550	0.230	<	0.420	OK	0.550	<	0.870	OK
1	15	0.320	0.320	<	0.450	OK	0.320	<	0.450	OK

Wind Drift in Y Direction

Seismic Supplementary Material

Occupancy Category	IV
Importance Factor (I)	1.5
S_s	0.170
S_1	0.056
Site Class	E
Total Building Height (ft)	57.5
T_a	0.716
T_L	12
Frequency (Hz)	1.40
Structural Behavior	Rigid Diaphragm

S_{ms}	0.425
S_{m1}	0.196
S_{ds}	0.283
S_{d1}	0.131
SDC	C
R	8.0
C_s	0.034
k	1.11
Base Shear (k)	678

Base Shear and Overturning Moment Distribution

Story	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.5	1093	97320	0.174	118	118	6790
3	43	2917	188250	0.337	228	347	14900
2	29	4074	169941	0.304	206	553	16029
1	15	5136	103204	0.185	125	678	10169
Main	0	6593	0	0.000	0	678	0
Total	57.5	19812	558715	1.000	678		47888
Base Shear =	**678**	**k**					

				Main Floor						
			Approx. Area =		48,030		SF			
			Floor to Floor Ht. =		15		ft			
Walls:			**Superimposed:**				**Slab:**			
Perimeter =	1220	ft	Partitions =	20	psf		Thk. =	6.0	in	
Height =	15	ft	MEP =	10	psf		Unit Wt. =	150	pcf	
Unit Wt. =	20	psf	Finishes =	5	psf					
Weight =	**366**	**k**	**Weight =**	**1681**	**k**		**Weight =**	**2567**	**k**	

		Columns:						**Beams:**		
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)		Shape	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W10x33	26	22	15	8.58		W8x10	10	17.5	0.18	
W10x45	2	45	15	1.35		W10x12	12	15.19	0.18	
W10x49	2	49	15	1.47		W8x13	13	18.11	0.24	
W12x65	4	65	15	3.90		W12x14	14	120	1.68	
W12x79	1	79	15	1.19		W12x16	16	78.73	1.26	
W10x60	7	60	15	6.30		W8x18	18	25	0.45	
W12x58	1	58	15	0.87		W12x19	19	165	3.14	
W10x54	1	54	15	0.81		W14x22	22	630	13.86	
W10x68	2	68	15	2.04		W16x26	26	170	4.42	
W10x88	2	88	15	2.64		W14x26	26	25	0.65	
W12x87	1	87	15	1.31		W16x36	36	30	1.08	
W14x132	2	132	15	3.96		W16x40	40	30	1.20	
W14x145	4	145	15	8.70		W18x40	40	360	14.40	
W14x159	2	159	15	4.77		W21x44	44	200	8.80	
W14x193	14	193	15	40.53		W21x48	48	25	1.20	
W14x211	8	211	15	25.32		W24x55	55	90	4.95	
			Weight =	**114**	**k**	W24x62	62	85	5.27	
						W21x62	62	25	1.55	
						W24x68	68	110	7.48	
						W24x84	84	60	5.04	
						W24x103	103	35	3.61	
						W24x104	104	70	7.28	
						W24x117	117	35	4.10	
								Weight =	**92**	**k**
		Main Floor Weight =			**4820**	**k**	**OR**	**100.4**	**psf**	

This table is provided to show the method used to compute floor weights. All tables are available upon request.

Story	Seismic Design					
	Story Loads (k)			Story Shears (k)		
	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	118	114.74	2.92	118	114.74	2.92
3	228	271.76	15.95	347	386.50	10.35
2	206	207.98	0.85	553	594.48	7.03
1	125	130.33	3.91	678	724.81	6.47
Total Base Shear (k)	**678**	**724.81**	**6.47**			
Overturning Moment (ft-k)	**47,888**	**51,329**	**6.70**			

Story	**Total Drift (in)**	**Story Drift (in)**	**Amplified Drift (in)**	**Reduction $(C_uT_a)/T_x$**		**Allowable Story Drift (in)**	
Roof	5.08	0.82	3.01	1.27	<	2.61	OK
3	4.26	1.12	4.09	1.73	<	2.52	OK
2	3.15	1.59	5.83	2.47	<	2.52	OK
1	1.56	1.56	5.70	2.42	<	2.70	OK

Seismic Drift in X Direction

Story	**Story Drift Ratio**	**0.7 x Story Drift Ratio**	**0.8 x Story Drift Ratio**	**Avg. Drift Ratio next 3 Stories**	**Soft Story Status**
Roof	0.00471	0.00330	0.00377	--	No
3	0.00664	0.00465	0.00531	--	No
2	0.00946	0.00663	0.00757	--	No
1	0.00864	0.00605	0.00691	0.00694	No

Soft Story Status in X Direction

Story	Total Drift (in)	Story Drift (in)	Amplified Drift (in)	Reduction $(C_uT_a)/T_y$		Allowable Story Drift (in)	
Roof	4.66	0.76	2.79	1.05	<	2.61	OK
3	3.90	1.04	3.81	1.44	<	2.52	OK
2	2.86	1.44	5.28	1.99	<	2.52	OK
1	1.42	1.42	5.21	1.96	<	2.70	OK

Seismic Drift in Y Direction

Story Drift Ratio	0.7 x Story Drift Ratio	0.8 x Story Drift Ratio	Avg. Drift Ratio next 3 Stories	Soft Story Status
0.00437	0.00306	0.00349	--	No
0.00619	0.00433	0.00495	--	No
0.00857	0.00600	0.00686	--	No
0.00789	0.00552	0.00631	0.00638	No

Soft Story Status in Y Direction

-End of Section-

Appendix B: *Existing Member Spot Checks*

Existing Steel Structure
Photo courtesy of Ruby + Associates

Existing Gravity & Lateral Member Spot Checks

Please reference Technical Reports I and III for this information.

Existing Vibration Criterion Spot Check

Floor Vibration for Sensitive Equipment--

INPUT YOUR DATA:

Loads:	Dead	85	psf	Slab:	w_c	150	pcf
	Live	11	psf		f'_c	3.5	ksi
Framing:					t_{slab}	4.5	in
Beam	Spacing	8.33	ft		t_{rib}	2	in
	Span	30	ft		wt	81	psf
Girder	Span	25	ft		E_c	3437	ksi
					n	6.25	<-- *modular ratio*
					Avg. Depth	5.5	in

Beam	W16x26		**Girder**	W24x68	
A	7.68	in^2	A	20.1	in^2
I_x	301	in^4	I_x	1830	in^4
d	15.7	in	d	23.7	in
b_{eff}	90	in	b_{eff}	75	in
y_{bar}	-0.968	in	wt	68	plf
	below top of form deck		y_{bar}	1.85	in
				below effective slab	

$w_j = S_{BM}(DL+LL+SPAN_G/S_{BM})$

$= 827$ plf

$I_{TR}=I_j= 1452$ in^4

$\Delta_j = 5w_jL^4/(384EI_j)$

$= 0.358$ in

$\Delta_{oj} = L^3/(96EI_j)$

$= 1.2E\text{-}05$ in/lb

$\Delta_{og} = L^3/(96EI_j)$

$= 1.7E\text{-}06$ in/lb

can use 1/96 because this is a simply supported geometric beam span

$w_g = SPAN_G(w_j/S_{BM})$+self weight

2549 plf

$I_{TR}=I_g= 5754$ in^4

$\Delta_g = 5w_gL^4/(384EI_g)$

0.134 in

$\Delta_g' = (SPAN_G/B_j)\Delta_g$

0.090 in

$D_s =$ 26.6 in^4/ft

$D_j =$ 174 Hz

$B_j =$ 37.5 ft

Since 0.018 ≤ de/S= 0.055 then,

$N_{eff} = 0.49+34.2d_e/S+(9.0x10^{-9})L_j^4/I_j-0.00059(L_j/S)2$

$=$ 2.47

Since:

$4.5 \times 10^6 \leq L_j^4/I_j =$ 1.16E+07 $\leq 257 \times 10^6$

$2 \leq L_j/S =$ 3.60 ≤ 30 then,

$\Delta_P = \Delta_{oj}/N_{eff} + \Delta_{gP}/2$

$=$ 4.68E-06 in/lb

From Table 6-2, based on 185 lb weight, for a 185 lb person walking at 100 steps/min (fast),

$F_m/W =$ 1.70

$F_m =$ 314.5 lb

From Table 6-2, the corresponding pulse rise frequency is,

$f_o =$ 5.0 Hz

$f_n = 0.18\sqrt{(g/(\Delta_j+\Delta_g'))}$

$=$ 5.29 Hz

$f_n/f_o =$ 1.06

Since $f_n > 5.0$ Hz,

$X_{max} = F_m \Delta_P f_o^2/(2f_n^2)$

$=$ 0.0006575 in

$=$ 657.5 µin

$U_v = \pi F_m f_o^2$

$=$ 24701 lb-Hz²

$V = U_v \Delta_P / f_n$

$=$ 2.18E-02 in-Hz

$=$ 2.18E-02 in/sec

$=$ 21843 µin/sec

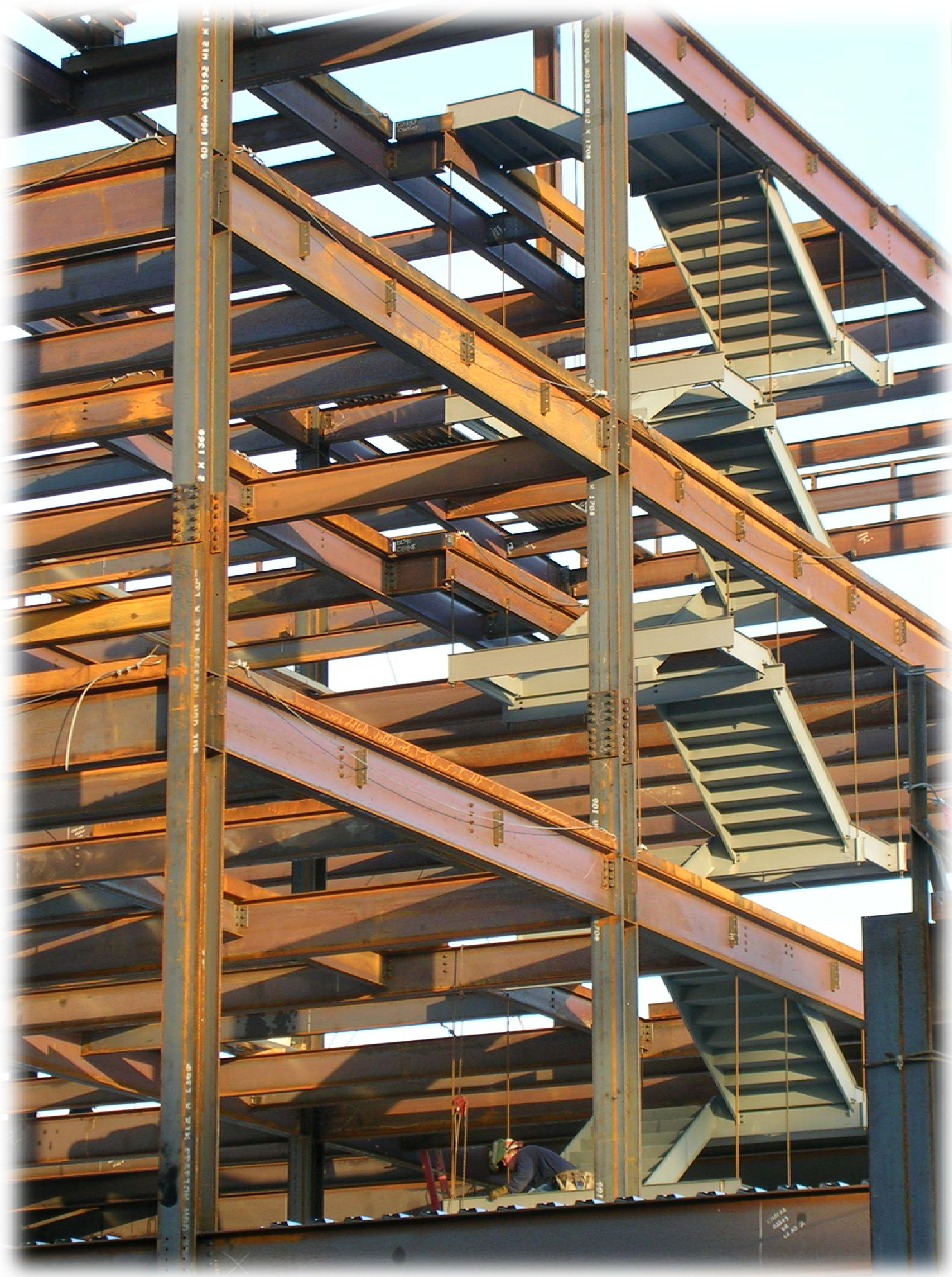
From Table 6-1,

$V_{allow} =$ 8000 µin/sec

NG **$V < V_{allow}$**

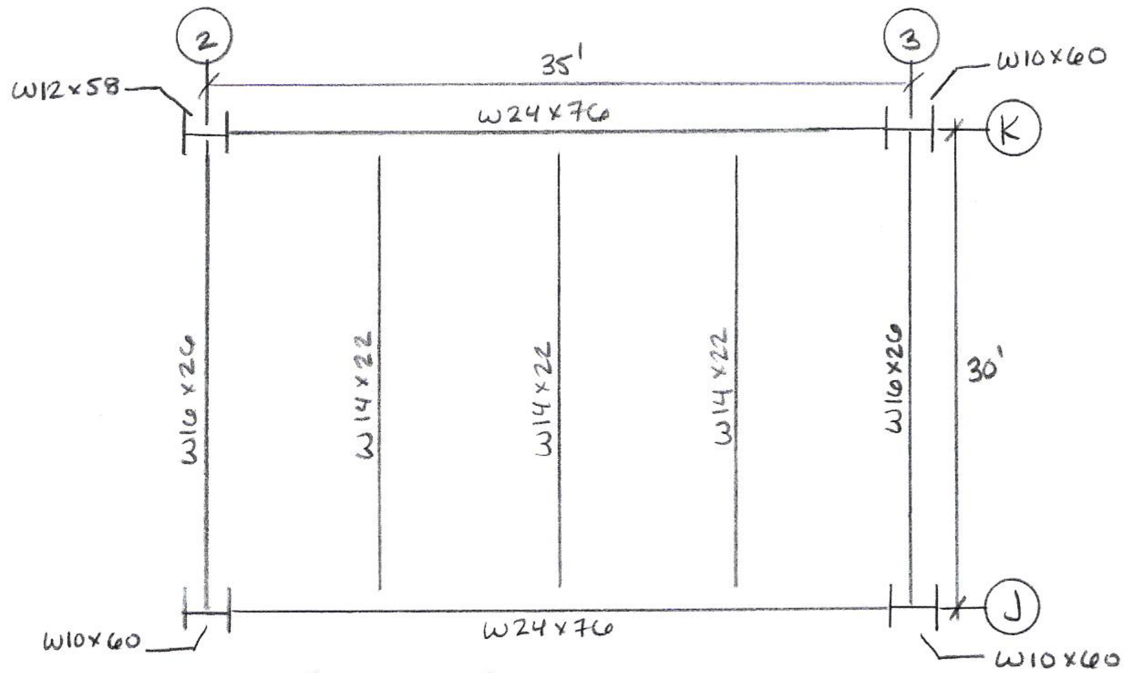
-End of Section-

Appendix C: ***Redesigned Member Spot Checks***



Existing Steel Structure
Photo courtesy of Ruby + Associates

TYPICAL INTERIOR BAY – 2^{ND} FLOOR



LOADS: LL = 60 psf (HOSPITAL)

DL = 110 psf (APPROXIMATE BASED ON SEISMIC CALC FLOOR WT.'S)

FLOOR SYSTEM: $4\frac{1}{2}''$ NWC

2'' x 20 GA DECK

LOAD COMBINATION: 1.2D + 1.6L

SPOT CHECK BEAM:

FACTORED LOAD: $w_u = 1.2D + 1.6L$

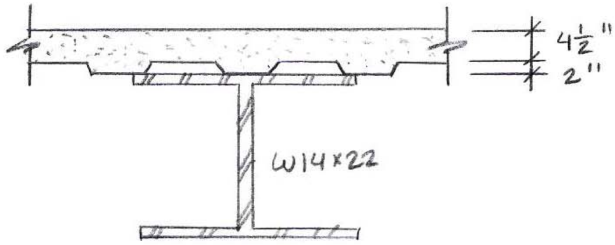
$w_u = 1.2(110\text{ psf}) + 1.6(60\text{ psf}) = 228\text{ psf}$

TRIB WIDTH $= 35'/4 = 8.75'$

$w_u = 228\text{psf}(8.75')/1000 = 2.00\text{ klf}$

$M_u = \dfrac{w_u l^2}{8} = \dfrac{2.00\text{ klf}(30)^2}{8} = 225^{\text{'k}}$

$b_{eff} = \min \left| \begin{array}{l} \text{SPACING} = 8.75'(12) = 105'' \\ \dfrac{\text{SPAN}}{4} = \dfrac{30'(12)}{4} = 90'' \quad \Leftarrow \text{CONTROLS} \end{array} \right.$



CHECK BENDING FOR CONSTRUCTION LOADING:

$w_{CONC} = 150\text{pcf}\,(4.5''/12) = 56.3\text{ psf}$

$w_{CONC} = 56.3\text{psf}\,(8.75') = 493\text{ plf} = 0.493\text{ klf}$

$w_{LIVE} = 20\text{ psf}\,(8.75') = 175\text{ plf} = 0.175\text{ klf}$

$w_u = 1.2(0.493) + 1.6(0.175) = 0.872\text{ klf}$

$M_u = \frac{w_u l^2}{8} = \frac{0.872\text{ klf}\,(30)^2}{8} = 98.1^{1k}$

$\phi M_{n_{14\times22}} = 125^{1k} > 98.1^{1k}$ ✓ OK

NOTE: COMPARE w/ ϕM_n VALUE IN TABLE 3-2 B/C SYSTEM IS NOT COMPOSITE UNTIL CONSTRUCTION IS COMPLETE.

FROM TABLE 3-19:

ASSUME $\Sigma Q_n = 157^k$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{157^k}{0.85(3.5)(90)} = 0.586''$

$Y2 = 6\tfrac{1}{2}'' - a/2 = 6\tfrac{1}{2}'' - 0.586''/2 = 6.21''$ (ROUND ↓ TO BE CONSERVATIVE)

USING TABLE 3-19:

W14x22 $\quad Y2 = 6'' \quad \Sigma Q_n = 157^k$ @ PNA BFL

$\phi M_n = 236^{1k} > M_u = 225^{1k}$ ✓ OK

CHECK # SHEAR STUDS:

TABLE 3-21:

SHEAR STUD DIAM. = 3/4" ; 1 STUD/RIB
DECK ⊥
$f'_c = 3000$ ksi (CONSERVATIVE)

$\Big\}\; Q_n = 17.2^k$

$\#\text{ STUDS REQ.} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{157}{17.2} \times 2 = 18.3 \rightarrow 19$ STUDS REQ.

STUDS PROVIDED = 30 (PLACED @ 12" O.C. OVER BM LENGTH)

STUDS PROVIDED > # STUDS REQ. ✓ OK

CHECK DEFLECTION:

TABLE 3-20:

$Y2 = 6'' \Longrightarrow I_{LB} = 550 \text{ in}^4$

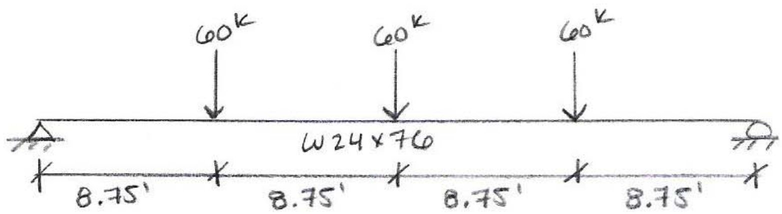
$$\Delta = \frac{5 w_{LL} l^4}{384 E I_{LB}} = \frac{5(0.525\text{ klf})(30)^4}{384(29000)(550)}(1728) = 0.60''$$

$$w_{LL} = 60\text{ psf}(8.75')/1000 = 0.525\text{ klf}$$

$$\Delta_{ALLOW} = l/360 = 35(12)/360 = 1.17''$$

$0.60'' < 1.17''$ ✓ OK

SPOT CHECK GIRDER:



$$w_{u,DL} = 1.2(110\text{ psf})(8.75')/1000 = 1.16\text{ klf}$$

$$P_{DL} = \frac{w_{u,DL}\, l}{2} = \frac{1.16\text{ klf}(30')}{2} = 17.4^k$$

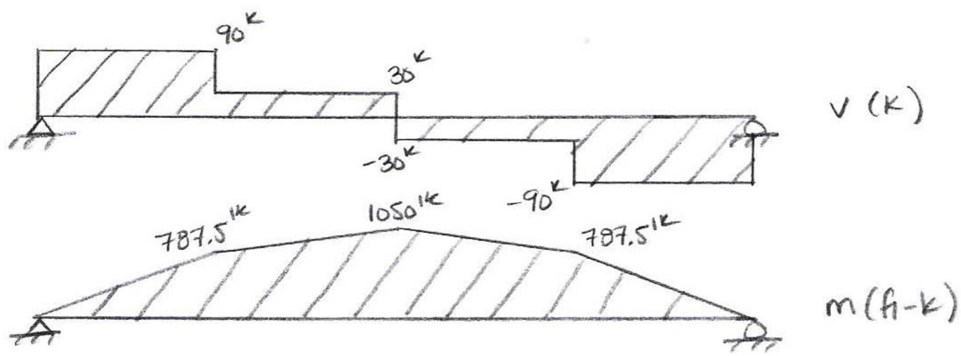
$$w_{u,LL} = 1.6(60\text{ psf})(8.75')/1000 = 0.840\text{ klf}$$

$$P_{LL} = \frac{w_{u,LL}\, l}{2} = \frac{0.840(30')}{2} = 12.6^k$$

BMS FRAME IN ON EA. SIDE

TOTAL P ON GIRDER $= (12.6^k + 17.4^k) \times 2 = 60^k$

DETERMINE M_{MAX} —



$M_{MAX} = 1050'^k$

ASSUME $Y2 = 3''$ REQUIRING PNA #6 (TABLE 3-19)

$\Sigma Q_n = 393^k$

$b_{eff} = \min \begin{cases} \text{SPACING} = 30' = 360'' \\ \frac{\text{SPAN}}{4} = \frac{25'}{4} = 6.25' = 75'' \leftarrow \text{CONTROLS} \end{cases}$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{393^k}{0.85(3.5)(75)} = 1.76''$$

GIRDERS PLACED 2" ABOVE BMS ∴ DEPTH = $4\frac{1}{2}''$ NOT $6\frac{1}{2}''$

$Y2 = 4\frac{1}{2}'' - a/2 = 4\frac{1}{2}'' - 1.76''/2 = 3.62''$ (ROUND ↓ TO BE CONSERVATIVE)

USING TABLE 3-19: (W24x76)

$Y2 = 3''$ $\Sigma Q_n = 393^k$ PNA #6

$\phi M_n = 1060'^k > M_{MAX} = 1050'^k$ ✓ OK

CHECK DEFLECTION:

$I_{LB} = 3400 \text{ in}^4$ (TABLE 3-20)

$$\Delta = \frac{5 w_{LL} l^4}{384 E I_{LB}}$$

$w_{LL} = 60psf(8.75')/1000 = 0.525 \text{ klf}$

$\Delta_{ALLOW} = l/360 = 30(12)/360 = 1''$

$$\Delta = \frac{5(0.525 \text{ klf})(30)^4}{384(29000)(3400)}(1728) = 0.01''$$

$0.01'' < 1''$ ✓ OK

SPOT CHECK COLUMN: COL LINE J-3

W10x60 COL

TRIB AREA = $30'(30') = 900 ft^2$ INFLUENCE AREA = $3(900 ft^2) = 2700 ft^2$

LL REDUCTION = $LL = L_o\sqrt{0.25 + 15/\sqrt{A_I}} = L_o\sqrt{0.25 + 15/\sqrt{2700}} = 0.734 L_o$

$LL = 0.734(60psf) = 44 psf$

$P_u = [1.2(110psf) + 1.6(44psf)](900 ft^2)(3) = 545{,}000 \text{ lb} = 545^k$

$\frac{KL}{r_x} = \frac{15(12)}{4.39} = 41.0$ $\frac{KL}{r_y} = \frac{15(12)}{2.57} = 70.0 \leftarrow$ CONTROLS

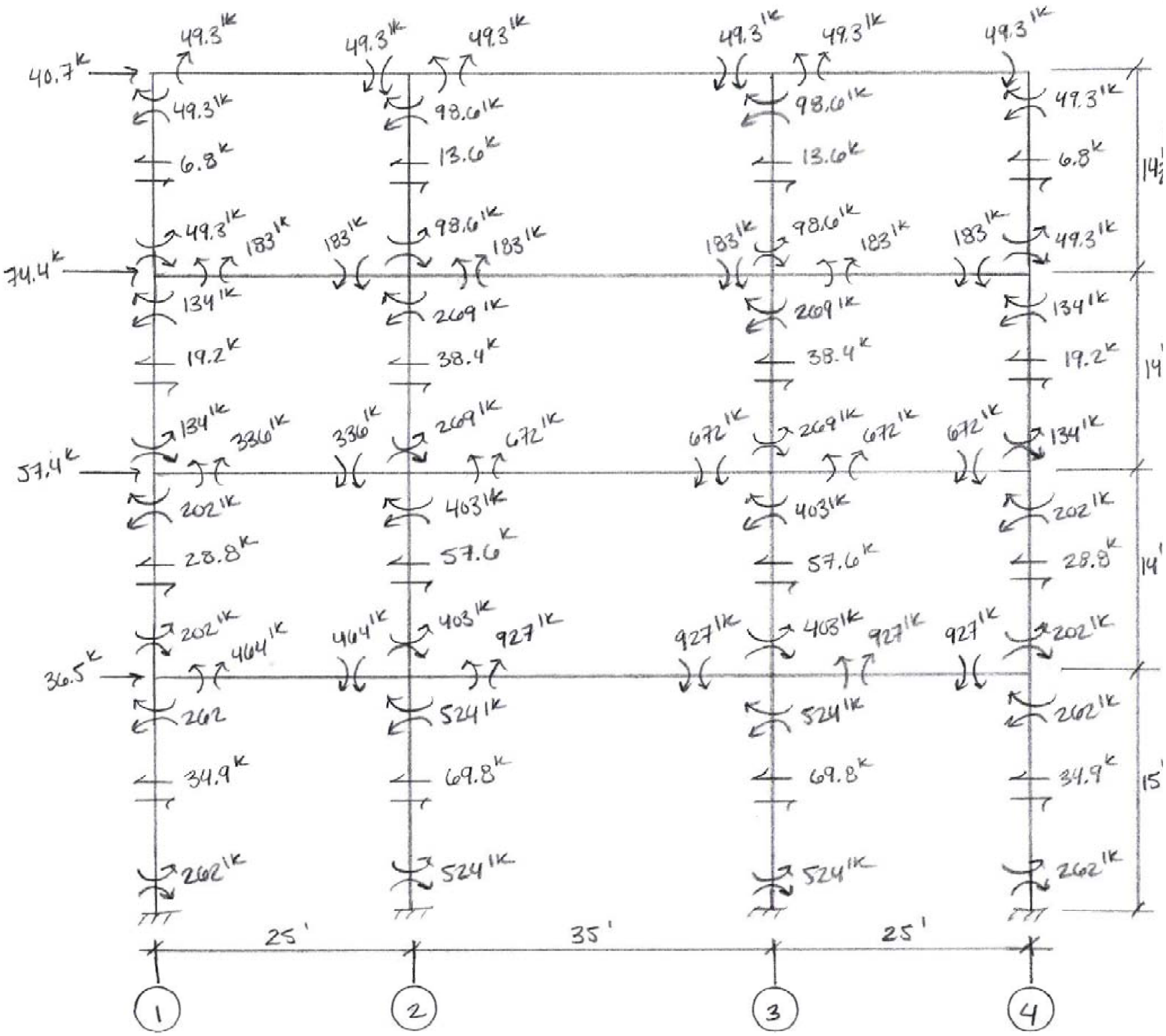
$\frac{KL}{r} \le 4.71\sqrt{E/F_y} = 4.71\sqrt{29000/50} = 113 > 70.0$ ∴ INELASTIC BEHAVIOR

$F_{CR} = [0.658^{F_y/F_e}]F_y = [0.658^{50/58.4}]50 = 34.9 \text{ ksi}$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{(70)^2} = 58.4 \text{ ksi}$

$\phi P_n = \phi F_{CR} A_g = 0.9(34.9)(17.6) = 553^k > P_u = 545^k$ ✓ OK

Redesigned Lateral Member Spot Checks — *SMF 3*



SMF 3 GIRDER CHECK :

GIRDER DOES NOT DIRECTLY TAKE LL + DL FROM FLOOR DIAPHRAGM DUE TO ORIENTATION OF GRAVITY BMS. THEREFORE, NO NEED TO ADD MOMENT DUE TO GRAVITY AT ENDS OF GIRDER TO MOMENT DUE TO LATERAL FORCES.

$M_{MAX} = 927^{IK}$ (FROM PORTAL ANALYSIS)

FROM TABLE 3-2 :

W24×94 — $\phi M_n = 953^{IK} > M_u = 927^{IK}$ ✓ OK

SMF 3 COLUMN CHECK :

W14×257

FROM PORTAL ANALYSIS : $M_u = 524^{IK}$

FIND P_u FROM DL + LL ON FLOOR —

$A_I = (30')(30')(3 \text{ FLOORS ABOVE}) = 2700 \text{ ft}^2$

$L_R = 60\text{psf}\left[0.25 + \frac{15}{\sqrt{2700}}\right] = 32.3 \text{ psf}$

$0.40(60\text{psf}) = 24\text{psf}$

$32.3\text{psf} > 24\text{psf}$ ∴ OK TO USE

$P_{LL} = 2700\text{ft}^2(32.3\text{psf}) = 87.2^K$

$P_{DL} = 110\text{psf}(4 \text{ FLOORS})(30 \times 30) = 396^K$

$P_u = 1.2 P_D + 1.6 P_L$

$= 1.2(396) + 1.6(87.2) = 615^K$

TABLE 6-1 :

$KL = 15' = $ STD. HT.

FOR W14×257 — $p = 0.338 \times 10^{-3}/K$

$b_x = 0.488 \times 10^{-3}/K$

$pP_u + b_x M_u = 0.338 \times 10^{-3}(615) + 0.488 \times 10^{-3}(524)$

$= 0.464 < 1.0$ ✓ OK

COLS ARE ADEQUATE FOR STRENGTH REQUIREMENTS
— LARGELY CONTROLLED BY SEISMIC DRIFT + "STRONG COL - WEAK BM" CRITERION FOR A REDUCED BM SECTION —
— THIS COLUMN SIZE ALSO ELIMINATES THE NEED FOR TRANSVERSE STIFFENERS + DOUBLER PLATES —

Redesigned Vibration Criteria Spot Check

Floor Vibration for Sensitive Equipment--

INPUT YOUR DATA:

Loads:	Dead	85	psf	Slab:	w_c	150	pcf
	Live	11	psf		f'_c	3.5	ksi
Framing:					t_{slab}	4.5	in
Beam	Spacing	6.25	ft		t_{rib}	2	in
	Span	30	ft		wt	81	psf
Girder	Span	25	ft		E_c	3437	ksi
					n	6.25	<-- *modular ratio*
					Avg. Depth	5.5	in

Beam	W18x40		**Girder**	W24x68	
A	11.8	in^2	A	20.1	in^2
I_x	612	in^4	I_x	1830	in^4
d	17.9	in	d	23.7	in
b_{eff}	90	in	b_{eff}	75	in
y_{bar}	-0.217	in	wt	68	plf
	below top of form deck		y_{bar}	1.85	in
				below effective slab	

$w_j = S_{BM}(DL+LL+SPAN_G/S_{BM})$

$= 627$ plf

$I_{TR}=I_j= 2497$ in^4

$\Delta_j = 5w_jL^4/(384EI_j)$

$= 0.158$ in

$\Delta_{oj} = L^3/(96EI_j)$

$= 6.7E\text{-}06$ in/lb

$\Delta_{og} = L^3/(96EI_j)$

$= 1.7E\text{-}06$ in/lb

$w_g = SPAN_G(w_j/S_{BM})$+self weight

2574 plf

$I_{TR}=I_g= 5754$ in^4

$\Delta_g = 5w_gL^4/(384EI_g)$

0.136 in

$\Delta_g' = (SPAN_G/B_j)\Delta_g$

0.111 in

can use 1/96 because this is a simply supported geometric beam span

$D_s = 26.6$ in^4/ft

$D_j = 400$ Hz

$B_j = 30.5$ ft

Since $0.018 \leq$ de/S= 0.073 then,

$N_{eff} = 0.49+34.2d_e/S+(9.0x10^{-9})L_j^4/I_j-0.00059(L_j/S)2$

$= 3.04$

Since:

$4.5 \times 10^6 \leq L_j^4/I_j =$ 6.73E+06 $\leq 257 \times 10^6$

$2 \le L_j/S=$	4.80	≤ 30	then,
$\Delta_P=$	$\Delta_{oj}/N_{eff} + \Delta_{gP}/2$		
=	2.20E-06	in/lb	

From Table 6-2, based on 185 lb weight, for a 185 lb person walking at 100 steps/min (fast),

$F_m/W=$	1.70	
$F_m=$	314.5	lb

From Table 6-2, the corresponding pulse rise frequency is,

$f_o=$	5.0	Hz
$f_n=$	$0.18\sqrt{(g/(\Delta_j+\Delta_g'))}$	
=	6.82	Hz
$f_n/f_o=$	1.36	

Since $f_n > 5.0$ Hz,

$X_{max}=$	$F_m\Delta_P f_o^2/(2f_n^2)$	
=	0.0001863	in
=	186.3	μin
$U_v=$	$\pi F_m f_o^2$	
=	24701	lb-Hz²
$V=$	$U_v\Delta_P/f_n$	
=	7.98E-03	in-Hz
=	7.98E-03	in/sec
=	7983	μin/sec

From Table 6-1,

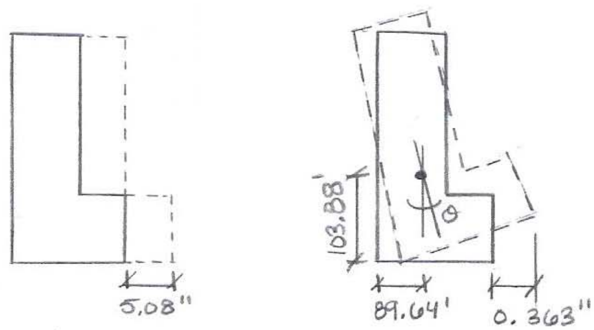
$V_{allow}=$	8000	μin/sec
	OK	**$V < V_{allow}$**

-End of Section-

TORSION IRREGULARITY CHECK:

ROOF: COR = (89.64, 103.88)

X-DIRECTION:



Δ_{V_X} $\Delta_{ACC.T.}$

FROM RAM FRAME –

$\theta = 0.0167°$

$\tan(0.0167) = \frac{X}{103.88(12)}$

$X = 0.363"$

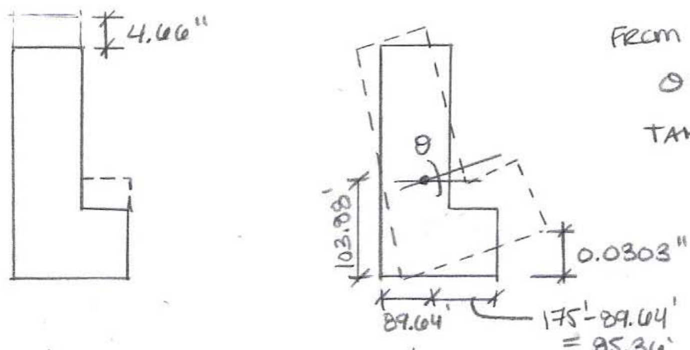
$\delta_{MAX} = 5.08" + 0.363" = 5.44"$

$\delta_{AVG} = 5.08"$

$A_X = \left[\frac{\delta_{MAX}}{\delta_{AVG}(1.2)}\right]^2 = \left[\frac{5.44}{5.08(1.2)}\right]^2 = 0.769 < 1.0$

$\therefore$ USE $A_X = 1.0$

Y-DIRECTION:



Δ_{V_Y} $\Delta_{ACC.T.}$

FROM RAM FRAME –

$\theta = 0.0017°$

$\tan(0.0017) = \frac{X}{85.36'(12)}$

$X = 0.0303"$

$\delta_{MAX} = 4.66" + 0.0303" = 4.69"$

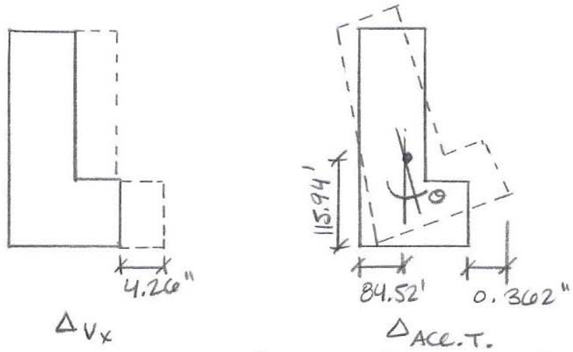
$\delta_{AVG} = 4.66"$

$A_X = \left[\frac{4.69}{4.66(1.2)}\right]^2 = 0.703 < 1.0$

$\therefore$ USE $A_X = 1.0$

STD. 3: COR = (84.52, 115.94)

X-DIRECTION:



FROM RAM FRAME —

$\theta = 0.0149°$

$TAN(0.0149) = \frac{X}{115.94(12)}$

$X = 0.362"$

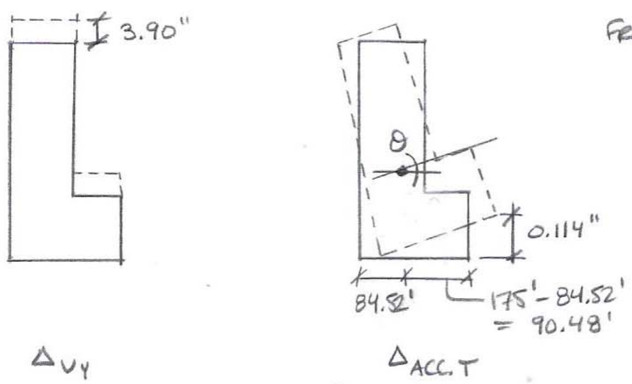
$\delta_{MAX} = 4.26" + 0.362" = 4.62"$

$\delta_{AVG} = 4.26"$

$A_X = \left[\frac{4.66}{4.26(12)}\right]^2 = 0.817 < 1.0$

$\therefore$ USE $A_X = 1.0$

Y-DIRECTION:



FROM RAM FRAME —

$\theta = 0.006°$

$TAN(0.006) = \frac{X}{90.48(12)}$

$X = 0.114"$

$\delta_{MAX} = 3.90" + 0.114" = 4.01"$

$\delta_{AVG} = 3.90"$

$A_X = \left[\frac{4.01}{3.90(1.2)}\right]^2 = 0.734 < 1.0$

$\therefore$ USE $A_X = 1.0$

CHECK HORIZ/VERT IRREG FOR ROOF-3RD STD. —

$\delta_{MAX} = (5.08 + 0.363) - (4.26 + 0.362) = 0.821"$

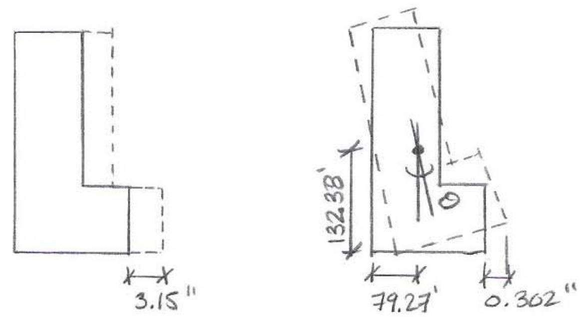
$\delta_{AVG} = 5.08 - 4.26 = 0.82"$

$\delta_{MAX}/\delta_{AVG} = 0.821"/0.82" = 1.00 < 1.2$ ✓ OK

$\therefore$ NO TORSIONAL IRREG.

STO. 2: COR = (79.27, 132.38)

X-DIRECTION:



Δ_{Vx} $\Delta_{ACC.T.}$

FROM RAM FRAME—

$\theta = 0.0109°$

$TAN(0.0109) = \frac{X}{132.38(12)}$

$X = 0.302"$

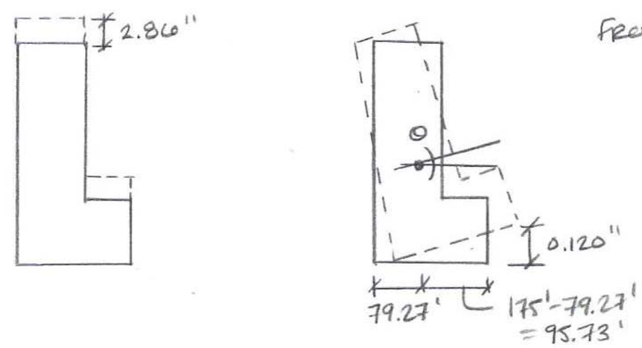
$\delta_{MAX} = 3.15" + 0.302" = 3.45"$

$\delta_{AVG} = 3.15"$

$A_X = \left[\frac{3.45}{3.15(1.2)}\right]^2 = 0.833 < 1.0$

$\therefore$ USE $A_X = 1.0$

Y-DIRECTION:



FROM RAM FRAME—

$\theta = 0.006°$

$TAN(0.006) = \frac{X}{95.73(12)}$

$X = 0.120"$

Δ_{VY} $\Delta_{ACC.T.}$

$\delta_{MAX} = 2.86" + 0.120" = 2.98"$

$\delta_{AVG} = 2.86"$

$A_X = \left[\frac{2.98}{2.86(1.2)}\right]^2 = 0.754 < 1.0$

$\therefore$ USE $A_X = 1.0$

CHECK HORIZ/VERT IRREG FOR 3RD STO. + 2ND STO. —

$\delta_{MAX} = (4.26 + 0.362) - (3.15 + 0.302) = 1.17"$

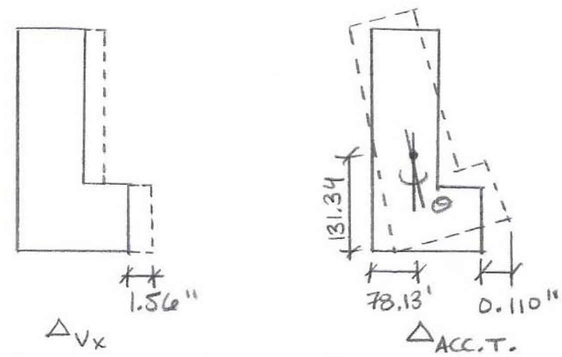
$\delta_{AVG} = 4.26 - 3.15 = 1.11"$

$\delta_{MAX}/\delta_{AVG} = 1.17"/1.11" = 1.05 < 1.2$ ✓ OK

$\therefore$ NO TORSIONAL IRREG.

STO. 1: COR = (78.13, 131.34)

X-DIRECTION:



FROM RAM FRAME —

$\theta = 0.004°$

$\tan(0.004) = \dfrac{X}{131.34(12)}$

$X = 0.110"$

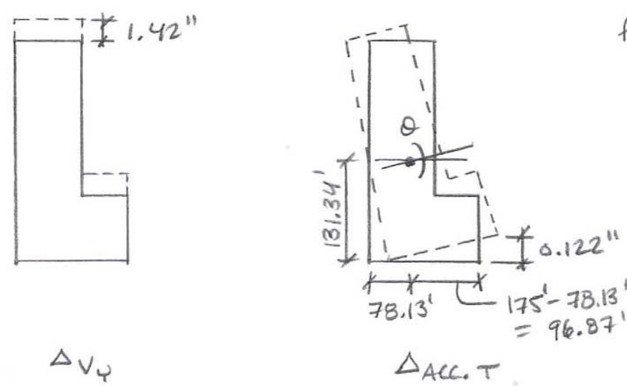
$\delta_{MAX} = 1.56" + 0.110" = 1.67"$

$\delta_{AVG} = 1.56"$

$A_X = \left[\dfrac{1.67}{1.56(1.2)}\right]^2 = 0.796 < 1.0$

$\therefore$ USE $A_X = 1.0$

Y-DIRECTION:



FROM RAM FRAME —

$\theta = 0.006°$

$\tan(0.006) = \dfrac{X}{96.87(12)}$

$X = 0.122"$

$\delta_{MAX} = 1.42" + 0.122" = 1.54"$

$\delta_{AVG} = 1.42"$

$A_X = \left[\dfrac{1.54}{1.42(1.2)}\right]^2 = 0.817 < 1.0$

$\therefore$ USE $A_X = 1.0$

CHECK FOR HORIZ/VERT IRREG FOR 2ND STO - 1ST STO. —

$\delta_{MAX} = (3.15 + 0.302) - (1.56 + 0.110) = 1.78"$

$\delta_{AVG} = 3.15 - 1.56 = 1.59"$

$\delta_{MAX}/\delta_{AVG} = 1.78"/1.59" = 1.12 < 1.2$ ✓OK

$\therefore$ NO TORSIONAL IRREG.

-End of Section-

Appendix D: *RBS Connection Design*

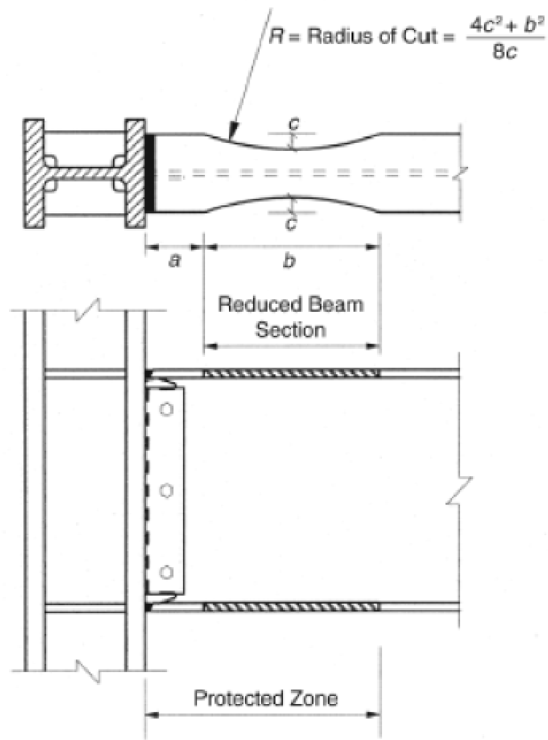


Fig. 5.1. Reduced beam section connection.

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications
AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

RBS Connection
Image courtesy of AISC, Inc.

RBS Connection Design for Chosen Configuration (Alternative 3)

SMF Column Design

INPUT YOUR DATA:

General:

SDC	C		E_s	29000	ksi
ρ	1.0		F_y	50	ksi
S_{DS}	0.283		L_{btop}	14.0	ft
$0.2*S_{DS}$	0.057		L_{bbot}	15.0	ft
A	44275	ft^2	(tributary area to leaning columns)		
A	20625	ft^2	(tributary area to stability/lateral columns)		

Loads:			W14x257:			W24x94:		
P_D	341	k	d	16.4	in	I_x	2700	in^4
P_L	69	k	t_w	1.18	in	L	35	ft
P_{Lr}	0	k	t_f	1.89	in	W14x176:		
P_{QE}	70	k	b_f	16.0	in	I_x	2140	in^4
V_D	1.9	k	I_x	3400	in^4	W14x145:		
V_L	0.69	k	I_y	1290	in^4	I_x	1710	in^4
V_{QE}	60	k	A_g	75.6	in^2			
M_{xDtop}	27	k-ft	r_x	6.71	in			
M_{xLtop}	9.6	k-ft	r_y	4.13	in			
M_{xLrtop}	0	k-ft	S_x	415	in^3			
M_{xQEtop}	837	k-ft	Z_x	487	in^3			
M_{xDbot}	0	k-ft						
M_{xLbot}	0	k-ft						
M_{xLrbot}	0	k-ft						
M_{xQEbot}	0	k-ft						

Assume there is no transverse loading between the column supports in the plane of bending.

Determine the factored loads--

1.2D+1.0E+0.5L+.2S

$V_u = (1.2+0.2S_{DS})V_D+\rho V_{QE}+0.5V_L+0.2V_S$

$= 63$ k

Determine the lateral-translation forces and nontranslation forces for subsequent calcs of secondary forces. It will be assumed that nontranslational forces are due to dead and live loads and translational forces are due to seismic load.

$P_u = (1.2+0.2S_{DS})P_D+\rho P_{QE}+0.5P_L+0.2P_S$

$P_{nt} = 463$ k

$P_{lt} = 70.00$ k

M_u= $(1.2+0.2S_{DS})M_D+\rho M_{QE}+0.5M_L+0.2M_S$

M_{ntxtop}=	38.7	k-ft
M_{ntxbot}=	0	k-ft
M_{ltxtop}=	837	k-ft
M_{ltxbot}=	0	k-ft

Check column element slenderness--

Width-Thickness Ratio for Flanges λ_f= $b_f/(2t_f)$

= 4.23

Flange Compactness λ_{ps}= $0.30\sqrt{(E_S/Fy)}$

7.22

$\lambda_f < \lambda_{ps}$

Flanges are seismically compact.

Width-Thickness Ratio for Web λ_w= $h/(t_w)$

13.90

Assuming B_2= 1.95

P_u= $P_{nt}+B_2P_{lt}$

600 k

C_a= $Pu/(\Phi_b P_y)$

= $P_u/(0.90F_yA_g)$

= 0.1762

λ_{ps}= $3.14\sqrt{(E_s/F_y)}(1-1.54C_a)$

55.1

$\lambda_w < \lambda_{ps}$

Web is seismically compact.

Check unbraced length (using Manual Table 3-2)--

L_p	14.6	ft
L_r	104	ft
L_b	15.0	ft

$L_p<L_b<L_r$ OK

Determine K--

For the x-x axis,

G_{top}= $\sum(I_c/L_c)/\sum(I_g/L_g)$

2.56

G_{bot}= $\sum(I_c/L_c)/\sum(I_g/L_g)$
3.04
From Commentary Figure C-C2.4,
Kx= 1.77
Leaning column amplifier:
$\sqrt{(1+\sum P_{leaning}/\sum P_{stability})}$=
= 1.77
Therefore,
K_x= 3.14
From § C1.3a and Commentary Table C-C2.2, Ky= 1.0

Determine the compression strength of the column--

K_xL_x/r_x= 78.6
K_yL_y/r_y= 40.7
Using Manual Table 4-22 with K_xL_x/r_x= 78.6

$\Phi_{cr}F_{cr}$= 28.6 ksi
P_n= $F_{cr}A_g$
Φ_cP_n= 2164 k

Interpolation

KL/r	$\Phi_{cr}F_{cr}$
78	28.8
78.6	28.62
79	28.5

Determine the flexural strength--

$M_n=M_p$= F_yZ_x
= 2029 k-ft
M_{cx}= Φ_bM_{nx}
= 1826 k-ft

Consider second order effects--

B_1= $C_m/(1-\alpha P_r/P_{e1}) \geq 1$
P_r= $P_{nt}+P_{lt}$
= 533 k
α= 1.0
C_m= $0.6-0.4(M_1/M_2)$
= 0.200
P_{e1x}= $\pi^2EI_x/(K_xL_x)^2$
= 34479 k
B_{1x}= 0.203 ≥ 1.0
= 1.0
B_2= $1/(1-\alpha P_{nt}/\sum P_{e2}) \geq 1$
P_D= 13239 k
LL Reduction= 0.71
P_L= 4741 k
P_{QE}= 0 k

$\sum P_{nt}=$ $(1.2+0.2S_{DS})*P_D+\rho P_{QE}+0.5P_L+0.2P_S$
= 21665 k

$P_{e2}=$ $\pi^2 EI_x/(K_xL_x)^2$ for larger sized columns on main and 1st floor
3498 k

To determine ∑Pe2 for the entire frame, determine the contribution of the 14x193 columns:

$G_{top}=$ $\sum(I_c/L_c)/\sum(I_g/L_g)$
3.56

$G_{bot}=$ $\sum(I_c/L_c)/\sum(I_g/L_g)$
3.83

Using Commentary Figure C-C2.4,
$K_x=$ 1.95
Adjust for the effects of leaning columns as before,

$K_x=$ 3.46

$P_{e2}=$ $\pi^2 EI_x/(K_xL_x)^2$ for smaller sized columns on 2nd and 3rd floor
= 1814 k

$\sum P_{e2}=$ 42490 k

$B_2=$ 2.04 Close to assumed value

$P_r=$ $P_{nt}+B_2P_{lt}$
= 606 k

$M_r=$ $B_1M_{nt}+B_2M_{lt}$
= 38.73 k-ft

Check combined loading--

$P_r/P_c=$ 0.280
Since $P_r/P_c >$
0.2,

$[P_r/(\Phi P_c)]+8/9*[M_{rx}/M_{cx}]+8/9*[M_{ry}/M_{cy}] \leq 1.0$ EQ. H1-1a
= 0.299 < 1.0 OK

Check shear strength of the column--

$2.24\sqrt{(E_S/F_{yw})}=$ 53.9

Since $h/t_w =$ 13.90 $< 2.24\sqrt{(E_S/F_{yw})}$

$\Phi V_n=$ $\Phi 0.6F_yA_wC_v$
= 581 k

$V_u=$ 62.7 k OK

The W14X257 is adequate to resist the loads given.

SMF Beam Design

INPUT YOUR DATA:

General:			Loads:		
SDC	C		M_D	-183	k-ft
ρ	1.0		M_L	-92	k-ft
S_{DS}	0.283		M_{QE}	-417	k-ft
$0.2*S_{DS}$	0.057		V_D	41	k
E_s	29000	ksi	V_L	18	k
F_y	50	ksi	V_{QE}	34	k
W24x94			M_A	233	k-ft
d	24.3	in	M_B	33.1	k-ft
Z_x	254	in^3	M_C	-184	k-ft
t_w	0.515	in	M_{midG}	33.1	k-ft
t_f	0.875	in	M_{midlat}	0	k-ft
r_y	1.98	in	M_{endlat}	417	k-ft
b_f	9.07	in	M_{qtptG}	24.8	k-ft
S_x	222	in^3	$M_{qtptlat}$	208.5	k-ft
h_o	23.64	in			
R_m	1.0				

Determine the factored loads--

$M_u = (1.2+0.2S_{DS})M_D+\rho M_{QE}+0.5M_L+0.2M_S)$ = -693 k-ft

$V_u = (1.2+0.2S_{DS})V_D+\rho V_{QE}+0.5V_L+0.2V_S)$ = 57.6 k

Check beam element slenderness--

Width-Thickness Ratio for Flanges $\lambda_f = b_f/(2t_f)$

= 5.18

Flange Compactness $\lambda_{ps} = 0.3\sqrt{(E_S/F_y)}$

= 7.22

$\lambda_f < \lambda_{ps}$

Flanges are seismically compact.

Width-Thickness Ratio for Web $\lambda_w = h/(t_w)$

= 43.8

Web Compactness $\lambda_{ps} = 2.45\sqrt{(E_S/F_y)}$

= 59.0

$\lambda_w < \lambda_{ps}$

Web is seismically compact.

Check lateral bracing requirements--

Per seismic provisions §9.8, both flanges must be laterally braced at intervals not to exceed:

$0.086r_y(E/F_y)=$	98.8	in
=	8.23	ft
=	8	ft

The diaphragm provides continuous lateral support to the top flange of the beam. However, the only lateral supports for the bottom flange occur at the end connections. Therefore, a bottom flange brace must be provided every 8 feet.

Provide a bottom flange brace every 8 feet.

Check unbraced length-- (using Table 3-2)

L_p	**6.99**	ft	
L_r	**21.2**	ft	
L_b	8.23	ft	
		$L_p<L_b<L_r$	OK

Determine the flexural strength at the full cross section--

$M_p = F_yZ_x$

$= 1058.33$ k-ft

$C_b = 12.5M_{max}R_m/(2.5M_{max}+3M_A+4M_B+3M_C)$

$= 4.30$

$M_n = C_b[M_p-(M_p-0.7F_yS_x(L_b-L_p)/(L_r-L_p)] \leq M_p$

$= 4398.57$ k-ft

$M_n = 1058.33$ k-ft

At the centerline of the reduced beam section:

$Z_e = Z_x-2ct_f(d-t_f)$

$= 167$ in^3

Comparing the reduced and unreduced section flexural strengths,

$M_{pr} < M_n$ OK

$\Phi_bM_{pr} = \Phi F_yZ_e$

$= 695$ k-ft

$Mu = 693$ k-ft

$M_u < \Phi_bM_{pr}$ OK

Check shear strength--

$2.24\sqrt{(E/F_{yw})} = 53.9$

Since $h/t_w < 2.24\sqrt{(E/F_y)}$ OK

$\Phi V_n = \Phi 0.6FyAwCv$

$=$ 375 k

$V_u < \Phi V_n$ OK

The W24x94 is adequate to resist the loads given.

Design lateral bracing--

Per seismic provisions §9.8, the required strength of the nodal lateral bracing away from an expected plastic hinge is:

$P_{br} = 0.02 M_r C_d / h_o$

$M_r = R_y F_y Z$

$=$ 13970 k-in

$=$ 1164.17 k-ft

$P_{br} =$ 11.5 k

The length of the brace is assumed to extend from the centerline of the bottom flange of the W24x94 BM to the CL of the top flange of the adjacent gravity beam. Beam spacing is 8'-4", so the length of the brace is,

$L = \sqrt{(spacing^2 + d_b^2)}$

$=$ 108 in

$=$ 8.98 ft

From Manual Table 4-12 for eccentrically loaded single angles with eccentricity equal to or less than 0.75 times the angle thickness, try a L5x5x5/16 with $K_z = 1.0$

$\Phi P_n =$ 17.5 k

$P_{br} =$ 11.5 k

$P_{br} < \Phi P_n$ OK

Seismic Provisions §9.8 also specifies a minimum stiffness for lateral bracing. Assuming a rigid brace support, the required brace stiffness is,

$\beta_{br} = 10 M_u C_d / (\Phi L_b h_o)$ =required brace stiffness

$\Phi =$ 0.75

$M_u = R_y F_y Z$

$=$ 13970 k-in

$C_d =$ 1.0

$L_b =$ 98.7624 in

$h_o =$ 23.64 in

$\beta_{br} =$ 79.7804 k/in

$k = A_g E / L \cos^2(\theta)$

$\theta = \tan^{-1}(d_b / spacing)$

$=$ 13.0305 °

$k =$ 651.946 k/in

$k > \beta_{br}$ OK

Use L5x5x5/16 kickers to brace the beam bottom flange at a maximum spacing of 8 feet.

SMF Beam to Column Connection Design

INPUT YOUR DATA:

Loads:			W24x94		
w_D	3.00	k/ft	d	24.3	in
w_L	1.80	k/ft	Z_x	254	in^3
W14x:257			t_w	0.515	in
d	16.4	in	t_f	0.875	in
t_w	1.18	in	b_f	9.07	in
t_f	1.89	in	R_y	1.1	
b_f	16.0	in	F_y	50	ksi
A_g	75.6	in^2	F_u	65	ksi
Z_x	487	in^3	a	5.5	in
k_{det}	3.1875	in	b	18	in

Determine the probable moment at the plastic hinge--

M_{pr}= $C_{pr}R_yF_yZ_e$

C_{pr}= $(F_y+F_u)/(2F_y) \le 1.2$

= 1.15

M_{pr}= 10556 k-in

Compute the expected shear force at the plastic hinge--

Required shear strength at the plastic hinge:

V_{RBS}= $2(M_{pr}/L')+V_{gravity}$

Factored uniform gravity load:

w_u= $1.2w_D+0.5w_L+0.2w_S$

= 4.50 k/ft

The distance from the column face to the assumed plastic hinge location is:

S_h= $a+b/2$

= 14.5 in

The distance between plastic hinges is:

L'= $L-2d_c/2-2S_h$

= 375 in

= 31.2 ft

Required shear strength at the plastic hinge due to gravity:

V_{gRBS}= $1/2w_uL'$

= 70.2 k

Expected shear at the plastic hinge:

V_{RBS}= $2M_{pr}/L'+V_{gRBS}$

= 126.6 k

V'_{RBS}= $2(-M_{pr})/L'+V_{gRBS}$

= 13.9 k

Compute the probable maximum moment at the column face--

Factored moment due to gravity load between the column flange and plastic hinge is:

$M_g = 1/2 w_u S_h^2$

$= 39.4$ k-in

Maximum probable moment at the face of the column is:

$M_f = M_{pr} + V_{RBS} S_h + M_g$

$= 12431$ k-in

$M'_f = -M_{pr} + V_{RBS} S_h + M_g$

$= -10315$ k-in

Compare M_f to M_{pe} at the column face--

Expected moment strength of the unreduced beam section at the column face is:

$M_{pe} = R_y F_y Z$

$= 13970$ k-in

$\Phi_d M_{pe} = 13970$ k-in $\Phi = 1.0$

$M_f = 12431$ k-in

$M_f \leq \Phi_d M_{pe}$ OK

Check column-beam moment ratio--

Strong column-weak beam criterion:

$\sum M_{pc}^* / \sum M_{pb}^* > 1.0$

$\sum M_{pc}^* = \sum [Z_c (F_y - P_{uc}/A_g)]$

$= 40895$ k-in

The expected flexural demand of the beam at the column centerline is:

$\sum M_v = (V_{RBS} + V_{RBS}')(a + b/2 + d_c/2)$

$= 2559$ k-in

$\sum M_{pb}^* = \sum (M_{pr} + M_v)$

$= 26229$ k-in

$\sum M_{pc}^* / \sum M_{pb}^* > 1.0$

$= 1.56 > 1.0$ OK

Strong column-weak beam criterion is met.

Check column panel zone strength--

Seismic Provisions § 9.3a requires panel zone shear strength to be calculated by summing the moments at the column faces:

$P_{uf} = R_u = \sum M_f / (d_b - t_f)$

$= 765$ k

$0.75 P_c = 0.75 F_y A_g =$

$= 2835$ k

Since

$P_r = 606$ k $< 0.75 P_c$, the design shear strength of the panel zone is defined as:

$\Phi R_n = \Phi 0.6F_y d_c t_w[1+(3b_{cf}t_{cf}^2)/d_b d_c t_w)]$
$= 792$ k

$R_u > \Phi R_n$

Web doubler plate not required.

To prevent seismic shear buckling in the panel-zone:

$t_{wmin} = (d_b+d_c-2t_f)/90$
$= 0.401$ in
$t_w = 1.18$ in

$t_w > t_{wmin}$ OK

Per Seismic Provisions § 9.2, the minimum thickness of each component of the panel zone, without the aid of intermediate plug welds between column web and the doubler is:

$d_z = d_b-2t_{bf}$
$= 22.6$ in
$w_z = d_c-2t_{cf}$
$= 12.6$ in
$t \geq (d_z+w_z)/90$
≥ 0.391 in
$t_w = 1.18 > 0.391$ OK

Determine the need for transverse stiffeners--

$P_y = F_y A$
$= 3780$ k
$P_u/P_y = 0.141$

Since this ratio is < 0.4,

$\Phi R_v = 0.9(0.6)F_y d_c t_w$
$= 871$ k
$P_{uf} = 765$ k

$\Phi R_v < P_{uf}$

OK

Determine the design strength of the flange and web to resist flange forces in tension:

For local flange bending:

$\Phi R_n = .9(6.25)t_f^2 F_y C_t$
$= 1005$ k
$P_{uf} = 765$ k

For local web yielding:

$\Phi R_n = 1.0[C_t(5k_{det})+N]F_y t_w$
$= 992$ k

P_{uf}= 765 k

OK, the web of the column can resist the tensile flange force

Determine the design strength of the web to resist flange forces in compression:

ΦR_n= 1005 k $<P_{uf}$

For web crippling,

N_d= $3N/d_c$

= 0.160

ΦR_n= $.75(135)C_t t_w^2[1+N_d(t_w/t_f)^{1.5}]\sqrt{(F_y t_f/t_w)}$

= 1361 k

P_{uf}= 765 k

OK, the web of the column can resist the compressive flange force

Panel-zone web shear and tensile/compressive flange forces can be resisted without reinforcement, therefore, transverse stiffeners are not needed.

Determine the need for continuity plates--

Unless this exception is met, continuity plates will be required:

$t_{cf} \geq$ $0.4\sqrt{(1.8 b_{bf} t b_f (F_{yb} R_{yb})/(F_{yc} R_{yc}))}$

$\geq$ 1.51 in

t_{cf}= 1.89 in

$t_{cf} \geq$ $b_{bf}/6$

$\geq$ 1.51 in

Minimum thickness requirements are met, continuity plates not required.

Design beam flange to column flange connection--

Per AISC 358 § 5.5, use a complete-joint-penetration groove weld

Factored shear force at the column face:

V_u= $2M_{pr}/L'+Vg_{ravity}$

= $V_{RBS}+w_u S_h$

132.0 k

Select a single plate connection with plate thickness 3/8" to support erection loads.

With the 3/8" single plate as backing, use a CJP groove weld to connect the beam web to the column flange.

Check beam web strength--

Minimum remaining web depth between weld access holes for the shear force is:

d_{min}= $V_u/(\Phi 0.6 F_y t_w)$ in

= 8.55

By inspection, a greater web depth remains. OK

SMF Column Splice Design

INPUT YOUR DATA:

General:			Loading on the upper shaft between 2^{nd} and 3^{rd} level is:		
SDC	C		P_D	140	k
ρ	1.0		P_L	29	k
S_{DS}	0.283		P_{Lr}	0	k
$0.2{*}S_{DS}$	0.057		P_{QE}	13	k
F_y	50	ksi	V_D	5	k
W14x176:			V_L	3	k
d	15.2	in	V_{Lr}	0	k
b_f	15.7	in	V_{QE}	45	k
t_f	1.31	in	M_{xDtop}	-31	k-ft
t_w	0.830	in	M_{xLtop}	-20	k-ft
A	51.8	in^2	M_{xLrtop}	0	k-ft
Z_x	320	in^3	M_{xQEtop}	126	k-ft
L_{top}	14	ft	M_{xDbot}	29	k-ft
W14x257:			M_{xLbot}	15	k-ft
Z_x	487	in^3	M_{xLrbot}	0	k-ft
			M_{xQEbot}	230	k-ft

Assume that there is no tranverse loading between the column supports in the plane of bending and that the connections into the column weak axis produce negligable moments on the column.

Determine the required axial strength of the splice--

$P_u = (1.2+0.2S_{DS})P_D+\rho P_{QE}+0.5P_L+0.2P_S$

203 k

Maximum tensile force in the column:

$T_u = (0.9-0.2S_{DS})P_D+\rho P_{QE}+1.6P_H$

105 k

According to ASCE 7-05, since $T_u>0$, there is no net uplift on the column. Therefore the requirements of Seismic Provisions § 8.4 do not apply. From Figure 4-10, the unbraced length of the column is 15 feet. Using Manual Table 4-1 with K=1.0,

$\Phi_cP_n=$ 2010 k

$P_u/\Phi_cP_n=$ 0.101

$T_u/\Phi_cP_n=$ 0.052

Since P_u/Φ_cP_n and T_u/Φ_cP_n are both < 0.4, Sesimic Provisions § 8.4 does not require consideration of amplified seismic loads in the design of the column. Therefore, the axial load for which the columnn is required to be designed is:

$P_u =$ 203 k

Determine the required flexural strength of the splice--

Per seismic Provisions § 9.9, the required flexural strength of the splice is equal to $R_yF_yZ_x$ of the smaller shaft or can be made with CJP groove welds.

Use CJP groove welds to splice the column webs and flanges.

Determine the required shear strength of the web splice--

From Manual Table 3-2,

$L_p =$ 14.2 ft

$\Phi M_p =$ 1200 k-ft

Per seismic Provisions § 9.9, the required shear strength of the web splice is:

$V_u = \sum M_{pc}/H$

$=$ 240 k

To develop this force through shear YIELDING of the web, the required web depth is:

$d_w = (V_u/(\Phi 0.6F_yt_w)$

9.65 in

Therefore, the maximum length of each weld access hole in the direction of the web is:

$l_{max,accesshole} = 1/2(d-2t_f-d_w)$

1.47 in

The access holes for the flange splice welds may not extend more than 1-1/2" measured perpendicular to the inside flange surface.

Check location of splice--

Per Seismic Provisions § 9.9, it is required that splices be located per § 8.4a. The height between the second and third levels is 14' and the column splices are shown to be located 48" above the finished floor elevation. The clear distance between the beam to column connections is approximately 11 feet. With a clear distance > 8 feet, § 8.4a requires that the splice be located a minimum of 4' from the beam to column connection.

The splice location is acceptable.

Connection Design for Alternative 1 *(Provided to show stiffener/doubler plate design, beam/column design and all calculations for Alternative 2 available upon request)*

SMF Beam to Column Connection Design

INPUT YOUR DATA:

Loads:			W24x94		
w_D	3.00	k/ft	d	24.3	in
w_L	1.80	k/ft	Z_x	254	in^3
W14x:211			t_w	0.515	in
d	15.7	in	t_f	0.875	in
t_w	0.98	in	b_f	9.07	in
t_f	1.56	in	R_y	1.1	
b_f	15.8	in	F_y	50	ksi
A_g	62.0	in^2	F_u	65	ksi
Z_x	390	in^3	a	5.5	in
k_{det}	2.875	in	b	18	in

Determine the probable moment at the plastic hinge--

$M_{pr}=$ $C_{pr}R_yF_yZ_e$

$C_{pr}=$ $(F_y+F_u)/(2F_y) \leq 1.2$

$=$ 1.15

$M_{pr}=$ 10556 k-in

Compute the expected shear force at the plastic hinge--

Required shear strength at the plastic hinge:

$V_{RBS}=$ $2(M_{pr}/L')+V_{gravity}$

Factored uniform gravity load:

$w_u=$ $1.2w_D+0.5w_L+0.2w_S$

$=$ 4.50 k/ft

The distance from the column face to the assumed plastic hinge location is:

$S_h=$ $a+b/2$

$=$ 14.5 in

The distance between plastic hinges is:

$L'=$ $L-2d_c/2-2S_h$

$=$ 375 in

$=$ 31.3 ft

Required shear strength at the plastic hinge due to gravity:

$V_{gRBS}=$ $1/2w_uL'$

$=$ 70.4 k

Expected shear at the plastic hinge:

V_{RBS}= $2M_{pr}/L'+V_{gRBS}$
= 126.6 k

V'_{RBS}= $2(-M_{pr})/L'+V_{gRBS}$
= 14.1 k

Compute the probable maximum moment at the column face--

Factored moment due to gravity load between the column flange and plastic hinge is:

M_g= $1/2 w_u S_h^2$
= 39.4 k-in

Maximum probable moment at the face of the column is:

M_f= $M_{pr}+V_{RBS}S_h+M_g$
= 12431 k-in

M'_f= $-M_{pr}+V_{RBS}S_h+M_g$
= -10312 k-in

Compare M_f to M_{pe} at the column face--

Expected moment strength of the unreduced beam section at the column face is:

M_{pe}= R_yF_yZ
= 13970 k-in

$\Phi_d M_{pe}$= 13970 k-in Φ=1.0

M_f = 12431 k-in

$M_f \leq \Phi_d M_{pe}$ OK

Check column-beam moment ratio--

Strong column-weak beam criterion:

$\sum M_{pc}^*/\sum M_{pb}^* > 1.0$

$\sum M_{pc}^*$= $\sum[Z_c(F_y-P_{uc}/A_g)]$
= 31423 k-in

The expected flexural demand of the beam at the column centerline is:

$\sum M_v$= $(V_{RBS}+V_{RBS}')(a+b/2+d_c/2)$
= 2514 k-in

$\sum M_{pb}^*$= $\sum(M_{pr}+M_v)$
= 26140 k-in

$\sum M_{pc}^*/\sum M_{pb}^* > 1.0$
= 1.20 > 1.0 OK

Strong column-weak beam criterion is met.

Check column panel zone strength--

Seismic Provisions § 9.3a requires panel zone shear strength to be calculated by summing the moments at the column faces:

$P_{uf}=R_u=$ $\sum M_f/(d_b-t_f)$
$=$ 765 k

$0.75P_c = 0.75F_yA_g=$
$=$ 2325 k

Since
$P_r=$ 602 k < $0.75P_c$, the design shear strength of the panel zone is defined as:

$\Phi R_n=$ $\Phi 0.6F_yd_ct_w[1+(3b_{cf}t_{cf}^2)/d_bd_ct_w)]$
$=$ 604 k

$R_u > \Phi R_n$

Web doubler plate is required.

To prevent seismic shear buckling in the panel-zone:

$t_{wmin}=$ $(d_b+d_c-2t_f)/90$
$=$ 0.400 in

$t_w=$ 0.98 in

$t_w > t_{wmin}$ OK

Per Seismic Provisions § 9.2, the minimum thickness of each component of the panel zone, without the aid of intermediate plug welds between column web and the doubler is:

$d_z=$ d_b-2t_{bf}
$=$ 22.6 in

$w_z=$ d_c-2t_{cf}
$=$ 12.6 in

$t \geq$ $(d_z+w_z)/90$
$\geq$ 0.390 in

$t_w=$ 0.98 > 0.390 OK

Determine the need for transverse stiffeners--

$P_y=$ F_yA
$=$ 3100 k

$P_u/P_y=$ 0.172

Since this ratio is < 0.4,

$\Phi R_v=$ $0.9(0.6)F_yd_ct_w$
$=$ 692 k

$P_{uf}=$ 765 k

$\Phi R_v < P_{uf}$

Column web is inadequate to resist panel-zone web shear without reinforcement.

Determine the design strength of the flange and web to resist flange forces in tension:
For local flange bending:

$\Phi R_n =$ $.9(6.25)t_f^2 F_y C_t$
$=$ 684 k
$P_{uf} =$ 765 k

For local web yielding:

$\Phi R_n =$ $1.0[C_t(5k_{det})+N]F_y t_w$
$=$ 747 k
$P_{uf} =$ 765 k

The flange and web of the column are inadequate to resist tensile flange force without reinforcement.

Determine the design strength of the web to resist flange forces in compression:

$\Phi R_n =$ 684 k $<P_{uf}$

For web crippling,

$N_d =$ $3N/d_c$
$=$ 0.167
$\Phi R_n =$ $.75(135)C_t t_w^2[1+N_d(t_w/t_f)^{1.5}]\sqrt{(F_y t_f/t_w)}$
$=$ 940 k
$P_{uf} =$ 765 k

OK, the web of the column can resist the compressive flange force

Panel-zone web shear and tensile flange forces cannot be resisted without reinforcement; therefore, transverse stiffeners are needed.

Calculate the transverse stiffener forces and web doubler plate shear force--

If Rust is negative, transverse stiffening is not required

$R_{ust} =$ $P_{uf} - \Phi R_{nmin}$
81 k

The required strength of the web doubler plate is:

$V_{udp} =$ $V_u - \Phi R_{vcw}$
$=$ 73 k

Unbalanced load in 1 transverse stiffener= 40.5 k

Therefore, the panel zone web shear force is more critical.

Design the web doubler plate and its associated welding:

$t_{pmin} \geq$ $V_u d_p/(0.9(0.6)F_y d_c)$
$\geq$ 0.17 in

Check t_{min} required to facilitate fillet welded joint detail between doubler plate and column flange

$t_{pmin} =$ $k - t_f - r_e$
$=$ 1.0 in

Doubler plate width and depth are selected based on the dimensions of the panel-zone and the edge details:

Transverse to the axis of the column, the doubler plate dimension is selected equal to the clear distance between the col flanges	=	12-9/16	in
Parallel to the axis of the column, the web doubler plate dimension is selected equal to the beam depth plus two times $2.5k_{det}$	=	31-1/2	in

Use PL 1" x 12-9/16" x 31-1/2".

$w_{min}=$ $1.70F_y t_{eff}/F_{exx} \geq t_{eff}\sqrt{2}$

1.23 ≥ 1.44

(See Seismic Provisions § 9.3c (pg. 6.1-32) weld can be CJP or fillet; shall be welded across top and bottom edges.

Use a CJP weld to connect the web doubler plate to the column web.

Design the transverse stiffeners and their associated welding--

$A_{stmin}=$ $R_{ust}/\Phi F_{yst}$

2.50 in^2

$b_{smin}=$ $b/3-t_{pz}/2$

2.53 in

$t_{smin}=$ $t/2 \geq b_s\sqrt{(F_{yst})}/95$

= 0.438 ≥ 0.189

Transverse stiffeners are required to match the configuration used in the qualifying cyclic tests. From Engelhardt et al., a pair of 1" x 5" stiffeners at each flange is adequate.

$t_{stiffener}=$ 1 in

$b_{stiffener}=$ 5 in

clips= 3/4 in

$A_{stiffener}=$ 4.50 in^2

$A_{stiffener} > A_{stmin}$ OK

The length of the transverse stiffeners is selected equal to the depth of the column minus two times the thickness of the column flange:

$l=$ d_c-2t_f

= 12 5/8 in

Check the shear strength of the transverse stiffener to transmit the unbalanced force in the stiffener to the column panel-zone:

$(R_{ust})_1+(R_{ust})_2 = (P_{uf}-\Phi R_{nmin})_1+(P_{uf}-\Phi R_{nmin})_2$

= 18 k

$t_s \geq$ $[(R_{ust})_1+(R_{ust})_2]/[0.9*0.6F_{yst}(l-2clip)*2]$

≥ 0.04 in

$t_{stiffener}$= 1 in OK

Use 2 PL 1" x 5" x 12-5/8" with two 3/4" x 3/4" corner clips each at each flange plate.

Complete-joint-penetration groove welds are used to connect the transverse stiffeners to the column flanges.

Use 1" CJP groove welds to connect the transverse stiffeners to the column flange.

For the double sided fillet welds connecting the transverse stiffeners to the web doubler plate:
For the limit state based on the strength of the transverse stiffener ends in tension,

ΦR_{nmax}= $0.9F_{yst}(4)(b_s\text{-clip})t_s$
= 551 k

For the limit state based on shear in the transverse stiffeners,
ΦR_{nmax}= $0.9(0.6)F_{yst}(l\text{-}2clip)(2t_s)$
= 431 k ***<-- Governs***

For the limit state based on shear in the column web, (one shear plane used because the force must be transmitted into the panel-zone)
ΦR_{nmax}= $0.9(0.6)F_y d_c t_{pz}$
= 839 k
R_{ust}= 431 k

w ≥ $R_{ust}/(0.75(0.6)F_{exx}(\text{length-2clip})(2)\sqrt{2})$
≥ 0.436 in
≥ 7/16 in
The minimum fillet weld size per Table J2.4 is 5/16 in.

Use 7/16" double-sided fillet welds to connect the transverse stiffeners to the doubler plate.

Determine the need for continuity plates--
Unless this exception is met, continuity plates will be required:
t_{cf} ≥ $0.4\sqrt{(1.8b_{bf}tb_f(F_{yb}R_{yb})/(F_{yc}R_{yc}))}$
≥ 1.51 in
t_{cf}= 1.56 in
t_{cf} ≥ $b_{bf}/6$
≥ 1.51 in
Minimum thickness requirements are met, continuity plates not required.

Design beam flange to column flange connection--
Per AISC 358 § 5.5, use a complete-joint-penetration groove weld
Factored shear force at the column face:

$V_u = \quad 2M_{pr}/L' + Vg_{ravity}$

$= \quad V_{RBS} + w_u S_h$

132.1 k

Select a single plate connection with plate thickness 3/8" to support erection loads.

With the 3/8" single plate as backing, use a CJP groove weld to connect the beam web to the column flange.

Check beam web strength--

Minimum remaining web depth between weld access holes for the shear force is:

$d_{min} = \quad V_u/(\Phi 0.6 F_y t_w)$ in

$= \quad 8.55$

By inspection, a greater web depth remains. OK

-End of Section-

APPENDIX E: *Geopier Element Supplementary Material*

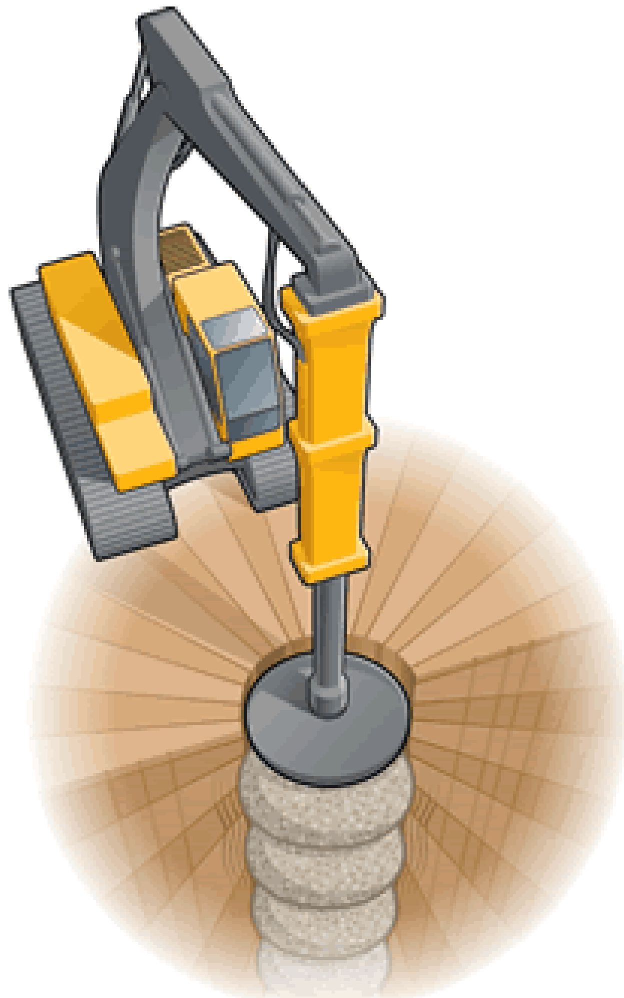
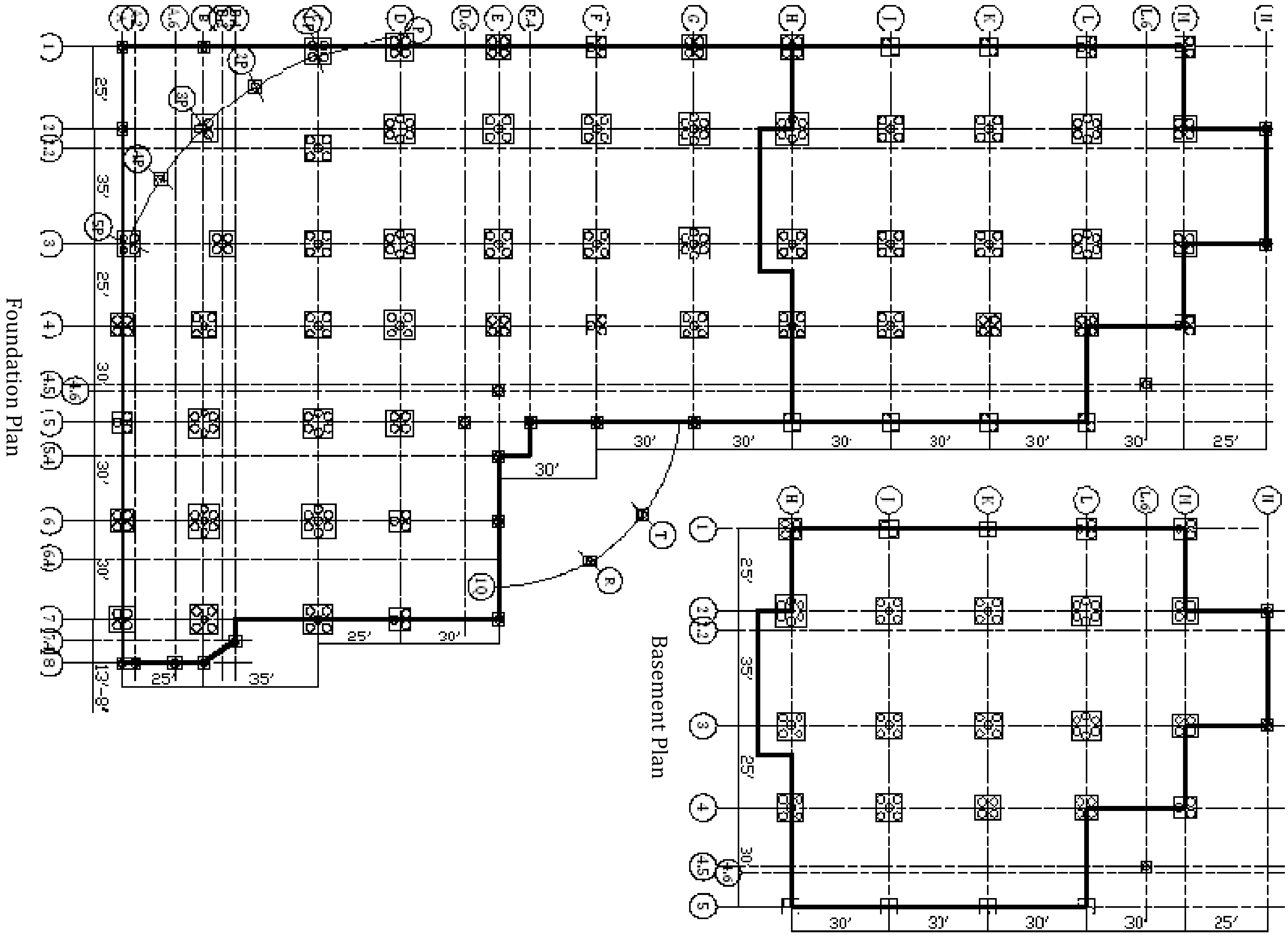


Image courtesy of www.geostructures.com


Foundation Plan
Basement Plan

GeoStructures

geopier™

Project: St. Vincent Hospital

Location: Ohio

Owner/GC: Penn State Student

GSI Proj. Number:

Total # of GPs: 290

Avg. GP Length, ft: 11.0

Issues/Revisions

Date	By	Remarks
2/16/2009	SMO	Preliminary Design

TABLE 1 SUMMARY OF SETTLEMENT CALCULATIONS

Notes	Ftg. Loc.	Spread Ftg. Mark	Column Load (kips) Dead DL	Live LL	Transient Load (100% LL)	Total TL	Ftg. Width B (ft)	Ftg. Length L (ft)	Ftg. Stress q_o (ksf)	GP Cap. Q_{gp} (kips)	Est'd GPs	Actual GPs GP#	GP Dia. d_{gp} (in)	GP Depth Z_{gp} (ft)	GP Mod. K_{gp} (pci)	Soil Mod. K_m (pci)	Stress Ratio H_s	Area Ratio H_a	GP Stress q_{gp} (ksf)	Soil Stress q_m (ksf)	Settlement TABLE2 UZ S_{UZ} (in)	LZ S_{LZ} (in)	Total S_{total} (in)
												OK		OK				OK	max = 76 kips			OK	OK
								max = 6.097 ksf			SCR min = 1.03			Hs max = 16'				0.40	max = 15.5 ksf			0.37 in	0.95 in
														DDmax = 26'									
	1-A		13.95			14	3.50	3.50	1.139	90	0.16	1	30	8	175	14	12.5	0.40	2.538	0.203	0.10		0.10
	2-A		19.57			20	3.50	3.50	1.598	90	0.22	1	30	8	175	14	12.5	0.40	3.561	0.285	0.14		0.14
	3-A		14.17			14	3.50	3.50	1.157	90	0.16	1	30	8	175	14	12.5	0.40	2.578	0.206	0.10		0.10
	4-A		177.81	28.46	124.15	330	7.50	7.50	5.874	90	3.67	4	30	10	175	14	12.5	0.35	14.644	1.171	0.58	0.25	0.83
	5-A		203.25	31.74	12.50	247	6.50	6.50	5.858	90	2.75	3	30	10	175	14	12.5	0.35	14.620	1.170	0.58	0.13	0.71
	6-A		209.42	76.36	12.50	298	7.00	7.00	6.087	90	3.31	4	30	10	175	14	12.5	0.40	13.568	1.085	0.54	0.19	0.73
	7-A		146.74	81.54	124.15	352	8.00	8.00	5.507	90	3.92	4	30	10	175	14	12.5	0.31	15.201	1.216	0.60	0.29	0.90
	8-A		41.77	12.93		55	3.50	3.50	4.465	90	0.61	1	30	8	175	14	12.5	0.40	9.953	0.796	0.39		0.39
	8-A.2		15.74	12.54		28	3.50	3.50	2.309	90	0.31	1	30	8	175	14	12.5	0.40	5.146	0.412	0.20		0.20
	1-B		19.84			20	3.50	3.50	1.620	90	0.22	1	30	8	175	14	12.5	0.40	3.610	0.289	0.14		0.14
	2-B		225.88	40.21		266	8.00	8.00	4.158	90	2.96	3	30	10	175	14	12.5	0.23	14.254	1.140	0.57	0.22	0.79
	3-B.2		268.55	56.86		325	8.00	8.00	5.085	90	3.62	4	30	10	175	14	12.5	0.31	14.036	1.123	0.56	0.27	0.83
	4-B		315.48	63.83		379	8.00	8.00	5.927	90	4.21	5	30	10	175	14	12.5	0.38	13.693	1.095	0.54	0.32	0.86
	5-B		356.16	70.33	70.78	497	9.50	9.50	5.510	90	5.53	6	30	12	175	14	12.5	0.33	14.491	1.159	0.58	0.34	0.92
	6-B		406.97	186.52		593	10.00	10.00	5.935	90	6.59	7	30	13	175	14	12.5	0.34	14.982	1.199	0.59	0.36	0.95
	7-B		226.69	133.98	60.80	421	9.00	9.00	5.203	90	4.68	5	30	11	175	14	12.5	0.30	14.503	1.160	0.58	0.33	0.91
	8-B		54.58	22.05		77	4.00	4.00	4.789	90	0.85	1	30	10	175	14	12.5	0.31	13.221	1.058	0.52		0.52
	1-C		13.82			14	3.50	3.50	1.128	90	0.15	1	30	8	175	14	12.5	0.40	2.515	0.201	0.10		0.10
	2.2-C		313.99	64.18		378	8.00	8.00	5.909	90	4.20	5	30	10	175	14	12.5	0.38	13.652	1.092	0.54	0.32	0.86
	3-C		308.97	70.45		379	8.00	8.00	5.928	90	4.22	5	30	10	175	14	12.5	0.38	13.697	1.096	0.54	0.32	0.86
	4-C		319.75	66.68		386	8.00	8.00	6.038	90	4.29	5	30	10	175	14	12.5	0.38	13.950	1.116	0.55	0.32	0.88
	5-C		363.68	70.34	52.45	486	9.00	9.00	6.006	90	5.41	6	30	12	175	14	12.5	0.36	14.489	1.159	0.57	0.30	0.88
	6-C		422.85	143.60		566	10.00	10.00	5.665	90	6.29	7	30	13	175	14	12.5	0.34	14.300	1.144	0.57	0.34	0.91
	7-C		281.24	79.88	46.54	408	8.50	8.50	5.642	90	4.53	5	30	10	175	14	12.5	0.34	14.374	1.150	0.57	0.37	0.94
	1-D		177.16	29.05	142.45	349	8.00	8.00	5.448	90	3.87	4	30	10	175	14	12.5	0.31	15.039	1.203	0.60	0.29	0.89
	2-D		333.44	64.39	72.70	471	9.00	9.00	5.809	90	5.23	6	30	11	175	14	12.5	0.36	14.014	1.121	0.56	0.37	0.92
	3-D		324.96	66.55	72.70	464	9.00	9.00	5.731	90	5.16	6	30	11	175	14	12.5	0.36	13.826	1.106	0.55	0.36	0.91
	4-D		256.71	42.62	142.45	442	9.00	9.00	5.454	90	4.91	5	30	11	175	14	12.5	0.30	15.202	1.216	0.60	0.35	0.95
	5-D		198.44	31.97	123.23	354	8.00	8.00	5.526	90	3.93	4	30	10	175	14	12.5	0.31	15.254	1.220	0.61	0.30	0.90
	6-D		193.44	31.97		225	6.50	6.50	5.335	90	2.50	3	30	10	175	14	12.5	0.35	13.316	1.065	0.53	0.12	0.65
	7-D		130.40	19.11	107.34	257	6.50	6.50	6.079	90	2.85	3	30	10	175	14	12.5	0.35	15.173	1.214	0.60	0.14	0.74

Notes	Ftg. Loc.	Spread Ftg. Mark	Column Load (kips) Dead DL	Live LL	Transient Load (100% LL)	Total TL	Ftg. Width B (ft)	Ftg. Length L (ft)	Ftg. Stress q_o (ksf)	GP Cap. Q_{gp} (kips)	Est'd GPs	Actual GPs GP#	GP Dia. d_{gp} (in)	GP Depth z_{gp} (ft)	GP Mod. K_{gp} (pci)	Soil Mod. K_m (pci)	Stress Ratio R_s	Area Ratio R_a	GP Stress q_{gp} (ksf)	Soil Stress q_m (ksf)	Settlement TABLE2 UZ S_{UZ} (in)	LZ S_{LZ} (in)	Total S_{total} (in)
												OK		OK				OK	max = 76 kips			OK	OK
								max = 6.097 ksf			SCR min =1.03			Hs max =16'				0.40	max = 15.5 ksf			0.37 in	0.95 in
	1-E		197.77	31.97	93.28	323	7.50	7.50	5.743	90	3.59	4	30	10	175	14	12.5	0.35	14.316	1.145	0.57	0.24	0.81
	2-E		375.60	73.80		449	9.00	9.00	5.548	90	4.99	5	30	12	175	14	12.5	0.30	15.464	1.237	0.61	0.28	0.89
	3-E		361.47	73.80		435	8.50	8.50	6.024	90	4.84	5	30	11	175	14	12.5	0.34	15.348	1.228	0.61	0.31	0.92
	4-E		199.22	31.97	87.52	319	7.50	7.50	5.666	90	3.54	4	30	10	175	14	12.5	0.35	14.125	1.130	0.56	0.24	0.80
	4.6-E		10.01			10	3.50	3.50	0.817	90	0.11	1	30	8	175	14	12.5	0.40	1.821	0.146	0.07		0.07
	5.4-E		10.27			10	3.50	3.50	0.838	90	0.11	1	30	8	175	14	12.5	0.40	1.869	0.149	0.07		0.07
	6-E		33.42			33	3.50	3.50	2.728	90	0.37	1	30	8	175	14	12.5	0.40	6.081	0.486	0.24		0.24
	7-E		23.28			23	3.50	3.50	1.900	90	0.26	1	30	8	175	14	12.5	0.40	4.236	0.339	0.17		0.17
	5-E.4		10.44			10	3.50	3.50	0.852	90	0.12	1	30	8	175	14	12.5	0.40	1.900	0.152	0.08		0.08
	1-F		202.76	31.97		235	6.50	6.50	5.556	90	2.61	3	30	10	175	14	12.5	0.35	13.866	1.109	0.55	0.12	0.67
	2-F		374.33	73.80		448	9.00	9.00	5.532	90	4.98	5	30	12	175	14	12.5	0.30	15.421	1.234	0.61	0.28	0.89
	3-F		361.27	73.80		435	8.50	8.50	6.022	90	4.83	5	30	11	175	14	12.5	0.34	15.341	1.227	0.61	0.31	0.92
	4-F		207.97	31.97		240	6.50	6.50	5.679	90	2.67	3	30	10	175	14	12.5	0.35	14.174	1.134	0.56	0.13	0.69
	5-F		33.81			34	3.50	3.50	2.760	90	0.38	1	30	8	175	14	12.5	0.40	6.152	0.492	0.24		0.24
	1-G		197.77	31.97	93.28	323	7.50	7.50	5.743	90	3.59	4	30	10	175	14	12.5	0.35	14.316	1.145	0.57	0.24	0.81
	2-G		426.46	148.32		575	10.00	10.00	5.748	90	6.39	7	30	13	175	14	12.5	0.34	14.510	1.161	0.58	0.35	0.92
	3-G		405.43	188.25		594	10.00	10.00	5.937	90	6.60	7	30	13	175	14	12.5	0.34	14.987	1.199	0.59	0.36	0.95
	4-G		234.50	75.61	87.52	398	8.50	8.50	5.504	90	4.42	5	30	10	175	14	12.5	0.34	14.021	1.122	0.56	0.36	0.92
	5-G		54.64	2.30		57	3.50	3.50	4.648	90	0.63	1	30	8	175	14	12.5	0.40	10.360	0.829	0.41		0.41
	1-H		161.53	23.57	97.84	283	7.00	7.00	5.774	90	3.14	4	30	10	175	14	12.5	0.40	12.870	1.030	0.51	0.18	0.69
	2-H		345.24	107.87	42.41	496	9.50	9.50	5.491	90	5.51	6	30	12	175	14	12.5	0.33	14.440	1.155	0.57	0.34	0.91
	3-H		267.72	98.95	42.41	409	8.50	8.50	5.662	90	4.55	5	30	10	175	14	12.5	0.34	14.424	1.154	0.57	0.37	0.94
	4-H		220.15	72.21	97.84	390	8.00	8.00	6.097	90	4.34	5	30	10	175	14	12.5	0.38	14.087	1.127	0.56	0.33	0.88
	5-H		109.56	12.69		122	5.00	5.00	4.890	90	1.36	2	30	8	175	14	12.5	0.39	11.081	0.887	0.44	0.07	0.51
	1-J		109.77	14.34	48.65	173	5.50	5.50	5.711	90	1.92	2	30	10	175	14	12.5	0.32	15.085	1.207	0.60	0.04	0.64
	2-J		316.88	70.25		387	8.00	8.00	6.049	90	4.30	5	30	15	175	14	12.5	0.38	13.976	1.118	0.55	0.04	0.59
	3-J		317.02	68.65		386	8.00	8.00	6.026	90	4.29	5	30	15	175	14	12.5	0.38	13.923	1.114	0.55	0.04	0.59

Notes	Ftg. Loc.	Spread Ftg. Mark	Column Load (kips) Dead DL	Live LL	Transient Load (100% LL)	Total TL	Ftg. Width B (ft)	Ftg. Length L (ft)	Ftg. Stress q_o (ksf)	GP Cap. Q_{gp} (kips)	Est'd GPs	Actual GPs GP#	GP Dia. d_{gp} (in)	GP Depth z_{gp} (ft)	GP Mod. K_{gp} (pci)	Soil Mod. K_m (pci)	Stress Ratio R_s	Area Ratio R_a	GP Stress q_{gp} (ksf)	Soil Stress q_m (ksf)	Settlement TABLE2 UZ S_{UZ} (in)	LZ S_{LZ} (in)	Total S_{total} (in)
								max = 6.097 ksf			SCR min = 1.03	OK		OK Hs max = 16'				OK 0.40	max = 76 kips max = 15.5 ksf			OK 0.37 in	OK 0.95 in
	4-J		273.88	55.95	49.02	379	8.00	8.00	5.920	90	4.21	5	30	15	175	14	12.5	0.38	13.677	1.094	0.54	0.04	0.58
	5-J		104.77	14.88		120	5.00	5.00	4.786	90	1.33	2	30	8	175	14	12.5	0.39	10.846	0.868	0.43	0.07	0.50
	1-K		110.97	14.34		125	5.00	5.00	5.012	90	1.39	2	30	8	175	14	12.5	0.39	11.359	0.909	0.45	0.07	0.52
	2-K		316.89	67.13		384	8.00	8.00	6.000	90	4.27	5	30	15	175	14	12.5	0.38	13.863	1.109	0.55	0.04	0.59
	3-K		317.46	67.88		385	8.00	8.00	6.021	90	4.28	5	30	15	175	14	12.5	0.38	13.911	1.113	0.55	0.04	0.59
	4-K		278.22	54.86		333	7.50	7.50	5.921	90	3.70	4	30	16	175	14	12.5	0.35	14.761	1.181	0.59		0.59
	5-K		108.59	14.88		123	5.00	5.00	4.939	90	1.37	2	30	8	175	14	12.5	0.39	11.192	0.895	0.44	0.07	0.52
	1-L		152.62	25.54	48.65	227	6.50	6.50	5.368	90	2.52	3	30	9	175	14	12.5	0.35	13.398	1.072	0.53	0.17	0.71
	2-L		378.71	100.81		480	9.00	9.00	5.920	90	5.33	6	30	15	175	14	12.5	0.36	14.282	1.143	0.57	0.12	0.69
	3-L		379.65	94.77		474	9.00	9.00	5.857	90	5.27	6	30	15	175	14	12.5	0.36	14.130	1.130	0.56	0.12	0.68
	4-L		216.06	26.26	49.62	292	7.00	7.00	5.958	90	3.24	4	30	9	175	14	12.5	0.40	13.280	1.062	0.53	0.26	0.79
	4.5-L.6		41.48	6.33		48	3.50	3.50	3.903	90	0.53	1	30	8.0	175	14	12.5	0.40	8.699	0.696	0.35		0.35
	5-L		85.20	10.89		96	5.00	5.00	3.844	90	1.07	2	30	8.0	175	14	12.5	0.39	8.710	0.697	0.35	0.06	0.40
	1-M		121.49	21.23	41.54	184	6.50	6.50	4.361	90	2.05	3	30	8.0	175	14	12.5	0.35	10.885	0.871	0.43	0.20	0.63
	2-M		230.41	51.84	18.41	301	7.50	7.50	5.345	90	3.34	4	30	9.0	175	14	12.5	0.35	13.325	1.066	0.53	0.30	0.82
	3-M		230.34	51.84	18.41	301	7.50	7.50	5.344	90	3.34	4	30	9.0	175	14	12.5	0.35	13.322	1.066	0.53	0.29	0.82
	4-M		137.05	22.00	41.54	201	6.50	6.50	4.748	90	2.23	3	30	9.0	175	14	12.5	0.35	11.850	0.948	0.47	0.15	0.62
	2-N		29.26	13.25		43	3.50	3.50	3.470	90	0.47	1	30	8.0	175	14	12.5	0.40	7.735	0.619	0.31		0.31
	3-N		29.26	13.25		43	3.50	3.50	3.470	90	0.47	1	30	8.0	175	14	12.5	0.40	7.735	0.619	0.31		0.31
	1P-P		149.81	28.11		178	6.00	6.00	4.942	90	1.98	2	30	10.0	175	14	12.5	0.27	14.936	1.195	0.59	0.07	0.66
	2P-P		64.55	12.67		77	4.00	4.00	4.826	90	0.86	1	30	9.0	175	14	12.5	0.31	13.323	1.066	0.53		0.53
	3P-P		225.88	40.21		266	7.00	7.00	5.430	90	2.96	3	30	10.0	175	14	12.5	0.30	15.233	1.219	0.60	0.17	0.78
	4P-P		66.23	13.08		79	4.00	4.00	4.957	90	0.88	1	30	10.0	175	14	12.5	0.31	13.683	1.095	0.54		0.54
	5P-P		171.30	32.66		204	6.50	6.50	4.827	90	2.27	3	30	9.0	175	14	12.5	0.35	12.049	0.964	0.48	0.16	0.63
	T-1Q		17.74			18	3.50	3.50	1.448	90	0.20	1	30	8.0	175	14	12.5	0.40	3.228	0.258	0.13		0.13
	R-1Q		17.90			18	3.50	3.50	1.461	90	0.20	1	30	8.0	175	14	12.5	0.40	3.257	0.261	0.13		0.13

GeoStructures

geopier™

Project: St. Vincent Hospital

GSI Proj. Number:

Location: Ohio

Total # of GPs: 290

Owner/GC: Penn State Student

Avg. GP Length, ft: 11

Issues/Revisions		
Date	By	Remarks
3/16/2009	SMO	Preliminary Design

TABLE 2 LOWER ZONE SETTLEMENT CALCULATIONS

Notes	Ftg. Loc.	Ftg. Type	SOG FF Elev. (ft, msl)	Constr. Grade Elev. (ft, msl)	Ftg. Thk. (ft)	Ftg. Ref. Depth (ft)	BOF Elev. (ft,msl)	Bot GP Shaft Elev. (ft,msl)	GP Drill Depth (ft)	Equiv. Ftg. Width B' (ft)	Ftg. Stress q_o (ksf)	GP Length H_s (ft)	Lower Zone Thickness Z_{4B} (ft)	Z_{2B} (ft)	Z_{LZ} (ft)	Center f(B) (%)	Westergaard LZ Stress q/q_o (%)	q_{LZ} (ksf)	Avg. LZ Mod. E_{LZ} (tsf)	Lower Zone Settlement S_{LZ} (in)	Remarks
		1-A	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.139	8.0	-						75		no lower zone
		2-A	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.598	8.0	-						75		no lower zone
		3-A	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.157	8.0	-						75		no lower zone
		4-A	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.50	5.874	10.0	-	5.0	5.0	1.67	11%	0.617	75	0.25	
		5-A	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.50	5.858	10.0	-	3.0	3.0	1.77	9%	0.545	75	0.13	
		6-A	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.00	6.087	10.0	-	4.0	4.0	1.71	10%	0.600	75	0.19	
		7-A	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.507	10.0	-	6.0	6.0	1.63	11%	0.613	75	0.29	
		8-A	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	4.465	8.0	-						75		no lower zone
		8-A.2	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	2.309	8.0	-						75		no lower zone
		1-B	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.620	8.0	-						75		no lower zone
		2-B	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	4.158	10.0	-	6.0	6.0	1.63	11%	0.463	75	0.22	
		3-B.2	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.085	10.0	-	6.0	6.0	1.63	11%	0.566	75	0.27	
		4-B	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.927	10.0	-	6.0	6.0	1.63	11%	0.659	75	0.32	
		5-B	608.00	610.00	2.00	-1.00	605.00	593.00	17.00	9.50	5.510	12.0	-	7.0	7.0	1.63	11%	0.608	75	0.34	
		6-B	608.00	610.00	2.00	-1.00	605.00	592.00	18.00	10.00	5.935	13.0	-	7.0	7.0	1.65	11%	0.638	75	0.36	
		7-B	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	9.00	5.203	11.0	-	7.0	7.0	1.61	11%	0.590	75	0.33	
		8-B	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	4.00	4.789	10.0	-						75		no lower zone
		1-C	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.128	8.0	-						75		no lower zone
		2.2-C	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.909	10.0	-	6.0	6.0	1.63	11%	0.657	75	0.32	
		3-C	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.928	10.0	-	6.0	6.0	1.63	11%	0.660	75	0.32	
		4-C	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	6.038	10.0	-	6.0	6.0	1.63	11%	0.672	75	0.32	
		5-C	608.00	610.00	2.00	-1.00	605.00	593.00	17.00	9.00	6.006	12.0	-	6.0	6.0	1.67	11%	0.631	75	0.30	
		6-C	608.00	610.00	2.00	-1.00	605.00	592.00	18.00	10.00	5.665	13.0	-	7.0	7.0	1.65	11%	0.609	75	0.34	
		7-C	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.50	5.642	10.0	-	7.0	7.0	1.59	12%	0.659	75	0.37	
		1-D	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.448	10.0	-	6.0	6.0	1.63	11%	0.606	75	0.29	

Notes	Ftg. Loc.	Ftg. Type	SOG FF Elev. (ft, msl)	Constr. Grade Elev. (ft, msl)	Ftg. Thk. (ft)	Ftg. Ref. Depth (ft)	BOF Elev. (ft,msl)	Bot GP Shaft Elev. (ft,msl)	GP Drill Depth (ft)	Equiv. Ftg. Width B' (ft)	Ftg. Stress q_o (ksf)	GP Length H_s (ft)	Lower Zone Thickness Z_{4B} (ft)	Z_{2B} (ft)	Z_{LZ} (ft)	Center f(B) (%)	Westergaard LZ Stress q/q_o (%)	q_{LZ} (ksf)	Avg. LZ Mod. E_{LZ} (tsf)	Lower Zone Settlement S_{LZ} (in)	Remarks
		2-D	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	9.00	5.809	11.0	-	7.0	7.0	1.61	11%	0.658	75	0.37	
		3-D	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	9.00	5.731	11.0	-	7.0	7.0	1.61	11%	0.650	75	0.36	
		4-D	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	9.00	5.454	11.0	-	7.0	7.0	1.61	11%	0.618	75	0.35	
		5-D	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	5.526	10.0	-	6.0	6.0	1.63	11%	0.615	75	0.30	
		6-D	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.50	5.335	10.0	-	3.0	3.0	1.77	9%	0.497	75	0.12	
		7-D	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.50	6.079	10.0	-	3.0	3.0	1.77	9%	0.566	75	0.14	
		1-E	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.50	5.743	10.0	-	5.0	5.0	1.67	11%	0.603	75	0.24	
		2-E	608.00	610.00	2.00	-1.00	605.00	593.00	17.00	9.00	5.548	12.0	-	6.0	6.0	1.67	11%	0.583	75	0.28	
		3-E	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	8.50	6.024	11.0	-	6.0	6.0	1.65	11%	0.650	75	0.31	
		4-E	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.50	5.666	10.0	-	5.0	5.0	1.67	11%	0.595	75	0.24	
		4.6-E	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	0.817	8.0	-						75		no lower zone
		5.4-E	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	0.838	8.0	-						75		no lower zone
		6-E	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	2.728	8.0	-						75		no lower zone
		7-E	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.900	8.0	-						75		no lower zone
		5-E.4	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	0.852	8.0	-						75		no lower zone
		1-F	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.50	5.556	10.0	-	3.0	3.0	1.77	9%	0.517	75	0.12	
		2-F	608.00	610.00	2.00	-1.00	605.00	593.00	17.00	9.00	5.532	12.0	-	6.0	6.0	1.67	11%	0.581	75	0.28	
		3-F	608.00	610.00	2.00	-1.00	605.00	594.00	16.00	8.50	6.022	11.0	-	6.0	6.0	1.65	11%	0.650	75	0.31	
		4-F	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.50	5.679	10.0	-	3.0	3.0	1.77	9%	0.529	75	0.13	
		5-F	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	2.760	8.0	-						75		no lower zone
		1-G	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.50	5.743	10.0	-	5.0	5.0	1.67	11%	0.603	75	0.24	
		2-G	608.00	610.00	2.00	-1.00	605.00	592.00	18.00	10.00	5.748	13.0	-	7.0	7.0	1.65	11%	0.618	75	0.35	
		3-G	608.00	610.00	2.00	-1.00	605.00	592.00	18.00	10.00	5.937	13.0	-	7.0	7.0	1.65	11%	0.638	75	0.36	
		4-G	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.50	5.504	10.0	-	7.0	7.0	1.59	12%	0.643	75	0.36	
		5-G	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	4.648	8.0	-						75		no lower zone
		1-H	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.00	5.774	10.0	-	4.0	4.0	1.71	10%	0.569	75	0.18	
		2-H	608.00	610.00	2.00	-1.00	605.00	593.00	17.00	9.50	5.491	12.0	-	7.0	7.0	1.63	11%	0.605	75	0.34	
		3-H	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.50	5.662	10.0	-	7.0	7.0	1.59	12%	0.661	75	0.37	
		4-H	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	8.00	6.097	10.0	-	6.0	6.0	1.63	11%	0.678	75	0.33	
		5-H	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	5.00	4.890	8.0	-	2.0	2.0	1.80	9%	0.440	75	0.07	

Notes	Ftg. Loc.	Ftg. Type	SOG FF Elev. (ft, msl)	Constr. Grade Elev. (ft, msl)	Ftg. Thk. (ft)	Ftg. Ref. Depth (ft)	BOF Elev. (ft,msl)	Bot GP Shaft Elev. (ft,msl)	GP Drill Depth (ft)	Equiv. Ftg. Width B' (ft)	Ftg. Stress q_o (ksf)	GP Length H_s (ft)	Lower Zone Thickness Z_{4B} (ft)	Z_{2B} (ft)	Z_{LZ} (ft)	Center f(B) (%)	Westergaard LZ Stress q/q_o (%)	q_{LZ} (ksf)	Avg. LZ Mod. E_{LZ} (tsf)	Lower Zone Settlement S_{LZ} (in)	Remarks
		1-J	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	5.50	5.711	10.0	-	1.0	1.0	1.91	8%	0.465	75	0.04	
Basement		2-J	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	8.00	6.049	15.0	-	1.0	1.0	1.94	8%	0.480	75	0.04	
Basement		3-J	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	8.00	6.026	15.0	-	1.0	1.0	1.94	8%	0.478	75	0.04	
Basement		4-J	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	8.00	5.920	15.0	-	1.0	1.0	1.94	8%	0.470	75	0.04	
		5-J	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	5.00	4.786	8.0	-	2.0	2.0	1.80	9%	0.431	75	0.07	
		1-K	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	5.00	5.012	8.0	-	2.0	2.0	1.80	9%	0.451	75	0.07	
Basement		2-K	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	8.00	6.000	15.0	-	1.0	1.0	1.94	8%	0.476	75	0.04	
Basement		3-K	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	8.00	6.021	15.0	-	1.0	1.0	1.94	8%	0.478	75	0.04	
Basement		4-K	593.00	600.00	2.00	-1.00	590.00	574.00	26.00	7.50	5.921	16.0	-						75		no lower zone
		5-K	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	5.00	4.939	8.0	-	2.0	2.0	1.80	9%	0.444	75	0.07	
		1-L	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	6.50	5.368	9.0	-	4.0	4.0	1.69	10%	0.543	75	0.17	
Basement		2-L	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	9.00	5.920	15.0	-	3.0	3.0	1.83	9%	0.517	75	0.12	
Basement		3-L	593.00	600.00	2.00	-1.00	590.00	575.00	25.00	9.00	5.857	15.0	-	3.0	3.0	1.83	9%	0.512	75	0.12	
		4-L	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	7.00	5.958	9.0	-	5.0	5.0	1.64	11%	0.647	75	0.26	
		4.5-L.6	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	3.903	8.0	-						75		no lower zone
		5-L	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	5.00	3.844	8.0	-	2.0	2.0	1.80	9%	0.346	75	0.06	
		1-M	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	6.50	4.361	8.0	-	5.0	5.0	1.62	11%	0.491	75	0.20	
		2-M	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	7.50	5.345	9.0	-	6.0	6.0	1.60	12%	0.615	75	0.30	
		3-M	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	7.50	5.344	9.0	-	6.0	6.0	1.60	12%	0.615	75	0.29	
		4-M	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	6.50	4.748	9.0	-	4.0	4.0	1.69	10%	0.480	75	0.15	
		2-N	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	3.470	8.0	-						75		no lower zone
		3-N	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	3.470	8.0	-						75		no lower zone
		1P-P	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	6.00	4.942	10.0	-	2.0	2.0	1.83	9%	0.432	75	0.07	
		2P-P	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	4.00	4.826	9.0	-						75		no lower zone
		3P-P	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	7.00	5.430	10.0	-	4.0	4.0	1.71	10%	0.535	75	0.17	
		4P-P	608.00	610.00	2.00	-1.00	605.00	595.00	15.00	4.00	4.957	10.0	-						75		no lower zone
		5P-P	608.00	610.00	2.00	-1.00	605.00	596.00	14.00	6.50	4.827	9.0	-	4.0	4.0	1.69	10%	0.488	75	0.16	
		T-1Q	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.448	8.0	-						75		no lower zone
		R-1Q	608.00	610.00	2.00	-1.00	605.00	597.00	13.00	3.50	1.461	8.0	-						75		no lower zone

GeoStructures
Engineered Earth Structures & Foundations

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TABLE 3 GEOPIER ELEMENT SHAFT CAPACITY

Project:	St. Vincent Hospital	GSI Proj. Number:	
Location:	Ohio	Total GPs:	290
Owner/ GC:	Penn State Student	Avg. Length, ft:	11.0

Issues/Revisions

Date	By	Remarks
2/16/2009	SMO	Preliminary Design

Notes	Ftg. Loc.	Spread Ftg. Mark	Actual GPs GP#	GP Dia. d_{gp} (in)	Embed Depth (ft)	GP Depth Z_{gp} (ft)	GP Stress q_{gp} (ksf)	SHAFT CAPACITY RATIO Phi (deg.)	Tan (phi)	Kp	Total Unit Weight (pcf)	BOGP Elev (ft, msl)	BOF Elev (ft, msl)	FF Elev (ft, msl)	GWL Elev (ft, msl)	Average Effective Unit Wt. (pcf)	Critical Depth (ft)	SCR min = 1.03 Shaft Capacity Ratio	OK?
		1-A	1	30	3.00	8.00	2.538	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	4.65	OK
		2-A	1	30	3.00	8.00	3.561	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	3.32	OK
		3-A	1	30	3.00	8.00	2.578	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	4.58	OK
		4-A	4	30	3.00	10.00	14.644	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.07	OK
		5-A	3	30	3.00	10.00	14.620	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.07	OK
		6-A	4	30	3.00	10.00	13.568	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.16	OK
		7-A	4	30	3.00	10.00	15.201	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		8-A	1	30	3.00	8.00	9.953	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.19	OK
		8-A.2	1	30	3.00	8.00	5.146	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	2.30	OK
		1-B	1	30	3.00	8.00	3.610	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	3.27	OK
		2-B	3	30	3.00	10.00	14.254	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.10	OK
		3-B.2	4	30	3.00	10.00	14.036	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.12	OK
		4-B	5	30	3.00	10.00	13.693	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		5-B	6	30	3.00	12.00	14.491	26	0.49	2.56	110	593.00	605.00	608.00	590.00	110.00	8.87	1.35	OK
		6-B	7	30	3.00	13.00	14.982	26	0.49	2.56	110	592.00	605.00	608.00	590.00	110.00	8.87	1.44	OK
		7-B	5	30	3.00	11.00	14.503	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.22	OK
		8-B	1	30	3.00	10.00	13.221	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.19	OK

Notes	Ftg. Loc.	Spread Ftg. Mark	Actual GPs GP#	GP Dia. d_{gp} (in)	Embed Depth (ft)	GP Depth Z_{gp} (ft)	GP Stress q_{gp} (ksf)	SHAFT CAPACITY RATIO										SCR min = 1.03	
								Phi (deg.)	Tan (phi)	Kp	Total Unit Weight (pcf)	BOGP Elev (ft, msl)	BOF Elev (ft, msl)	FF Elev (ft, msl)	GWL Elev (ft, msl)	Average Effective Unit Wt. (pcf)	Critical Depth (ft)	Shaft Capacity Ratio	OK?
		1-C	1	30	3.00	8.00	2.515	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	4.70	OK
		2.2-C	5	30	3.00	10.00	13.652	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		3-C	5	30	3.00	10.00	13.697	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		4-C	5	30	3.00	10.00	13.950	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.13	OK
		5-C	6	30	3.00	12.00	14.489	26	0.49	2.56	110	593.00	605.00	608.00	590.00	110.00	8.87	1.35	OK
		6-C	7	30	3.00	13.00	14.300	26	0.49	2.56	110	592.00	605.00	608.00	590.00	110.00	8.87	1.51	OK
		7-C	5	30	3.00	10.00	14.374	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.09	OK
		1-D	4	30	3.00	10.00	15.039	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.05	OK
		2-D	6	30	3.00	11.00	14.014	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.26	OK
		3-D	6	30	3.00	11.00	13.826	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.28	OK
		4-D	5	30	3.00	11.00	15.202	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.16	OK
		5-D	4	30	3.00	10.00	15.254	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		6-D	3	30	3.00	10.00	13.316	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.18	OK
		7-D	3	30	3.00	10.00	15.173	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.04	OK
		1-E	4	30	3.00	10.00	14.316	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.10	OK
		2-E	5	30	3.00	12.00	15.464	26	0.49	2.56	110	593.00	605.00	608.00	590.00	110.00	8.87	1.27	OK
		3-E	5	30	3.00	11.00	15.348	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		4-E	4	30	3.00	10.00	14.125	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.11	OK
		4.6-E	1	30	3.00	8.00	1.821	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	6.49	OK
		5.4-E	1	30	3.00	8.00	1.869	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	6.32	OK
		6-E	1	30	3.00	8.00	6.081	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.94	OK
		7-E	1	30	3.00	8.00	4.236	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	2.79	OK
		5-E.4	1	30	3.00	8.00	1.900	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	6.22	OK

Notes	Ftg. Loc.	Spread Ftg. Mark	Actual GPs GP#	GP Dia. d_{gp} (in)	Embed Depth (ft)	GP Depth Z_{gp} (ft)	GP Stress q_{gp} (ksf)	SHAFT CAPACITY RATIO										SCR min = 1.03	
								Phi (deg.)	Tan (phi)	Kp	Total Unit Weight (pcf)	BOGP Elev (ft, msl)	BOF Elev (ft, msl)	FF Elev (ft, msl)	GWL Elev (ft, msl)	Average Effective Unit Wt. (pcf)	Critical Depth (ft)	Shaft Capacity Ratio	OK?
		6-E	1	30	3.00	8.00	6.081	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.94	OK
		7-E	1	30	3.00	8.00	4.236	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	2.79	OK
		5-E.4	1	30	3.00	8.00	1.900	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	6.22	OK
		1-F	3	30	3.00	10.00	13.866	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.13	OK
		2-F	5	30	3.00	12.00	15.421	26	0.49	2.56	110	593.00	605.00	608.00	590.00	110.00	8.87	1.27	OK
		3-F	5	30	3.00	11.00	15.341	26	0.49	2.56	110	594.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		4-F	3	30	3.00	10.00	14.174	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.11	OK
		5-F	1	30	3.00	8.00	6.152	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.92	OK
		1-G	4	30	3.00	10.00	14.316	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.10	OK
		2-G	7	30	3.00	13.00	14.510	26	0.49	2.56	110	592.00	605.00	608.00	590.00	110.00	8.87	1.49	OK
		3-G	7	30	3.00	13.00	14.987	26	0.49	2.56	110	592.00	605.00	608.00	590.00	110.00	8.87	1.44	OK
		4-G	5	30	3.00	10.00	14.021	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.12	OK
		5-G	1	30	3.00	8.00	10.360	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.14	OK
		1-H	4	30	3.00	10.00	12.870	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.22	OK
		2-H	6	30	3.00	12.00	14.440	26	0.49	2.56	110	593.00	605.00	608.00	590.00	110.00	8.87	1.36	OK
		3-H	5	30	3.00	10.00	14.424	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.09	OK
		4-H	5	30	3.00	10.00	14.087	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.12	OK
		5-H	2	30	3.00	8.00	11.081	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.07	OK
		1-J	2	30	3.00	10.00	15.085	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.04	OK
		2-J	5	30	3.00	15.00	13.976	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.05	OK
		3-J	5	30	3.00	15.00	13.923	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.05	OK
		4-J	5	30	3.00	15.00	13.677	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.07	OK
		5-J	2	30	3.00	8.00	10.846	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.09	OK
		1-K	2	30	3.00	8.00	11.359	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.04	OK
		2-K	5	30	3.00	15.00	13.863	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.06	OK
		3-K	5	30	3.00	15.00	13.911	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.06	OK

Notes	Ftg. Loc.	Spread Ftg. Mark	Actual GPs GP#	GP Dia. d_{gp} (in)	Embed Depth (ft)	GP Depth z_{gp} (ft)	GP Stress q_{gp} (ksf)	SHAFT CAPACITY RATIO										SCR min = 1.03	
								Phi (deg.)	Tan (phi)	Kp	Total Unit Weight (pcf)	BOGP Elev (ft, msl)	BOF Elev (ft, msl)	FF Elev (ft, msl)	GWL Elev (ft, msl)	Average Effective Unit Wt. (pcf)	Critical Depth (ft)	Shaft Capacity Ratio	OK?
		4-K	4	30	3.00	16.00	14.761	26	0.49	2.56	110	574.00	590.00	593.00	590.00	47.60	20.51	1.13	OK
		5-K	2	30	3.00	8.00	11.192	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.06	OK
		1-L	3	30	3.00	9.00	13.398	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		2-L	6	30	3.00	15.00	14.282	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.03	OK
		3-L	6	30	3.00	15.00	14.130	26	0.49	2.56	110	575.00	590.00	593.00	590.00	47.60	20.51	1.04	OK
		4-L	4	30	3.00	9.00	13.280	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.04	OK
		4.5-L.6	1	30	3.00	8.00	8.699	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.36	OK
		5-L	2	30	3.00	8.00	8.710	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.36	OK
		1-M	3	30	3.00	8.00	10.885	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.09	OK
		2-M	4	30	3.00	9.00	13.325	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		3-M	4	30	3.00	9.00	13.322	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		4-M	3	30	3.00	9.00	11.850	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.16	OK
		2-N	1	30	3.00	8.00	7.735	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.53	OK
		3-N	1	30	3.00	8.00	7.735	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	1.53	OK
		1P-P	2	30	3.00	10.00	14.936	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.05	OK
		2P-P	1	30	3.00	9.00	13.323	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		3P-P	3	30	3.00	10.00	15.233	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.03	OK
		4P-P	1	30	3.00	10.00	13.683	26	0.49	2.56	110	595.00	605.00	608.00	590.00	110.00	8.87	1.15	OK
		5P-P	3	30	3.00	9.00	12.049	26	0.49	2.56	110	596.00	605.00	608.00	590.00	110.00	8.87	1.14	OK
		T-1Q	1	30	3.00	8.00	3.228	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	3.66	OK
		R-1Q	1	30	3.00	8.00	3.257	26	0.49	2.56	110	597.00	605.00	608.00	590.00	110.00	8.87	3.63	OK

-End of Section-

APPENDIX F: *Façade Study Supplementary Material*

THERMAL GRADIENT CALCULATIONS

BRICK WALL: (1) BRICK
(2) 3/4" AIR SPACE
(3) CONTINUOUS MEMBRANE FLASHING
(4.) FULL HT. BATTING INSULATION
(5) 5/8" GYPSOM BOARD

(ALL R VALUES OBTAINED FROM MECHANICAL + ELECTRICAL EQUIPMENT FOR BUILDINGS)

BRICK:
- DIRECTION OF HEAT FLOW: HORIZ.
- MEAN TEMP = 0°F
- TEMP. DIFF. = 10°F
- THK = 3/4"
- E = 0.82

AIR SPACE:
- DIRECTION OF HEAT FLOW: HORIZ.
- MEAN TEMP = 0°F
- TEMP. DIFF. = 20°F
- THK = 3/4"
- E = 0.82

R = 1.32 FOR CONT. MEMBRANE FLASHING

BATT. INSUL.:
- DIRECTION OF HEAT FLOW = HORIZ.
- MEAN TEMP. = 50°F
- TEMP. DIFF. = 30°F
- THK = 5 5/8"
- E = 0.82

R = 0.56 FOR GYP. BOARD

R_o	0.17
R_1	0.11
R_2	1.26
R_3	1.32
R_4	19
R_5	0.56
R_i	0.68
ΣR	23.10
u	0.0433

$$T_x = T_o + (T_i - T_o)\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}$$

$$T_1 = 0 + (70 - 0)\left(\frac{0.28}{23.10}\right) = 0.848°F$$

$$T_2 = 70\left(\frac{1.54}{23.10}\right) = 4.67°F$$

$$T_3 = 70\left(\frac{2.86}{23.10}\right) = 8.67°F$$

$$T_4 = 70\left(\frac{21.86}{23.10}\right) = 66.2°F$$

$$T_5 = 70\left(\frac{22.54}{23.10}\right) = 68.3°F$$

- THERMAL GRADIENT —

BTW.	ΣR_{o-x}	TEMP (°F)
0–1	0.17	0
1–2	0.28	0.848
2–3	1.54	4.67
3–4	2.86	8.67
4–5	21.86	66.2
5–i	22.54	68.3
	23.10	70

CURTAINWALL: ① GLASS (R VALUE OBTAINED FROM MANUFACTURER)
② 1/2" AIR SPACE
③ GLASS

R= 2.63 FOR GLASS PANEL

AIR SPACE:
- DIRECTION OF HEAT FLOW: HORIZ.
- MEAN TEMP.: 0°F
- TEMP. DIFF.: 10°F
- THK = 1/2"
- E = 0.82
- R = 1.15

R_o	0.17
R_1	2.63
R_2	1.15
R_3	2.63
R_i	0.68
ΣR	7.09
U	0.141

$$T_x = T_o + (T_i - T_o)\frac{\Sigma R_{o-x}}{\Sigma R_{o-i}}$$

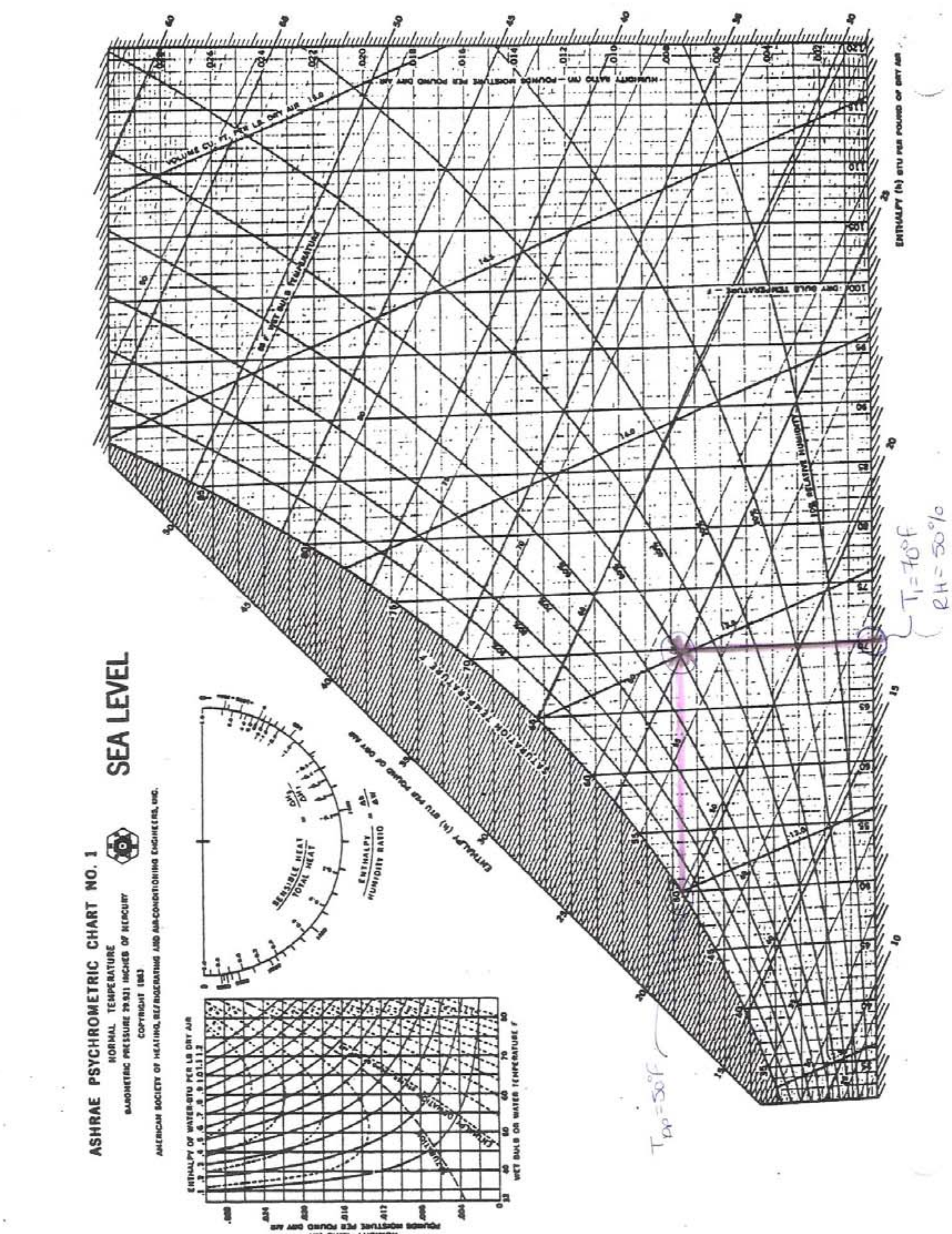
$$T_1 = 0 + (70-0)\left(\frac{2.80}{7.09}\right) = 27.6°F$$

$$T_2 = 70\left(\frac{3.95}{7.09}\right) = 40.0°F$$

$$T_3 = 70\left(\frac{6.58}{7.09}\right) = 65.0°F$$

• THERMAL GRADIENT —

BTW.	ΣR_{o-x}	TEMP (°F)
0-1	0.17	0
1-2	2.80	27.6
2-3	3.95	40.0
3-i	6.58	65.0
	7.09	70



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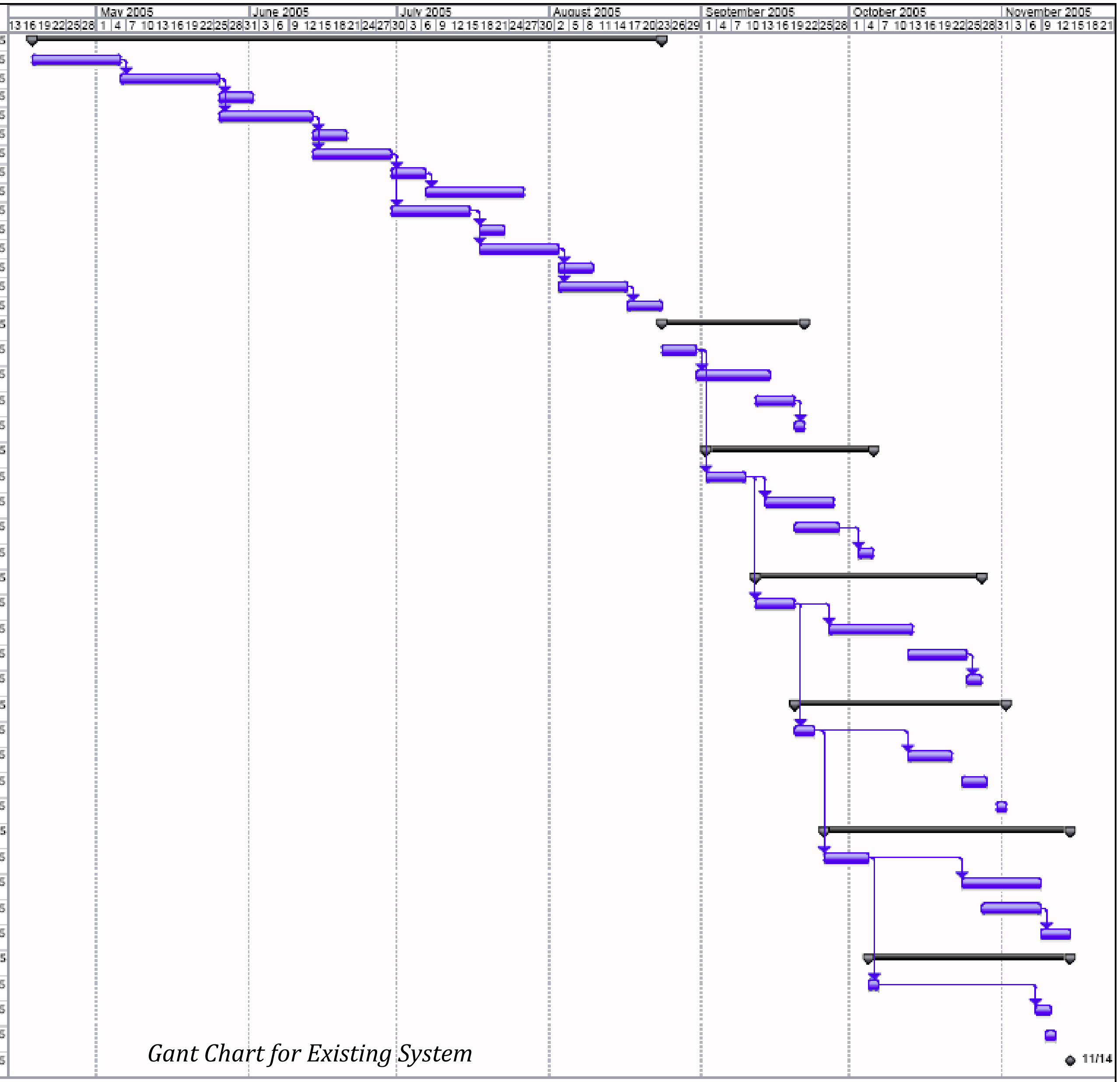
APPENDIX G: *Construction Management Supplementary Material*

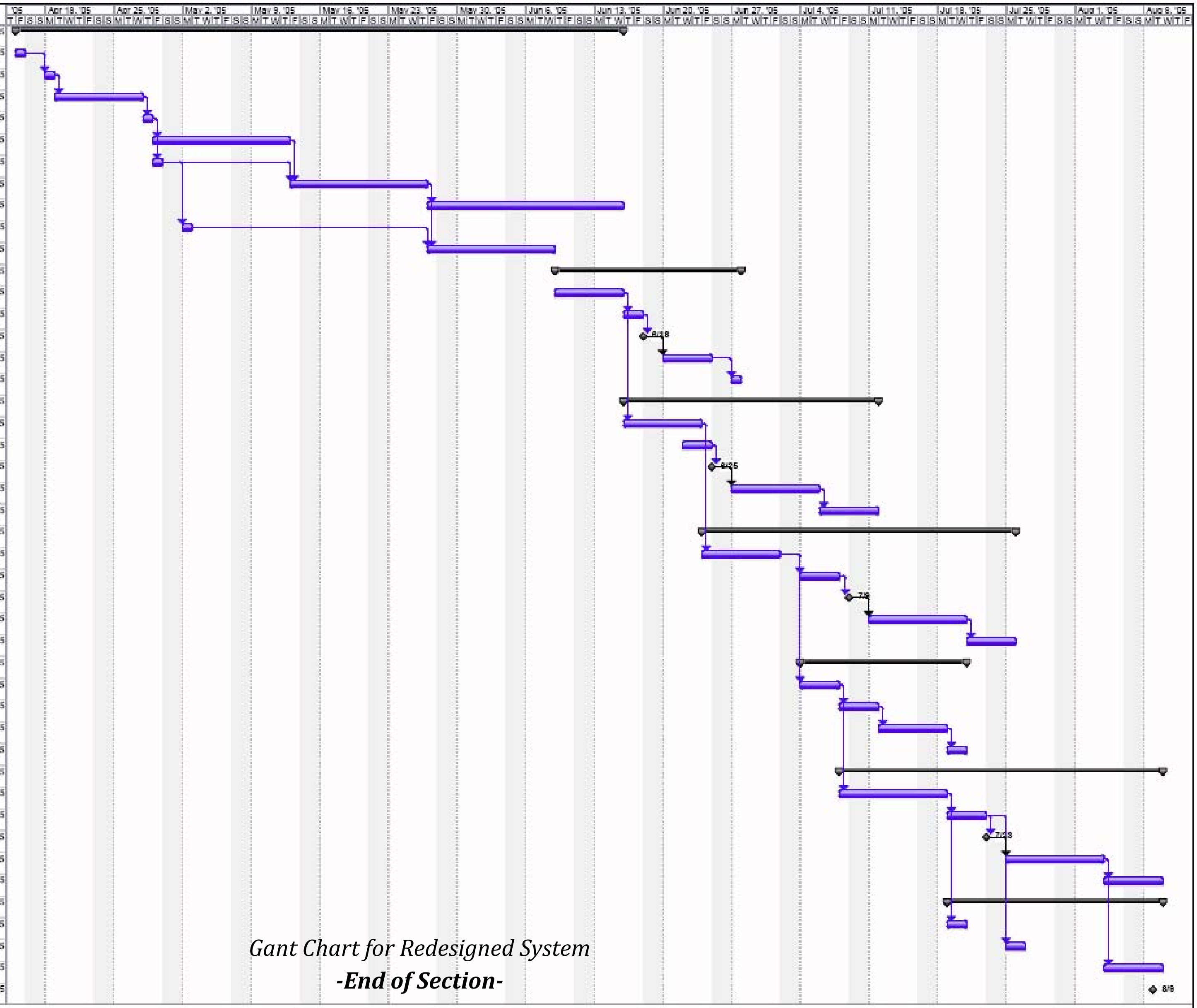
Photos courtesy of
Ruby + Associates

ID	Task Name	Duration	Start	Finish
1	**Foundations**	**92 days**	**Mon 4/18/05**	**Tue 8/23/05**
2	Excavate	14 days	Mon 4/18/05	Thu 5/5/05
3	Caissons- Seq. 1	14 days	Fri 5/6/05	Wed 5/25/05
4	SOG Seq. 1	5 days	Thu 5/26/05	Wed 6/1/05
5	Caissons- Seq. 2	13 days	Thu 5/26/05	Mon 6/13/05
6	SOG Seq. 2	5 days	Tue 6/14/05	Mon 6/20/05
7	Caissons- Seq. 3	12 days	Tue 6/14/05	Wed 6/29/05
8	SOG Seq. 3	5 days	Thu 6/30/05	Wed 7/6/05
9	Basement Walls	14 days	Thu 7/7/05	Tue 7/26/05
10	Caissons- Seq. 4	12 days	Thu 6/30/05	Fri 7/15/05
11	SOG Seq. 4	5 days	Mon 7/18/05	Fri 7/22/05
12	Caissons- Seq. 5	12 days	Mon 7/18/05	Tue 8/2/05
13	SOG Seq. 5	5 days	Wed 8/3/05	Tue 8/9/05
14	Caissons- Seq. 6	10 days	Wed 8/3/05	Tue 8/16/05
15	SOG Seg. 6	5 days	Wed 8/17/05	Tue 8/23/05
16	**Steel Sequence 1**	**21 days**	**Wed 8/24/05**	**Wed 9/21/05**
17	Erect	5 days	Wed 8/24/05	Tue 8/30/05
18	Detail	11 days	Wed 8/31/05	Wed 9/14/05
19	Deck	6 days	Mon 9/12/05	Mon 9/19/05
20	Studs	2 days	Tue 9/20/05	Wed 9/21/05
21	**Steel Sequence 2**	**24 days**	**Fri 9/2/05**	**Wed 10/5/05**
22	Erect	6 days	Fri 9/2/05	Fri 9/9/05
23	Detail	10 days	Wed 9/14/05	Tue 9/27/05
24	Deck	7 days	Tue 9/20/05	Wed 9/28/05
25	Studs	3 days	Mon 10/3/05	Wed 10/5/05
26	**Steel Sequence 3**	**34 days**	**Mon 9/12/05**	**Thu 10/27/05**
27	Erect	6 days	Mon 9/12/05	Mon 9/19/05
28	Detail	13 days	Tue 9/27/05	Thu 10/13/05
29	Deck	8 days	Thu 10/13/05	Mon 10/24/05
30	Studs	3 days	Tue 10/25/05	Thu 10/27/05
31	**Steel Sequence 4**	**31 days**	**Tue 9/20/05**	**Tue 11/1/05**
32	Erect	4 days	Tue 9/20/05	Fri 9/23/05
33	Detail	7 days	Thu 10/13/05	Fri 10/21/05
34	Deck	5 days	Mon 10/24/05	Fri 10/28/05
35	Studs	2 days	Mon 10/31/05	Tue 11/1/05
36	**Steel Sequence 5**	**36 days**	**Mon 9/26/05**	**Mon 11/14/05**
37	Erect	7 days	Mon 9/26/05	Tue 10/4/05
38	Detail	12 days	Mon 10/24/05	Tue 11/8/05
39	Deck	8 days	Fri 10/28/05	Tue 11/8/05
40	Studs	4 days	Wed 11/9/05	Mon 11/14/05
41	**Steel Sequence 6**	**29 days**	**Wed 10/5/05**	**Mon 11/14/05**
42	Erect	2 days	Wed 10/5/05	Thu 10/6/05
43	Detail	3 days	Tue 11/8/05	Thu 11/10/05
44	Deck	2 days	Thu 11/10/05	Fri 11/11/05
45	No Studs	0 days	Mon 11/14/05	Mon 11/14/05



Gant Chart for Existing System

ID		Task Name	Duration	Start	Finish
1		Foundations	44 days	Fri 4/15/05	Wed 6/15/05
2		Install Test Pier	1 day	Fri 4/15/05	Fri 4/15/05
3		Test Pier	1 day	Mon 4/18/05	Mon 4/18/05
4		Excavate	7 days	Tue 4/19/05	Wed 4/27/05
5		Install Geopiers- Seq. 1, 2	1 day	Thu 4/28/05	Thu 4/28/05
6		Place SOG- Seq.1, 2	10 days	Fri 4/29/05	Thu 5/12/05
7		Install Geopiers- Seq. 3, 4	1 day	Fri 4/29/05	Fri 4/29/05
8		Place SOG- Seq. 3, 4	10 days	Fri 5/13/05	Thu 5/26/05
9		Basement Walls	14 days	Fri 5/27/05	Wed 6/15/05
10		Install Geopiers- Seq. 5, 6	1 day	Mon 5/2/05	Mon 5/2/05
11		Place SOG- Seq. 5, 6	9 days	Fri 5/27/05	Wed 6/8/05
12		Steel Sequence 1	13 days	Thu 6/9/05	Mon 6/27/05
13		Erect	5 days	Thu 6/9/05	Wed 6/15/05
14		Detail	2 days	Thu 6/16/05	Fri 6/17/05
15		U.T. Testing	0 days	Sat 6/18/05	Sat 6/18/05
16		Deck	5 days	Mon 6/20/05	Fri 6/24/05
17		Studs	1 day	Mon 6/27/05	Mon 6/27/05
18		Steel Sequence 2	18 days	Thu 6/16/05	Mon 7/11/05
19		Erect	6 days	Thu 6/16/05	Thu 6/23/05
20		Detail	3 days	Wed 6/22/05	Fri 6/24/05
21		U.T. Testing	0 days	Sat 6/25/05	Sat 6/25/05
22		Deck	7 days	Mon 6/27/05	Tue 7/5/05
23		Studs	4 days	Wed 7/6/05	Mon 7/11/05
24		Steel Sequence 3	22 days	Fri 6/24/05	Mon 7/25/05
25		Erect	6 days	Fri 6/24/05	Fri 7/1/05
26		Detail	4 days	Mon 7/4/05	Thu 7/7/05
27		U.T. Testing	0 days	Sat 7/9/05	Sat 7/9/05
28		Deck	8 days	Mon 7/11/05	Wed 7/20/05
29		Studs	3 days	Thu 7/21/05	Mon 7/25/05
30		Steel Sequence 4	13 days	Mon 7/4/05	Wed 7/20/05
31		Erect	4 days	Mon 7/4/05	Thu 7/7/05
32		Detail	2 days	Fri 7/8/05	Mon 7/11/05
33		Deck	5 days	Tue 7/12/05	Mon 7/18/05
34		Studs	2 days	Tue 7/19/05	Wed 7/20/05
35		Steel Sequence 5	23 days	Fri 7/8/05	Tue 8/9/05
36		Erect	7 days	Fri 7/8/05	Mon 7/18/05
37		Detail	4 days	Tue 7/19/05	Fri 7/22/05
38		U.T. Testing	0 days	Sat 7/23/05	Sat 7/23/05
39		Deck	8 days	Mon 7/25/05	Wed 8/3/05
40		Studs	4 days	Thu 8/4/05	Tue 8/9/05
41		Steel Sequence 6	16 days	Tue 7/19/05	Tue 8/9/05
42		Erect	2 days	Tue 7/19/05	Wed 7/20/05
43		Detail	2 days	Mon 7/25/05	Tue 7/26/05
44		Deck	4 days	Thu 8/4/05	Tue 8/9/05
45		No studs	0 days	Tue 8/9/05	Tue 8/9/05



Gant Chart for Redesigned System

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