Kevin Wigton – Structural Option

AE Senior Thesis Final Report: Expansion Design Scenario Study

Simmons College School of Management, Boston, Ma

Advisor: Professor Parfitt
4/7/2010

Simmons College School of Management
Boston, Massachusetts

Project Team

Owner:	Simmons College
CM:	Lee Kennedy
Architect:	Cannon Design
Engineer:	Cannon Design
Geotech:	McPhail Associates

Building Statistics

Size:	65,000 SF
Height:	81'-10"
Cost:	$63 Million Total Cost
Delivery Method:	CM at Risk, Design, Bid, Build
Construction:	Summer 2006 - Winter 2007

Architecture

Five story educational facility
Below grade parking
New courtyard space created
Limestone, Glass, Aluminum Facade

Sustainability

Project Achieved LEED Gold Rating
Reduced Energy usage by 38%
Reduced water usage by 34%
Total 40 Green Points Achieved

Structure

Post Tensioned sub grade parking
3'-0" Slurry wall retaining system
Structural Steel framing above grade
Composite action floor system
Steel brace frame lateral system

Mechanical/Electrical

9000 CFM Dedicated Outside Air system
Radiant Ceiling Beams on all Levels
Power supplied by 15KV Campus Loop

Kevin Wigton Structural

www.engr.psu.edu/ae/thesis/portfolios/2010/ktw5007

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Acknowledgements

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Executive Summary

The Simmons College School of Management is a newly constructed five story educational facility located in Boston, Massachusetts. The building is 65,000 SF and sits on the south east corner of a five level below grade parking garage. Accommodations have been made in the original design for a future expansion of the building which would top out at nine stories.

Structural allowances for the expansion included increased gravity load carrying capacity through select areas of the superstructure as well as the below grade parking garage. The existing plans indicate that the expansion will occur primarily on the west end of the building. During the initial design, the expansion was considered to be separated from the existing building with a building expansion joint, or to be isolated a distance away from the base building.

Design information was not available for the exact layout of the expansion. Only the allotments for additional gravity loads as well as the plan and height restrictions. Therefore, the layout of the expansion studied in this report was treated as complying within the plan dimensions and load allotments made for the original designers' expansion program.

Through this study, a structurally tied expansion scenario was developed with proposed revisions to the existing lateral system. It was determined that with minimal revisions to the structure, a solution can be reached that satisfies the requirements of both the existing and expanded building layouts. There are several benefits to considering a structurally tied expansion as an initial design scenario. This alternative allows the owner of the building an additional option as their facility needs change. With respect to structure, there will be no need for additional lateral force resisting elements within the new layout. Using a structurally tied system will increase the architectural freedom in the expansion areas by not requiring space for braced frame lateral elements. There is also the potential for a lower comparative cost by excluding the material and labor costs associated with additional moment frames.

A façade study was conducted to ensure that the new design parameters could be accommodated in the revised system. Detailing requirements for typical glazing sizes in the building façade were revised as a result of increased seismic drift. Additionally, alterations to the west wall façade connection were proposed to allow for the expansion to be tied into the existing structure.

A study of the expansion constructability was also conducted. It was determined through this evaluation that the expansion would be able to be constructed with few additional accommodations. Ultimately the additional expansion design scenario was determined to be feasible. It is therefore concluded that with some additional system evaluation during the design process, an alternative expansion option could be provided for the building owners to consider.

Introduction

The Simmons College School of Management is a newly completed five story educational facility located on the Simmons College campus in Boston, Massachusetts. The $63 million building which was completed in December of 2008 was designed by Cannon Design.

As part of the project a five level below grade parking structure was provided to replace the parking lot that previously occupied the site. This relocation of parking allowed for the creation of a new green space quad to serve the school.

When the building was completed it achieved the LEED Gold rating by the USGBC. The project received 40 LEED points which included recognition for significant reductions in water and energy usage.

The project includes design considerations for a future building expansion. This design parameter was considered from the beginning of the design process including the original geotechnical evaluation of the site. Specific information regarding the layout for the building expansion was not available during this study. The expansion of the building will be discussed in greater detail in the following sections.

Architecture

Summary

The Simmons College School of Management building is a five story educational facility with an additional five levels of sub grade parking. Vehicles access the building under its southwest corner and enter into a centrally planned garage. Two way and one-way traffic patterns are utilized to access the 147 parking spaces per floor. The parking garage transitions to the building at the plaza level. Here, much of the 222 foot square garage is covered by the landscaped quad to the north of the building. The superstructure is positioned on the southeast corner of the garage. Primary pedestrian access to the building is from the quad into the main lobby area. Interior spaces include classrooms, offices, and administrative areas. A green roof patio overlooking the quad is accessible from the fifth floor. A curving metal screen hides mechanical units on the roof. See Figure One for Simmons College north façade.

Figure 1 North Façade Elevation

Building Façade

The facade of the Simmons College School of Management utilizes a combination of different materials to develop its architectural aesthetic. Curtain wall system veneers are hung from steel angles with metal stud backups for the exterior wall. A limestone veneer is used for the light colored façade elements with a brick veneer used as the darker infill at floors two through four. An aluminum curtain wall system with a kaynar finish and insulating glass is used for the fifth floor and the main entrance. The glazing system uses a combination of tempered insulating vision glass, and an insulating vision glass unit that is tempered and 100% fritted. Granite bases and aluminum flashing are used throughout the building. For a view of the building materials see Figure One above.

Roofing

On the roof of the building membrane roofing with tapered insulation is used to form the weather barrier. Built up from the concrete roof decking is a vapor barrier, tapered rigid insulation, dens-deck board, and a top layer of membrane sheathing.

Sustainability Features

The design of the Simmons College School of Management building was evaluated by the US Green Building Council according to the LEED rating system. The building obtained all of the prerequisite requirements as well as all 40 credits that were attempted during design to obtain a LEED Gold rating. Reductions in energy usage and potable water consumption were among the design features of the building.

Building Systems

Mechanical System

The cooling loads for the Simmons College School of Management are approximately 165 tons and are handled by the existing campus chilled water loop. The necessary upgrades to the chilled water plant were made prior to the completion of the building project. Spaces are serviced by terminal fan coil units and chilled radiant ceiling panels. The heating loads of the building are met by the circulation hot water through the building to fan coil units serviced by the campus central boiler plant.

A single 9000 CFM dedicated outside air, air handling unit will service the ventilation demands of the building. The below grade parking garage is controlled by a carbon monoxide monitoring system and is ventilated but not heated.

Electrical and Lighting Systems

Main electoral service is provided by the existing 15kV campus loop. Electrical systems in the building, similar to the structural systems, are designed for the current load with considerations for future building expansion. Equipment dedicated for electrical distribution is located in electrical closets on each individual floor. The 480V provides service for lighting with 208Y/120 volts, 3 phase, servicing lower voltage needs such as wall receptacles.

The lighting system is designed to integrate daylighting as well as sensing controls to increase the energy efficiency of the building. Sensors are connected to the HVAC system as well to maximize energy efficiency. Interior spaces use a combination of fluorescent lighting, halogen, and LED systems. Typical interior fluorescent lighting is provided by T5 and T8 lamps with electronic ballasts. Cold temperature areas are serviced by T12 high output lamps with magnetic ballasts.

Special Construction Considerations

The excavation of the parking garage was executed in a top down construction method. Construction of the slurry wall and installation of interior column and load bearing element foundations were done prior to the main excavation of the sight. Post tension slabs when installed provide the lateral support for the slurry wall as excavation continued. Special considerations were made for a crane to be located at the plaza level of the sight during the construction of the above grade levels.

Existing Structural System Description

Foundations

The below grade parking structure was constructed by the top down method with the installation of a slurry wall and load bearing elements (LBE) prior to excavation. Slurry wall panels have varying widths ranging from 10'-0" to 25'-0" with the typical panel width being 24'-0". Penetration of the 10'-0" centerbite into marine sands on site ranges from 1'-0" to 43'-0" depending on the bearing capacity demands of the wall section. See Figure 4 for typical slurry wall panel elevation.

Load bearing elements are constructed with W14 columns from the garage embedded in concrete shafts. Depths of the concrete shafts are divided into four categories summarized in Figure 2. W14 column embedment into the concrete shafts ranges from 16' to 27'. Typical shear studs are 4" long ¾" diameter and arranged in patterns of eight, ten, or 12 studs per foot seen in Figure 3. See Figure 5 for typical LBE configuration below the slab on grade.

LBE INSTALLATION CRITERIA CATEGORIES	
CATEGORY 1	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT
CATEGORY 2	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL DEPOSIT OR MINIMUM EMBEDMENT OF TWO (2) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF TEN (10) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSITS
CATEGORY 3	MINIMUM EMBEDMENT OF FIVE (5) FEET BELOW THE TOP OF THE BEDROCK DEPOSIT AND A MINIMUM TOTAL EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF THE GLACIAL TILL/BEDROCK DEPOSIT
CATEGORY 4	MINIMUM EMBEDMENT OF FIFTEEN (15) FEET BELOW THE TOP OF BEDROCK DEPOSIT

Figure 2 Typical LBE Configuration

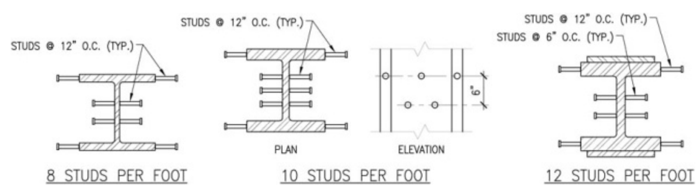


Figure 3 Typical LBE Configuration

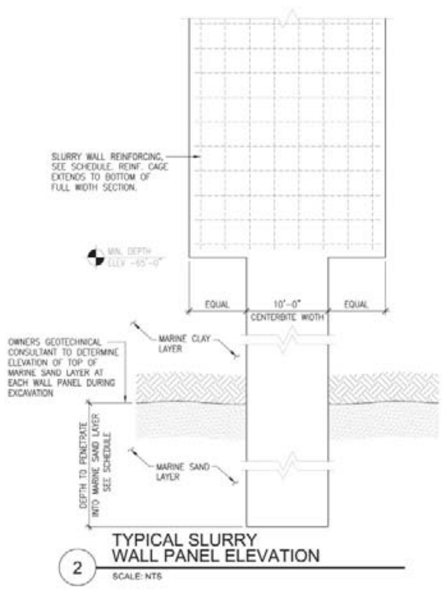


Figure 4 Slurry Wall Foundation Detail

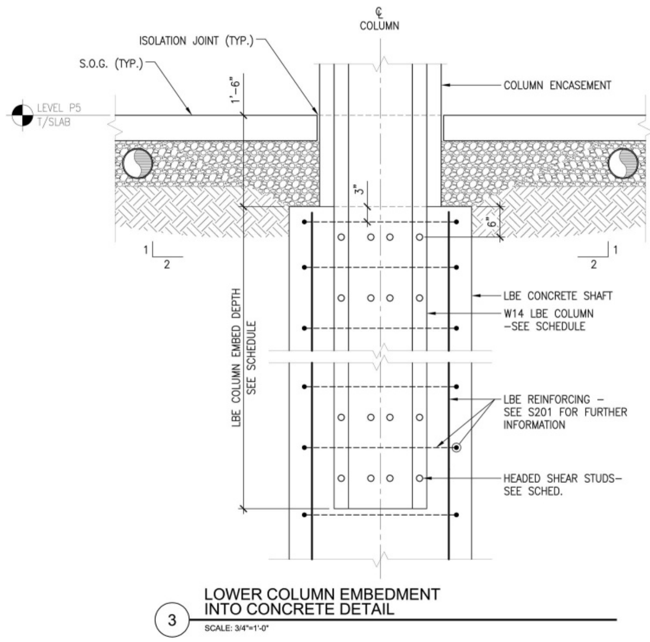


Figure 5 Load Bearing Element Foundation Detail

Beneath the area of the superstructure that is not located on top of the parking garage .365" thick, 10.75" diameter concrete filled steel pipe piles are used for foundations at column locations. Arrangements of piles include three, four, five, and eleven pile configurations. This foundation type is used below the braced frame which will be assessed for its load carrying capacity in the following sections. See Figure 6 for a typical layout of the pipe pile foundation.

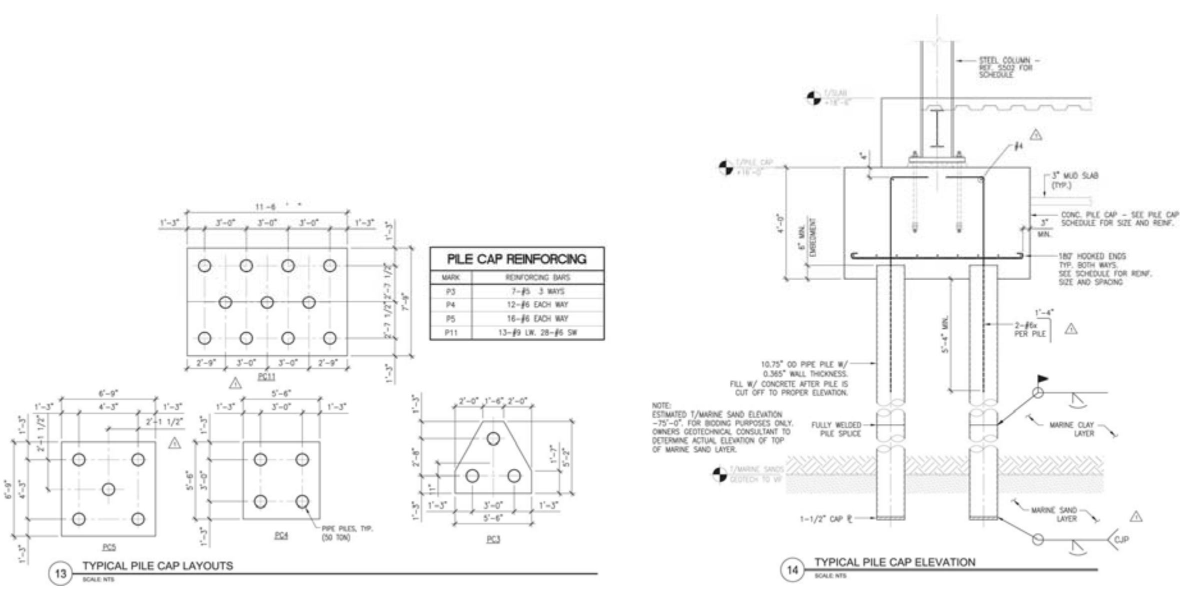


Figure 6 HSS Pile Foundation Detail

Floor Systems

Post tensioned concrete slabs are utilized for the typical floor system in the sub grade parking garage. Slab thickness in levels P1 through P4 is 14" with 6500 psi concrete. Bay sizes in the parking garage range from 36'x32' to 42'x49'.

Banded reinforcement spans in the north south direction of the parking garage plan with the typical bottom drape in each tendon meeting the minimum concrete cover at 1.75 inches. The typical force after all losses in these tendons is 1600 kips. Distributed reinforcement is placed in the east west direction at a maximum of 48 inches on center. At the column connections various patterns of stud rail arrangements and additional mild reinforcement are provided. For the lower four parking levels steel columns are encased in concrete to form a round 2'-8" diameter round column. The post tensioned slabs provide the permanent lateral bracing for the foundation slurry wall to resist the lateral soil pressures.

At the plaza and first floor level the structural floor system changes from post tensioned concrete to steel beams with composite floor slabs. In the main quad area typical bay sizes remain the same. Typical horizontal framing in this area ranges from W24x76 beams with 52 shear studs to W36x135 beams with 80 shear studs. Three inch deck with 9" of 3000psi concrete is typical for all horizontal surfaces at the main quad space. Plate girders are used to transfer load from superstructure columns above this level to the columns extending through the parking garage. All plate girders are either 48 or 54 inches deep with weights ranging from 330 to 849 lb/ft.

The floor system in the above grade building uses steel beams with composite action. This framing system allows the steel framing to engage the floor plate and take advantage of the compressive strength of the concrete. Both girders and beams act as composite members. In these bays the floor is composed of 5 ½" light weight concrete on 2" metal deck. Shear studs provide the mechanical connection between the steel and concrete with the number used varying based on strength requirements.

One of the distinct advantages of this system is the ability to have varied floor framing and column layout. The geometry and architecture of the building necessitates changes in the column grid as well as the layout of floor framing members. See the third floor framing in Figure 7 for a typical plan and framing layout. Typical members were checked under critical load combinations.

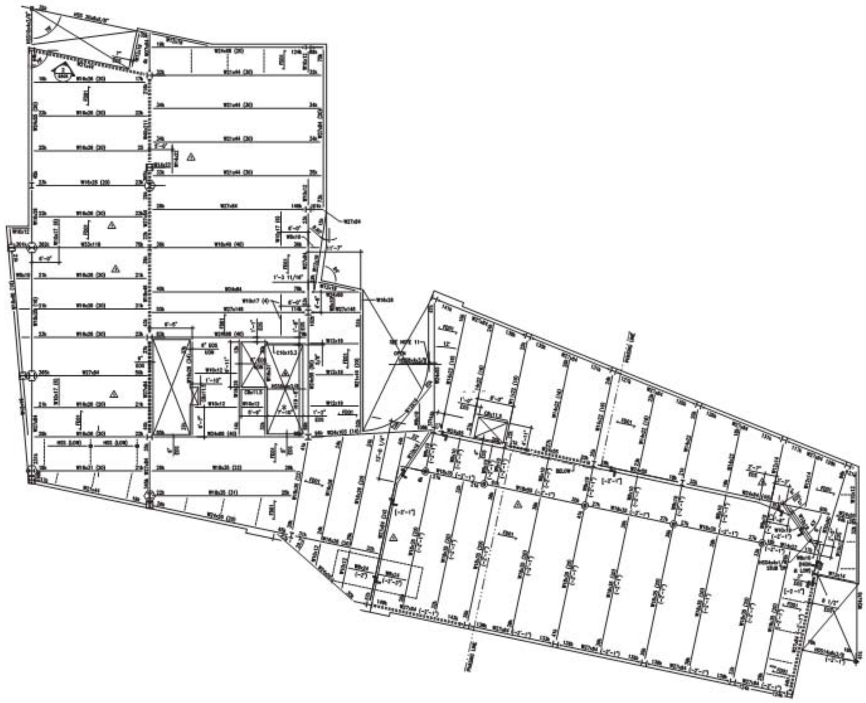


Figure 7 Second Floor Framing Layout

Columns

Typical column sections for the superstructure of the Simmons College School of Management are wide flange sections with some usage of hollow structural steel (HSS) sections. Wide flange sections are all W14s with weights varying from 43 to 109 lb/ft. The most commonly used wide flange column is a W14X90. HSS sections are either HSS6x6 or HSS8x8. In addition to carrying gravity loads many of the columns participate in the lateral force resisting systems as part of either the moment frames or braced frames.

Once the building column loads have been transferred by the plate girders W14 column sections continue to carry the load through the parking garage. Weights vary from 159 to 398 lbs/ft. In two different locations W14x398 with side plates or W14x500 columns are used. Here all columns below the first parking garage level are encased in concrete to form a 2’-8” diameter round column.

Supplementary Structural Systems

Two supplementary structural systems are used in the building in addition to the main load carrying elements. At the roof a braced frame screen is used to hide the penthouse and mechanical equipment. HSS sections are used for vertical and horizontal members while angles form the diagonal bracing.

In the parking garage reinforced concrete members are used to form the ramp access to all parking levels. Edge beams span the length of the length of the ramp with a 12 inch slab bridging the 21’-2” for the driving surface. Girders are 2’-7” deep and span below the slab at columns locations.

Exiting Lateral Systems

Two structural systems are used in the Simmons College School of Management to resist lateral forces applied to the building. In the north south direction of the building steel braced frames carry lateral loads. The lateral force resisting system in the east west direction is a combination of steel braced frames and steel moment frames. Locations of steel braced frames can be seen in Figure 8 and steel moment frames are noted in Figure 9. The number of steel braced frames used is reduced in the upper floors of the building. In some areas of the building, moment frames are used in combination with braced frames to control building drift. The majority of the braced and moment frames transfer load at their bases to transfer girders which frame to the garage columns where lateral loads are then transferred out to the exterior slurry walls. An area of note is the offset of the moment frame from the moment frame noted as number 3 in Figure 10 to its location at 4 in the fifth story. The moment frame changes from column line ZE to ZD to accommodate the building setback at this location. Lateral loads are then transferred to the moment frame on column line ZE by W33x141 beams.

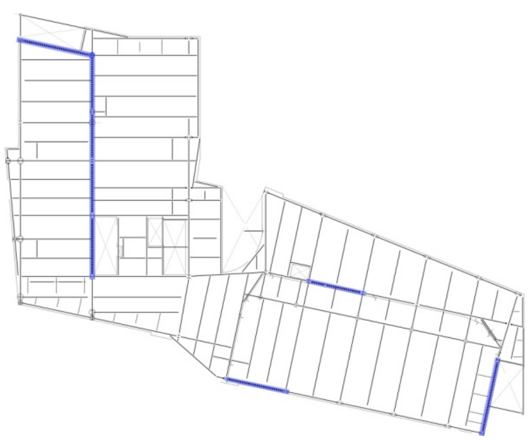


Figure 8 Braced Frame Locations

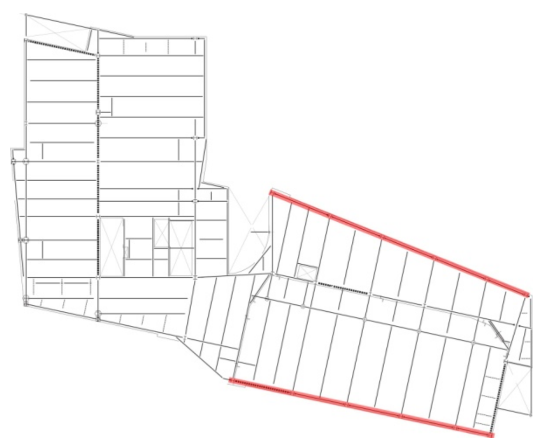


Figure 9 Moment Frame Locations

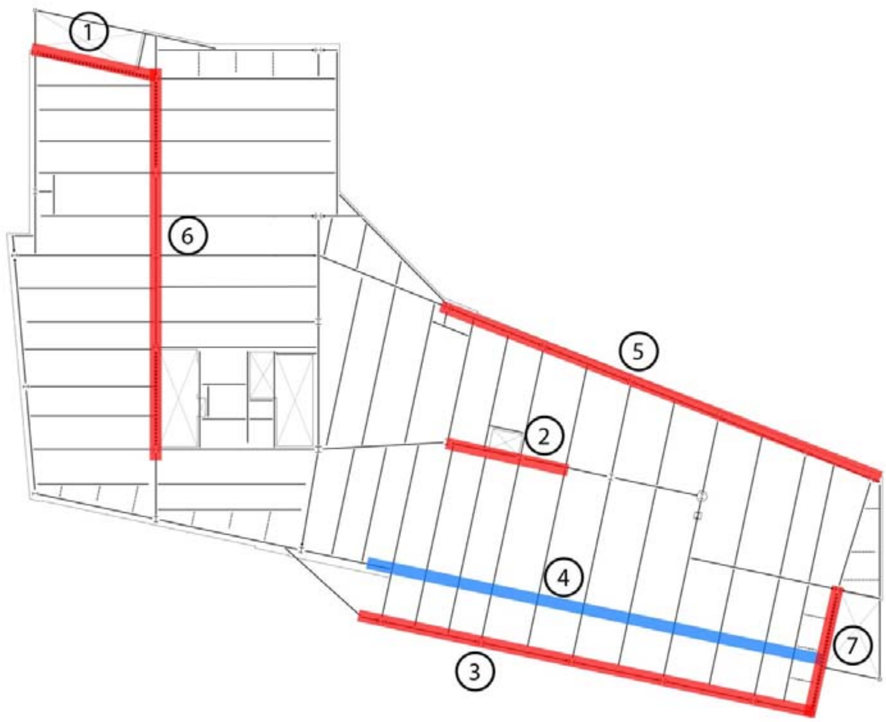


Figure 10 Lateral Frame Identifications

1. BF-EW-1
2. BF-EW-2
3. B/MF-EW-3
4. MF-EW-4 (offset of B/MF-EW-3 at 5th story)
5. MF-EW-5
6. BF-NS-1
7. BF-NS-2

At all levels the concrete floor deck forms a ridged diaphragm which transfers lateral load to either the braced or moment frames. The amount of force that each lateral load resisting element receives is dependent on that element's relative stiffness in the system.

Due to the arrangement of the lateral elements throughout the building, the effect of torsion becomes increasingly important. When lateral loads are applied to the building all elements participate in the resistance of load even when the loads are applied only in the primary directions.

Code Requirements

Design Codes

Building Code, Design Loads:	Massachusetts State Building Code CMR 780 6th Addition
Reinforced Concrete:	American Concrete Institute (ACI) 318
Structural Steel:	American Institute of Steel Construction (AISC)

Substitute Codes for Thesis

Building Code:	International Building Code (IBC) 2006
Building Loads:	American Society of Civil Engineers (ASCE) 7-05
Structural Steel:	American Institute of Steel Construction (AISC) 13th Edition 2005
Reinforced Concrete:	American Concrete Institute (ACI) 318-08
Seismic Design:	AISC Seismic Design Manual
Diaphragm Design	Steel Deck Institute, Diaphragm Design Manual 3rd Edition

Load Combinations

Lateral load combinations that would apply to this building determined from ASCE 7-05. These loads are listed below as well as the load case inputs for ETABS. Load cases which include dead and live loads were combined through additional analysis methods. ETABS allowed for the assessment of the four wind load cases from section 6.5.12.3 of ASCE 7-05 to determine the critical loading of the structure. The 3D model was developed to model the lateral system and did not include the effects of gravity loads.

ASCE 7-05 Lateral Load Cases

1.2D + 1.6(Lr or S or R) + 0.8W
1.2D + 1.6W + L + 0.5(Lr or S or R)
1.2D + E + L + 0.2S
0.9D + 1.6W + 1.6H
0.9D + E + 1.6H

Wind and seismic loads were determined for the building in the primary X and Y direction. Wind loads were applied to the building at the center of pressure while seismic loads were applied to the center of mass of each floor diaphragm. Using each load and the load cases, the following load combination inputs were developed for the 3D ETABS model. These combinations only include the unfactored lateral loads. The load factors and effects of gravity loads were to be assessed through additional analysis methods.

ETABS Load Combinations

±Ex
±Ey
±Ex±Ext
±Ey±Eyt
±Ex±0.3Ey (Only critical combinations causing the most torsion considered for this case)
±Ey±0.3Ex (Only critical combinations causing the most torsion considered for this case)
±Wx
±Wy
±0.75Wx ±Mtx
±0.75Wy ±Mty
±0.75Wx ±0.75Wy
±0.563Wx ±0.563Wy ±0.563Mtx ±0.563Mty (Moments only considered acting in the same direction)

Building Loads

Dead Loads

(All Values in PSF)

FD01	43.2
FD02	42.7
FD03	69.0
FD04	96.8
PT floor slab	175
Structural Steel	Per AISC Manual
Green Roof	100
Superimposed Dead loads:	
MEP	10
Partitions	20
Finishes/Misc.	5
Curtain Wall	10

Live Loads

(All Values in PSF)

Space:	Design Value	ASCE 7-05
Parking Floors	50	40
Plaza	100	100
	300 Construction	
Exit Corridors	100	100
Stairs	100	100
Lobbies	100	100
Typical Floor	50	50 (office load)
Corridors above 1st Floor	80	80
Roof Garden	100	100
Flat Roof	-	20
Mechanical Areas	150	

Existing Building Lateral Loads

Lateral loads acting on the structure were determined according to ASCE 7-05. The original loading for the building was in accordance with the sixth addition of the Massachusetts State Building Code. This is one source of variance that is observed between design loads that those calculated in this study. Seismic loads were the controlling lateral force on the building. Both base shear and overturning moment values for seismic design were higher than the values for wind design.

Wind Load Analysis

Wind loads were calculated using method two, the analytical procedure from section 6.5 of ASCE 7-05. Given the configuration of the building, loads were assumed to act on projected widths of the building. In this technical report the wind load was analyzed in the primary directions as seen in Figure 11. The structure was assumed to be rigid for this analysis procedure. A summary of the calculations can be reviewed in APPENDIX B.

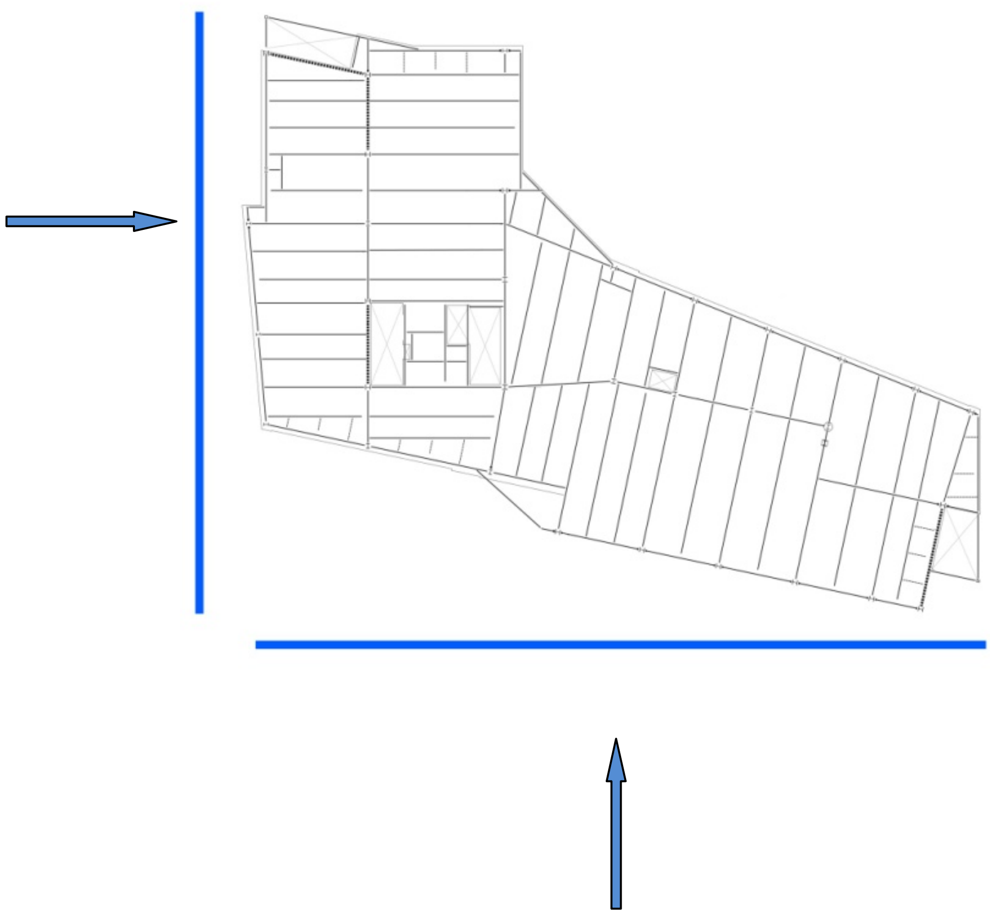


Figure 11 Wind Loading Directions

Design Wind pressures p EAST WEST direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.57	5.78	27.35	15.79
	60	30.6	20.56	5.78	26.34	14.78
	50	29.2	19.62	5.78	25.40	13.84
	40	27.4	18.41	5.78	24.19	12.63
	30	25.2	16.93	5.78	22.71	11.15
	25	23.8	15.99	5.78	21.77	10.21
	20	22.3	14.99	5.78	20.77	9.21
	15	20.5	13.78	5.78	19.56	8.00
Leeward	All	32.1	-8.09	5.78	-2.31	-13.87
Side	All	32.1	-18.87	5.78	-13.09	-24.65
Roof	70.5	32.1	-24.26	5.78	-18.48	-30.04
	70.5	32.1	-13.48	5.78	-7.70	-19.26
	70.5	32.1	-8.09	5.78	-2.31	-13.87

East
West

Pressure	height	width	moment arm	Shear	overturning moment
8.1	70.5	140	35.25	79.95	2818.13
13.8	15	140	7.5	28.98	217.35
15	5	140	17.5	10.50	183.75
16	5	140	22.5	11.20	252.00
16.9	5	140	27.5	11.83	325.33
18.4	10	140	35	25.76	901.60
19.6	10	140	45	27.44	1234.80
20.6	10	140	55	28.84	1586.20
21.6	10	140	65	30.24	1965.60
				254.74	9484.76

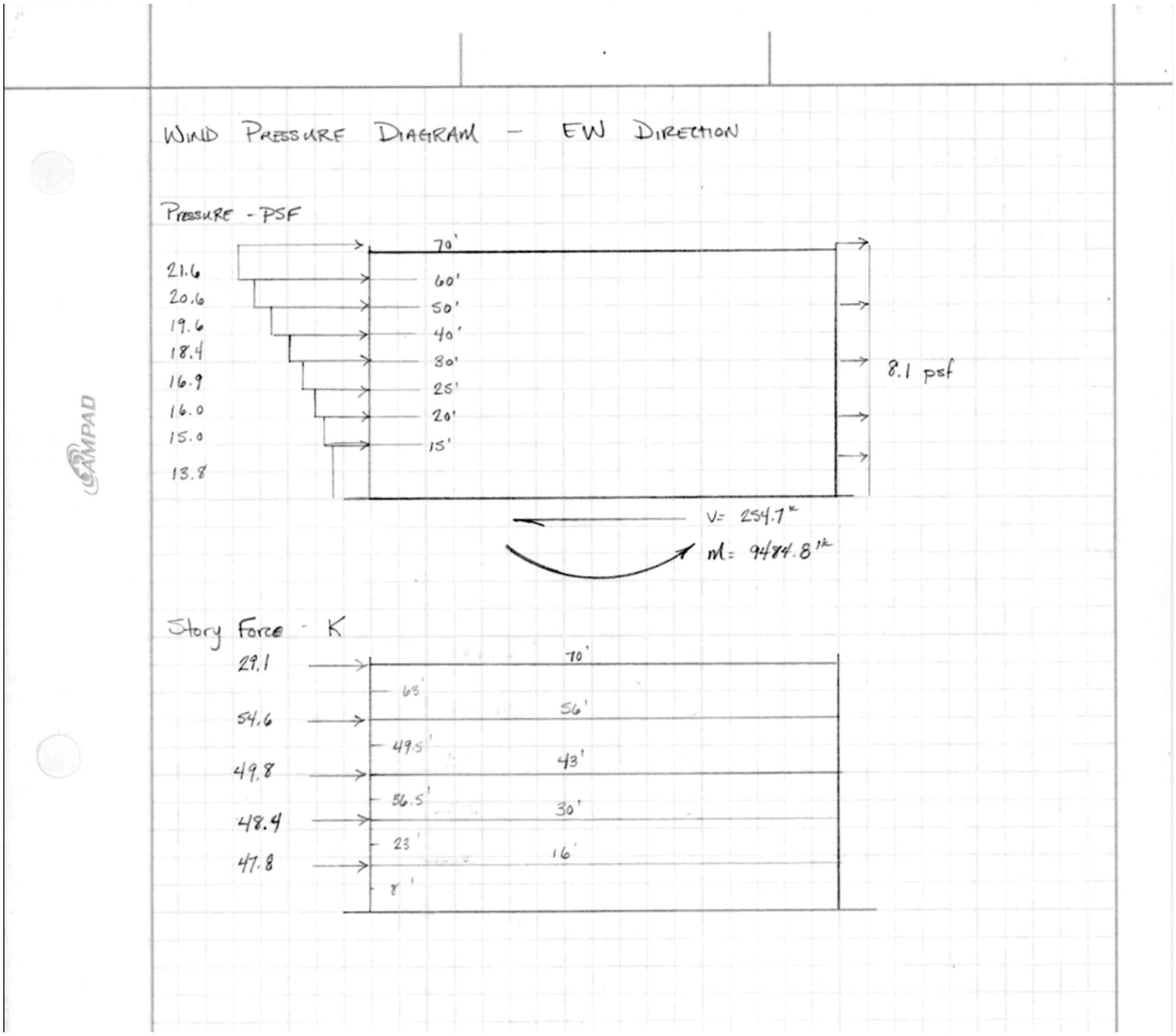


Figure 12 Wind Pressures, East-West

Design Wind pressures p NORTH SOUTH direction

Location	Height above ground	q (psf)	External Pressure qGCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
					+(Gcpi)	-(Gcpi)
Windward	70	32.1	21.06	5.78	26.84	15.28
	60	30.6	20.07	5.78	25.85	14.29
	50	29.2	19.16	5.78	24.94	13.38
	40	27.4	17.97	5.78	23.75	12.19
	30	25.2	16.53	5.78	22.31	10.75
	25	23.8	15.61	5.78	21.39	9.83
	20	22.3	14.63	5.78	20.41	8.85
	15	20.5	13.45	5.78	19.23	7.67
Leeward	All	32.1	-13.16	5.78	-7.38	-18.94
Side	All	32.1	-18.42	5.78	-12.64	-24.20
Roof	70.5	32.1	-31.58	5.78	-25.80	-37.36
	70.5	32.1	-18.42	5.78	-12.64	-24.20
	70.5	32.1	-18.42	5.78	-12.64	-24.20

North South

Pressure	height	width	moment arm	Shear	overturning moment
13.2	70.5	180	35.25	167.51	5904.66
13.5	15	180	7.5	36.45	273.38
14.6	5	180	17.5	13.14	229.95
15.6	5	180	22.5	14.04	315.90
16.5	5	180	27.5	14.85	408.38
18	10	180	35	32.40	1134.00
19.2	10	180	45	34.56	1555.20
20.1	10	180	55	36.18	1989.90
21.1	10	180	65	37.98	2468.70
				387.11	14280.06

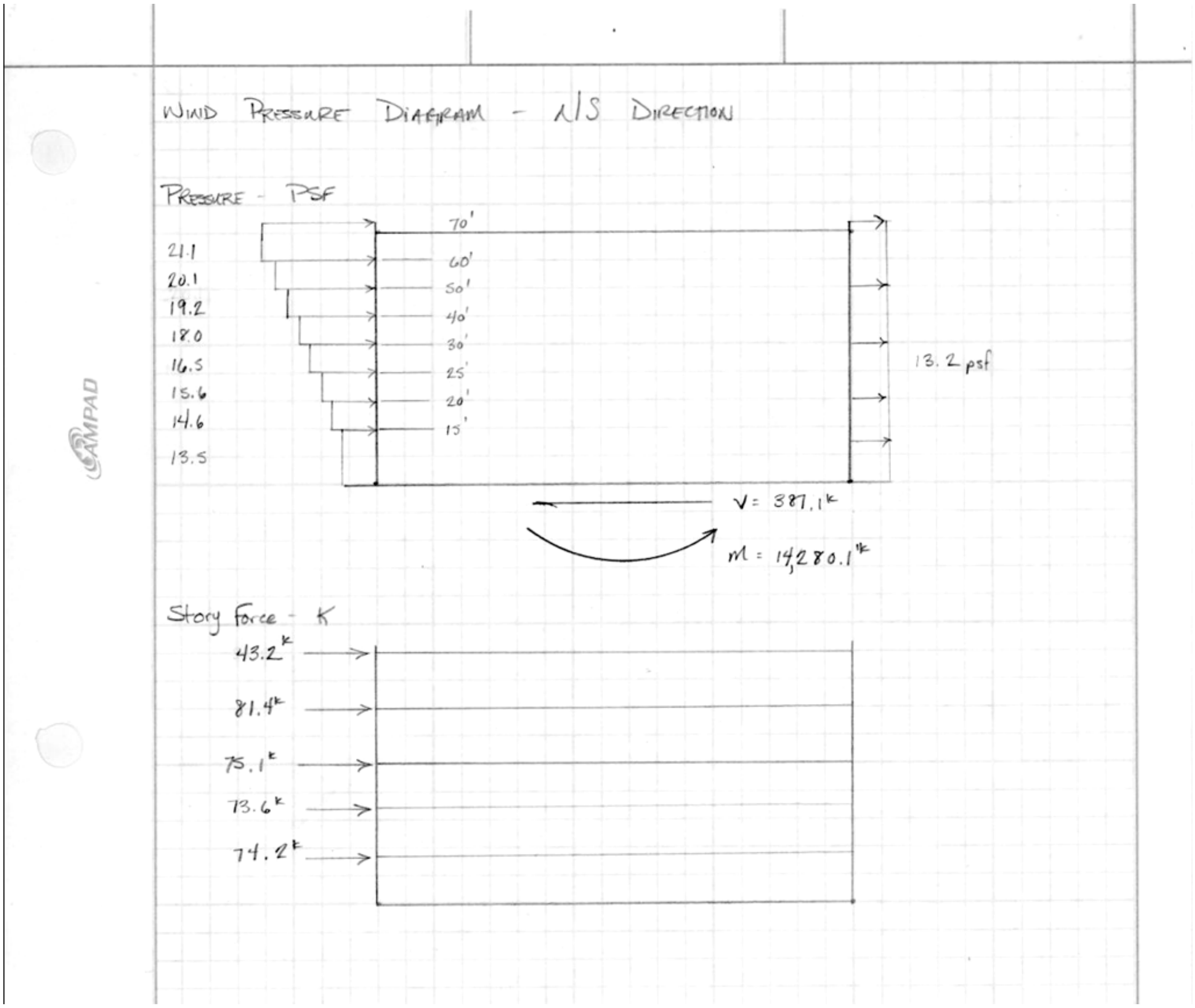


Figure 13 Wind Pressures, North-South

Seismic Load Analysis

Seismic loads, similar to the wind loads, were determined in accordance with ASCE 7-05 rather than the Massachusetts State Building Code. As a result some differences are present in the design calculations and those presented in this report. Site class E was used as a conservative approximation for the soil classification. This was determined to be the closest to the S3 soil classification that was used during design. The R-factor in each direction was determined to be a 5 when using the Massachusetts State Building Code. ASCE 7-05 categorizes the lateral systems differently which resulted in an R-factor of 6 in the EW direction and 3.25 in the NS direction. The seismic design category in this analysis was determined to be SDC = C. The ground motion acceleration values used in this report were determined with the USGS Ground Motion Parameter Calculator

Seismic Forces in the North/South Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	258.18	258.18	17899.62
5	1832	56	102592.0	0.34	373.46	631.64	20913.53
4	1438	43	61834.0	0.21	225.09	856.72	9678.80
3	1449	30	43470.0	0.14	158.24	1014.96	4747.19
2	1404	15.66	21986.6	0.07	80.04	1095.00	1253.36
				Total:	1095.00		54492.51

Seismic Forces in the East/West Direction							
Level	Story weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Cvx	Lateral force Fx (kips)	Story Shear Vx (Kips)	Moment contribution (ft-K)
R	1023	69.33	70924.6	0.24	127.32	127.32	8827.21
5	1832	56	102592.0	0.34	184.17	311.49	10313.52
4	1438	43	61834.0	0.21	111.00	422.49	4773.11
3	1449	30	43470.0	0.14	78.04	500.53	2341.08
2	1404	15.66	21986.6	0.07	39.47	540.00	618.10
				Total:	540.00		26873.02

Existing Building 3D Building Model

It was determined to be important to develop a 3D building model to account for all effects of the building lateral system arrangement as well as the eccentric loading on the building. To perform the modeling ETABS was used to accurately determined the center of mass, center of rigidity, and distribution of lateral loads. Additionally the 3D model outputs the building's primary periods of vibration which can be used to develop dynamic response characteristics. The modeling procedure for ETABS is summarized below.

ETABS Modeling Assumptions:

1. All basses are pinned
2. Braces and beams not participating in moment frames have the 3-3 moment released
3. Rigid end offsets = 1.0 (moment frames)
4. Panel Zone Explicit Modeling (moment frames)
5. Rigid Diaphragm Constraint at all levels
6. Diaphragm mass based on a typical 100psf floor dead weight
7. Beam insertion points with modified stiffness: Top Center = -5.25

In this preliminary model, the bases were assumed not to resist moment with the effects of vertical and horizontal displacements neglected. This assumption was later addressed in the following study of the lateral system. Typical floor dead weights including the exterior walls as a uniform distributed load ranged from 92 – 101 psf. For consistency a 100 psf floor load was used as the input for building mass. The critical building output from the ETABS model is summarized in the following charts and tables.

First Mode Period of Vibration

Period of Vibration (s)	
X	1.0016
Y	0.6962
Z	0.5489

Center of Mass and Center of Rigidity
Top Left of Floor Diaphragm as (0,0) See Figure 14

Story	XCM	YCM	XCR	YCR
STORY5	1025.94	-1032.845	982.088	-893.621
STORY4	908.083	-899.85	1076.668	-822.085
STORY3	908.083	-899.85	1014.08	-766.483
STORY2	908.083	-899.85	975.33	-726.267
STORY1	907.257	-907.941	770.768	-704.372

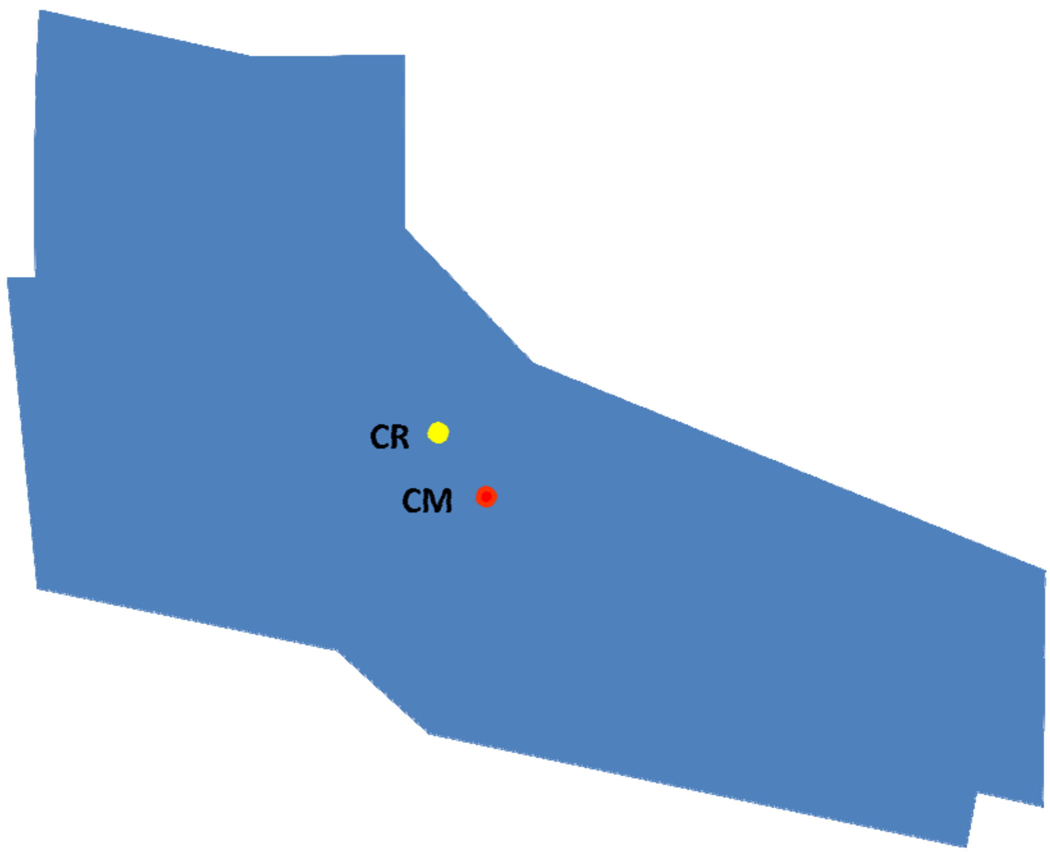


Figure 14 Center of Mass and Center of Rigidity

As part of the design checks for the existing structure the building drift was analyzed under lateral loading. Building drift due to wind load was compared to the typical industry standard for wind drift, h/400. Seismic story drift was compared to ASCE 7-05 allowable story drift values from Table 12.12-1. It was important to address the total movement when assessing the drift values. Torsion in the building caused the drift and displacement in both the X and Y directions. Therefore the resultant of these two components was used to compare against accepted values. Both seismic and wind design checks were verified to meet code and industry standard. Below is a summary of the critical drift and displacement values in the existing building layout.

Wind Drift

Story	Point	Load	DispX	DispY	DispTOT	Allowable Building Drift Δ=h/400
STORY5	84	+0.563Wx +0.563Wy +0.563Mtx +0.563Mty	0.4147	0.4195	0.589878	2.1

Seismic Story Drift

Drift Summary		Max Drift Percent		Max. Drift (in.)		Allowable Drift	
Story	Height	X	Y	X	Y	%	in.
5	13.25	0.53	0.89	0.85	1.42	1.50	2.39
4	13	0.42	0.82	0.66	1.27	1.50	2.34
3	13	0.44	0.77	0.69	1.21	1.50	2.34
2	14.25	0.43	0.58	0.73	1.00	1.50	2.57
1	15.75	0.15	0.31	0.28	0.59	1.50	2.84

Lateral System Design Problem Statement

When designed, allowances were included in the building with the intent that it could later be expanded to top out at nine stories. This allows for the building to have a longer life, adaptable to the changing needs of the college.

Accommodations in the existing structure include allowances for the increased gravity load to be carried through select areas of the superstructure as well as the below grade parking garage. The existing plans indicate that the expansion will occur primarily on the west end of the building. During the initial design, the expansion was considered to be separated from the existing building with a building expansion joint, or to be isolated a distance away from the base building. See Figure 15 for the proposed area for future expansion. The area limit for the expansion is highlighted in red and the columns with future load consideration are circled.

Figure 15 Partial Plaza Plan with Future Expansion Area

Specific design information was not available for the particular layout of the expansion. Only the allotments for additional gravity loads as well as the anticipated plan and height restrictions. Therefore, the layout of the expansion studied in this report was treated as complying within the plan dimensions and load allotments made for the original designers' expansion program.

Solution Method – Design Criteria

A design study will evaluate the consideration of an alternative option for the building expansion. This design scenario will consider an expansion that will be structurally tied to the existing structure. Therefore, this expansion scenario will be limited to five stories in order to be able to rely completely on the existing lateral system. The problem will be addressed as if it were being considered in the initial design rather than an alteration to the constructed building. The goal of this investigation is to increase the future options for the owners with the least cost impact, without compromising any other design programs.

Throughout the alternative expansion study there will be several design criteria that will be adhered to for the acceptability of the revised system. The revised structural system must be able to adequately resist loads applied by both the existing, and expanded systems. Critical to achieving this will be the reduction in building torsion that is developed by a westward expansion resisted by the existing lateral system layout. Building codes and standard engineering practice will govern for the building drift limitations under seismic and wind loadings. Finally, the system must not impinge on the ability for any other expansion design scenario to be carried out, rather than the scenario under investigation in this report.

There are several benefits to considering a structurally tied expansion as an initial design scenario. This alternative allows the owner of the building an additional option as their facility needs change. With respect to the structure, there will be no need for additional lateral force resisting elements within the new layout. Using a structurally tied system will increase the architectural freedom in the expansion areas by not requiring space for braced frame lateral elements. There is also the potential for a lower comparative cost by excluding the material and costs associated with additional moment frames.

Structural Depth Study

Building Expansion Layout

A study of potential layouts for a structurally tied expansion was conducted based on site layout and existing future column loading. The maximum expansion building area was considered, nine stories and engaging the full tributary area of the garage columns. Conservative gravity load were estimated as acting on these columns. With the estimated loading condition, it was determined that the allowable future factored loads would be exceeded. Next, a five story expansion that encompassed the full expansion area was considered. This scenario fell within the limitations of the future factored loading allotments. However, with consideration of the existing site layout, it was assumed that this layout would not be a desirable arrangement.

A third expanded system layout was then explored now working with the loading and layout limitations of the site. This considered an expansion 50 feet westward along the full length of the building's west façade. This system was viewed as reasonable for the site layout limitations, also meeting the future factored loads restrictions. In total, this expansion increases the existing building's floor area by approximately one third, adding 5000 square feet to the typical floor layout. See Figure 16 for the finalized layout of the expansion to be studied.

Figure 16 New Expansion Layout (Orange), Existing Building (Blue)

Base Condition Modeling

Important to the study of a structurally tied expansion was the proper assessment of the lateral system center of rigidity. Given the arrangement of lateral force resisting elements, it was difficult to correctly assess the system center of rigidity by relative stiffness and hand calculation. A 3D ETABS model was the primary tool used to investigate the system properties.

As discussed previously in the existing lateral system modeling section, the base condition for all lateral force resisting elements was assumed to be a pinned connection. While this correctly recognizes the building plaza level as the base of the system, the flaw in this approach is that the vertical interaction between the above and below grade structures is ignored. Many of the lateral system columns are positioned on built up plate girders that transfer load through flexure to the garage columns below. This was viewed as potentially impacting the performance of the entire lateral system. As a result, different methods for the implementing parking garage structure into the building model were considered. See Figure 17 for the significant transfer girders at the plaza level.

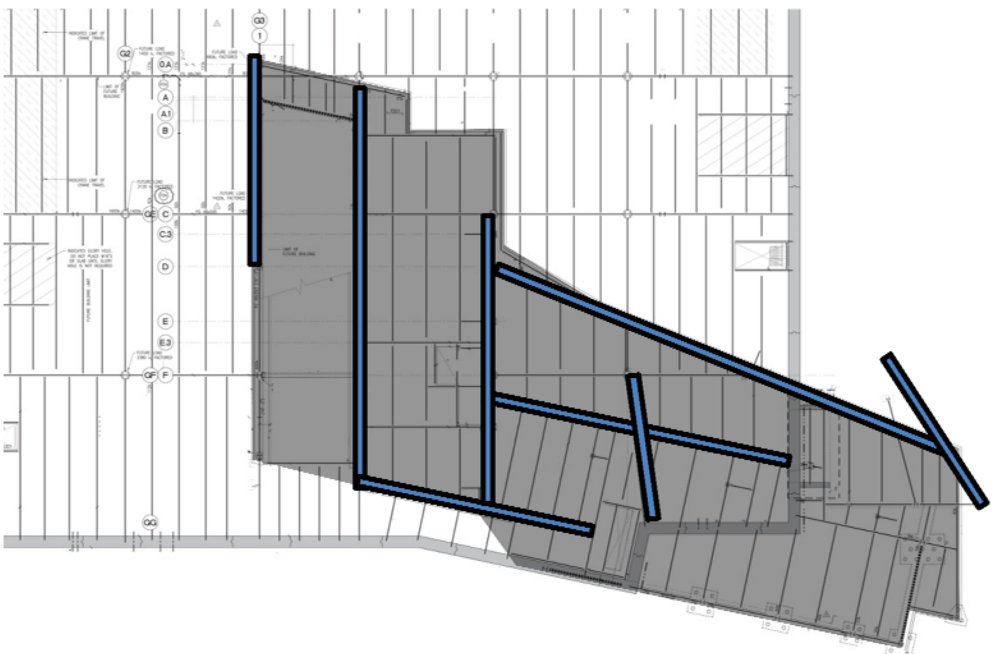


Figure 17 Selected Built-Up Plate Transfer Girders

One method under consideration was the explicit modeling of all critical structural elements in the parking garage. This would include the built up plate girders as well as the columns within the garage. The difficulty with this method is that now additional steps need to be taken to force the model to behave as intended. At the plaza level, horizontal restrains would need to be added to simulate the effects of the slurry walls. Also, intermediate bracing for the parking garage columns would need to be established to limit P-Delta Effects.

The second method for base condition modeling considered was the use of vertical stiffness modifiers to simulate the effects of the above and below grade structure interaction. This uses equivalent systems which uses spring constraints with stiffness modifiers rather than the explicit modeling of built up members.

Ultimately the modeling implementation for the base condition relied on a combination of explicit member modeling as well as spring constraints. Plate girders that had multiple columns from a lateral force resisting element framing into it were explicitly modeled. This was to ensure the vertical interaction between these columns was most correctly modeled. All built up plate girders that had a single column framing into it were investigated in SAP 2000 to determine an equivalent vertical stiffness. Each girder was modeled with a unit load applied at the given location of the column. Stiffness was then related by the equation K = P/Δ, or in this case K = 1/Δ. The stiffnesses were then implemented into the model as spring constraints at the corresponding framing location. See Figure 18 for the base condition implementation into the model. A summary of built up plate girder stiffnesses is shown below.

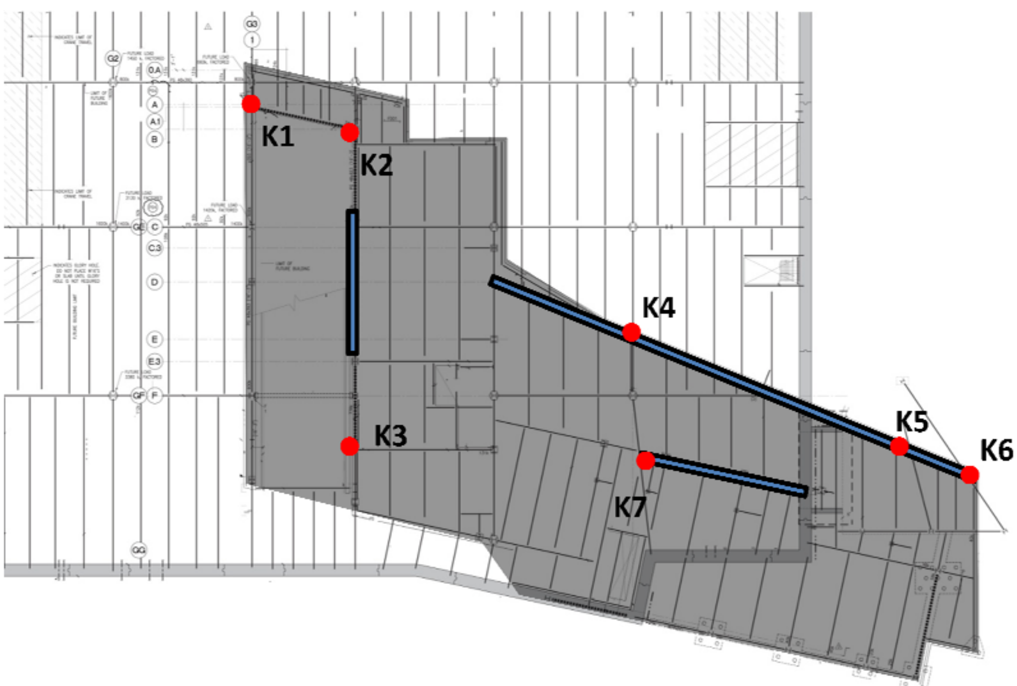


Figure 18 Base Condition Modeling Implementation
Plate Girders (Blue), Spring Elements (Red)

Spring Stiffnesses at Base Condition

Spring	Transfer Girder	Length	Load Location	Deflection (in)	Stiffness (K/in)
K1	PG48X393	36’-0”	29’-10”	.0005885	1699.235
K2	PG48X823	36’-0”	24’-4”	.00058667	1704.448
K3	PG48X849	42’-0”	28’-6”	.0006829	1464.343
K4	PG48X823	42’-0”	14’-0”	.0008951	1117.194
K5	W33X354	28’-0”	28’-0”	.0009743	1026.378
K6	W33X387	45’-0”	28’-0”	.00437	228.833
K7	PG48X823	40’-0”	14’-0”	.0008221	1216.397

As a result of the revised base condition, lateral frames with modified vertical stiffness, mostly located in the northwest of the building, now have a lower relative stiffness in the system. This causes the center of rigidity to move away from these elements and toward the southeast of the building, passing the center of mass for the building. The effects of inherent torsion as well as the critical torsional moment are all reversed as a result of the center of rigidity moving from one side of the center of mass to the other. See Figure 19 for the existing and revised locations of the center of rigidity.

When considering the westward expansion, the modeling of the base condition interaction between the above and below grade structures becomes a necessity for this building. The expanded building eccentricity would be much smaller if only pinned bases were used at the base of all lateral frame elements. This then results in an unconservative estimation of the load effects on the building. As a result, the modifications to the bases were used in the remainder of the 3D modeling studies.

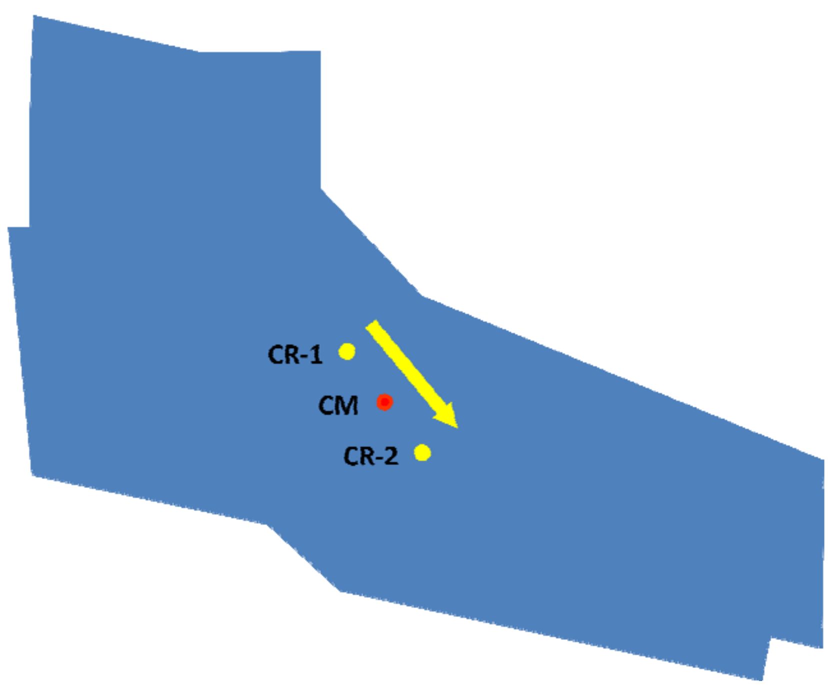


Figure 19 Center of Mass vs. Center of Rigidity
Unmodified Base (CR-1), Modified Base (CR-2)

Load Analysis - Expanded Layout

Lateral loads for the structure were determined for the new building configuration. ASCE 7-05 again was used as the primary resource for lateral loads and their application on the building. Much of the same process that was used in the existing building loading was followed in calculating the new loading condition for the expanded layout. However, the 3D model was used to accurately predict the building period to be used in the load calculations. This resulted in a varied approach for load determination than that presented in previous sections.

Wind Analysis, Flexible Structure

Wind loads were calculated acting on the modified tributary widths of the expanded building. This had the largest effect on the loads applied in the Y-Direction of the structure due to the expansion primarily taking place perpendicular to this axis.

The real building periods were used in the determination of the wind loads. Given that the primary axis periods for the building, the natural frequency is now less than one. This results in a categorization of the building as a flexible structure. This is in contrast with the initial loading condition which used a rigid structure categorization, and the corresponding approach to determining gust factor. The charts below summarize the revised wind loading acting on the building. See Appendix C for the revised wind load calculations.

East – West Design Wind Pressures

Location	Height above ground	Kz	qz (psf)	External Pressure q_zG_fCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
						+(Gcpi)	-(Gcpi)
Windward	70	0.89	32.1	22.68	5.76	28.44	16.92
	60	0.85	30.6	21.66	5.76	27.42	15.90
	50	0.81	29.2	20.64	5.76	26.40	14.88
	40	0.76	27.4	19.37	5.76	25.13	13.61
	30	0.70	25.2	17.84	5.76	23.60	12.08
	25	0.66	23.8	16.82	5.76	22.58	11.06
	20	0.62	22.3	15.80	5.76	21.56	10.04
	15	0.57	20.5	14.52	5.76	20.28	8.76
Leeward	All		32.1	-11.32	5.76	-5.56	-17.08
Side	All		32.1	-19.80	5.76	-14.04	-25.56
Roof	70.5		32.1	-24.26	5.76	-18.50	-30.02
	70.5		32.1	-13.48	5.76	-7.72	-19.24
	70.5		32.1	-8.09	5.76	-2.33	-13.85

Pressure	height	width	moment arm	Shear	overturning moment
22.68	70	140	65	31.75	2063.66
21.66	60	140	55	30.32	1667.69
20.64	50	140	45	28.89	1300.27
19.37	40	140	35	27.11	948.89
17.84	30	140	27.5	12.49	343.35
16.82	25	140	22.5	11.77	264.87
15.80	20	140	17.5	11.06	193.52
14.52	15	140	7.5	30.50	228.75
-11.32	70	140	35	110.89	3881.11
				294.78	10892.11

East/West, X-Direction Loading					
	Pw	Pl	TOTAL P	CUM	M
R	22224	11089	33.3	33.3	8394.9
5	41291	21386	62.7	96.0	15794.5
4	36939	20594	57.5	153.5	14498.2
3	34816	21386	56.2	209.7	14162.9
2	32355	23762	56.1	265.8	14141.5

North – South Design Wind Pressures

Location	Height above ground	Kz	qz (psf)	External Pressure q_zG_fCp (psf)	Internal Pressure qh(Gcpi) (psf)	Net Pressure p (psf)	
						+(Gcpi)	-(Gcpi)
Windward	70	0.89	32.1	22.32	5.76	28.08	16.56
	60	0.85	30.6	21.27	5.76	27.03	15.51
	50	0.81	29.2	20.30	5.76	26.06	14.54
	40	0.76	27.4	19.05	5.76	24.81	13.29
	30	0.70	25.2	17.52	5.76	23.28	11.76
	25	0.66	23.8	16.55	5.76	22.31	10.79
	20	0.62	22.3	15.50	5.76	21.26	9.74
	15	0.57	20.5	14.25	5.76	20.01	8.49
Leeward	All		32.1	-13.90	5.76	-8.14	-19.66
Side	All		32.1	-19.47	5.76	-13.71	-25.23
Roof	70.5		32.1	-31.58	5.76	-25.82	-37.34
	70.5		32.1	-18.42	5.76	-12.66	-24.18
	70.5		32.1	-18.42	5.76	-12.66	-24.18

Pressure	height	width	moment arm	Shear	overturning moment
22.32	70	230	65	51.33	3336.23
21.27	60	230	55	48.93	2691.05
20.30	50	230	45	46.69	2101.03
19.05	40	230	35	43.81	1533.40
17.52	30	230	27.5	20.15	554.04
16.55	25	230	22.5	19.03	428.12
15.50	20	230	17.5	17.83	312.00
14.25	15	230	7.5	49.17	368.76
-13.90	70	230	35	223.85	7834.90
				520.78	19159.54

North/South, Y-Direction Loading					
	Pw	Pl	TOTAL P	CUM	M
R	35929	22385	58.3	58.3	24142.0
5	66661	43172	109.8	168.1	45470.7
4	59689	41573	101.3	269.4	41922.5
3	56235	43172	99.4	368.8	41154.6
2	52190	47969	100.2	469.0	41465.8

Seismic Loading

A new seismic loading condition was determined acting on the expanded building layout, again using ASCE 7-05. Given that there was no specific design information, an area calculation was used to relate the weight of each expanded floor to the weight of the exiting floor. In the most cases this factor was 20,000sf/15,000sf or 1.33. Actual building period was used to determine the Cs factor for each direction.

Once the revised lateral loads were applied to the building, the effects of torsional irregularities were examined. At all levels an extreme torsional irregularity was now present. This is characterized by the maximum lateral displacement on a floor being greater than 1.4 times the average displacement. Therefore it was necessary to include the proper provisions for Seismic Design Category C from Table 12.3-1, Horizontal Structural Irregularities, of ASCE 7-05. The amplification of accidental torsion factor, Ax, was now considered at each level. The charts below summarize the initial seismic loading condition for the expanded building. See Appendix C for complete calculations.

North – South Direction Seismic Loads

T	0.768
k	1.134
Vb	805

Seismic Forces in the North/South Direction										
Level	wx (kips)	hx (ft)	wxhxk	Cvx	Fx (kips)	Vx (Kips)	Bx (ft)	5% By (ft)	Ax	Mz (ft-K)
R	1238	69.33	151490.6	0.24	191.06	191.06	230.00	11.50	2.46	5405.19
5	2291	56	220015.9	0.34	277.49	468.55	230.00	11.50	2.49	7945.91
4	1890	43	134538.7	0.21	169.68	638.24	230.00	11.50	2.47	4819.86
3	1905	30	90148.8	0.14	113.70	751.93	230.00	11.50	2.46	3216.51
2	1863	15.66	42169.7	0.07	53.19	805.12	230.00	11.50	2.39	1461.80
				Total:	805.12					22849.28

East – West Direction Seismic Loads

T	0.768
k	1.134
Vb	433

Seismic Forces in the East/West Direction										
Level	wx (kips)	hx (ft)	wxhxk	Cvx	Fx (kips)	Vx (Kips)	By (ft)	5% Bx (ft)	Ax	Mz (ft-K)
R	1238	69.33	151490.6	0.24	102.70	102.70	140.00	7.00	1.00	718.88
5	2291	56	220015.9	0.34	149.15	251.85	140.00	7.00	1.05	1096.26
4	1890	43	134538.7	0.21	91.20	343.05	140.00	7.00	1.02	651.20
3	1905	30	90148.8	0.14	61.11	404.16	140.00	7.00	1.05	449.18
2	1863	15.66	42169.7	0.07	28.59	432.75	140.00	7.00	1.21	242.13
				Total:	432.75					3157.65

Lateral System Analysis and Redesign

Prior to beginning the redesign process, the properties of the expanded building were studied to develop an understanding of how the controlling load cases were affecting the structure. In all cases, the controlling load condition for the expanded building layout was a seismic load case. This corresponds to the controlling load cases for the existing layout.

The first parameter that was investigated for the building was the revised center of mass, center of rigidity, and the resulting eccentricity. It was expected that the new center of rigidity would follow the addition of mass, and move to the west. The new eccentricity for the expanded layout was on average 35’. This was identified as a major contributing factor for the existence of an extreme torsional irregularity. See Figure 20 for the revised center of mass in the expanded layout.

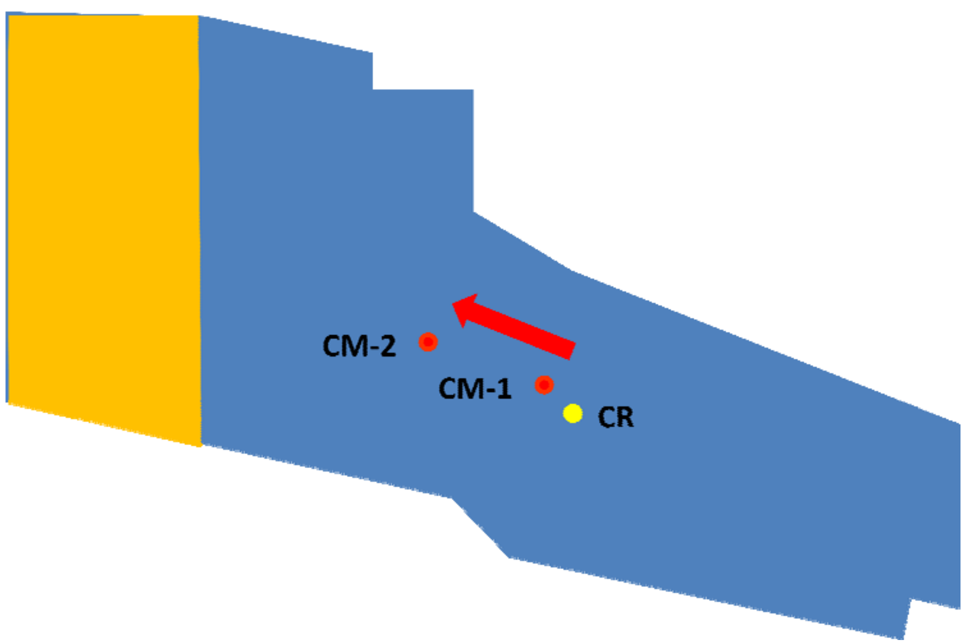


Figure 20 Center of Mass vs. Center of Rigidity
Existing Layout (CM-1), Expanded Layout (CM-2)

The controlling load case for BF-NS-1 on the west end of the building was carefully examined. See Figure 10 above for complete lateral frame identification. Seismic Design Category C requires that Seismic loading must consider 100% load acting in one direction and 30% load acting in the perpendicular direction along with the corresponding accidental torsional moments. Under this loading condition the strength capacity of braces within the frame were being overloaded. On the east end of the building, BF-NS-2 was resisting a very small amount of shear. While this was not the controlling load case for this frame, it did indicate that the moments caused by inherent and accidental torsion were enough cancel out the direct shear applied. See Figure 21 for the critical load condition as it applies to the expanded structure.

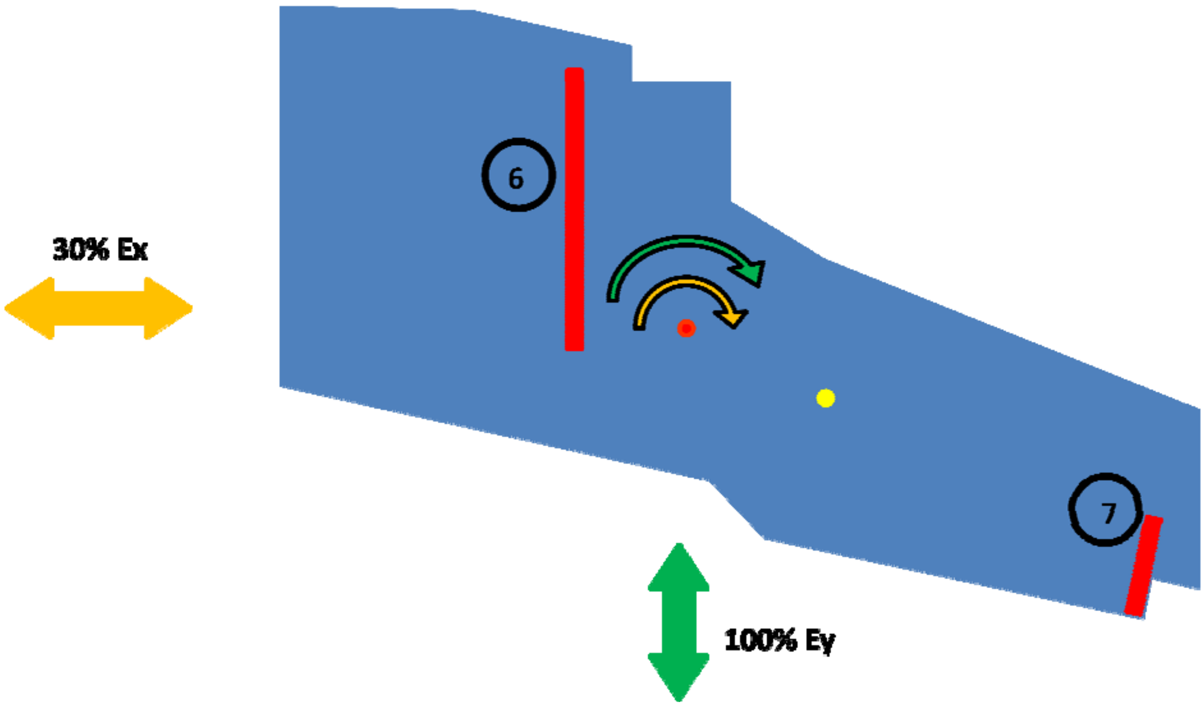


Figure 21 Critical Load Application

6. BF-NS-1
7. BF-NS-2

Various methods were considered to resolve the critical loading condition. Ultimately it was determined that the relocation of the structure center of rigidity would reduce the inherent torsional moment as well as counteract the extreme torsional irregularity, reducing the Ax factor. This results in a decrease in the loads that would need to be resisted by the system.

Careful study of each lateral frame was conducted to determine the potential methods to move the center of rigidity westward, reducing the X-direction eccentricity seen in Figure 22. One method would be to increase the stiffness of BF-NS-1, see Figure 23 for elevation. After preliminary studies of this solution method, it was determined that member sizes would need to be increase significantly to achieve desired performance. In some cases, a limit of the ability to increase stiffness in this frame was reached. As the stiffness was increased, more load was drawn to this frame causing additional members to fail. Although a solution was possible with this approach, alternative methods with less structural and architectural implications were studied.

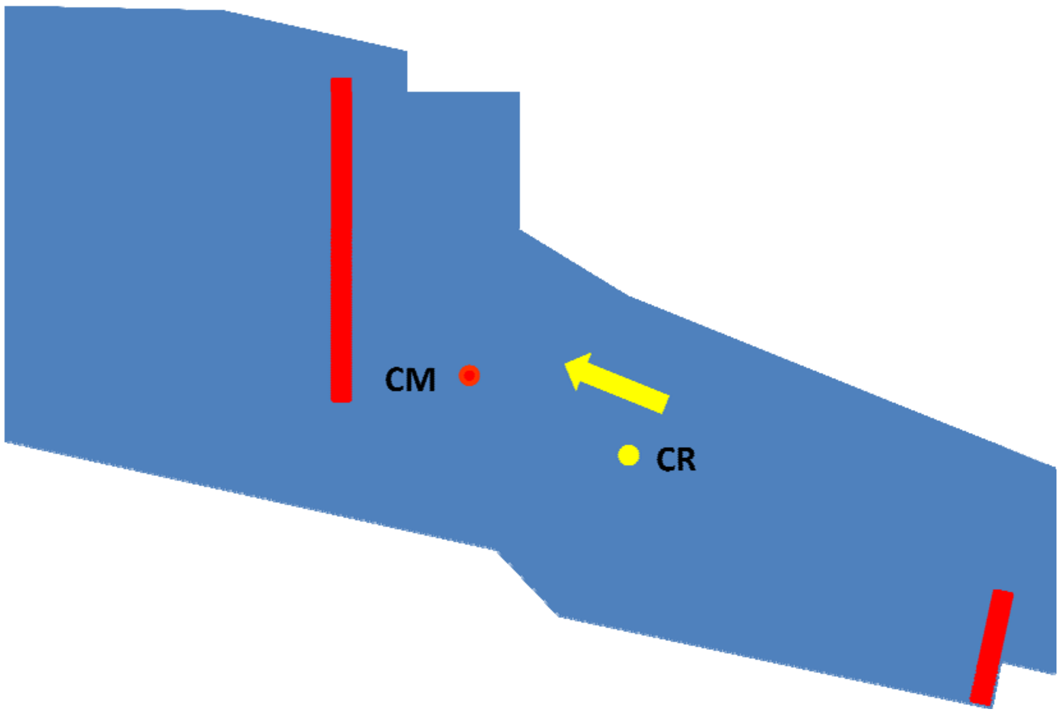


Figure 22 Desired Center of Rigidity Relocation

A second option that would result in the relocation of the center of rigidity is to decrease the stiffness of BF-NS-2, see Figure 23 for elevation. Decreasing the stiffness of the braced frame would move the center of rigidity of the structure away from this frame and to the west of the building. Preliminary studies of this solution showed that this was an extremely effective way to move the center of rigidity, given the layout of lateral elements in the building. However, this solution is limited by the structural requirements of the expanded and existing building layouts and would become a key consideration to the new member sizing. When possible, the reduction in stiffness of BF-NS-2 was considered before member sizes were increased to satisfy either strength or stiffness requirements.

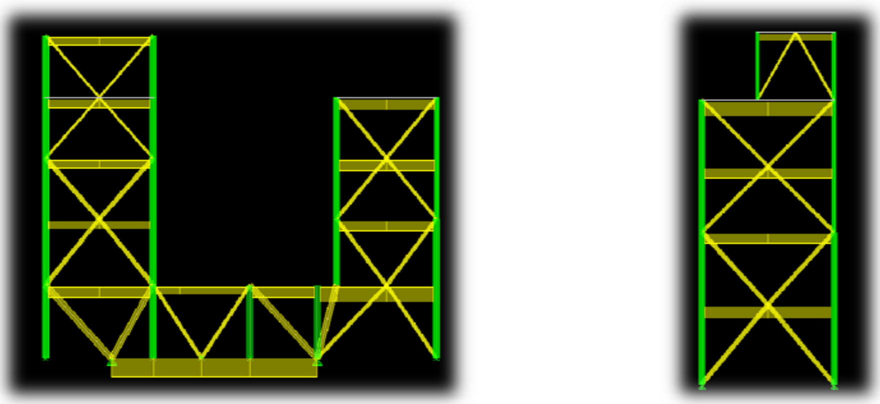


Figure 23 BF-NS-1 (Left) BF-NS-2 (Right)

Iterative Design Process for Load Application

An iterative approach was taken to develop a design solution for the additional parameters that the structural system now has to meet. It is difficult to directly determine the required structural strength and stiffness required by the system. With each member that is changed in the lateral system, the load distribution for direct shear force changes as a result of the altered relative stiffness. Also, the center of rigidity is moved to a new location. This causes the response of the structure to change in two ways. First, inherent torsion induced by the center of mass, center of rigidity eccentricity is reduced, and therefore changing the loads applied to the system. Second, the torsional displacement response is altered which allows for a revision to the accidental torsion amplification factor.

A process was developed for the investigation of varied structural system arrangements. Using the loads determined from the previous system, revised brace elements were chosen from Table 4-4 of the AISC Manual for the strength characteristics, as well as member area as an indicator of axial stiffness. Knowing these two parameters of the revised brace members allows for an estimation of the new system response with respect to load carrying capability and the center of rigidity.

At this stage of the iteration the new system center of rigidity was inspected to determine if the desired revised system properties were obtained by the revision. Comparing the system eccentricity to the starting model allows for an indication of percent reduction in inherent torsion. Following this, new accidental torsion amplification factors were developed for the revised system. Again these revised values can be compared to the starting values of the expanded system to give an indication as to the percent reduction in torsional moment.

Due to the repetitive nature to the design process used in this study, multiple EXCEL spread sheets were used to aid in the necessary calculations. Spreadsheets were set up to assess the center of rigidity, determine the torsional irregularities for each floor, and develop the new loading condition for the system under investigation. Below are the steps in developing the eighth revision to the lateral system that was being investigated. This system ultimately failed due to the connection requirements of the Seismic Design Manual.

Center of Mass vs. Center of Rigidity

Story	XCM	YCM	XCR	YCR	Ex	Ey
STORY5	61	-81	82	-84	-21	3
STORY4	51	-68	83	-83	-31	16
STORY3	51	-68	75	-79	-23	11
STORY2	51	-68	64	-73	-13	5
STORY1	51	-68	46	-71	4	3

Comparison of Eccentricity in the X-Direction

X-Direction Eccentricity			
Base	Expansion	Mod 8	% diff
6.8	-30.4	-21.1	30.4
-5.1	-49.7	-31.2	37.2
2.8	-45.2	-23.2	48.7
13.3	-40.8	-12.8	68.6
26.9	-20.8	4.1	119.8
		Avg.	60.9

Example of Accidental Torsional Irregularity Amplification Factor Determination
(calculated individually for each floor)

Story	Point	Load	UX	UY		Dmax	Davg	1.2Davg	1.4Davg	Ax
STORY4	52	1EY	0.20	0.63		0.66	1.57	1.88	2.19	1.73
STORY4	662	1EY	1.01	2.26		2.47				
STORY4	52	EYTA	0.29	0.92		0.97	1.49	1.79	2.09	1.28
STORY4	662	EYTA	0.77	1.87		2.03				
STORY4	52	EYTB	0.10	0.35		0.36	1.64	1.97	2.30	2.20
STORY4	662	EYTB	1.26	2.64		2.92				

Revised Seismic Load in the Y-Direction

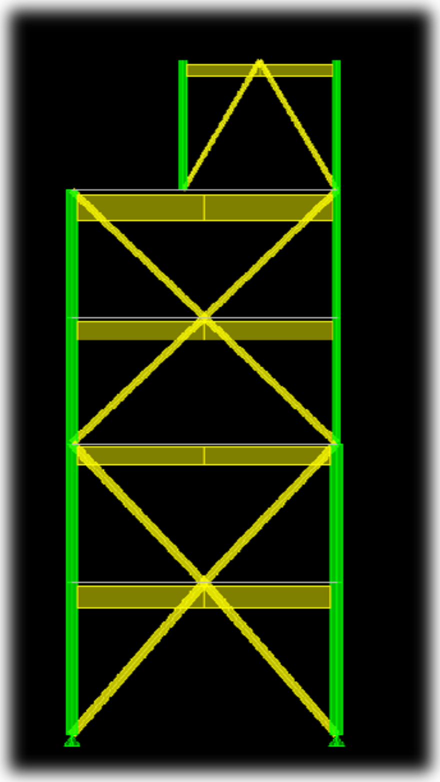
Seismic Forces in the North/South Direction										
Level	w_x (kips)	Height (ft)	$w_x h_x^k$	Cvx	Fx (kips)	Vx (Kips)	Bx (ft)	5% By (ft)	Ax	Mz (ft-K)
R	1238	69.33	151490.6	0.24	191.06	191.06	230.00	11.50	2.23	4899.83
5	2291	56	220015.9	0.34	277.49	468.55	230.00	11.50	2.20	7020.48
4	1890	43	134538.7	0.21	169.68	638.24	230.00	11.50	2.13	4156.40
3	1905	30	90148.8	0.14	113.70	751.93	230.00	11.50	2.04	2667.35
2	1863	15.66	42169.7	0.07	53.19	805.12	230.00	11.50	1.76	1076.47
				Total:	805.12					19820.53

Ax Mod 8	Ax Expansion	% DIFF
2.23	2.46	9.3
2.20	2.49	11.6
2.13	2.47	13.8
2.04	2.46	17.1
1.76	2.39	26.4
	Ave dec.	15.6

Finalized Structural System

Ultimately a structural system was finalized that met all design criteria that were set forth at the onset of the design scenario study. This system is characterized by the reduction in stiffness of BF-NS-2 with few member sizes that needed to be increased to accommodate strength requirements in BF-NS-1.

Figure 24 summarizes member size revisions in BF-NS-2. At all levels, members were either reduced in stiffness or kept the same to accommodate strength requirements in either the existing or expanded building layouts. As a result, eccentricity in the X-Direction was reduced at each level, which reduced the inherent torsional moment of the system. The effects of the extreme torsional irregularity in the expanded layout were reduced, but not removed with this system.



Existing Braces	Revised Braces
HSS6X6X3/8	HSS6X6X1/4
HSS8X8X3/8	HSS6X6X1/4
HSS8X8X3/8	Remain the same
HSS10X10X5/8	HSS8X8X1/2
HSS10X10X5/8	HSS8X8X1/2

Figure 24 BF-NS-2 Member Revision Summary

Figure 25 summarizes the revisions to member sizes in BF-NS-1. Two members needed to be increased to accommodate the strength requirements in the expanded building layout. Reductions to stiffness of BF-NS-2 nearly removed the necessity to revise these member sizes. Ultimately it was necessary to increase these member sizes. The revised braces remained within the column dimensions and did not require further study into architectural implications.

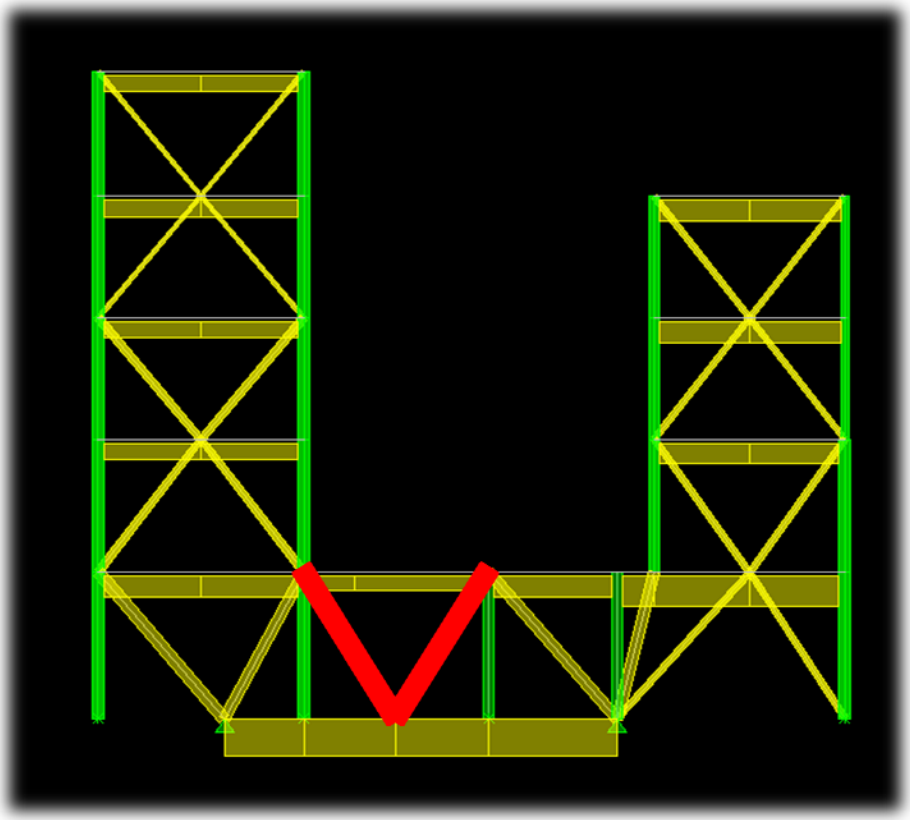


Figure 25 BF-NS-1 Member Size Revision
Change: HSS8X8X5/8 to HSS10X10X5/8 (Red)

All other structural elements were checked by hand using the member forces from the 3D ETABS model or studied in 2D models using SAP 2000. For the case of the moment frames, critical moments were checked using 2D modeling procedures considering the effects of live load patterns. In all cases the existing lateral system members had sufficient strength to resist loads applied in the expanded building layout. Story and building drift increased but did not exceed code limitations. Therefore, changes were not made to these elements.

Properties of the revised system, similar to those of modification 8, are summarized below. The finalized system was not as effective at reducing the effects of inherent torsion and the amplification factor as modified system 8. This was a result of limitations on member stiffness reduction at BF-NS-2 due to connection requirements of the Seismic Design Manual. Members within modified system 8, while meeting the required strength for the system, failed when the connections were detailed. This necessitated revisions which resulted in the increase of member sizes.

Center of Mass vs. Center of Rigidity

Story	XCM	YCM	XCR	YCR	Ex	Ey
STORY5	61	-81	86	-86	-25	5
STORY4	51	-68	90	-88	-39	20
STORY3	51	-68	85	-85	-34	18
STORY2	51	-68	76	-80	-25	12
STORY1	51	-68	58	-80	-7	12

Comparison of Eccentricity in the X-Direction

X-Direction Eccentricity			
Base	Expansion	Mod 7	% diff
6.8	-30.4	-24.8	18.2
-5.1	-49.7	-38.7	22.2
2.8	-45.2	-34.0	24.9
13.3	-40.8	-24.8	39.2
26.9	-20.8	-7.4	64.3
		Avg.	33.8

Loading Condition for Finalized System

Seismic Forces in the East/West Direction										
Level	w_x (kips)	h_x (ft)	$w_x h_x^k$	Cvx	Fx (kips)	Vx (Kips)	Bx (ft)	5% By (ft)	Ax	Mz (ft-K)
R	1238	69.33	151490.6	0.24	102.70	102.70	140.00	7.00	1.00	718.88
5	2291	56	220015.9	0.34	149.15	251.85	140.00	7.00	1.00	1044.05
4	1890	43	134538.7	0.21	91.20	343.05	140.00	7.00	1.00	638.43
3	1905	30	90148.8	0.14	61.11	404.16	140.00	7.00	1.00	427.79
2	1863	15.66	42169.7	0.07	28.59	432.75	140.00	7.00	1.11	222.12
				Total:	432.75					3051.28

Seismic Forces in the North/South Direction										
Level	w_x (kips)	h_x (ft)	$w_x h_x^k$	Cvx	Fx (kips)	Vx (Kips)	Bx (ft)	5% By (ft)	Ax	Mz (ft-K)
R	1238	69.33	151490.6	0.24	191.06	191.06	230.00	11.50	2.34	5141.52
5	2291	56	220015.9	0.34	277.49	468.55	230.00	11.50	2.34	7467.24
4	1890	43	134538.7	0.21	169.68	638.24	230.00	11.50	2.32	4527.15
3	1905	30	90148.8	0.14	113.70	751.93	230.00	11.50	2.27	2968.08
2	1863	15.66	42169.7	0.07	53.19	805.12	230.00	11.50	2.14	1308.90
				Total:	805.12					21412.90

Ax	Ax Expansion	% DIFF
2.34	2.46	4.9
2.34	2.49	6.0
2.32	2.47	6.1
2.27	2.46	7.7
2.14	2.39	10.5
	Ave dec.	7.0

Expanded Building Response

A requirement for this study was the compliance with drift standards for both wind and seismic loading. The building seismic response was checked in accordance with ASCE 7-05 to determine if the values met the 1.5% story drift requirement. In comparison to the existing building layout, seismic drift was increased substantially in each direction. Provided that the correction for real building period is used, all story drift ratios are within the limitations of the code. If period correction is not included, the maximum building drift in the Y-Direction increases from 1.23% to 2.35%. This value is now well beyond the drift limitation presented in Table 12.12-1 of ASCE 7-05. See the summary of the expanded building drift ratios below.

Drift Summary		Max Drift Percent		Δa (%)	Max. Drift (in.)		Δa (in.)
Story	Height	X	Y	1.5	X	Y	X/Y
5	13.25	0.59	1.23	OK	0.94	1.96	2.39
4	13	0.82	1.19	OK	1.28	1.86	2.34
3	13	0.78	1.14	OK	1.22	1.78	2.34
2	14.25	0.71	0.97	OK	1.21	1.66	2.57
1	15.75	0.41	0.31	OK	0.77	0.59	2.84

Total building drift under wind loading was checked at the top story of the building. This did not consider the effects of wind applied in each direction, only that that maximum building drift was compliant with the industry standard h/400. Again the maximum building drift was increased as a result of the expanded layout and additional loading on the structure, but was less than the limitation. See the maximum loading case and deflection below for building drift under wind loading.

Story	Point	Load	UX	UY	UTT	Δ=h/400
STORY5	664	0.563Wx+0.563Wy-0.563Mtx-0.563Mty	0.7771	1.0284	1.288988	2.1

Existing Building Layout – Modified Structural System Response

Additional to the requirements of the expanded building layout and loading conditions, the revised structural system was considered in the existing building layout. All members were checked determined to have sufficient strength in the existing building layout. The response for seismic and wind drift were checked to ensure that these values remained within acceptable standards.

All values met the code requirements for both wind and seismic drift. It is notable however that the wind drift did increase in the base building as a result of the revised structural system. This would require additional attention in an expansion scenario that is not structurally tied. The larger drift would need to be accommodated within the building expansion joint under these circumstances. See the summary of the seismic story drift and wind building drift below.

Drift Summary		Max Drift Percent		Δa (%)	Max. Drift (in.)		Δa (in.)
Story	Height	X	Y	1.5	X	Y	X/Y
5	13.25	0.54	0.69	OK	0.86	1.10	2.39
4	13	0.58	0.68	OK	0.90	1.06	2.34
3	13	0.60	0.65	OK	0.94	1.01	2.34
2	14.25	0.55	0.54	OK	0.94	0.92	2.57
1	15.75	0.30	0.23	OK	0.57	0.43	2.84

Story	Point	Load	UX	UY	UTT	Δ=h/400
STORY5	664	0.563Wx+0.563Wy-0.563Mtx-0.563Mty	0.5726	0.6063	0.8339	2.1

Connection Detailing

Important to the implementation of this design was the detailing of connections for the system. Given that the seismic loading condition was the controlling load case for both the existing and expanded building systems it was pivotal to meet the requirements of the Seismic Design Manual. Proper detailing of the system ensures the validity of the categorization of the system and the R-factor used throughout analysis. As discussed previously, on design iteration was rejected in the final detailing stages due to the inability of the elements to develop the required strength at the connections.

The connection that was studied was the central connection of the lower two story X brace configuration of BF-NS-2 seen in Figure 26. The connection allowed for the development of member forces induced by both building layouts. The analysis resulted in a 5/8” gusset plate that was 5’ long and 20” high on top and bottom of the W27X84 beam. All welds in the system were 7/16” fillet welds with brace members requiring 13” to on each side to develop the required forces. See Appendix C to see the full calculations for this connection. See Figure 27 for the connection detail.

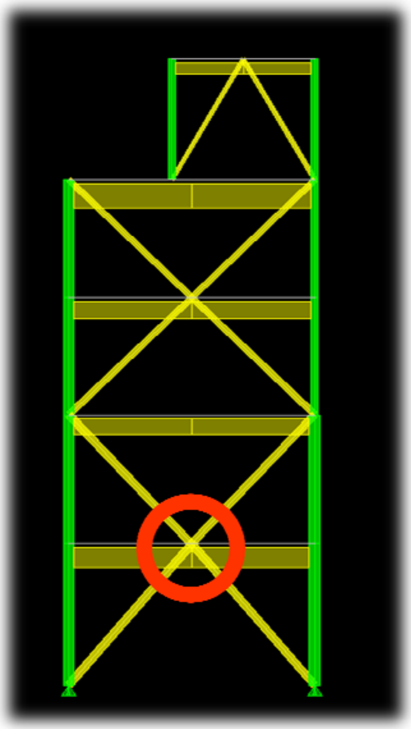


Figure 26 Connection Location at BF-NS-2

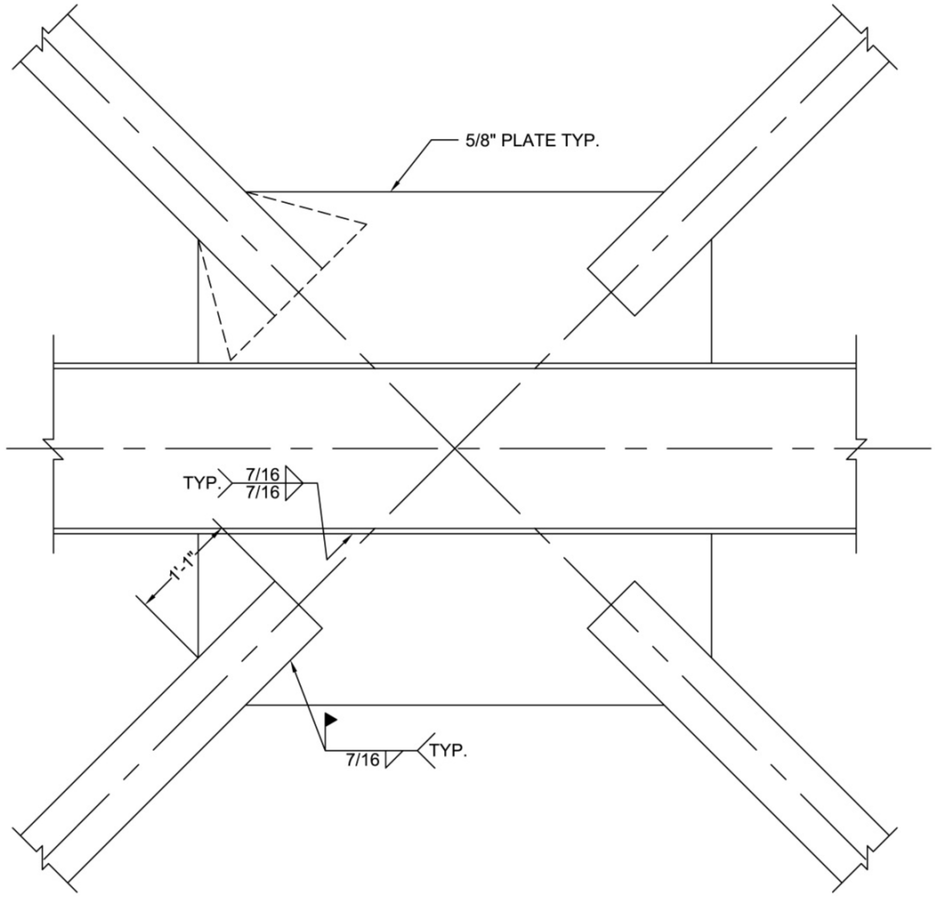


Figure 27 Connection Detail at BF-NS-2

Summary of Revised Structural System

Through minor revisions to the structural system, the building can now resist applied lateral loads in both the existing building layout, as well as a structurally tied expanded building layout. All strength and serviceability requirements were met in both scenarios. There are several issues that need to be addressed to determine the feasibility of this expansion scenario. One issued that needs to be addressed is the ability to transfer diaphragm forces from the expansion into the existing floor layout and lateral system. The second is the ability the façade to accommodate the substantial increase in story drift. Finally the third issue is the ability of this expansion to be constructed on site given the existing site constraints. These issues will be addressed in the following studies.

Façade Study - Breadth Study One

The façade of the Simmons College School of Management was one system that needed to be addressed as a result of the altered design parameters for the building. Due to the increase in story drift ratios, the system now needs to have a revised detail to accommodate this change. The façade connection at the building's west wall was also addressed to develop a new detail that would be more constructable and allow for the development of diaphragm forces in the expanded system. Finally a review of the thermal properties of the façade system was investigated to determine any necessary revisions to the system.

Glazing Detailing to Resist Earthquakes

As a result of the building expanded layout and loading conditions, there was a substantial increase in story drift ratio, from 0.54% to 1.23%. It was necessary to now revise the detailing requirements for the typical windows used in the building façade. To steps taken in this study followed the procedures presented in "Design of Architectural Glazing to Resist Earthquakes" by Richard Behr. The glass was detailed in order to provide adequate clearances to avoid glass to frame contact in a seismic event. Figure 28 shows the condition which is to be avoided during an earthquake. Two typical glazing panel sizes are repeated in the façade, a 2' by 11' panel and a 4' by 13' panel. The calculations assumed that the horizontal and vertical clearances would be equal in each detail. The 2' by 11' panel now requires a 3/16"clearance on all sides and the 4' by 13' panel requires a 5/16" clearance.

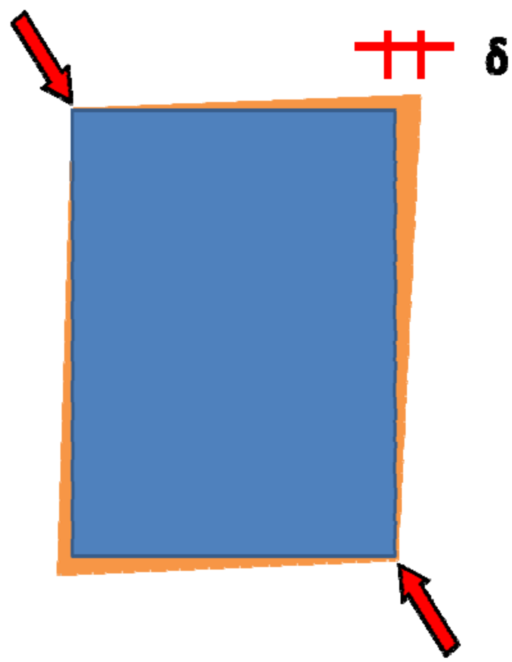


Figure 28 Seismic Drift Acting on Façade Glazing

Façade Connection Detail

A requirement of the structural system is that the lateral forces developed in the expansion area are able to be transferred through the diaphragm and into the lateral force resisting system. This necessitated a study of the façade connection at the west wall. Important considerations for this revised detail were the ability of diaphragm forces to be developed at this location, as well construction process and safety concerns.

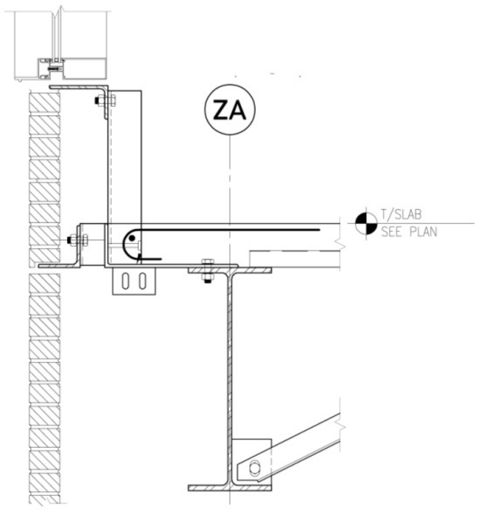


Figure 29 Existing Façade Detail

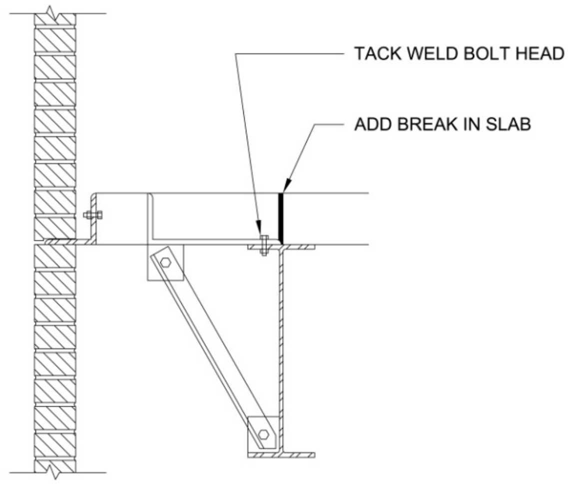


Figure 30 Revised Façade Detail

Figure 29 shows a typical connection at the west wall of the building, where the expansion is to take place. If unmodified, this detail requires that the slab be saw cut out of place to allow for the expansion diaphragm to be connected at this location. This can be a time consuming and labor intensive process. Therefore, a revised detail was proposed to allow for easier system disassembly and the development of the diaphragm connection. Figure 30 shows the revisions to the diaphragm connection detail. In order to allow for easy disassembly, a break was provided in the slab. It was then necessary to provide a diagonal angle to maintain stability in the slab extension. Bolt heads that provide the connection to the top of the wide flange would also now require a tack weld keep the bolts in place during disassembly. With this configuration, only the angle and nut at the top of the wide flange need to be removed in order to disassemble the system. The diaphragm connection is then able to be developed along the existing beam at the west wall.

Façade System Performance Assessment

An additional study of the thermal performance of the wall systems was conducted for typical wall sections of the building. Two parameters were checked to assess if there was a need for an alteration to the wall system configuration. H.A.M. Toolbox was the primary tool used to assess each system property that was investigated. Reference Appendix D for the system models used during this evaluation.

First, a dew point analysis was conducted to determine if the dew point would be reached on the interior of the wall system. In both wall systems that were checked, the winter condition dew point intersected the interstitial wall temperature within the spray on foam insulation. This does not present a problem for two reasons. First, the foam insulation is a closed cell material, which acts as a drainage plane for the system and will not allow for the penetration of water into the material. Second, if water does penetrate the material it will be able to drain out at the bottom flashing.

The second façade system property that was investigated was a condensation analysis. In both systems that were checked there was no condensation that developed in either summer or winter conditions. Therefore no alterations to the wall systems were recommended.

Construction Management - Breadth Study Two

The ability to construct the expansion was a key concern when performing the study of the expansion design scenario of the Simmons College School of Management. Constructability considerations pertaining to the disassembly of the façade and diaphragm connection were discussed in the previous section. Site access as well as physical constraints imposed by existing buildings, were both guiding considerations in the site layout. Figure 31 shows an aerial view of the site while construction was taking place for the existing building.

Figure 31 Aerial Image of Initial Construction Site

The initial design of the building included provisions at the plaza level to allow for crane loading. It was desired from the onset of the expansion study that the existing loading zone be used for the crane layout during construction of the expansion. The geometry of the expansion was determined with this site constraint in mind. See Figure 32 for the crane loading zone in relation to the existing building and expanded layout.

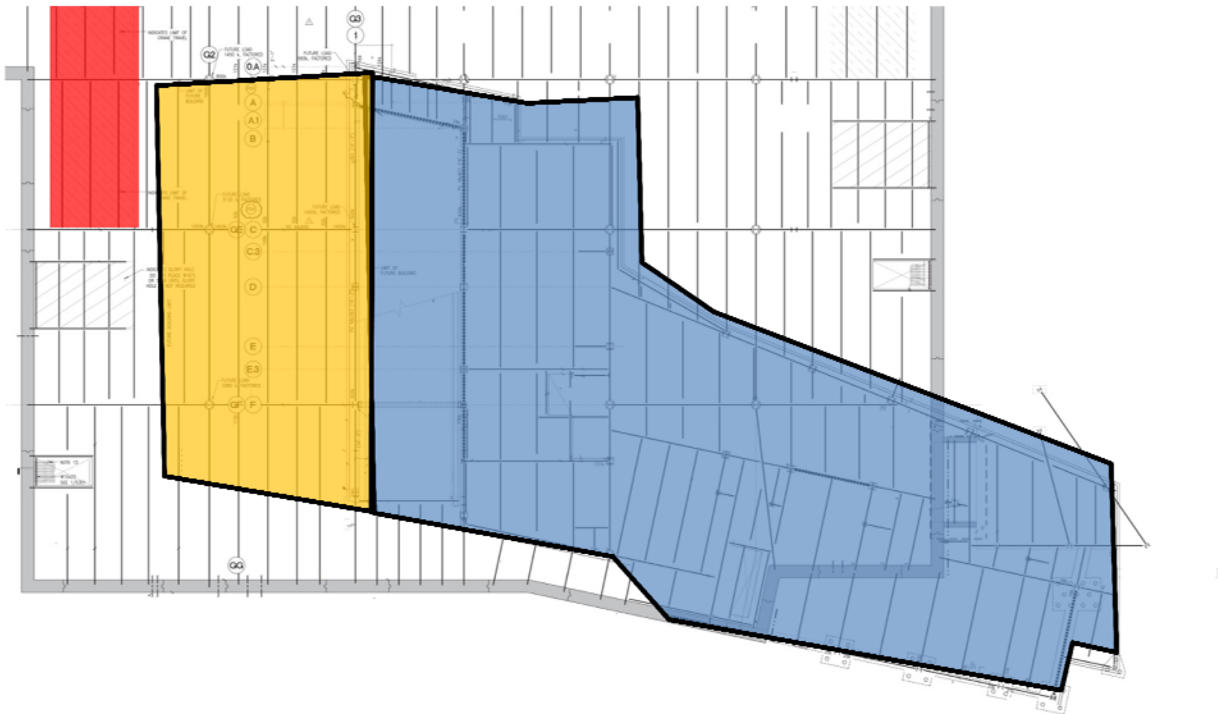


Figure 32 Construction Loading Zone for Crane (Red)
in relation to the Existing Building and Expanded Layout

The requirements for site access and material delivery were the next parameters for site layout that were evaluated. It needed to be determined if the campus access road would be able to be used to deliver materials to the site, or if additional accommodations needed to be made for deliveries. Figure 33 shows the unloading zone on the site access road and the required Crane swing radius. The furthest reach requirement for the crane given these conditions is 100'. Two options for cranes were considered at this time. One option is a stationary tower crane that could handle the required picks at the given distances. The second option is a small mobile crane that would be able to reach the required distances. See Appendix E for an example of swing radius for the AC40/2L mobile crane.

Material layout would likely need to be staged to the west of the expansion, between the crane and material delivery zone. Additional site requirements would exist in two locations. The zone to the north of the expansion at the plaza level would be used for temporary toilets as well as trash and recycling dumpsters. Areas in the parking garage would also need to be utilized for contractor trailers and construction worker parking.

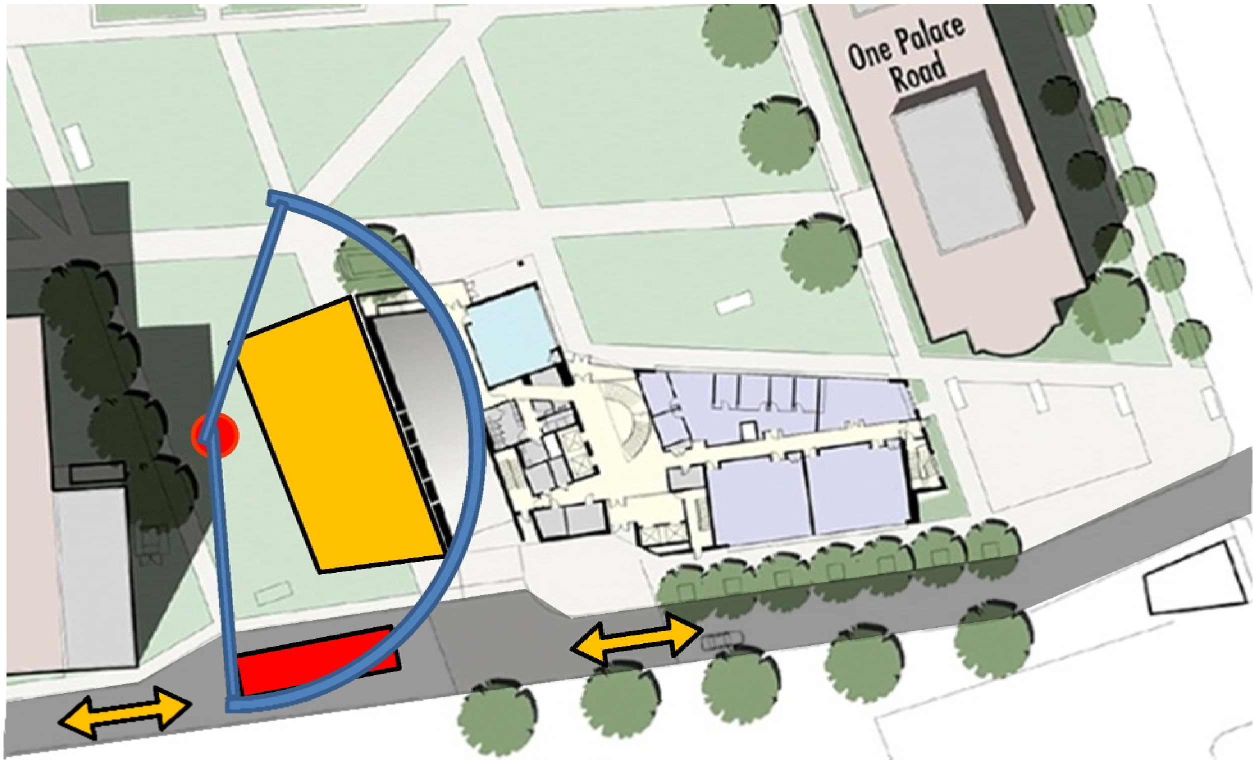


Figure 33 Site Layout Requirements

As a result of this study, it was determined that the proposed building expansion layout could be constructed with the existing site constraints. The existing construction loading zones can be utilized, removing the need for additional accommodations to be made in the plaza level structure. Site access would rely on the existing roadway crossing the campus. This would require coordination throughout the day with the school when road shut downs are necessary during deliveries. Ultimately the constructability of the expansion was considered to be feasible under exiting site conditions.

Master of Architectural Engineering Requirements

Integral to the completion of the expansion study will be analysis of the building through computer modeling. The additional load imparted on the system will cause significant torsional effects. The analysis of load effects caused by inherent and accidental torsion will be assessed through system properties of the 3D model. Two programs were used throughout the structural system investigation, ETABS v14 was used for the full 3D model and SAP2000 was used for individual frame analysis as necessary.

Detailing requirements of the finalized structural system were developed through connection design principles. Methods of hand calculation, as well as the use of design aids in the AISC Steel Construction Manual were utilized for this section.

The façade analysis and assessment was based on design principles developed through the coursework of several graduate level classes. Building Enclosures allowed for the proper assessment of the physical properties of the façade system while Building Failures was the basis for the constructability and good construction practice assessment.

Conclusion and Recommendations

The goal of this study was to determine the feasibility of an additional expansion scenario for the Simmons College School of Management. The problem was approached as if it were considered in the initial stages of design. A study of the structural system determined that with minimal revisions to the structure, a solution can be reached that satisfies the requirements of both the existing and expanded building layouts.

A façade study was conducted to ensure that the new design parameters could be accommodated in the revised system. Detailing requirements for typical glazing sizes in the building façade were revised as a result of increased seismic drift. Additionally, alterations to the west wall façade connection would need to be made to allow for the expansion to be tied into the existing structure.

A study of the expansion constructability was also conducted. It was determined through this evaluation that the expansion would be able to be constructed with few additional accommodations. Ultimately the additional expansion design scenario was determined to be feasible. It is therefore determined that with some additional system evaluation during the design process, an alternative expansion option could be provided for the building owners to consider.

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Appendix A: Building Information

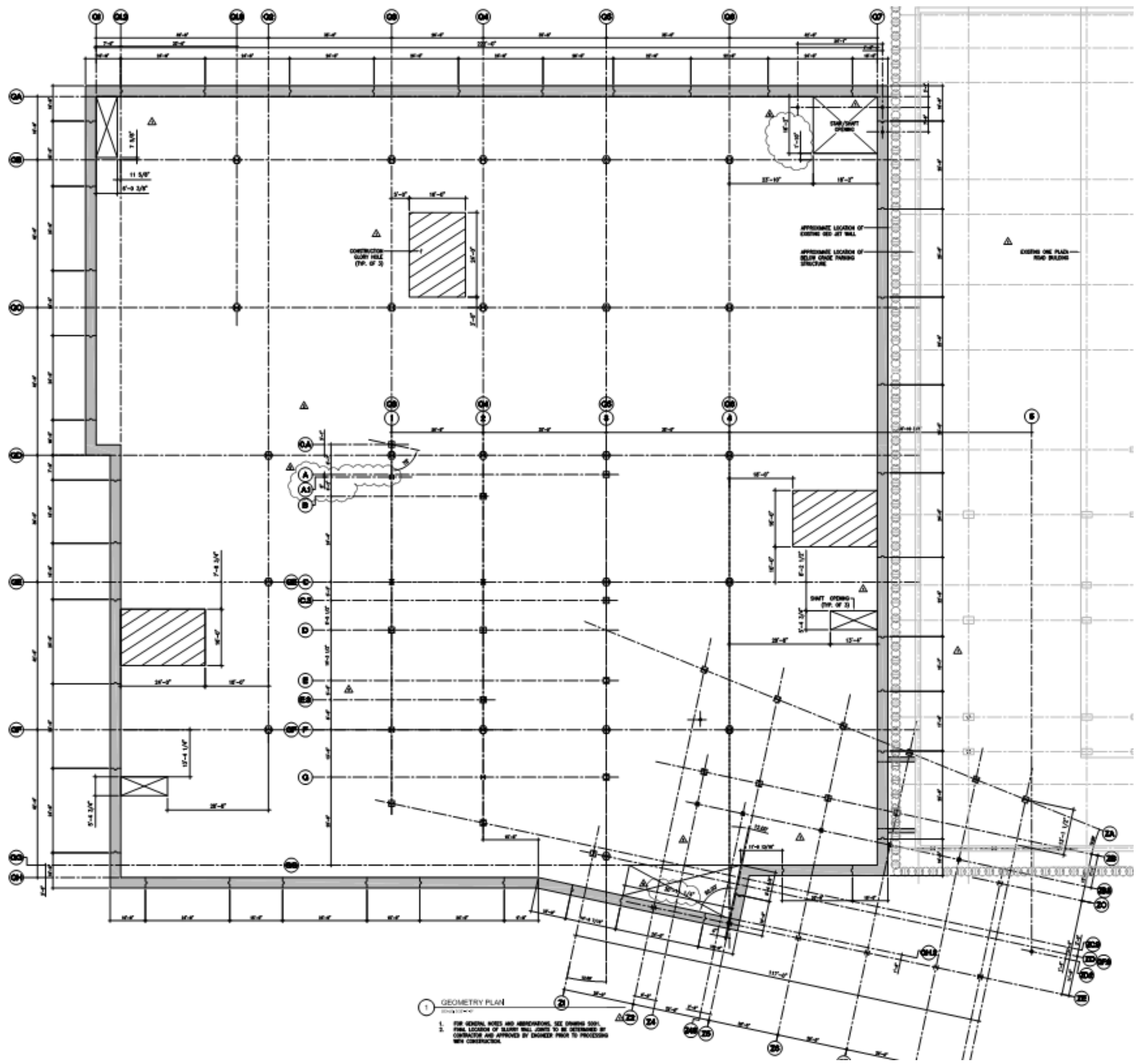

Figure 34 Sub Grade Parking Garage Layout

Lateral Frame Elements – Existing Building

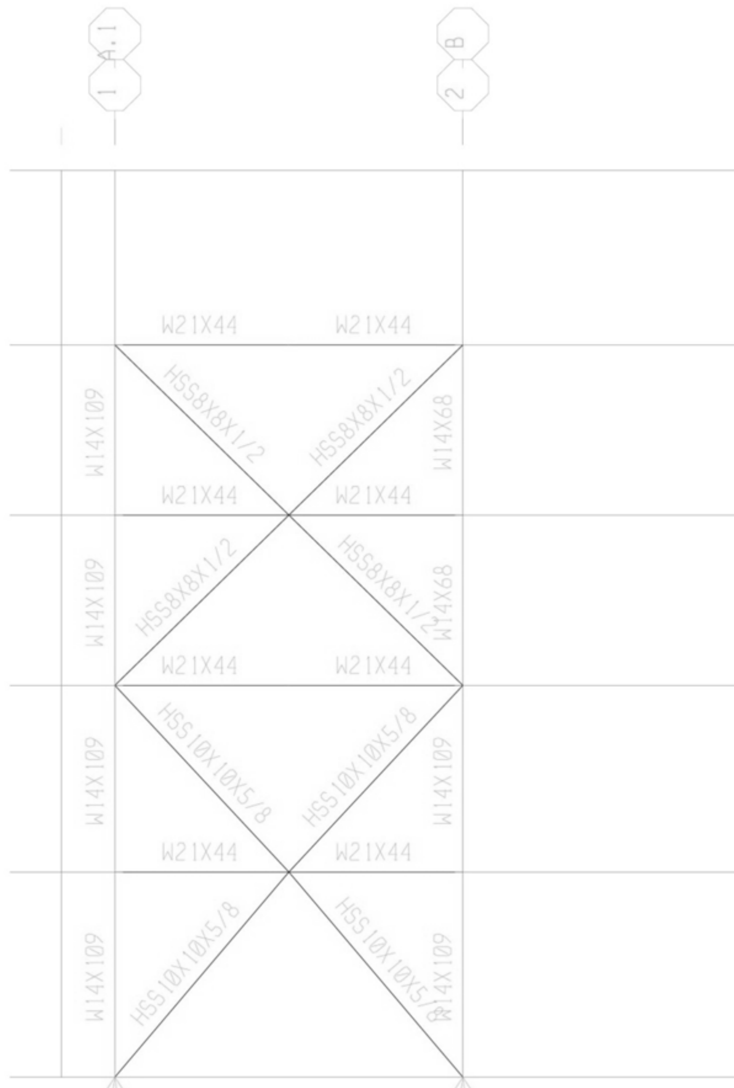

Figure 37 BF-EW-1

Figure 38 BF-EW-2

Figure 39 B/MF-EW-3

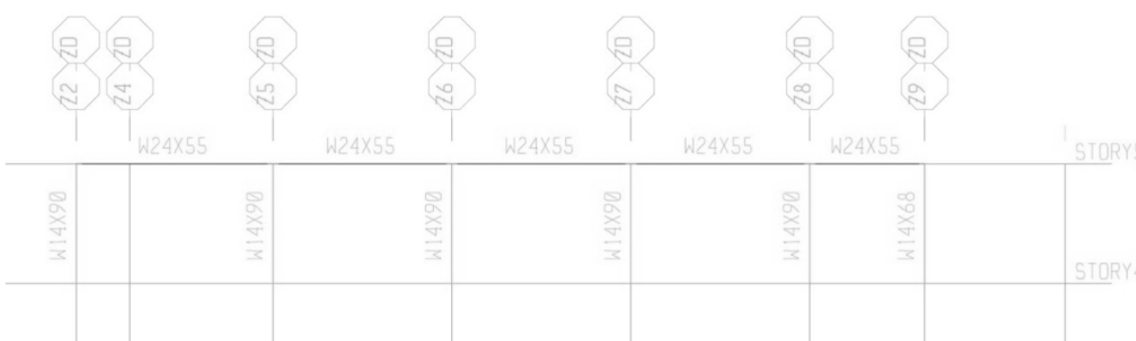

Figure 40 MF-EW-4

Figure 41 MF-EW-5

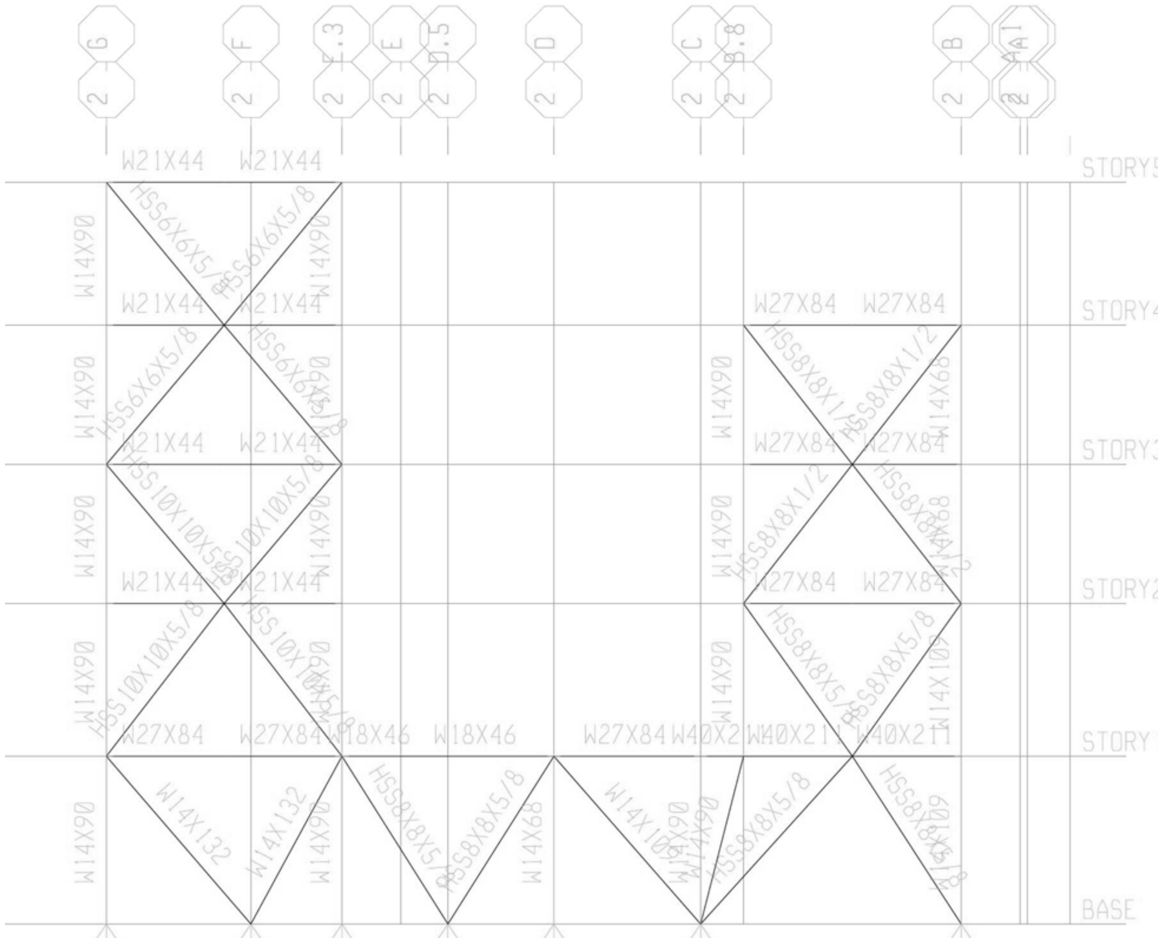

Figure 42 BF-NS-1

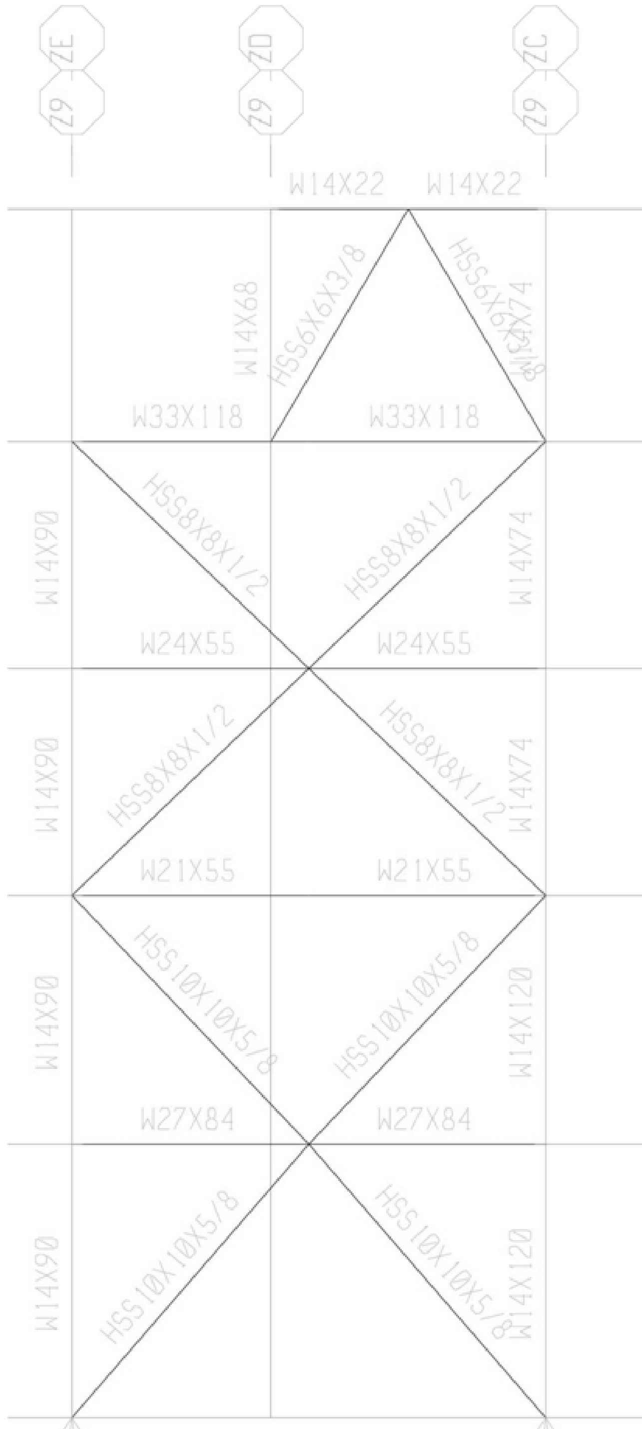

Figure 43 BF-NS-2

Appendix C: Lateral Loads Existing Building

Wind Loads

Wind Loads - ASCE 7-05
Mean h = 70.5' > 60 ∴ Cannot Apply MFWRS
Method 2 - Analytical Procedure § 6.5

Simmons College - SOM
Design Loads
9/28/09
KTW

Location: Boston, Ma., Exposure B

Basic Wind Speed	$V = 120$ mph	Figure 6-1
Wind Directionality Factor	$K_d = 0.85$	Table 6-4
Occupancy Category: III		Table 1-1
Importance Factor I:	$I = 1.15$	Table 6-1
Not Located on Hill, Ridge, Escarpment	$K_{zt} = 1.0$	§ 6.5.7.1
Exposure Coefficient	$K_{z,h} = 0.89$	Table 6-3

$q_p = 0.00256\, K_z K_{zt} K_d V^2 I$
$= 0.00256(0.89)(1.0)(0.85)(120)^2(1.15)$
$q_p = 32$ psf

$q_p = 32$ psf — Eq. 6-15

Parapet Net Pressure Coefficient
WW $GC_{pn} = 1.5$ — § 6.5.12.2.4
LW $GC_{pn} = -1.0$ — " "

Design Pressure on Parapet
Windward: $p_p = q_p GC_{pn} = (32)(1.5)$
$p_p = 48$ psf

$p_p = 48$ psf (WW)

Leeward: $p_p = q_p GC_{pn}$
$= 32(-1.0)$

$p_p = -32$ psf (LW)

Natural Frequency
EW - Steel Moment Resisting Frame
$n_1 = 22.2/H^{0.8} = 22.2/(70.5)^{0.8} = 0.74$

NS/EW - Steel braced frame
Use (C6-17 (average value)
$n_1 = 100/H = 100/70.5 = 1.42 \geq 1$

Check Lower Bound
$n_1 = 75/70.5 = 100/70.5 = 1.06 \geq 1$

Since the average value and Lower bound are both greater then 1:

ASSUME STRUCTURE IS RIGID.

$g_Q = g_v = 3.4$

$\bar{z} = 0.6h = 0.6(70.5)$
$\bar{z} = 42.3$ ft $> z_{min} = 30'$

$\bar{z} = 42.3$ ft.

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{42.3}\right)^{1/6}$
$= 0.29$

$I_{\bar{z}} = 0.29$

WIND LOADS 2

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{42.3}{33}\right)^{(1/3)}$$

$$L_{\bar{z}} = 347.6'$$

$L_{\bar{z}} = 347.6'$

$$Q_{E\text{-}W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$Q_{E\text{-}W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{95 + 70.5}{347.6}\right)^{0.63}}} =$$

$Q_{E\text{-}W} = 0.85$

$$Q_{N\text{-}S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{170 + 70.5}{347.6}\right)^{0.63}}} = 0.82$$

$Q_{N\text{-}S} = 0.82$

$$G = 0.925\left[\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$G_{EW} = 0.925\left[\frac{1 + 1.7(3.4)(0.29)(0.85)}{1 + 1.7(3.4)(0.29)}\right] = 0.84$$

$G_{EW} = 0.84$

$$G_{N\text{-}S} = 0.925\left[\frac{1 + 1.7(3.4)(0.29)(0.82)}{1 + 1.7(3.4)(0.29)}\right] = 0.82$$

$G_{NS} = 0.82$

Velocity Pressure Coefficients

Height Above Ground	K_z
70	0.89
60	0.85
50	0.81
40	0.76
30	0.70
25	0.66
20	0.62
< 15	0.57

WIND LOADS 3

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$
$= 0.00256\, K_z (1.0)(0.85)(120^2)(1.15)$
$= 36.03\, K_z$

Height Above Ground	K_z	q_z (psf)
70	0.89	32.1
60	0.85	30.6
50	0.81	29.2
40	0.76	27.4
30	0.70	25.2
25	0.66	23.8
20	0.62	22.3
<15	0.57	20.5

Pressure Coefficient (E-W)

Windward Wall: $C_p = 0.8$ w/ q_z

Leeward Wall
$(L/B) = (170/95) = 1.8 \Rightarrow 2$ $C_p = -0.3$ w/ q_h

Side Walls $C_p = -0.7$ w/ q_h

Roof Pressures
$h/L = 70.5/170 = 0.41 < 0.5$

from WW edge to $h = 70.5$ $C_p = -0.9, -0.18$ w/ q_h
from 70.5 to $2h = 141$ $C_p = -0.5, -0.18$ w/ q_h
from 141 to 170 $C_p = -0.3, -0.18$ w/ q_h

Pressure Coefficients (N-S)

Windward Wall $C_p = 0.8$ w/ q_z

Leeward Wall
$(L/B) = (95/170) = 0.56$ $C_p = -0.5$ w/ q_h

Side Walls $C_p = -0.7$ w/ q_h

Roof Pressures
$h/L = 70.5/95 = 0.74$
$(h/2)(w) = 6000 \Rightarrow$ Reduction factor $= 0.9$
from WW edge to $h/2 = 35.25$ $C_p = -1.2, -0.18$ w/ q_h
from 35.25 to 95 $C_p = -0.7, -0.18$ w/ q_h

Windward walls, side walls, leeward walls, roofs
$q_i = q_h = 32.1$ psf

Internal Pressure Coefficient $GC_{pi} = \pm 0.18$

EAST - WEST

Windward Walls
$p_z = q_z G C_p - q_h GC_{pi}$
$= q_z (0.84)(0.8) - 32.1(\pm 0.18) = 0.672\, q_z \pm 5.78 = p_z$

Leeward Side walls & Roof
$p_h = q_h G C_p - q_h (GC_{pi})$
$= C_p (32.1)(0.84) - (32.1)(\pm 0.18) = 26.96\, C_p \pm 5.78 = p_h$

WIND LOADS 4

North South

Windward Walls

$p_z = q_z(0.82)(0.8) - 32.1(\pm 0.18)$

$p_z = 0.656\, q_z \pm 5.78$

Leeward, Side walls & Roof

$p_h = C_p(32.1)(0.82) \pm 5.78$

$p_h = 26.32\, C_p \pm 5.78$

Seismic Loads

SEISMIC LOADS 1

Building Data
Location : Boston, Ma (Latitude 42.35; Longitude -71.1°)
Soil Classification : S_3 Mass. State Bldg. Code, Site Class E Assumed as a conservative approximation.
Occupancy : III
Material : Structural Steel
Structural System
N-S : Ordinary Concentric Braced Frames
E-W : Dual System, Ordinary Moment Resisting frames with Ordinary Concentric Braced Frames.

Seismic Ground Motion Values
Mapped Accelerations : USGS Ground Motion Parameter Calculator $S_s = 0.277$
$S_1 = 0.068$

Soil Modified Accelerations
Site Class E, $S_s = 0.277$ Table 11.4-1 interpolation $F_a = 2.0$
Site Class E, $S_1 = 0.068 < 0.1$ Table 11.4-2 $F_v = 3.5$
$S_{MS} = F_a S_s = 2.0(0.277) = 0.55$ $S_{MS} = 0.55$
$S_{M1} = F_v S_1 = 3.5(0.068) = 0.24$ $S_{M1} = 0.24$

Design Accelerations
$S_{DS} = 2/3\, S_{MS} = 2/3(0.55) = 0.37$ $S_{DS} = 0.37$
$S_{D1} = 2/3\, S_{M1} = 2/3(0.24) = 0.16$ $S_{D1} = 0.16$

Determine SDC
Check if $T_a < 0.8 T_s$
N-S Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec
E-W Direction
$T_a = C_t h_n^x = 0.02(69.25)^{0.75} = 0.48$ sec $T_a = 0.48$ sec
$T_s = S_{D1}/S_{DS} = 0.16/0.37 = 0.43$ $T_s = 0.43$

T_a is not less than $0.8\,T_s$ ∴ use 11.6-1 & 11.6-2
Table 11.6-1 SDS = C
Table 11.6-2 SDS = C SDC = C

Determine Analytical Process
Check if $T < 3.5\,T_s$
$3.5\,T_s = 3.5(0.43) = 1.5 > T$

Determine if the Structure is Regular
Determine Response Modification Coefficient
E-W Direction (E1) No limit $R_{EW} = 6$
N-S Direction (B4) No Limit $R_{N-S} = 3.25$
Importance Factor
$I = 1.25$ $I = 1.25$
Long term period
$T_L = 6$ sec $T_L = 6$ sec

SEISMIC LOADS 2

Seismic Response Coefficient

E-W Direction

$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{(6/1.25)} = 0.077$

$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.16}{(0.48)(6/1.25)} = \underline{0.069}$

$\geq C_s = 0.01$

E-W: $C_s = 0.069$

N-S Direction

$C_s = \frac{S_{DS}}{R/I} = \frac{0.37}{3.25/1.25} = \underline{0.14}$

$\leq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.37}{(0.48)(3.25/1.25)} = 0.28$

$\geq C_s = 0.01$

N-S $C_s = 0.14$

Effective Seismic Weight

See Spread Sheet

$W = 7{,}820^k$

Base Shear

N-S Direction

$V = C_s W = 0.14(7{,}820) = 1095^k$

$V_{N\text{-}S} = 1095^k$

E-W Direction

$V = C_s W = 0.069(7{,}820) = 540^k$

$V_{EW} = 540^k$

Exponent for Structural Period

$T_{EW} = T_{N\text{-}S} < 0.5 \quad \therefore \; k = 1.0$

Appendix C: Modified Structural System Calculations

Wind Load Calculation – Flexible Structure

Wind Loads – ASCE 7-05 — MODIFIED LAYOUT / UNMODIFIED LAT SYSTEM

Method 2 – Analytical Procedure.

Location: Boston, MA.

Basic Wind Speed	$V = 120$ mph	Figure 6-1
Wind Directionality Factor	$K_d = 0.85$	Table 6-4
Occupancy Category III	III	Table 1-1
Importance Factor	$I = 1.15$	Table 6-1
Exposure Category	Category B	§ 6.5.6
Topographic Factor	$K_{zt} = 1.0$	§ 6.5.7.2
Exposure Coefficient	$K_{z,h} = 0.89$	Table 6-3

Velocity Pressure

$q_h = 0.00256\, K_z K_{zt} K_d V^2 I$

$q_h = 0.00256(0.89)(1.0)(0.85)(120)^2(1.15)$

$q_h = 32$ psf

$q_z = 36.03\, K_z$

$q_h = 32$ psf — Eq. 6-15

$q_z = 36.03\, K_z$

Period of Vibration — $T_x = 1.15$ s

Natural Frequency $n_1 = \frac{1}{1.15} =$ — $n_1 = 0.87$

∴ Flexible Structure.

Gust Factor:

$g_Q = 3.4$

$g_v = 3.4$

$g_r = \sqrt{2\ln(3{,}600\, n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600\, n_1)}}$ — $g_r = 4.15$

$\bar{z} = 0.6h = 0.6(70) = 42'$ — $\bar{z} = 42'$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{(1/6)} = 0.3\left(\frac{33}{42}\right)^{(1/6)}$ — $I_{\bar{z}} = 0.288$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{42}{33}\right)^{(1/3)}$ — $L_{\bar{z}} = 347'$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ — $Q_{EW} = 0.827$, $Q_{NS} = 0.797$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$

$= 0.45\left(\frac{42}{33}\right)^{(0.25)}(120)\left(\frac{88}{60}\right)$ — $\bar{V}_{\bar{z}} = 84$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.87(347)}{84}$ — $N_1 = 3.59$

$R_n = \frac{7.47\, N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.59)}{(1 + 10.3(3.59))^{5/3}}$ — $R_n = 0.0625$

$\eta = 4.6\, n_1 h / \bar{V}_{\bar{z}} = 4.6(0.87)\, 70/84$ — $\eta = 3.34$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ — $R_h = 0.255$

$\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.87)140/84$ $\qquad \eta_B = 6.67$

$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ $\qquad R_B = 0.139$

$\eta_L = 15.4 n_1 L / \bar{V}_{\bar{z}}$ $\qquad \eta_L = 36.7$

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ $\qquad R_L = 0.0269$

Assume 1% Damping $\qquad \beta = 0.01$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ $\qquad R = 0.347$

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$ $\qquad G_{f\,EW} = 0.884$ $\qquad G_{f\,NS} = 0.869$

Design Pressures

Parapet $z = 72'$ $K_z = 0.90$

$q_p = 32.43$

$p_p = q_p\, GC_{pn}$ Windward $GC_{pn} = +1.15$ $p_p = 48.6$

Leeward $GC_{pn} = -1.0$ $p_p = -32.4$

Windward Walls

$q_z = K_z\, 36.03$ $C_p = 0.8$

$GC_{pi} = \pm 0.18$

Velocity Pressure Coefficient

Height Above Ground (ft)	K_z
70	0.89
60	0.85
50	0.81
40	0.76
30	0.70
25	0.66
20	0.62
<15	0.57

$p_z = q_z\, G_f\, C_p - q_h\,(GC_{pi})$

Leeward Walls

EW $L/B = 230/140 = 1.64$ $\Rightarrow C_p = -0.4$

NS $L/B = 140/230 = 0.6$ $\Rightarrow C_p = -0.5$

$GC_{pi} = \pm 0.18$

$p_h = q_h\, G_f\, C_p - q_h\, GC_{pi}$

Side Walls

$GC_{pi} = \pm 0.18$

$C_p = -0.7$

$p_h = q_h G_f\, C_p - q_h\, GC_{pi}$

Roof Pressure.

Seismic Load Calculation

Seismic Loads - Expanded Building

Design Parameters - ASCE 7-05

$S_S = 0.0277$ $\quad F_a = 2.0$

$S_1 = 0.068$ $\quad F_v = 3.5$

$I = 1.25$ $\quad T_L = 6s$

Occupancy Category: III

Seismic Design Category: C

$W = 10{,}064^k$

X-Direction	Y-Direction
$R = 6$	$R = 3.25$
$C_d = 5$	$C_d = 3.25$
$C_t = 0.02$	$C_t = 0.02$
$h_n = 70'$	$h_n = 70$
$x = 0.75$	$x = 0.75$
$C_u = 1.6$	$C_u = 1.6$

$T_a = 0.48$

$T_s = 0.043$

$T_x = 1.4488$

$T_y = 1.0872$

$T_z = 0.6677$

Story Shear X-Direction

$$T \le \begin{cases} C_u T_a = 1.6(0.48) = 0.768 & \leftarrow \text{Controls} \\ T_b = 1.4488 \end{cases}$$

$$C_s = \min \left| \begin{array}{ll} S_{DS}/(R/I) & = 0.077 \\ S_{D1}/[(T \cdot R)/I] & = 0.043 \quad \leftarrow \text{Controls} \\ S_{D1} \cdot R/[T^2 \cdot R/I] & = 0.339 \end{array} \right.$$

Base Shear

$$V_{bx} = C_s \cdot W = 0.043(10{,}064) = 433^K$$

Story Shear Y-Direction

$$T = C_u T_a = 0.768$$

$$C_s = \min \left| \begin{array}{ll} S_{DS}/(R/I) & = 0.14 \\ S_{D1}/[(T \cdot R)/I] & = 0.08 \quad \leftarrow \text{Controls} \\ S_{D1} \cdot R/[T^2 \cdot R/I] & = 0.339 \end{array} \right.$$

Base Shear

$$V_{by} = C_s \cdot W = 0.08(10{,}064) = 805^k$$

Seismic Drift Calculations

Drift Determination - Base Building, Existing Structure 3/28

Period of Vibration (w/ P-Delta)

$T_x = 1.2222$ s
$T_y = 0.8281$ s
$T_z = 0.6066$ s

Building Story Drifts

X-Direction

$$Drift = \frac{C_d}{I}(\%\,drift)\left(\frac{C_u T_a}{T_x}\right)$$

$$Drift = \frac{5}{1.25}(\%\,drift)\left(\frac{0.768}{1.2222}\right)$$

$$Drift = 2.51(\%\,drift)$$

Y-Direction

$$Drift = \frac{C_d}{I}(\%\,drift)\left(\frac{C_u T_a}{T_y}\right)$$

$$Drift = \frac{3.25}{1.25}(\%\,drift)\left(\frac{0.768}{0.8281}\right)$$

$$Drift = 2.41(\%\,drift)$$

Percent Story Drift

Story	X-Direction	Y-Direction	Limit = 1.5%
5	0.45	0.54	OK
4	0.50	0.53	
3	0.51	0.50	
2	0.48	0.43	
1	0.28	0.19	

Exp. Bldg. - MoD Sys 9

Seismic Story Drift - MoD 9 4/4

Period of Vibration - P-Delta

$T_x = 1.4648$ s
$T_y = 1.0944$ s
$T_z = 0.7448$ s

$$\text{Drift} = \frac{C_d}{I}\,(\%\,\text{drift})\,\frac{C_u T_a}{T_b}$$

Building Drift

X - Direction

$$\text{Drift} = \left(\frac{5}{1.25}\right)(\%\,\text{drift})\left(\frac{0.768}{1.4648}\right)$$
$$\text{Drift} = 2.10\ (\%\,\text{drift})$$

Y - Direction

$$\text{Drift} = \left(\frac{3.25}{1.25}\right)(\%\,\text{drift})\left(\frac{0.768}{1.0944}\right)$$
$$\text{Drift} = 1.82\ (\%\,\text{drift})$$

Percent Story Drift

Story	X-Direction	Y-Direction	Limit 1.5%
5	0.59	1.23	OK
4	0.82	1.19	
3	0.78	1.14	
2	0.71	0.97	
1	0.41	0.31	

Base Bldg - ModSys 9

Seismic Story Drift - MoD 9 4/4

Period of Vibration - w/ P-Delta
T_x = 1.2073 s
T_y = 0.8224 s
T_z = 0.6584

$\%Drift = \frac{C_d}{I}(\%drift)\frac{C_u T_a}{T_b}$

Bldg Drift

X-Direction
$Drift = \left(\frac{5}{1.25}\right)(\%drift)\left(\frac{0.768}{1.2073}\right)$
$Drift = 2.54\ (\%drift)$

Y-Direction
$Drift = \left(\frac{8.25}{1.25}\right)(\%drift)\left(\frac{0.768}{0.8224}\right)$
$Drift = 2.43\ (\%drift)$

Percent Story Drift

Story	X-Direction	Y-Direction	Limit 1.5%
5	0.54	0.69	OK
4	0.58	0.68	
3	0.60	0.65	
2	0.55	0.54	
1	0.30	0.23	

Connection Detail

MOD SYSTEM 9

Connection Detail – See Example 3.4, Seismic Design Manual 4/3 page 1

$S_{DC} = C$

$\Omega_o = 2.0$

$\rho = 1.0$

$S_{DS} = 0.57$

HSS 8x8x½ A500 Grade B.

$R_y = 1.4$

Required Tension Strength, § 14.4

$T_u = R_y F_y A_g$

$T_u = (1.4)(46)(13.5)$

$T_u = 869.4^K$

Not Greater than actual Load Amplified by Ω_o

$Q_E = 202^K$

$P_u = \Omega_o Q_E = 2.0(202)$

$P_u = 404^K$ ← Required tensile Strength.

Brace to Gusset weld (Set, Weld Strength = Shear Rupture, Base Metal)

$\phi\left(\frac{1}{\sqrt{2}}\right)\left(\frac{D}{16}\right)(0.60\ F_{EXX}) = \phi\left(0.60\ F_{u_{HSS}}\right) t_{des}$

$D \le \frac{F_u \times t_{des}}{3.09} = \frac{58\ (0.465)}{3.09}$

$D \le 8.7$ sixteenths (Cannot be greater than t_{des})

→ USE (4) 7/16" welds

Weld length

$l_w \ge \frac{P_u}{4(1.392)D} = \frac{404}{4\ (1.392)7}$

$l_w = 10.4"$

→ Use 13" welds

Weld Strength

$\phi R_n = \phi F_w A_w$

$\phi R_n = 1.392\ (4)(7)(13)$

$\phi R_n = 506.7^K > 404^K$

Minimum Gusset Plate Thickness, Weld Rupture

$t_{min} = \frac{P_u}{2\phi(0.6F_u)L}$

$t_{min} = \frac{404}{2(0.75)(0.6)(58)(13)}$

$t_{min} = 0.595"$ ⇒ USE 5/8" PL

Shear Lag Rupture for Brace

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page 2

$A_n = A_g - 2(5/8 + 1/8) t_{w}$

$A_n = 13.5 - 2(0.75)(0.465)$

$A_n = 12.8 \text{ in}^2$

$\bar{X} = \dfrac{B^2 + 2BH}{4(B+H)} = \dfrac{3B}{8}$

$\bar{X} = 3''$

$U = 1 - \dfrac{\bar{X}}{L} = 1 - \dfrac{3}{13}$

$U = 0.77$

$A_e = UA_n = 0.77(12.8)$

$A_e = 9.86 \text{ in}^2$

$\phi P_n = \phi F_u A_e = 0.75(58)(9.86)$

$\phi P_n = 429^k > P_u = 404$ $\therefore$ ok

Whitmore Section (Assume only Properties of PL)

$\phi R_n = \phi F_y A_g$

$A_g = t(2 l_w \tan(30°) + B) = (5/8)(23)$

$A_g = 14.4$

$\phi R_n = 0.9(36)(14.4)$

$\phi R_n = 466.6^k > 404^k$ $\therefore$ ok

BRACE CONNECTION, Center X

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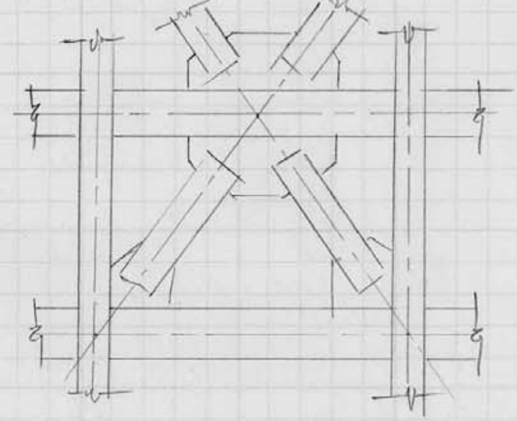
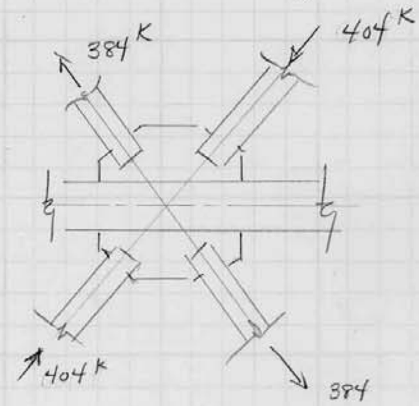
page 3





Forces – Shear

$V = (404 + 384)(1/\sqrt{2}) = 556^{K}$

Tension

$T = (404 - 384)(1/\sqrt{2}) = 16^{K}$

Moment

$M = V\left(\frac{d_b}{2}\right) = 556\left(\frac{24.8}{2}\right) = 6894.4^{\text{''-K}}$

Gusset to Beam Weld

$S_w = \frac{l_w^2}{6} = \frac{(61)^2}{6} = 620 \text{ in}^3/\text{in}$

$f_v = \frac{V}{l} = \frac{556}{61} = 9.1 \text{ K/in}$

$f_a = \frac{T}{l} = \frac{16}{61} = 0.26 \text{ K/in}$

$f_b = \frac{M}{S_w} = \frac{6894.4}{620} = 11.12 \text{ K/in}$

$f_{peak} = \sqrt{f_v^2 + (f_a + f_b)^2} = \sqrt{9.1^2 + (0.2 + 11.12)^2}$

$f_{peak} = 14.6 \text{ K/in}$

$f_{avg} = \frac{1}{2}\left[\sqrt{f_v^2 + (f_a - f_b)^2} + \sqrt{f_v^2 + (f_a + f_b)^2}\right]$

$f_{avg} = 14.4 \text{ K/in}$

$\frac{f_{peak}}{f_{avg}} = \frac{14.6}{14.4} = 1.00 < 1.25$

$\therefore f_r = 1.25 f_{avg} = 1.25(11.1) = 18$

$D \geq \frac{18}{2(1.392)} = 6.5$

Use 7/16" welds each side.

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page 4

Compression Buckling - Gusset Plate.
Avg. Unbraced Length = 12" (along brace axis)

$r = \frac{t}{\sqrt{12}} = \frac{(5/8)}{\sqrt{12}} = 0.18$

$\frac{KL}{r} = \frac{0.65\,(12")}{0.18} = 43.3$

$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29000)}{(43.3)^2} = 152.65$

$F_{cr} = 32.6$ ksi

Required Whitmore Width

$L_{w_{min}} = \frac{P_u}{\phi F_{cr}\, t_g} = \frac{404}{(0.9)(32.6)(5/8)} = 22"$

$L_{w_{actual}} = 23" > L_{w\,min} = 22"$ ∴ ok

Beam Web Yielding, Check Beam Yield vs. Peak tensile Load, Quick Check

$\phi F_{yw} t_w = 0.9\,(50)(0.46) = 20.7$ k/in > 14.6 k/in ∴ ok

Beam Web Crippling.

$L_t = \left(\frac{f_t}{f_t + f_c}\right)(L_g) = \left(\frac{7.76}{7.76 + 8.2}\right)72$

$L_t = 35"$

$R_u = \frac{1}{2}(L_g - L_t) f_c = \frac{1}{2}(72 - 35)(8.2) = 151.7^k$

$R_n = 0.8\, t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$

$= 0.8\,(0.46)^2\left[1 + 3\left(\frac{35}{24.8}\right)\left(\frac{0.46}{0.64}\right)^{1.5}\right]\sqrt{\frac{29000\,(50)(0.64)}{0.46}}$

$R_n = 860.7^k$

$\phi R_n = 0.75(860.7) = 645.5^k > 151.7$

Appendix D: Façade Calculations

Seismic Drift Detailing

FAÇADE STUDY - MOD SYS 9 4/

Source - "Design of Architectural Glazing to Resist Earthquakes"

Max. Drift = 0.0123 ⇒ 1.23%

$\Delta_i = 0.0123\ h_s$

$D_p = 0.0123\ h_w$

① Typical Window 1 - 2' x 11'

$D_p = 0.0123\,(11\,(12/1)) = 1.624''$

$\Delta_{fallout} \geq 1.25 \cdot I \cdot D_p$

$\geq 1.25(1.25)(1.624)$

$\Delta_{fo} \geq 2.54'$

Assume $C_1 = C_2$ (vertical = horizontal gap)

$D_{clr} > \Delta_{f.o.}$

$D_{clr} = 2c_1\left(1 + \frac{h_p\,c_2}{b_p\,c_1}\right)$

$2.54' = 2c_1\left(1 + \frac{11}{2}\right)$

$C = 0.195'' \Rightarrow C_1 = C_2 = 1/4''$ clearance all sides

② Typical Window 2 - 4' x 13'

$D_p = 0.01213\,(13\,(12/1)) = 1.92''$

$\Delta_{fo} \geq 1.25(1.25)(1.92)$

$\Delta_{fo} \geq 3.0''$

Assume $C_1 = C_2$

$D_{clr} \geq \Delta_{fo}$

$3.0'' = 2c_1\left(1 + \frac{13}{4}\right)$

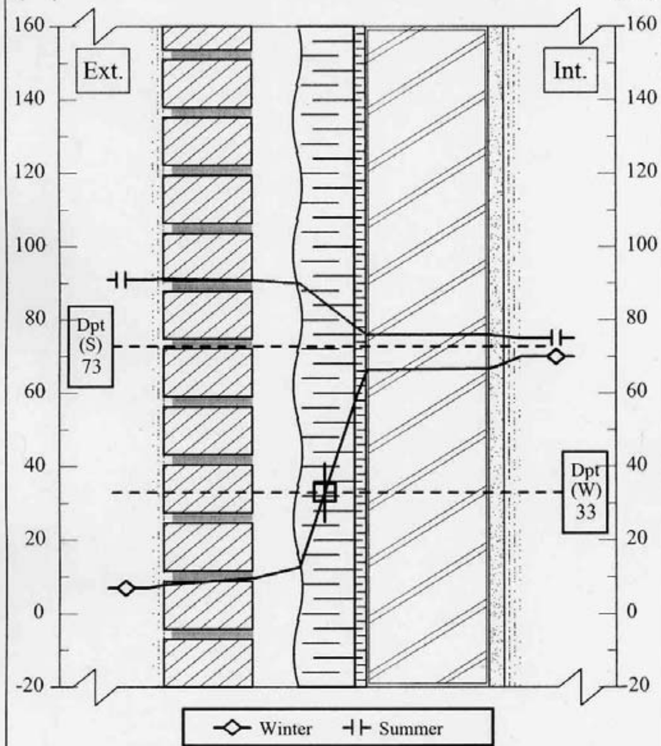
$C_1 = 0.353 \Rightarrow 3/8''$ clearance, all sides.

Thermal Performance Study

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Simmons College SOM
Number	1
City	Boston, MA
Date	4/6/2010
Analysis by:	Kevin Wigton
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	7	75	91
RH (%)	25	80	50	53
DPT (°F)	33	3	56	73

PENNSYLVANIA STATE UNIVERSITY

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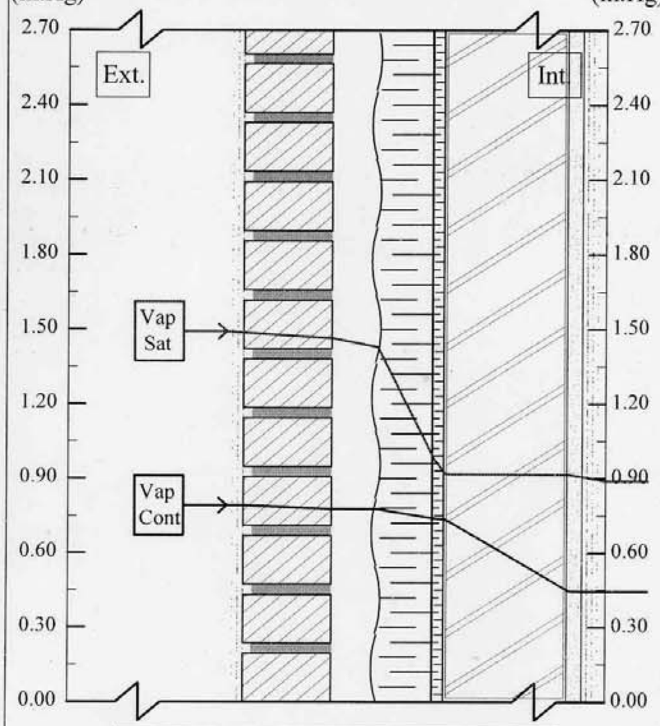
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.75	0.17	7.9	91.3
2	brick (TTW), 4 in.	No Recor...	Generic...	4.00	0.64	9.8	90.8
3	cavity, 2 in.	No Recor...	Generic...	2.00	0.98	12.7	90.0
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	2.50	15.43	58.7	78.0
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.50	2.57	66.4	75.9
6	steel stud, 5-1/2 in.	No Recor...	Generic...	5.51	0.12	66.7	75.9
7	gypsum bd., 5/8 in., (#2)	No Recor...	Generic...	0.63	0.46	68.1	75.5
8	air film (int), 3/4 in.	No Recor...	Generic...	0.75	0.64	70.0	75.0
				15.14	21.00	(7.3)	(91.4)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name: Simmons College SOM

Number: 1

City: Boston, MA

Date: 3/13/2010

Analysis by: Kevin Wigton

Wall Type   Option

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	—	—	75	91
RH (%)	—	—	50	53
DPT (°F)	—	—	56	73

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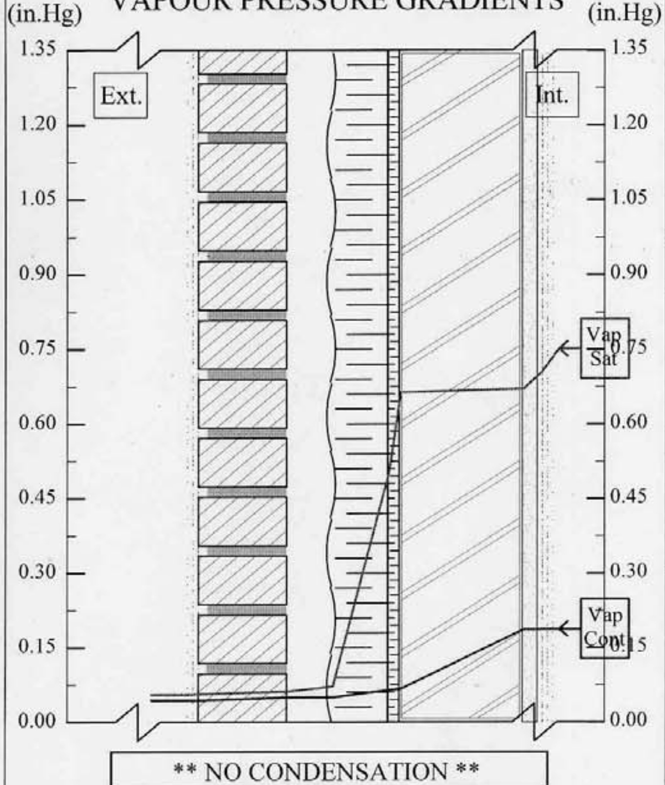
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	90.9	1.463	0.778
2	brick (TTW), 4 in.	No Recor...	Generic...	1.430	90.4	1.441	0.764
3	cavity, 2 in.	No Recor...	Generic...	0.016	89.6	1.407	0.764
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	3.576	77.9	0.964	0.728
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.430	75.9	0.903	0.724
6	steel stud, 5-1/2 in.	No Recor...	Generic...	28.607	75.8	0.901	0.440
7	gypsum bd., 5/8 in., (#2)	No Recor...	Generic...	0.229	75.5	0.890	0.438
8	air film (int), 3/4 in.	No Recor...	Generic...	0.006	75.0	0.876	0.438
9							
10							
11							
12							
	TOTAL or (Layer 0)			34.437	(91.0)	(1.469)	(0.778)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



PROJECT

Name	Simmons College SOM
Number	1
City	Boston, MA
Date	3/13/2010
Analysis by:	Kevin Wigton
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	7	–	–
RH (%)	25	80	–	–
DPT (°F)	33	3	–	–

PENNSYLVANIA STATE UNIVERSITY

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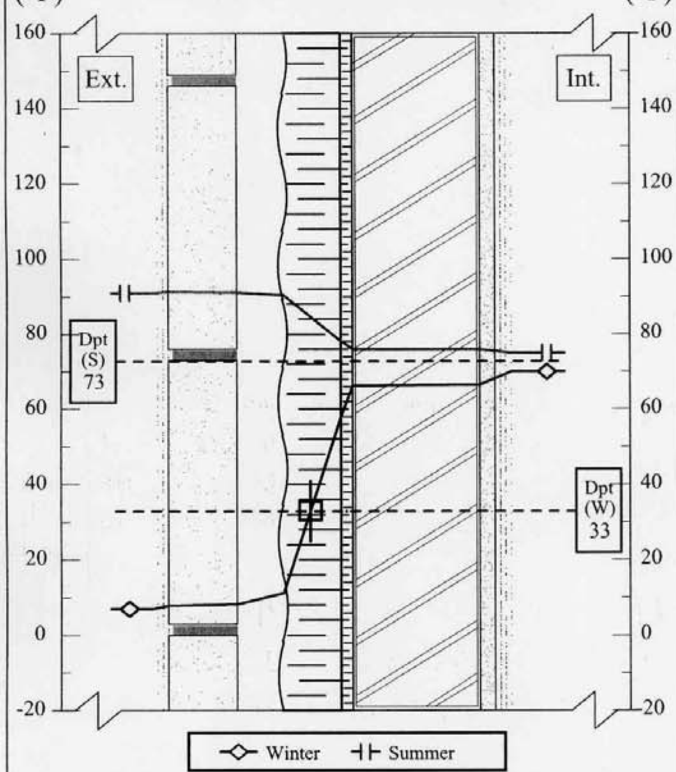
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	7.5	0.056	0.043
2	brick (TTW), 4 in.	No Recor...	Generic...	1.430	9.4	0.061	0.049
3	cavity, 2 in.	No Recor...	Generic...	0.016	12.4	0.071	0.049
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	3.576	58.6	0.497	0.064
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.430	66.4	0.653	0.066
6	steel stud, 5-1/2 in.	No Recor...	Generic...	28.607	66.7	0.661	0.184
7	gypsum bd., 5/8 in., (#2)	No Recor...	Generic...	0.229	68.1	0.693	0.185
8	air film (int), 3/4 in.	No Recor...	Generic...	0.006	70.0	0.740	0.185
9							
10							
11							
12							
	TOTAL or (Layer 0)			34.437	(7.0)	(0.054)	(0.043)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Simmons College SOM
Number	1
City	Boston, MA
Date	4/6/2010
Analysis by:	Kevin Wigton
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	7	75	91
RH (%)	25	80	50	53
DPT (°F)	33	3	56	73

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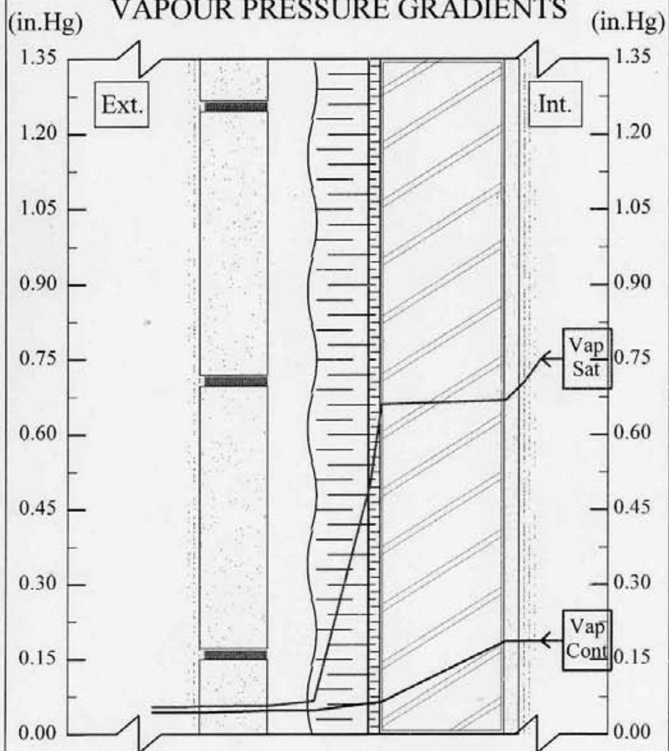
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.75	0.17	7.9	91.3
2	stone, limest. (unvntd), 3 in.	No Recor...	Generic...	3.00	0.14	8.3	91.1
3	cavity, 2 in.	No Recor...	Generic...	2.00	0.98	11.3	90.4
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	2.50	15.43	58.4	78.0
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.50	2.57	66.3	76.0
6	steel stud, 5-1/2 in.	No Recor...	Generic...	5.51	0.12	66.6	75.9
7	gypsum bd., 5/8 in., (#2)	No Recor...	Generic...	0.63	0.46	68.1	75.5
8	air film (int), 3/4 in.	No Recor...	Generic...	0.75	0.64	70.0	75.0
	Total or (Layer 0)			14.14	20.51	(7.3)	(91.4)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



** NO CONDENSATION **

PROJECT

Name	Simmons College SOM
Number	1
City	Boston, MA
Date	3/13/2010
Analysis by:	Kevin Wigton
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	70	7	—	—
RH (%)	25	80	—	—
DPT (°F)	33	3	—	—

PENNSYLVANIA STATE UNIVERSITY

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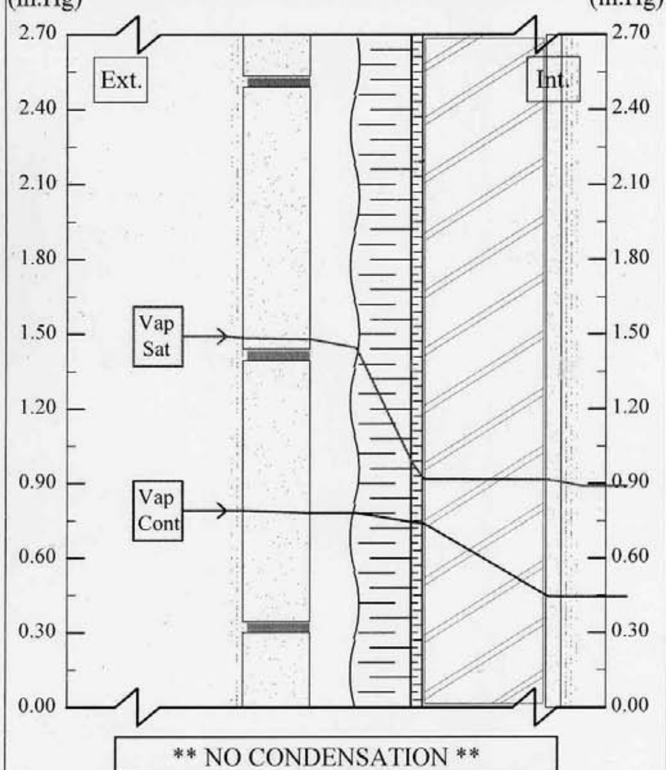
	Material	Manufacturer	Model No.	Rvap (1/M)	Temp (°F)	VapSat (in.Hg)	VapCont (in.Hg)
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	7.5	0.056	0.043
2	stone, limest. (unvntd), 3 in.	No Recor...	Generic...	0.954	8.0	0.057	0.047
3	cavity, 2 in.	No Recor...	Generic...	0.016	11.0	0.066	0.047
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	3.576	58.4	0.493	0.062
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.430	66.3	0.651	0.064
6	steel stud, 5-1/2 in.	No Recor...	Generic...	28.607	66.6	0.659	0.184
7	gypsum bd., 5/8 in., (#1)	No Recor...	Generic...	0.229	68.0	0.692	0.185
8	air film (int), 3/4 in.	No Recor...	Generic...	0.006	70.0	0.740	0.185
9							
10							
11							
12							
	TOTAL or (Layer 0)			33.959	(7.0)	(0.054)	(0.043)

CONDENSATION ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11a)

WALL SECTION AND VAPOUR PRESSURE GRADIENTS



PROJECT

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CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
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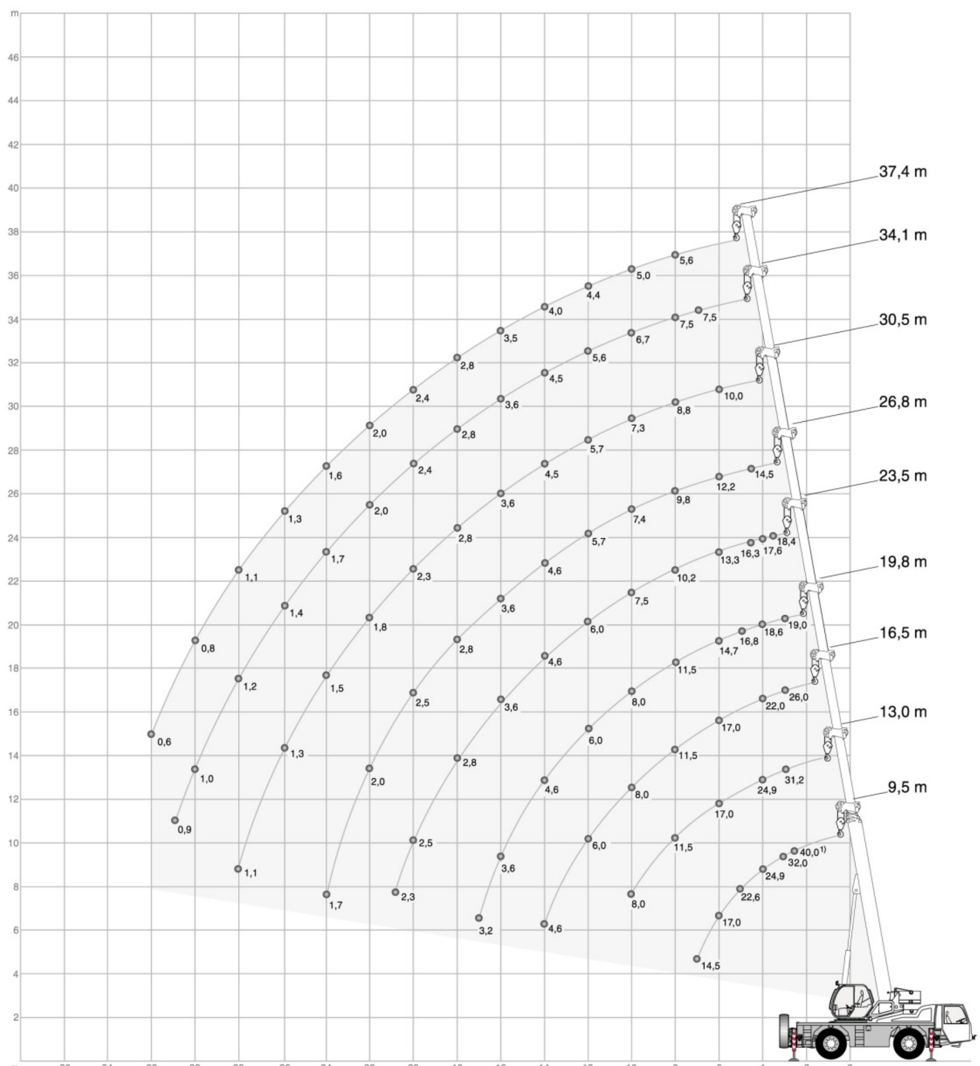
1	air film (ext), 3/4 in.	No Recor...	Generic...	0.001	90.9	1.462	0.778
2	stone, limest. (unvntd), 3 in.	No Recor...	Generic...	0.954	90.8	1.457	0.769
3	cavity, 2 in.	No Recor...	Generic...	0.016	90.0	1.423	0.769
4	ureth.(ext.) insul., 2-1/2 in.	No Recor...	Generic...	3.576	78.0	0.966	0.733
5	rigid ins.,(extru.), 1/2 in.	No Recor...	Generic...	0.430	75.9	0.904	0.728
6	steel stud, 5-1/2 in.	No Recor...	Generic...	28.607	75.9	0.901	0.440
7	gypsum bd., 5/8 in., (#1)	No Recor...	Generic...	0.229	75.5	0.890	0.438
8	air film (int), 3/4 in.	No Recor...	Generic...	0.006	75.0	0.876	0.438
9							
10							
11							
12							
	TOTAL or (Layer 0)			33.959	(91.0)	(1.469)	(0.778)

Appendix E: Construction Information

TEREX®

AC40/2L
ALL TERRAIN CRANE

HA WORKING RANGES · ARBEITSBEREICHE · PORTÉES · CAMPO DI LAVORO · RANGOS DE TRABAJO



1) over rear · nach hinten · sur l'arrière · sul retro · hacia atrás

AC40/2L

7