2012

Final Report

Thomas J Kleinosky – Structural Option
Patient Care Pavilion; Albany, NY
4/4/2012
Advisor: Dr. Hanagan

Albany Medical Center
Patient Care Pavilion

Albany, NY

Phase One
6-Story Hospital

General Information

Owner:
Albany Medical Center

Function: Hospital

Size:
348,000ft² + Future expansion

Height:
75ft + 75ft for future expansion

Stories: 6 + 4 for future exansion

Cost: $180 million

Construction: Sept. 2010 to June 2013

Delivery Method: Construction Manager at Risk

Phase Two – Four Story Vertical Expansion

Project Team

CM & GC:
Gilbane Building Co.

Architect:
TRO Jung|Brannen

Structure:
Ryan-Biggs Associates

HVAC:
ICOR Associates, LLC

Civil:
Clark Patterson Lee

Elevators:
Lerch Bates Inc.

Structure

- 3'-0" Reinforced concrete mat foundations and near existing buildings are 20''-0" deep piles
- Steel structure utilizing braced and moment frames for transferring lateral loads
- Lightweight concrete slab on metal decking for composite floor construction
- 62" Plate girder cantilevering floors 2-10 18'-0" over the site
- Thickenede floor slabs in vibration sensitive areas
- 2-story struss supporting a pedestrian access bridge to existing buildings

Architecture

- Traditional stacked brick and stone façade with punched windows
- Integrates the use of metal panels and glazed curtain-walls
- The Patient Pavilion is to be constructed to 6 stories and expanded to 10 in the future
- Four story atrium connecting the major entrances to public elevators

MEP

- 7 air handling units rangings from 8,500-77,600 supply CFM
- Constant volume boxes used throughout and 3 Variable air volume boxes on the entry level
- 20 humidifer units one per operating room
- Main power is 480/277 3-phase, 4 wires
- (2) 90 HP generators on the roof and (1) future 90 HP generator
- Flourescent, LED, and metal halide lighting

Thomas J. Kleinosky – Structural

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Table of Contents

Executive Summary

The goal of this thesis is to perform a progressive collapse analysis per Unified Facilities Criteria 4-023-03 to consider the removal of load bearing elements. Efficient members were designed to resist the collapse of the Patient Pavilion for these column removals. Per occupancy category IV the UFC requires three design methods that must be performed to resist collapse of the Patient Pavilion, these are:

1. Tie-Forces Method
2. Alternate Path Method
3. Enhanced Local Resistance

The tie-force method is an indirect method to resisting progressive collapse, because it does not resist the collapse of the structure it takes the load from the damaged structure and distributes it to the undamaged structure. The tie-force method mechanically ties the structure together by placing additional reinforcement within the slab. The ties that were designed in the slab had enough strength to “tie” the building together and therefore help redirect the additional load from the removed load bearing element.

The alternate path method performed is a direct method because it directly resists the collapse of the structure by localizing the collapse with moment frames. A virtual work plastic analysis was utilized to determine preliminary beam sizes around the perimeter of the building. The assumptions with the virtual work method were that the member would hinge at the column faces and that internal work equals external work.

A computer model was made in ETABS and the preliminary beam sizes designed by the virtual work method were deemed adequate when the progressive collapse analysis was performed. The columns were checked next and the initial column design was under designed which was expected, therefore multiple iterations were made until columns’ axial and moment interaction were less than unity (Chapter H of AISC).

A façade study will performed for the existing façade, based on those results, a new façade precast façade was proposed increasing the R-value of the wall and decreasing the shading coefficient of the glazing. Studies were performed to determine the amount of energy savings per year that would result from the new performance enhanced system

A site logistics study was performed for the precast façade to determine the construction feasibility of the façade. It was determined the constructability for the precast façade was very difficult therefore the idea was disregarded and a site logistics was created for the construction on the Patient Pavilion.

Two connections were designed to meet MAE requirements for the masters portion of this thesis. The two connections designed were an extended shear tab connection and a welded unreinforced flange moment connection.

Acknowledgements

I would like to extend my acknowledgements and gratitude to the following people and companies for their input and support throughout the completion of this thesis:

- Steve Florio, thank you for all your help and guidance when it was needed these past two semesters
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I can't thank you enough for the support you have given me when I was faced with a tough choice or decision, you've made me who I am today.

In Loving Memory of Joseph T. Sefcheck Sr.

Introduction

The Patient Pavilion is located in Albany, NY, at the intersection of New Scotland Avenue and Myrtle Avenue, on the eastern end of the existing Albany Medical Center Hospital (AMCH) campus. Constructed as an expansion to the AMCH, the Patient Pavilion utilizes pedestrian bridges to tie into an existing parking structure across New Scotland Avenue, as well as tying into an existing building on the AMCH campus as shown in *Figure 1.*

The Patient Pavilion will retain the architectural style, forms, and materials of downtown Albany and the AMCH campus, as specified in the City of Albany Zoning Ordinance. The façade primarily consists of brick and stone with punched windows and white stone accenting the upper levels. To add emphasis to the pedestrian walkway over New Scotland Avenue, metal paneling and glazed aluminum curtain-walls added an integrated modern look to the traditional façade.

Figure 1 – Pedestrian Bridges

The Patient Pavilion consists of two phases; Phase 1, contains the demolition of an existing building on the AMCH campus, and the construction of a six story medical center see *Figure 2* and Phase 2 is a future four story vertical expansion of the Patient Pavilion see *Figure 3*. The building height of Phase 1 is 75 feet above grade and the vertical expansion of Phase 2 will increase the building height to 145 feet above grade. Due to a small site and large square footage demands, the building cantilevers over the site on the side of New Scotland Avenue, demanding for the design of cantilevered plate girders to support a column load from stories 2-10.

Figure 2 – Phase 1 of Patient Pavilion; Initial Design

This patient care facility, contributes 229 patient beds, 20 operating rooms, and 1000 new permanent jobs to the AMCH campus. The 348,000 square foot expansion consists of six stories above grade with a four story vertical expansion in the future. Phase 1 construction on the Patient Pavilion began in September of 2010 and projects to finish in June of 2013.

To better understand the terminology used for referring to designated levels, an architectural elevation is provided on the next page.

Figure 3 – Phase 2 of the Patient Pavilion; Vertical Expansion

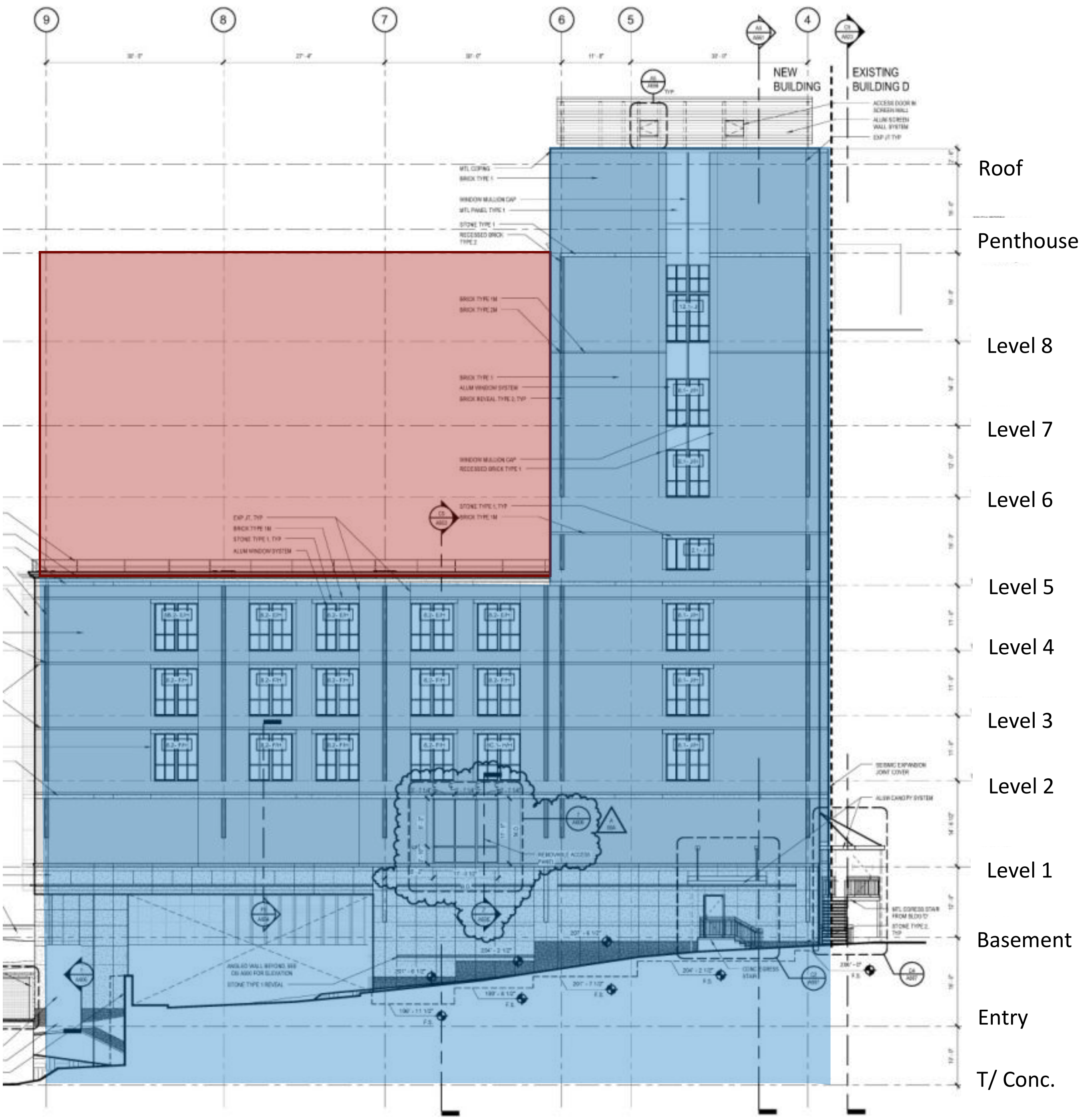


Figure 4 – South Elevation

Phase 1

Phase 2

Figure 5 – Site Plan

New Scotland Avenue

Myrtle Avenue

Structural Overview

The majority of the Patient Pavilion rests on a 36" thick mat foundation, and some piles located near existing buildings. The floor system utilizes composite beams, girders, and slabs to carry the loads derived from ASCE07-02. The lateral forces are collected on the brick non-bearing façade, transfers into the slab and is distributed to the foundation/grade by the integration of braced and moment frames. On the southern end of the site, 62" deep plate girders are utilized to cantilever nine stories over the edge of the site. Multi-story trusses are utilized to carry multiple levels with a large clear span, these are located over the emergency access ramp and at the pedestrian bridge that ties into an existing AMCH building see *Figure 6.*

Figure 6 - Span over Emergency Access Ramp and Street Labels

Foundation

Vernon Hoffman PE Soil and Foundation Engineering supplied the geotechnical report for the Patient Pavilion site. Procedures used were site boring, vane shear testing, pressure testing, and cone testing. Soil testing concluded that foundations must be designed to a net bearing pressure of 3000psf. Design ground water level was reported to be between 4' and 10' throughout the site. After a full analysis of the site, the geotechnical report recommended the building to sit on a mat foundation resting on a controlled fill.

Because of the relatively low allowable soil bearing pressure, the majority of the Patient Pavilion sits on a 36" mat foundation resting on a 4" mud slab with a 12" compacted aggregate base. Alternatively, 20'-0" deep piles are utilized in order to prevent unwanted settlement of the existing buildings. Piles are utilized in place of shallow foundations because piles will control settlements and provide uplift resistance more effectively than shallow foundations.

Foundation walls are utilized along existing building C and along New Scotland Avenue to lessen the demand on the excavation shoring; these walls also serve the purpose of shear walls in the lateral system. Backfilling behind these walls was needed to provide construction access for equipment and materials to build the pile caps and grade beams.

Floor System

The Patient Pavilion utilizes 3"x20ga galvanized composite steel deck with 3 1/2" lightweight topping, reinforced with #4's at 16" O.C. for shrinkage and temperature, this floor system is typical throughout the levels, unless otherwise noted. On level 2, the floor slab is thickened with a 3" lightweight concrete topping in order to reduce floor vibrations in the operating rooms. The entry level utilizes an 8" lightweight concrete slab on 3 1/2"x16ga composite metal deck because of longer deck spans and larger live loads. In areas where radiation is prevalent, the slabs above and below that level are stiffened with a steel plate anchored to the slab with angles. These plates are located on levels 2 and 3 and their function is to provide a shield from the radiation for adjacent areas, refer to *Figure 7* for radiation slab details.

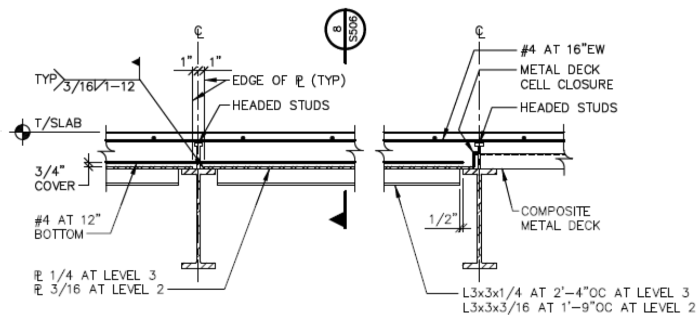


Figure 7 – Slab Detail; Radiation Shielding Plate

Typical beam spacing throughout is 10'-0" O.C., creating a 10'-0" deck span requirement, all beams are composite beams, typically W12's. However, on the Basement Level and Level 2, typical beams range from W16's to W18's. Reasons for deeper beams are that the live load requirements on the Entry Level through Level 2 are greater than the other floors. However, the Basement Level and Level 2 utilize deeper beams than the Entry Level and Level 1 due to greater floor-to-floor heights.

Typical beams span 27'-4", these beams sit on girders that typically span 30'-0". Girder sizes range from W14's to W18's; however, on the Basement Level and Level 2 girder sizes fluctuate from W18's to W24's, refer to *Figure 8* for a typical bay on Level 3.

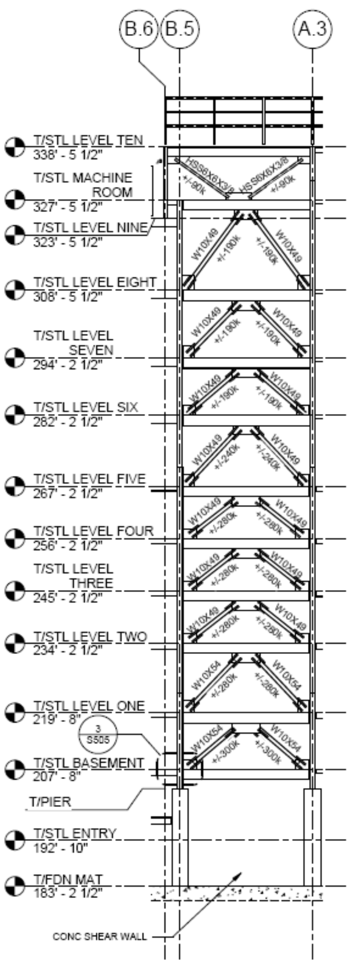
A demand for specialty framing is needed in certain areas in this project; on the southern end of the site, a column is cantilevered 18' over the edge of the site resting on a 62" plate girder. The pedestrian bridge on the tying into the existing AMCH building spans 83' over another existing AMCH building. A two-story truss was designed on bottom two levels of this pedestrian bridge, consisting of W10x77's and W10x100's.

Figure 8 – Typical Floor Plan

Lateral System

The lateral system for the Patient Pavilion predominantly consists of braced frames, with some moment frames. Within the structure, there are 14 braced frames and 5 moment frames, because of the locations of the braced frames, Chevron bracing is utilized to allow openings for doorways and corridors. See *Figure 8* for a typical braced frame. Figure *7* shows the locations of the braced and moment frames, the location of some braced frames fluctuate from level to level. For instance, braced frame 13 is braced between the Basement Level through Level 2 and above Level 2 is a moment frame.

The braced frames along the western side of the site sit on retaining walls in the basement, which also act as concrete shear walls. A strong connection is required to transfer the shear load as well as to resist upift, for these connections a 30”x30”x3½” baseplate with a 2” diameter anchor bold anchored 42” into the wall is specified. Diagonal bracing on the lower levels range from W10’s to W12’s and HSS8x6’s to HSS8x8’s on the upper levels. Heavier bracing on the lower levels provides a greater resistance to shear, which increases as the force moves down the frame. Columns used in these lateral resisting frames range from W14x43 to W 14x233.



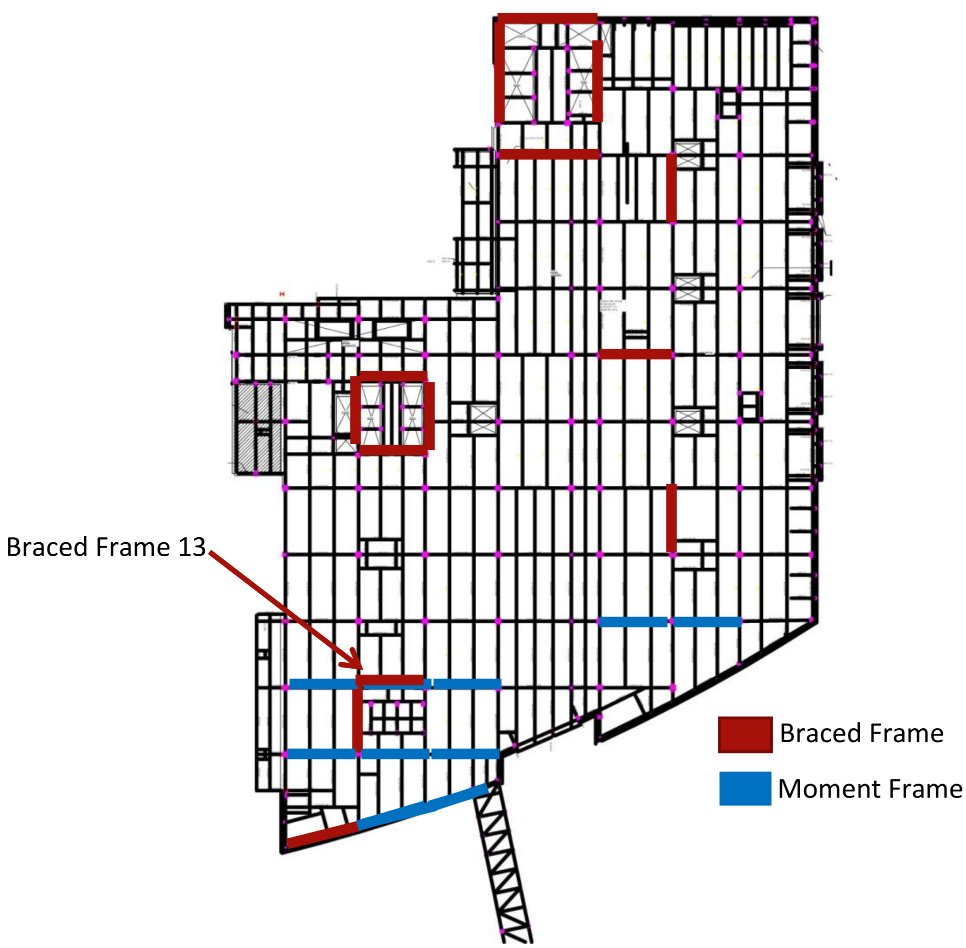


Figure 9 – Typical Layout of Lateral System

Figure 10 – Typical Braced Frame

Design Codes and Standards

Ryan-Biggs Associates abided by these standards and codes when developing the design of the Patient Pavilion:

- AISC 13th Edition Manual
- AISC Specification 360-05
- 2007 Building Code of New York State (BCNYS)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-02)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Standards and codes utilized for this report:

- AISC 14th Edition Manual
- AISC Specification 360-10
- 2006 International Building Code (IBC 2006)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-05)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)
- Unified Facilities Criteria 4-023-03
- ASCE41-06 Seismic Rehabilitation of Existing Buildings

Materials

The structural materials designated by the AISC 13th Ed. were used in the design of the Patient Pavilion by Ryan-Biggs; see *Table 1* for the capacities of the large variety of structural elements. The materials were specified on the General Notes page, S001, on the Construction Documents provided via Gilbane Building Company. All steel materials below are according to ASTM standards.

Table 1 – Material Properties

Material Properties		
Material		**Strength**
Rolled Steel	**Grade**	**f_y = ksi**
W Shapes	A 992	50
C, S, M, MC, and HP Shapes	A 36	36
Plates, bars, and angles	A 36	36
HSS pipe	A53 type E or S Grade B	35
Reinforcing Steel	A 615	60
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Footings/mat foundation	145	3,000
Interior S.O.G or Slab on Deck	145	3,500
Foundation Walls, Shear walls, Piers, Pile caps, and Grade beams	145	4,000
Exterior S.O.G.	145	4,500
Masonry	**Grade**	**f'_m = psi**
Concrete Block	C 90	2,800
Mortar	C 270 Type S	n/a
Unit Masonry	n/a	2,000
Grout	C 476	2,500
Brick	C 216 type FBS Grade SW	3,000
Welding Electrodes	E70 XX	70 ksi

Loads

In the following tables, dead and live loads that were used to analyze and design the Patient Pavilion are listed as well as the loads used for this thesis. Live loads interpreted by the designer were derived from ASCE7-02, live loads used in this thesis were derived from ASCE 7-05; dead loads were assumed or calculated and verified with specified dead loads on the structural general notes.

Dead Loads

The dead loads listed on the general notes of the structural drawings are listed below in *Table 2.* Upon further analysis shown in *Table 3* and *Table 4*, the assumptions of these loads were verified to be accurate and conservative in some cases. The MEP is larger than typical because in a hospital the MEP weight is to be assumed larger than typical.

Table 2 – Superimposed Dead Loads

Dead Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Roof Without Conc. Slab	30
Roof With Conc. Slab	95
Roof Garden	325
Floor	95
Level 9 Mechanical Penthouse	125

Table 3 – Roof without Conc. Slab Verification

Roof Without Conc. Slab Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x16ga decking	5
Rigid Insulation (tapered starting at 8”)	.75psf per in thickness=(.75x8x.5)= 12
Total	**29**

Table 4 – Roof with Conc. Slab and Floor Verification

Roof With Conc. Slab and Floor Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x20ga Composite Decking	48
Steel Framing	13
Finishes and Partitions	15
Fireproofing	2
Miscellaneous	5
Total	**95**

Live Loads

See *Table 5* for the controlling live load description per each level with the exception of elevator lobbies and stairs. The live loads given on the structural general notes were obtained using ASCE7-02, they were rechecked according to ASCE7-05 and were deemed accurate, see *Table 6.*

Table 5 – Live Loads

Live Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Entry	100
Basement	100
Level 1	100
Level 2	100
Level 3	80
Level 4	80
Level 5	80
Level 6	80
Level 7	80
Level 8	80
Level 9 (Mechanical Penthouse)	125
Elevator Lobbies and Stairs	100

Table 6 – Verifying Live Loads per ASCE7-05

Level 1 – Level 2; Verification (ASCE7-05)	
Occupancy	**Weight (psf)**
Assembly Areas – Lobby	100
Hospitals – OR Rooms	60 + Partitions
Hospitals – Patient Rooms	40 + Partitions
Hospitals – Corridors above 1st Floor	80

Snow Load

The snow load for the Patient Pavilion was determined per ASCE7-05 section 7.3. Following the procedure described in this section, the flat roof snow load was calculated to be 37 psf, approximately 40psf, which was listed on the structural general notes.

Upon finding the density of the snow, and back figuring the density to find the height, it was determined the flat roof snow load height was 2 feet; this eliminates drift along the parapets, which are 2 feet high. Snowdrifts were calculated against the stair towers (See *Figure 9)* where windward drift loads control because of a larger l_u. Due to the windward forces control, the height of the snow load was reduced by using 3/4 of h_d, however after interpretation of the code the full h_d was used to calculate the drift width W. The height and weight of the drift is shown below in *Figure 9*, the location of each drift calculated is shown in *Figure 10.*

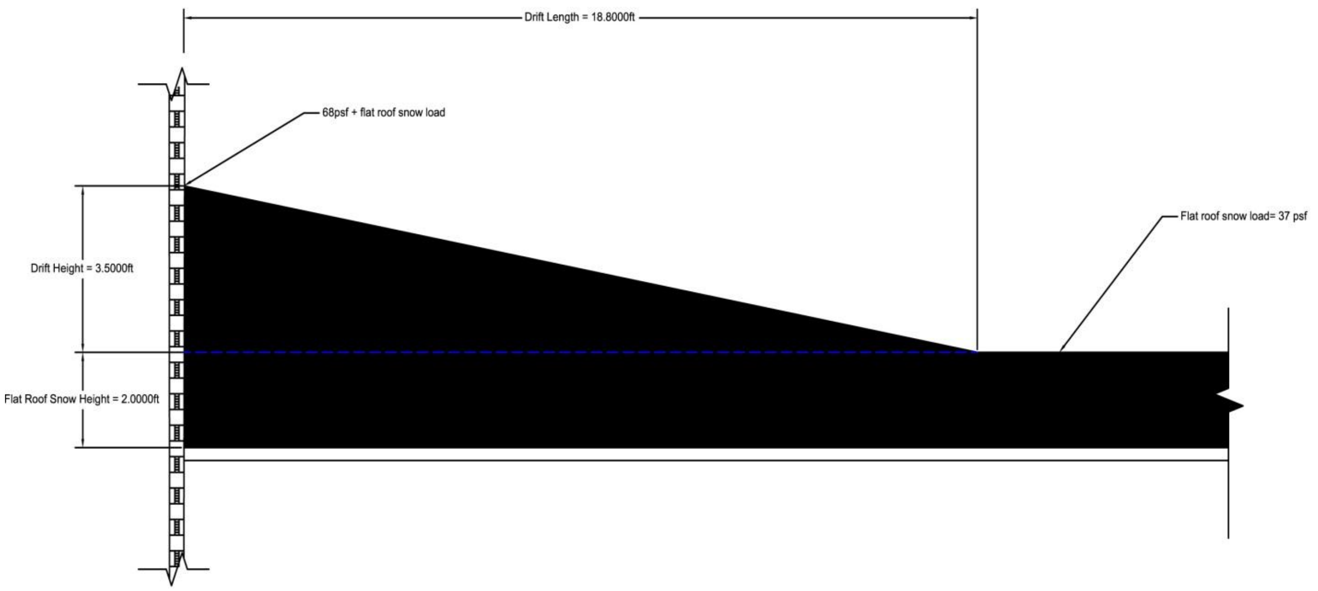


Figure 11 – Snow Drift

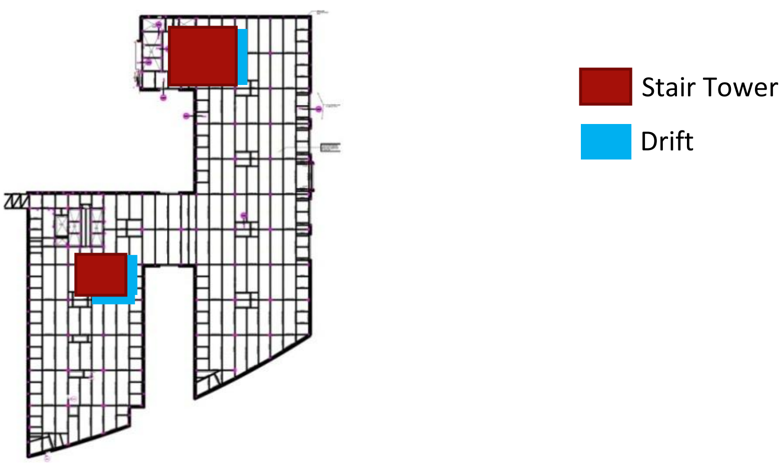


Figure 12 Drift and Stair Tower Locations

Wind Loads

Wind loads were calculated by Method 2, Main Wind Force Resisting System (MWFRS), provided in ASCE7-05 Chapter 6 to determine wind pressures in both the North-South direction and East-West direction. Initial assumptions had to be made for this procedure; the building footprint had to be projected into a rectangle, which is a valid assumption because the lateral systems run in two orthogonal directions (*See Figure 11*).

A flexible building is defined in the ASCE7-05 as building with a frequency of 1Hz or less, equations to calculate the natural frequency are provided in the commentary in the ASCE7-05. Calculating the lower bound frequency (Eq C6-17) and the Average Value frequency (Eq C6-18), the natural frequency was less than 1Hz, the assumption of a flexible building was verified.

The calculations required for this analysis are redundant and time consuming; to simplifying the redundant process, a Microsoft Excel spreadsheet was created. The spreadsheet calculates windward and leeward forces, as well as story shear and overturning moment, in the North-South direction and East-West direction. The final forces in the North-South direction and East-West direction are shown in the following tables, as well as a schematic depiction showing the wind pressures and wind forces along the building height.

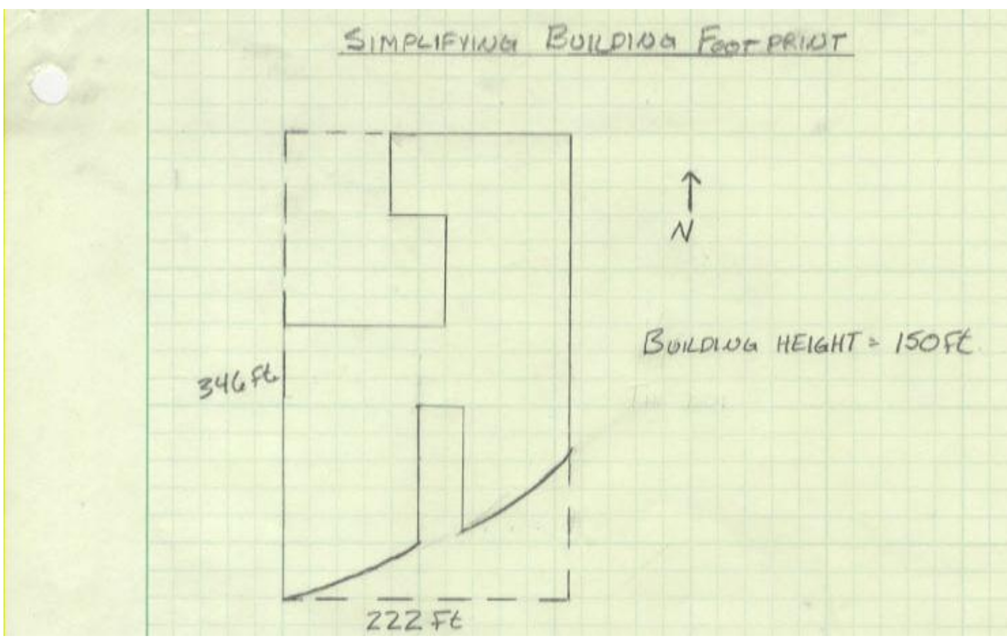


Figure 13 – Simplified Building Footprint

Table 7 – Wind Pressures; North-South Direction

Wind Pressure					
	Windward (psf)	Leeward (psf)	Internal Pressures (+/-)	Net Pressure	
				($+GC_{pi}$)	($-GC_{pi}$)
Entry Level	7.77	-7.27	4.01	3.75	11.78
Basement	7.77	-7.27	4.01	3.75	11.78
Level 1	9.21	-7.27	4.01	5.20	13.22
Level 2	10.46	-7.27	4.01	6.45	14.48
Level 3	11.17	-7.27	4.01	7.16	15.18
Level 4	11.77	-7.27	4.01	7.76	15.78
Level 5	12.37	-7.27	4.01	8.36	16.38
Level 6	13.08	-7.27	4.01	9.07	17.09
Level 7	13.49	-7.27	4.01	9.47	17.50
Level 8	14.03	-7.27	4.01	10.02	18.05
Level 9	14.58	-7.27	4.01	10.56	18.59

Table 8 – Roof Uplift; North-South Direction

Roof	Uplift (psf)	Internal Pressures (+/-)	($+GC_{pi}$)	($-GC_{pi}$)
0 to 75 ft	-16.86	4.01	-20.87	-12.85
75 to 150 ft	-16.86	4.01	-20.87	-12.85
150 to 300 ft	-9.37	4.01	-13.38	-5.35
>300 ft	-5.62	4.01	-9.63	-1.61

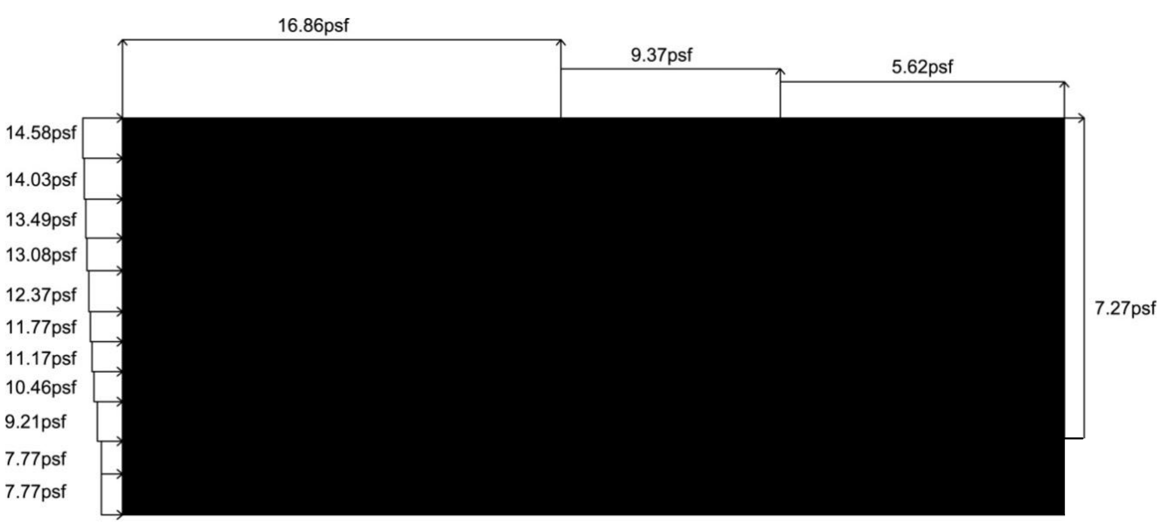


Figure 14 – Wind Pressures; North-South Direction

Table 9 – Wind Forces; North-South Direction

Wind Forces							
	Trib Heights		**Elevation**	**Wall Width (Perp. To N-S)**	**Trib. Area**	**Story Force (kips)**	**Story Shear (kips)**
	Below	Above					
Entry Level	0	7.5	0	222	1665	25.03	616.67
Basement	7.5	6	15	222	2997	45.06	591.64
Level 1	6	7.25	27	222	2941.5	48.47	546.58
Level 2	7.25	5.5	41.5	222	2830.5	50.19	498.11
Level 3	5.5	5.5	52.5	222	2442	45.03	447.93
Level 4	5.5	5.5	63.5	222	2442	46.49	402.90
Level 5	5.5	7.5	74.5	222	2886	56.68	356.40
Level 6	7.5	6	89.5	222	2997	60.98	299.73
Level 7	6	7.125	101.5	222	2913.75	60.48	238.75
Level 8	7.125	7.5	115.75	222	3246.75	69.16	178.27
Level 9	7.5	7.5	130.75	222	3330	72.74	109.12
Level 10	7.5	0	145.75	222	1665	36.37	37.22
						Total Base Shear=	**616.67**

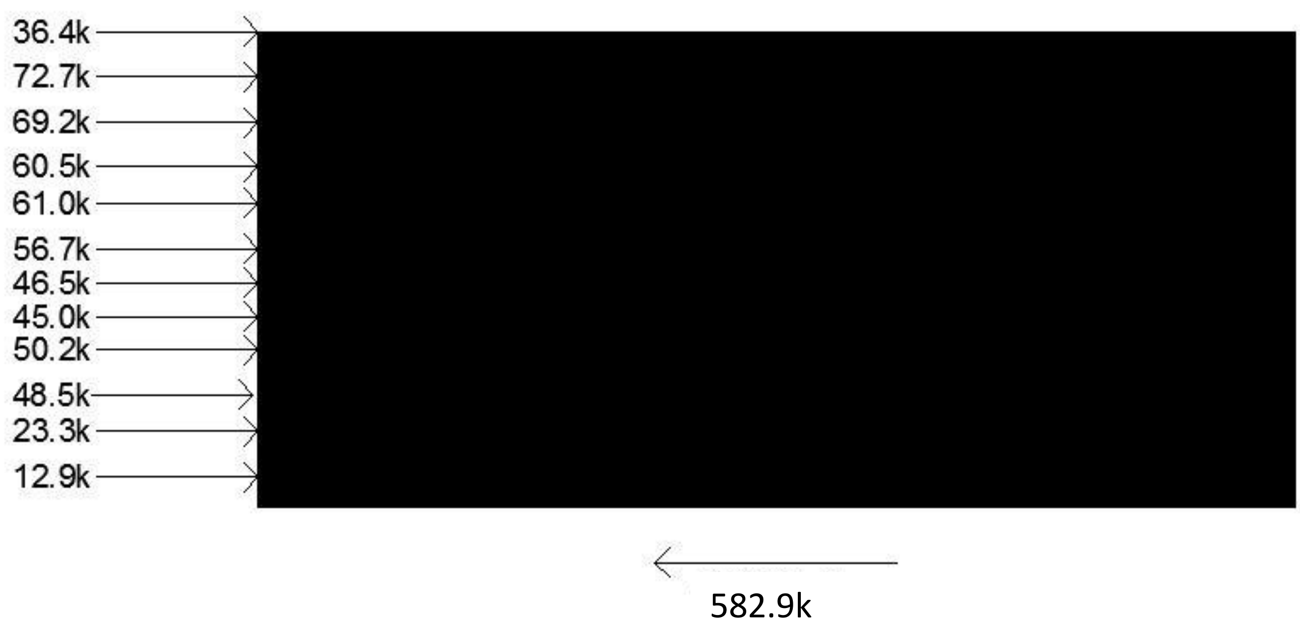


Figure 15 – North-South Wind Forces

Table 10 –Wind Pressures; East-West Direction

Wind Pressure					
	Windward (psf)	Leeward (psf)	Internal Pressures (+/-)	Net Pressure	
				(+GC$_{pi}$)	(-GC$_{pi}$)
Entry Level	7.56	-9.11	4.01	3.54	11.57
Basement	7.56	-9.11	4.01	3.54	11.57
Level 1	8.96	-9.11	4.01	4.95	12.97
Level 2	10.18	-9.11	4.01	6.17	14.19
Level 3	10.87	-9.11	4.01	6.86	14.88
Level 4	11.45	-9.11	4.01	7.44	15.47
Level 5	12.04	-9.11	4.01	8.02	16.05
Level 6	12.73	-9.11	4.01	8.71	16.74
Level 7	13.12	-9.11	4.01	9.11	17.14
Level 8	13.65	-9.11	4.01	9.64	17.67
Level 9	14.18	-9.11	4.01	10.17	18.20

Table 11 – Roof Uplift; East West Direction

Roof	Uplift (psf)	Internal Pressure (+/-)	(+GC$_{pi}$)	(-GC$_{pi}$)
0 to 75 ft	-19.48	4.01	-23.49	-15.47
75 to 150ft	-15.55	4.01	-19.56	-11.53
150 to end	-10.68	4.01	-14.69	-6.66

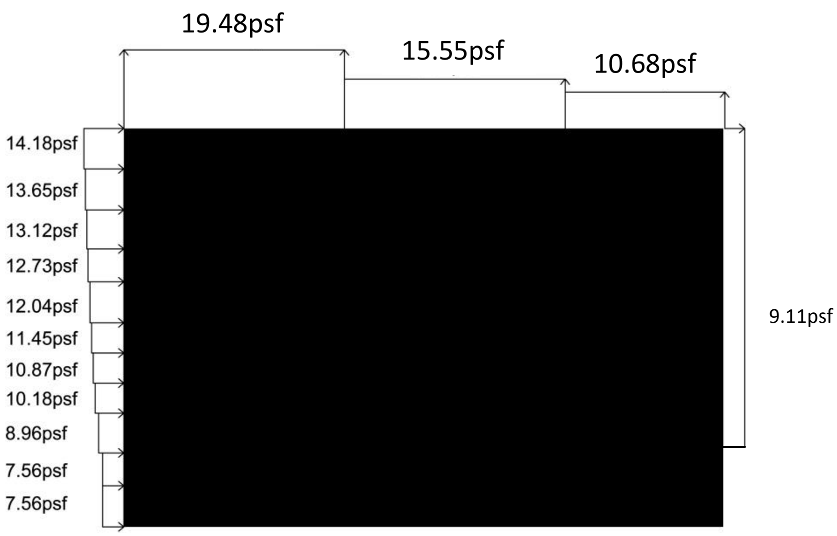


Figure 16 – Wind Pressures; East-West Direction

Table 12 – Wind Forces; East-West Direction

Wind Forces							
	Trib Heights		Elevation (ft)	Wall Width (ft)	Trib. Area (sf)	Story Force (k)	Story Shear (k)
	Below	Above					
Entry	0	7.5	0	346	2595	19.6	971
Basement	7.5	6	15	346	4671	35.3	952.33
Level 1	6	7.25	27	346	4584.5	82.86	917.03
Level 2	7.25	5.5	41.5	346	4411.5	85.12	834.17
Level 3	5.5	5.5	52.5	346	3806	76.06	749.05
Level 4	5.5	5.5	63.5	346	3806	78.28	672.99
Level 5	5.5	7.5	74.5	346	4498	95.13	594.72
Level 6	7.5	6	89.5	346	4671	102.01	499.58
Level 7	6	7.125	101.5	346	4541.25	100.99	397.57
Level 8	7.125	7.5	115.75	346	5060.25	115.21	296.58
Level 9	7.5	7.5	130.75	346	5190	120.92	181.37
Level 10	7.5	0	145.75	346	2595	60.46	60.46
						Total Base Shear=	**972**

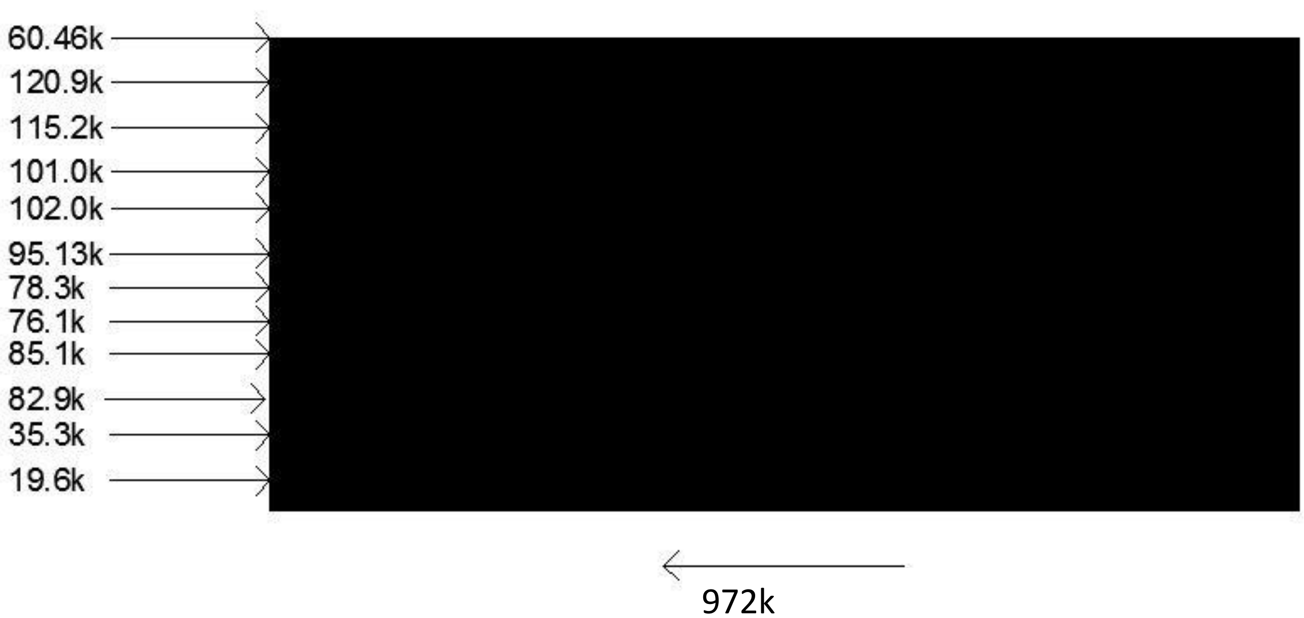


Figure 17 – Wind Forces; East-West Direction

Eleven serviceability load combinations are used to check total and story drifts for wind, these wind load cases are defined in Fig. 6-9, Chapter 6 in ASCE7-05. By inspection and knowledge of the center of rigidity and center of mass of a structure, several of these load combinations can be disregarded. However the load cases that are disregarded vary from project to project, they depend on the moment induced in the structure, which causes additive and subtractive forces in the lateral frames. Below in *Table 13* are the eleven load cases specified in the ASCE7-05, Chapter 6.

Table 13 – Wind Load Cases

Case 1	**$PW_x + PL_x$**
	$PW_y + PL_y$
Case 2	$0.75P_{Wx} + 0.75P_{Lx} + M_T$
	$0.75P_{Wx} + 0.75P_{Lx} - M_T$
	$0.75P_{Wy} + 0.75P_{Ly} + M_T$
	$0.75P_{Wy} + 0.75P_{Ly} - M_T$
Case 3	$0.75(P_{Wx} + P_{Lx})+0.75(P_{Wy} + P_{Ly})$
Case 4	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(+e_x,+e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(-e_x,-e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(+e_x,-e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(-e_x,+e_y)$

Seismic Loads

The seismic design of the Patient Pavilion follows the Equivalent Lateral Force Procedure (ASCE7-05) described in Chapter 12. Seismic Ground Motion Values were obtained per ASCE7-05, Chapter 11.4, the initial parameter necessary for the Equivalent Lateral Force Procedure were calculated, and parameters S_s and S_1 were found using this online reference (*http://earthquake.usgs.gox/research/hazmaps/design/*) provided in graduate course AE597A. After reviewing the geotechnical report, it was determined that the average shear wave velocity, $\overline{v_s}$, was 716 feet per second, from table 20.3-1 a $\overline{v_s}$ of 716 feet per second classifies the soil as class D, stiff soil.

Following the Equivalent Lateral Force Procedure, the building weight must be determined in order to find the seismic response coefficient, C_s. This was performed by counting the beams and columns and multiplying the length by their unit weights. The tributary height of the columns was found by taking half of the height to next level up plus half the height from the lower level. Using the Vulcraft Metal Decking catalog a floor load of 48psf was determined for 3 1/2"x20ga composite decking with lightweight concrete. Superimposed dead loads were determined by subtracting the floor dead load of 45psf from the given floor dead load on the structural general notes. The weight of the exterior façade was determined by assuming dead load of 48psf for exterior stud walls with brick veneers via table C3-1 (ASCE7-05). To apply this load to each level the self-weight was multiplied by the perimeter and the tributary height of each level. Summarized in *Table 14* below are the weights of each element contributing to the seismic calculation.

Table 14 – Building Weight

	Framing	Floor	Columns	Façade	Dead	20% snow	Total Weight (k)
Entry	375.9115885	2138.454	211.5	789.6	2093.903		5609
Basement	375.9115885	2138.454	211.5	789.6	2093.903		5609
Level 1	581.5651741	2559.648	213.7	838.2394	2506.322		6699
Level 2	570.97604	2565.843	165.32	1198.337	2483.01		6983
Level 3	534.66928	2092.368	136.4	1108.8	2048.777		5921
Level 4	396.15239	2114.496	135.6	1064.448	2070.444		5781
Level 5	396.15239	2113.872	157	1257.984	2069.833		5995
Level 6	396.15239	2113.872	154.64	1306.368	2069.833		6041
Level 7	396.15239	2113.872	148.7	1270.08	2069.833		5999
Level 8	396.15239	2113.872	166.1	1415.232	2069.833		6161
Level 9	396.15239	2113.872	88.84	1451.52	2069.833	352.312	6473
Level 10	25.62584	88.992	2.9	180	87.138	14.832	399
						Total Weight=	**67671**

After obtaining the weights of each level, the seismic coefficient was determined using equation 12.8-3 (ASCE) because the value calculated from equation, 12.8-2 was larger than the allowable upper limit defined in equation 12.8-3. An excel spreadsheet (provided in AE597A) was utilized to determine the shear distribution and overturning moment for each level, refer to *Table 15* below for the Excel spreadsheet. Provided below is a schematic description showing the story forces, base shear, and overturning moment.

Table 15 – Seismic Distribution

Direction	i	h_i (ft)	h (ft)	W (kips)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	Bx (ft)	5%Bx	Ax	M_z (k-ft)
E-W Direction	10	15.0	155	399	3985254	0.018	43	43	339	17	1.0	737
	9	15.0	140	6473	53721073	0.244	586	630	339	17	1.0	9941
	8	14.3	125	6161	41605387	0.189	454	1084	339	17	1.0	7699
	7	12.0	111	6000	32512839	0.148	355	1439	339	17	1.0	6016
	6	15.0	99	6040	26570786	0.121	290	1729	339	17	1.0	4917
	5	11.0	84	5995	19551044	0.089	213	1942	339	17	1.0	3618
	4	11.0	73	5781	14600735	0.066	159	2101	339	17	1.0	2702
	3	11.0	62	5921	11108047	0.051	121	2223	339	17	1.0	2056
	2	14.5	51	6983	9182133	0.042	100	2323	339	17	1.0	1699
	1	12.0	37	6700	4785959	0.022	52	2375	339	17	1.0	886
	Basement	15.0	25	5609	1941543	0.009	21	2396	339	17	1.0	359
	Entry	9.6	10	5609	349616	0.002	4	2400	339	17	1.0	65
			S	67671	219914416		2400					40695
N-S Direction	i	h_i (ft)	h (ft)	W (kips)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	Bx (ft)	5%Bx	Ax	M_z (k-ft)
	10	15.0	155	399	3985254	0.018	43	43	216	11	1.0	470
	9	15.0	140	6473	53721073	0.244	586	630	216	11	1.0	6332
	8	14.3	125	6161	41605387	0.189	454	1084	216	11	1.0	4904
	7	12.0	111	6000	32512839	0.148	355	1439	216	11	1.0	3832
	6	15.0	99	6040	26570786	0.121	290	1729	216	11	1.0	3132
	5	11.0	84	5995	19551044	0.089	213	1942	216	11	1.0	2304
	4	11.0	73	5781	14600735	0.066	159	2101	216	11	1.0	1721
	3	11.0	62	5921	11108047	0.051	121	2223	216	11	1.0	1309
	2	14.5	51	6983	9182133	0.042	100	2323	216	11	1.0	1082
	1	12.0	37	6700	4785959	0.022	52	2375	216	11	1.0	564
	Basement	15.0	25	5609	1941543	0.009	21	2396	216	11	1.0	229
	Entry	9.6	10	5609	349616	0.002	4	2400	216	11	1.0	41
			S	67671	219914416		2400					25920

Serviceability load combinations for seismic are shown in *Table 15* and are to be used to calculate total drift and story drift. The M_a which is defined in ASCE7-05 12.8.4.2, is the accidental moment due to an eccentricity of 5% the width of the floor plan. For example, seismic loading in the X-Direction, the eccentricity of the accidental moment will be 5% of the Y-Direction.

Table 16 – Seismic Servicability Load Cases

X-Direction	$E_{Qx+}M_a$
Y-Direction	$E_{Qy+}M_a$

Proposal Objective

Structural Depth

As designed, the steel structure for the Patient Pavilion is adequate and very economical for its location. Redesigning the steel structure to concrete would benefit the building because it would be easier to meet the low floor-to-floor height requirements. However, in Albany, New York concrete is much more expensive than steel therefore the redesign would be more costly. Finding improvements for the Patient Pavilion was difficult, and considering the author of this proposal is interested in working on the East coast this rules out seismic considerations. Having the capacity to hospitalize a large amount of people, an in depth progressive collapse analysis will be made in order to prevent the possible catastrophic event of an explosion collapsing the entire building.

Lining two sides of the Patient Pavilion are vehicle streets, as well as an emergency access ramp that runs under the Northeast corner of the building. These areas are very accessible for a vehicle and an explosion could easily destroy the exposed exterior columns, see Figure 13 below. These are not the only critical areas of the hospital for explosion, contained within the hospital are oxygen and gas tanks which have the capability of exploding and destroying a column. Due to time restrictions and in order to perform a thorough research on progressive collapse only the exterior columns will be studied, it should be noted that interior columns should also be considered in areas where gas tanks are prevalent.

Figure 18 - Areas of Interest Courtesy of Gilbane Construction

Mechanical – Façade Study

The existing façade for the Patient Care Pavilion is hand-stacked brick tied into each floor. A heat transfer analysis will be performed for the patient rooms on the exterior of the building to determine their thermal performance. Based on the results of the analysis a new curtain wall façade and glazing type will be chosen to increase the thermal performance for the facade. The TRACE computer modeling program will be utilized to model an exterior patient room in the Patient Pavilion to obtain more in depth results faster. Finally, a cost analysis will be performed to determine the upfront costs and the annual energy cost savings.

Construction Management – Site Logistics Study

The focus of the construction management breadth is to utilize concrete precast panels for the façade rather than the existing façade that is constructed of hand-stacked brick. The location of the Patient Pavilion makes a difficult site to use precast paneling for the façade, due to existing buildings around its western perimeter and a high traffic area on the southern side. If it is determined it is not feasible to use a precast façade a site logistics of the building with the existing hand-stacked brick will be performed.

MAE Incorporation

The exterior façade has been designed with all moment frames and per the UFC, simple partially restrained connections must be used for the secondary members and fully restrained connections for the primary members. The two connections to be designed are an extended shear tab connection and a welded unreinforced flange moment connection.

Structural Depth

Introduction

Progressive collapse is defined by the American Society of Civil Engineers 7 as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". Designing a building for progressive collapse infers that any column within a building can be removed and the structure will not collapse and the occupants will be able to safely evacuate the building. However this is a very rare event for a column to be completely removed, if a building isn't designed for the column removal it can turn in a catastrophic event in minutes.

Due to recent attacks in the United States such as the Oklahoma bombing, in Figure 19, and 9/11, developments have been made to provide guidelines and controlled practices to design against collapse. The Oklahoma bombing was a catastrophic event that could have been prevented if proper considerations were taken to resist the collapse of the structure. The explosive used was not sufficient to do significant damage to the structure or to the occupants however, due to a lack of continuity, ductility and redundancy in the slab the removal of that single column took the lives of many. The failure was due to the slab reinforcement terminating at the columns instead of being continuous and spliced together throughout the building.

Figure 19 - Aftermath of Oklahoma Bombing Image courtesy of NYdailynews.com

Design Approach

The design approach used in this thesis is in accordance with United Facilities Criteria (UFC) 4-023-03, the guidelines provide two general approaches for design against progressive collapse, the Indirect and Direct Method.

1. Indirect Design- The method used for this design is the Tie-Force Method; this design provides internal ties for the structure to give it continuity, ductility, and redundancy. The Tie-Force method distributes the additional load from the removed column and spreads it from the damaged structure to the undamaged structure.

2. Direct Design- This method explicitly resists the collapse of the structure by enabling the structure to span over the removed column.

    a. Alternate Path Method- This method localizes the failure cause by the removal of load bearing element.

    b. Enhanced Local Resistance- This method is performed after the Alternate Path Method to ensure the members provide sufficient strength to resist a specific failure case.

Before performing these design methods the occupancy category must be determined from ASCE7-05 based on building function, level of occupancy, and criticality. Once an occupancy category is determined, Table 2-2 of the UFC, see Figure 20, can be used to determine what methods must be performed to perform a complete progressive collapse analysis.

Table 2-2. Occupancy Categories and Design Requirements

Occupancy Category	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

Figure 20 – Design Criteria per Occupancy (United Facilities Criteria)

The building occupancy of the Patient Pavilion is occupancy IV per ASCE7-05; therefore, all three methods described above must be utilized to mitigate progressive collapse.

Tie-Force Method

Introduction

Per occupancy category IV the designer must mechanically tie the structure together by utilizing internal, peripheral, and vertical ties to meet develop adequate tie strength, see Figure 21 below. The structure must have adequate tie force strength before proceeding to the Alternate Path Method, if it does not have adequate tie force strength, the structure must be redesigned. Existing structural elements designed for gravity loading typically have adequate tie-force strength; however, the floor member must be able to withstand a 0.20-rad rotation while applying their tie-force simultaneously.

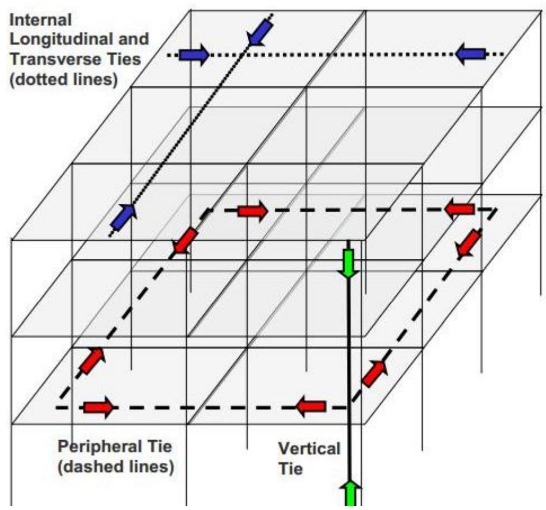


Figure 21 – Internal Ties Image from UFC 4-023-03

In a composite steel structure, the horizontal tie forces are resisted by placing additional reinforcement in the slab to transfer the load from the damaged portion of the building to the undamaged portion. Where continuity of the ties is interrupted such as elevator shafts, stairs, atriums, etc. a peripheral tie must be used around the opening for the longitudinal and transverse ties to tie into it. Ties may run perpendicular to floor members but they may not run parallel to the member if the member is not capable of the 0.20-rad rotation. The internal ties within this area must be placed on either side of the beam, not directly above it, see Figure 22.

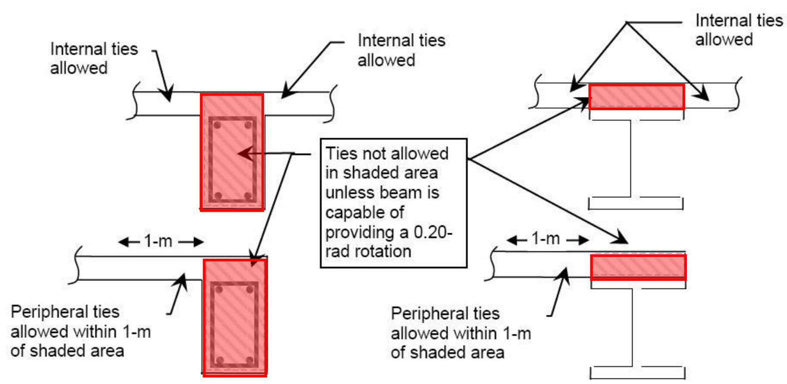


Figure 22 – Location Restrictions for Internal and Peripheral Ties Image from UFC 4-023-03

Design Process

This method was performed using the Load and Resistance Factor Design approach, the design tie strength is the product of the strength reduction factor F and the nominal tie strength R_n. The nominal strength R_n is the product of the area of the reinforcement, the expected yield stress of the reinforcement which is R_y*F_y (R_y=1.1), and an over strength factor of 1.25 per ASCE41-06 Table 6-4. Tie-Force method results are shown in Tables 17 through 20, detailed hand calculations are provided in Appendix A.

The load w_f used to determine the required tie strengths is calculated using this load combination:

$$\mathbf{w_f = 1.2D + 0.5L}$$

The dead and live load does not vary throughout the plan of the floor therefore there is no need to consider non-uniform loading. For the horizontal ties, once w_f is calculated the tie force in pounds per linear foot, F_i, can be calculated using the equation below:

$$\mathbf{F_i = 3 \cdot w_f \cdot L_1}$$

L_1= the greatest distance from column to column parallel to the direction of reinforcement

This calculation was performed for both directions of horizontal ties, longitudinal and transverse; once the loading was obtained, the reinforcement could be sized. To size the reinforcement, F_i is divided by F, F_{ye}, and the over strength factor to obtain the area of reinforcement per linear foot. No. 4s were used for the tie reinforcement, the spacing required was found by dividing the area required per linear foot by the area of a No. 4 bar. This spacing was checked against the max allowable spacing that is equal to 0.2L; L is the smallest of the directions orthogonal to the direction of reinforcement.

Peripheral reinforcement runs around the perimeter of the building and any opening within the building and the transverse and longitudinal ties tie into it. The calculated tie force, F_P, for peripheral ties can be calculated using the equation below:

$$\mathbf{F_P = 6 \cdot w_f \cdot L_1 \cdot L_P}$$

L_1= the greatest distance from column to column parallel to the direction under consideration

L_P= 3 feet

The same process is used to determine the required area of reinforcement which is spaced evenly across a 3-foot strip around the perimeter of the building. See Figure 22 below for a more detailed layout of horizontal ties.

Results

Table 17 – Base to 2nd Level Reinforcement Results

Basement to 2nd Level - w_f = 164 psf						
Direction	Length (ft.)	F_i/F_P (kip/ft)/(kip)	$A_{s\,min}$ (in.^{2})	Bar Used	Spacing (in.)	Max Spacing (in.)
East - West	30	14.8	0.219	No. 4	10	65
North - South	27.33	13.5	0.2	No. 4	12	72
Peripheral (East - West)	30	88.7	1.314	(4) No. 6	9	N/A
Peripheral (North - South)	27.33	80.7	1.2	(3) No. 6	12	N/A

Table 18 – 3rd to 8th Level Reinforcement Results

3rd to 8th Level - w_f = 154 psf						
Direction	Length (ft.)	F_i/F_P (kip/ft)/(kip)	$A_{s\,min}$ (in.^{2})	Bar Used	Spacing (in.)	Max Spacing (in.)
East - West	30	13.9	0.205	No. 4	11	65
North - South	27.33	12.6	0.187	No. 4	12	72
Peripheral (East - West)	30	83.2	1.23	(3) No. 6	12	N/A
Peripheral (North - South)	27.33	75.8	1.12	(3) No. 6	12	N/A

Table 19 – Penthouse Level Reinforcement Results

Penthouse Level - w_f = 213 psf						
Direction	Length (ft.)	F_i/F_P (kip/ft)/(kip)	$A_{s\,min}$ (in.^{2})	Bar Used	Spacing (in.)	Max Spacing (in.)
East - West	30	19.2	0.284	No. 4	8	65
North - South	27.33	17.5	0.259	No. 4	9	72
Peripheral (East - West)	30	115	1.7	(4) No. 6	9	N/A
Peripheral (North - South)	27.33	104.8	1.55	(4) No. 6	9	N/A

Table 20 – Roof Level Reinforcement Results

Roof Level - w_f = 124 psf						
Direction	Length (ft.)	F_i/F_P (kip/ft)/(kip)	$A_{s\,min}$ (in.^{2})	Bar Used	Spacing (in.)	Max Spacing (in.)
East - West	30	11.2	0.166	No. 4	14	65
North - South	27.33	10.2	0.151	No. 4	15	72
Peripheral (East - West)	30	67	0.99	(3) No. 6	12	N/A
Peripheral (North - South)	27.33	61	0.9	(3) No. 6	12	N/A

Alternate Path Method

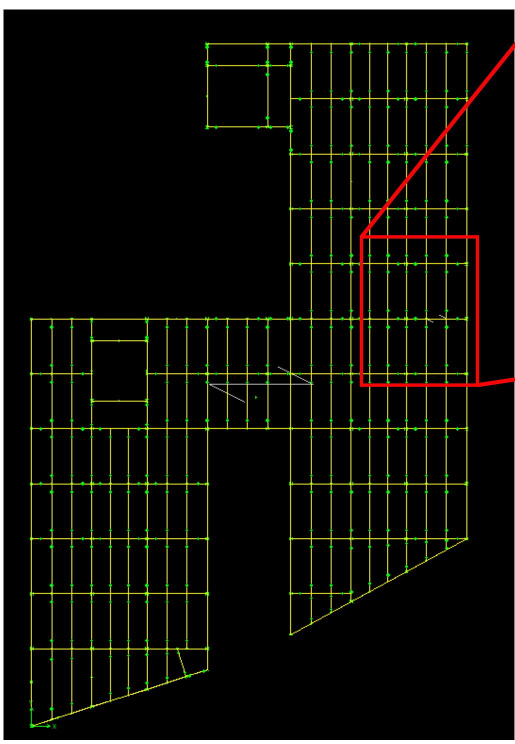
Introduction

Per occupancy category IV the structure must be able to bridge over a removed load-bearing vertical element, if it cannot then it must be redesigned to bridge over the element. Figure 23 below shows a plan and elevation view of removed column. The UFC specifies which columns must be removed in plan according to what occupancy level the building falls into, for occupancy IV the columns removed are as follows but not limited to:

1. Middle of short side
2. Middle of long side
3. At the corners of the building
4. Geometry of the structure changes
    a. Bay size changes
    b. Reentrant corners
    c. Locations where adjacent bays are lightly loaded resulting in small members than other bays

For each column removed in plan, the Alternate Path analyses must be performed for:

1. First story above grade
2. Story directly below roof
3. Story at mid-height



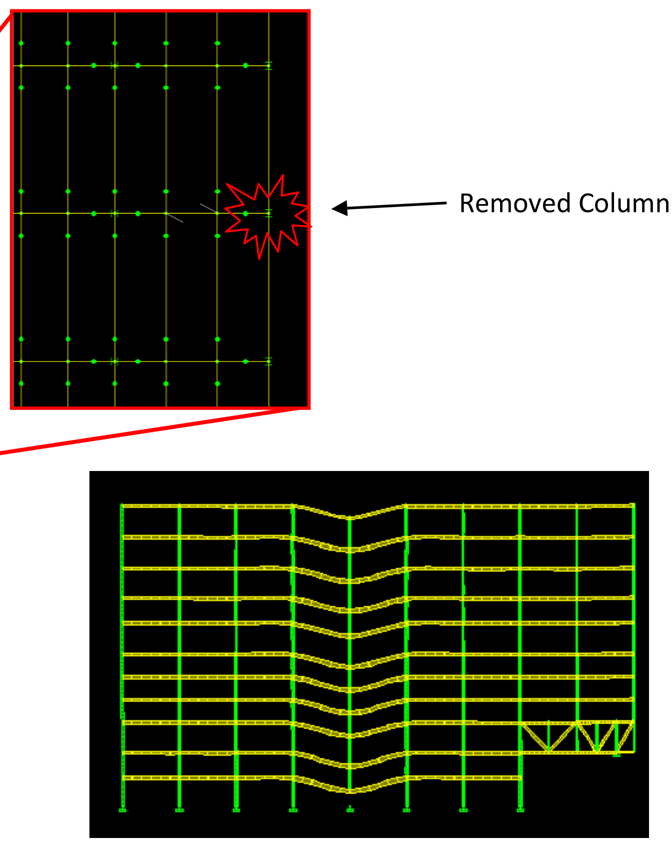


Figure 23 – Location of Removed Column Plan and Elevation

Unified Facilities Criteria 4-023-03

Design Process

Primary and Secondary Components

After the preliminary gravity analysis and the computer model has been generated, the next step is to define members as primary or secondary components. Primary components are members that are essential to resist collapse when removing a load-bearing element. Examples of primary components are beams, girders, and columns that directly resist collapse, these members have fixed connections. Secondary components are members that do not contribute in resisting collapse of the structure; examples of these are beams and girders which have zero fixity for their connections.

Force-Controlled and Deformation-Controlled Models

After the computer model is made, a second copy must be made in order to analyze the deformation and force controlled actions separately. Shown in Figure 24 below is a table provided by UFC summarizing deformation and force controlled actions per frame type and action.

Component	Deformation-Controlled Action	Force- Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V[1]
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M[2]	P, V, M

Figure 24 – Summary of Deformation- and Force- Controlled Actions

For example, when designing a column in a moment frame where the load-bearing element has been removed, the design moment will come from the Deformation-Controlled model and the design axial load will be obtained from the Force-Controlled model. If the axial load in columns if $P/P_{CL} \geq 0.5$ then it is considered a force-controlled member and the axial load and moment will both be obtained from the force-controlled model.

m-Factors

Each member and connection within the structure is assigned an m factor, also known as a demand modifier; this value greatly reduces the moment portion of the axial and bending capacity interaction diagram, see equation below:

For $P_r/P_c<0.2$:

$$\frac{P_r}{2P_c}+\left[\frac{\left[\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right]}{m\,factor}\right]\le 1.0$$

For $P_r/P_c>0.2$:

$$\frac{P_r}{P_c}+\frac{8}{9}\left[\frac{\left[\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right]}{m\,factor}\right]\le 1.0$$

These m-factors are based on the member slenderness, Chapter 5 in ASCE41-06 provides a table to calculate member m-factors based on component, action, primary/secondary components, and safety criteria see Figure 25 below. The primary and secondary components are broken down into Life Safety (LS) and Collapse Prevention (CP) categories, per UFC 4-023-03 the columns are designed as collapse prevention members and the beams and secondary members are designed as life safety members.

m-Factors for Linear Procedures

Component/Action	IO	Primary LS	Primary CP	Secondary LS	Secondary CP
Beams—Flexure					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolatio n between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				
Columns—Flexure [11,12]					
For $P/P_{CL} < 0.2$					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	1.25	1.25	2	2	3
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				
For $0.2 \le P/P_{CL} \le 0.5$					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	1.25	—[1]	—[2]	—[3]	—[4]
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	1.25	1.25	1.5	2	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				

[1] $m = 9(1 - 5/3\ P/P_{CL})$.
[2] $m = 12(1 - 5/3\ P/P_{CL})$.
[3] $m = 15(1 - 5/3\ P/P_{CL})$.
[4] $m = 18(1 - 5/3\ P/P_{CL})$.

Figure 25 – m-factors per ASCE41-06

Connection m-factors are provided for various types of connections; fully restrained, partially restrained moment connections, and partially restrained simple connections. The UFC provides a table, see Figure 26 below, to calculate the m-factor based upon connection type and primary or secondary components, if another connection is needed, Chapter 5 in the ASCE41-06 provides more connection types and their m-factors. The m-factors for the fully restrained moment connections are based upon the depth of the member and for the partially restrained members it is based upon the depth of the bolt group.

Connection Type	Linear Acceptance Criteria *m*-factors	
	Primary(1)	Secondary(1)
Fully Restrained Moment Connections		
Improved WUF with Bolted Web	2.3 – 0.021d	4.9 – 0.048d
Reduced Beam Section (RBS)	4.9 – 0.025d	6.5 – 0.025d
WUF	4.3 – 0.083d	4.3 -0.048d
SidePlate®	6.7 – 0.039d(2)	11.1 – 0.062d
Partially Restrained Moment Connections (Relatively Stiff)		
Double Split Tee		
a. Shear in Bolt	4	6
b. Tension in Bolt	1.5	4
c. Tension in Tee	1.5	4
d. Flexure in Tee	5	7
Partially Restrained Simple Connections (Flexible)		
Double Angles		
a. Shear in Bolt	$5.8 – 0.107d_{bg}$(3)	$8.7 – 0.161d_{bg}$
b. Tension in Bolt	1.5	4
c. Flexure in Angles	$8.9 – 0.193d_{bg}$	$13.0 – 0.290d_{bg}$
Simple Shear Tab	$5.8 – 0.107d_{bg}$	$8.7 – 0.161d_{bg}$

(1) Refer to Section 3-2.4 for determination of Primary and Secondary classification
(2) d = depth of beam, inch
(3) d_{bg} = depth of bolt group, inch

Figure 26 – Connection m-factors per UFC 4-023-03

Load Combinations

The gravity load combinations for both the deformation- and force-controlled models are shown below:

$$G=[1.2D + (0.5L \text{ or } 0.2S)]$$

For bays that are immediately adjacent to a removed column have a load increase factor applied to the above equation. Deformation-controlled actions have a multiplier, Ω_{LD}, the load increase factor for calculating gravity loads using the linear static analysis. The load increase factor depends on m_{LIF}, which is the smallest m-factor of any primary member or connection that is directly above the removed columns. For steel framing the equation for the load increase factor are as follows:

$$\Omega_{LD}=0.9\, m_{LIF} + 1.1$$

Force-controlled actions have a load increase factor Ω_{LF} which is equal to 2.0 for all material types and functions.

$$\Omega_{LF}=2.0$$

**Live load reduction may be applied per ASCE7-05.

In addition to the gravity loads, a lateral load assignment is made for deformation- and force-controlled actions. The lateral load is applied to each side of the building one side at a time and at each floor level. The lateral load L_{LAT} is the product of 0.002 and $\sum P$, the gravity loads (dead and live) associated with that floor, in the Table 21 below are the notional lateral loads per floor.

Table 21 - Notional Lateral Loads

Level	Area (Sq ft)	Dead (psf)	Live (psf)	L_{LAT} (kips)
Basement	44550	95	100	17.3745
Level 1	53326	95	100	20.79714
Level 2	48840	95	100	19.0476
Level 3	43591	95	80	15.25685
Level 4	44040	95	80	15.414
Level 5	44040	95	80	15.414
Level 6	44040	95	80	15.414
Level 7	44040	95	80	15.414
Level 8	44040	95	80	15.414
Penthouse	44039	125	125	22.0195
Roof	39500	95	30	9.875

Progressive Collapse Analysis – Primary Members

Virtual Work – Plastic Failure

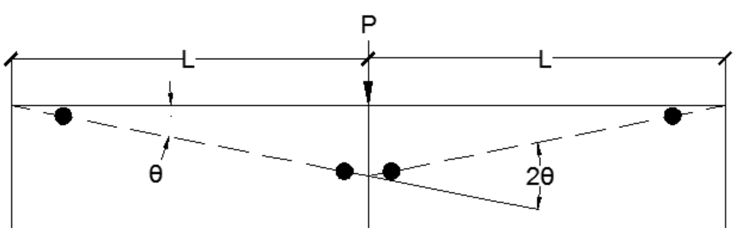
Before beginning the Alternate Path method, preliminary member calculations were made in order to start the analysis. Assumptions made during these calculations were:

1. A point load in the middle of the span is the product of the tributary width of the beam multiplied by their length
2. No live load reduction was performed
3. Deflections are neglected

The virtual-work method is based upon the external work equaling the internal work and for this method the small-angle theory is applied. The small angle theory means that the sine of a small angle equals the tangent of that angle which also equals that same angle expressed in radians.

The collapse mechanism for this structure involves having hinges form at the interior face of every column, therefore creating four hinges within the structure. The external work done on this structure is the resulting point load in the middle of the small and the internal work is the product of the sum of plastic moments at each hinge times the angle in which it works.

Once the plastic moment is calculated, the required section modulus can be calculated and a preliminary member can be chosen. See Figure 27 for calculations used to find the plastic moment required to induce a collapse mechanism for a two bay frame and section modulus, also see Appendix B for hand calculations.



$$(M_p \cdot \theta) + (M_p \cdot \theta) + (M_p \cdot 2\theta) + (M_p \cdot 2\theta) = P \cdot \delta$$

$$tan\theta \cong \theta = \frac{\delta}{\frac{L}{2}} \therefore \ \delta = \frac{\theta \cdot L}{2}$$

$$6(M_p \cdot \theta) = P \cdot \left(\frac{\theta \cdot L}{2}\right)$$

$$6M_p = \frac{P \cdot L}{2}$$

$$M_p = \frac{P \cdot L}{12}$$

$$M_p = \emptyset \cdot F_y \cdot Z$$

$$\frac{M_p}{\emptyset} = F_y \cdot Z \quad (LRFD\ approach)$$

$$\therefore \ Z = \frac{M_p}{\emptyset \cdot F_y}$$

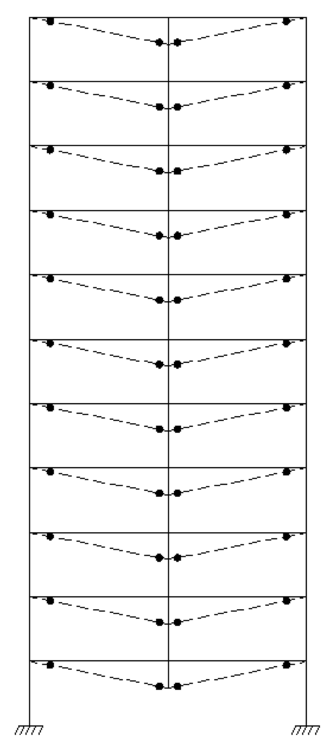


Figure 27 - Virtual Work; Plastic Analysis Method

Computer Model

ETABS was used to create a computer model for this analysis; the gravity system for the Patient Pavilion was added to the lateral model made for Technical Assignment 3. The gravity model was created using the same floor members from the existing design provided by Ryan Biggs Associates. The gravity members were modeled with the assumption they have zero resistance by releasing the moments for each interior member. In order to resist progressive collapse, the exterior bays around the perimeter building were modeled from pin-pin gravity frames to fully restrained moment frames.

New load combinations were created for this computer model to perform the progressive collapse analysis. According to the UFC, the two bays on either side of the removed column and above it must be loaded with a special load case specific for progressive collapse; all other bays are to be loaded with load combinations per the ASCE7-05. The new load combinations and their locations are shown below in Table 22 and Figure 28:

Table 22 - Load Combinations

	Load Combinations
PCLATX	$\Omega[1.2D + (0.5L \text{ or } 0.2S)]+0.002\sum P_x$
PCLATY	$\Omega[1.2D + (0.5L \text{ or } 0.2S)]+0.002\sum P_y$
GRAVITY	1.2D + 1.6L + 0.2S

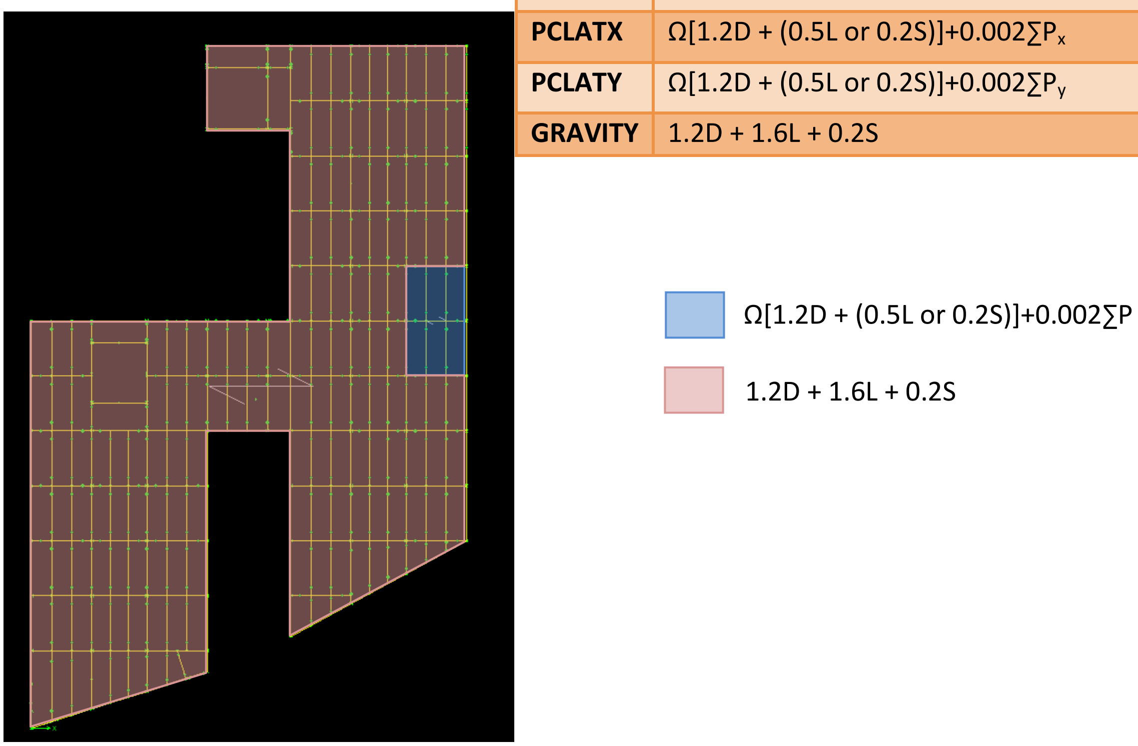


Figure 28 - Load Case Locations

PCLATX and PCLATY are the same gravity case, where they differ is the notional lateral load that is applied to each floor level, one is applying the load in the X-direction, the other applies the load in the Y-direction. These combinations are both applied to the two adjacent bays in order to simultaneously consider each load case, reducing iteration time.

In order to analyze deformation- and force-controlled members simultaneously a copy of the computer model was made and the load increase factors were applied to the gravity loads accordingly. These two computer models were replicated three more times a piece to analyze a column removed at the base, mid-height, and the roof level for both deformation- and force-controlled members. Figure 29 below shows the elevation view of the three different locations where the column was removed to perform this analysis.

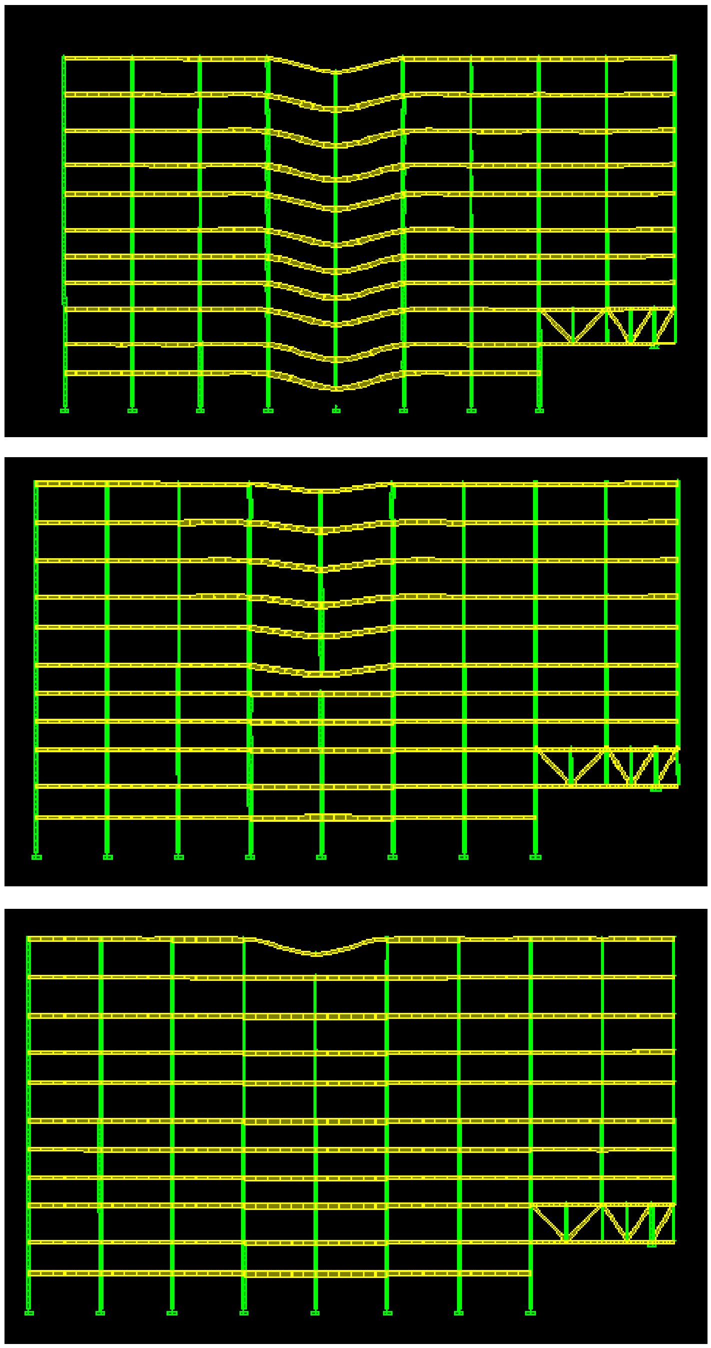


Figure 29 - Locations of Removed Columns

m-Factors

Once the computer model was created and the preliminary members were designed, the m-factors for these members and their connections must be calculated in order to determine the load increase factor and apply it to the gravity loading in the computer model. The m_{LIF} factor can then be determined after all the m factors are calculated; using the m_{LIF} the load increase factor can be calculated. See Tables 23 to 25 for tabulated beam, girder and connection m factors and the calculated load increase factors.

Table 23 – Component m factors for Primary Deformation-Controlled Actions

Level	Beam/Girder	Beam/Girder m factor	Connection m factor
Roof	W21x50	6.0	2.57
Penthouse	W24x76	6.0	2.32
Basement to 8th	W24x62	6.0	2.33

Table 24 – Load Increase Factors: Deformation-Controlled

m_{LIF} (Smallest m factor)	Ω_{LD}=0.9 m_{LIF} + 1.1	$\Omega_{LD} \cdot$ 1.2D	$\Omega_{LD} \cdot$ 0.5L	$\Omega_{LD} \cdot$ 0.2S
2.32	3.18	3.82D	1.59L	0.64S

Table 25 – Load Increase Factors: Force-Controlled

m_{LIF} (Smallest m factor)	Ω_{LF}=0.9 m_{LIF} + 1.1	$\Omega_{LD} \cdot$ 1.2D	$\Omega_{LD} \cdot$ 0.5L	$\Omega_{LD} \cdot$ 0.2S
2.0	2.9	3.48D	1.45L	0.58S

Beam Design

Preliminary members sizes for the exterior beams were calculated by using the virtual work method with the assumption that the members hinge at the face of every column, creating four hinges in the two bays. The results from the virtual work method are shown below in Table 26. Figure 30 shows the resulting interactions for the beams per column removal location and beam location.

The results from the virtual method for the beams were deemed sufficient when input into the computer model and the progressive collapse analysis was run. The axial load was negligible in the beams therefore the deformation-controlled model was used to analyze these members. The interaction equation H1(b) from the AISC 14^{th} Edition as well as the m factor was used to determine whether or not the members were adequate for progressive collapse.

H11(b) For $P_r/P_c<0.2$:

$$\frac{P_r}{2P_c}+\left[\frac{\left[\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right]}{m\,factor}\right]\leq 1.0$$

Table 26 - Virtual Work Results

Level	Z_{req} (in^3)	Member
Roof Level	101.6	W21x50
Penthouse Level	191.5	W24x76
1st to 3rd Level	149	W24x68

Removed Column	Member Location	Member	Bending Axis	F_y	Z	Area	M_r	P_r	M_c	P_c	m-factor	Interaction	
F-9 Base	Roof	W21X50	Z_x	50	110	14.7	268.6	0	412.5	661.5	6	0.10852525	PASS
	Basement to 8th	W24X62	Z_x	50	153	18.2	1582.4	0	573.75	819	6	0.45966594	PASS
	Penthouse	W24X76	Z_x	50	200	22.4	1347.1	0	750	1008	6	0.29935556	PASS
F-9 Middle	Roof	W21X50	Z_x	50	110	14.7	302.7	0	412.5	661.5	6	0.12230303	PASS
	Basement to 8th	W24X62	Z_x	50	153	18.2	1284.1	0	573.75	819	6	0.3730138	PASS
	Penthouse	W24X76	Z_x	50	200	22.4	1481.6	0	750	1008	6	0.32924444	PASS
F-9 Roof	Roof	W21X50	Z_x	50	110	14.7	916.4	0	412.5	661.5	6	0.37026263	PASS
	Basement to 8th	W24X62	Z_x	50	153	18.2	85.4	0	573.75	819	6	0.02480755	PASS
	Penthouse	W24X76	Z_x	50	200	22.4	109.3	0	750	1008	6	0.02428889	PASS

Figure 30 - Beam Design Interaction Results

Column Design

The column design was a more iterative process due to the method of determining the m factor, the process took four iterations to determine members that were adequate for the bay under consideration. Column m factors depend on the ratio of P_r/P_c, if that ratio is above 0.5 then the member is defined as force-controlled therefore the m factor is 1.0 and there is no reduction in the moments. The key to getting members to work was to design large heavy W14's so their axial capacity was greater than two times the axial load on the member. Expediting the process of calculating axial loads, an Excel spreadsheet was created to quickly obtain the axial loads at every level for a given tributary width and influence area.

For the first iteration, the initial gravity column design was analyzed first and deemed insufficient for progressive collapse, which was expected. The second and third iteration both failed when analyzing the removal of the base level column, the issue was then determined that the P_r/P_c was greater than 0.5 therefore the member was a force controlled member not deformation controlled and the m factor was 1.0. The fourth and final iteration, the axial load on the column and the allowable axial load of the column were taken into greater consideration. All the members except for two were force-controlled members; however, the force-controlled members passed the interaction unity equation and the design was deemed sufficient. The different iterations and results for the column design are shown on the next page in Figure 31, more detailed calculations from Excel spreadsheets are provided in the Appendix C.

Removed Column	Interaction		Original Design	Iteration 1	Iteration 2	Iteration 3	Iteration 4
F-9 Base	0.07670476	PASS	~~W14x74~~	~~W14x109~~	~~W14x145~~	W14x176	
	0.14522504	PASS					
	0.32948002	PASS					
	0.42507717	PASS					
	0.62502045	PASS					
	0.31413079	PASS	~~W14x109~~	~~W14x145~~	~~W14x311~~	W14x342	
	0.37123647	PASS					
	0.43088985	PASS					
	0.75773136	PASS					
	0.57200347	PASS	~~W14x176~~	~~W14x193~~	W14x370	W14x370	
	0.65259382	PASS					
F-9 Middle	0.06715387	PASS				~~W14x176~~	W14x193
	0.11388562	PASS					
	0.08612531	PASS					
	0.2812003	PASS					
	0.38307581	PASS					
	0.12405204	PASS				W14x342	W14x342
	0.22183674	PASS					
	0.23617975	PASS					
	0.3028998	PASS					
	0.2595143	PASS				W14x370	W14x370
	0.28329749	PASS					
F-9 Roof	0.0386231	PASS					W14x193
	0.05212593	PASS					
	0.06467891	PASS					
	0.07330856	PASS					
	0.09720434	PASS					
	0.05621943	PASS					W14x342
	0.06487096	PASS					
	0.07174589	PASS					
	0.09718122	PASS					
	0.0867758	PASS					W14x370
	0.0988021	PASS					

Figure 31 - Primary Column Results

Enhanced Local Resistance

Introduction

Enhanced local resistance is the final check in the progressive collapse primary member check. This final check is comprised of two parts, enhanced flexural resistance and enhanced shear resistance. Hand calculations for both enhanced flexural resistance and enhanced shear resistance can be found in Appendix D.

Enhanced flexural resistance is the first part of final check in the progressive collapse analysis method described by the United Facilities Criteria. Per occupancy level IV two flexural resistances must be calculated and compared against each other. First the baseline flexural resistance is considered, this is the flexural resistance of the gravity columns that were designed prior to the progressive collapse design. Secondly the existing flexural resistance is to be calculated, which is the flexural resistance of the columns after the progressive collapse analysis.

For occupancy IV if the existing flexural resistance of the columns is greater than two times the baseline flexural resistance the members are sufficient. However, if the existing flexural resistance is less than two times the baseline model, the members must be redesigned until this check passes.

Enhanced shear resistance is the shear resistance of the load bearing member considered in the existing flexural resistance calculations. The shear force is calculated from the flexural condition described above for the existing flexural resistance of a given column; the AISC was used next to calculate the allowable shear for that column. If the member is not sufficient for the shear, doubler plates must be welded to the web to resist the shear force.

Design Process/Results

Occupancy IV requires that enhanced local resistance must be performed on the lower two levels around the perimeter of the building. The lower two story members for the baseline model are both W14x176s and for the existing model they are W14x370s. The plastic moment of these two members is the product of the expected yield strength, F_{YE}, and the section modulus, Z_x, the existing structure was 2.3 times larger than the baseline structure, therefore it is sufficient.

The enhanced shear resistance shear load was determined using a simplified ETABS computer model. A single eleven story column was modeled using the same member properties determined from the alternate path method. At each story level the node was restrained in all directions and rotations except rotation around the z-axis, to essentially create a pin-pin connection, eleven bays long. The load was applied to the column as a distributed load and was increased until the lower two members failed flexure by 1.0 or greater. Once the member failed, the shear values were taken from the computer model and used to perform the enhanced shear resistance analysis.

The shear capacity for the wide flange was calculated per equation G2-1 per the AISC 14th Edition (shown below). For the base of the first story column the shear capacity was exceeded and a 3/8" plate needed to be welded to the web of the column. Next the length of the plate needed to be determined, this was performed using geometry, see Figure 32 for the calculations that were used to determine the length of the doubler plate.

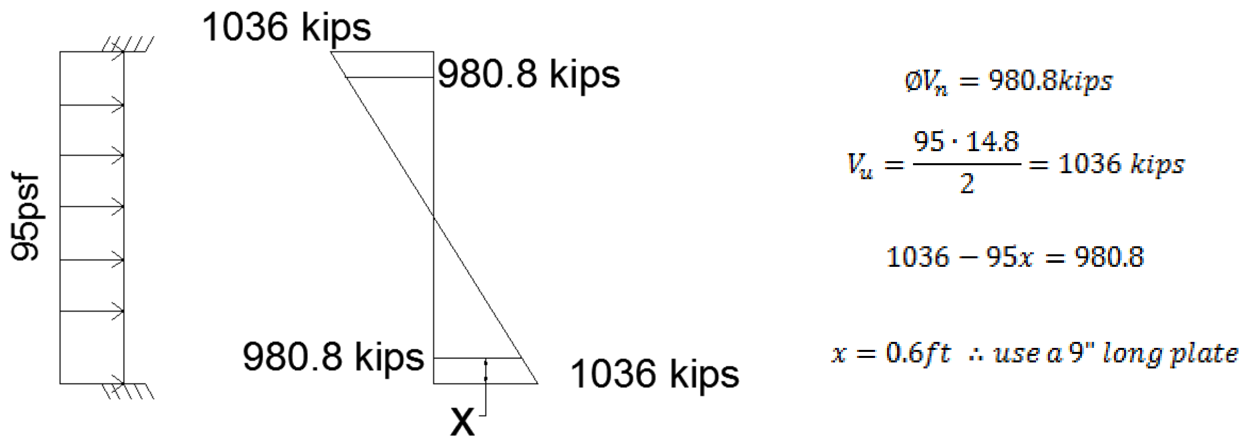


Figure 32 - Doubler Plate Calculations

Secondary Components

Following the alternate path method analysis a secondary component check must be performed to verify the members not directly resisting collapse are sufficient when the vertical load-bearing member is removed. The secondary beams must be analyzed to verify they are sufficient in bending, and their connections will also be analyzed. See Appendix G for secondary beam m-factors.

Beam Design

A simple shear tab connection is assumed for the connection of all secondary members and their m factors have been provided in Table 27. An assumption was made that the depth of the bolt group was equal to half the member depth prior to designing the connections. The process for analyzing the secondary members is the very similar to analyzing primary members, the difference is the upper and lower bounds of the m factors provided in Figure 25 on page 36 in this report. The results of the analysis are shown on the next page in Figure 33.

Table 27 - Shear Tab Connection m-factors

Member Size	d	m-factor
W24X68	23.7	6.79215
W18X143	19.5	7.13025
W24X94	24.3	6.74385
W16X89	16.8	7.3476
W24X76	23.9	6.77605
W18X35	17.7	7.27515

Removed Column	Level	Member	Bending Axis	F_y	Z	Area	M_r	P_r	M_c	P_c	m-factor	Interaction	
F-9 Base	Roof	W18X35	Z_x	50	66.5	10.3	761.4	0	249.375	463.5	7.41	0.41198478	PASS
	Penhouse	W24X76	Z_x	50	200	22.4	1432.5	0	750	1008	10.00	0.191	PASS
	8th-3rd	W16X89	Z_x	50	175	26.2	1021.5	0	656.25	1179	10.00	0.15565714	PASS
	2nd	W24X94	Z_x	50	254	27.7	1102.8	0	952.5	1246.5	10.00	0.11577953	PASS
	1st	W18X143	Z_x	50	322	42	1101.1	0	1207.5	1890	10.00	0.09118841	PASS
	Basement	W24X68	Z_x	50	177	20.1	1154.4	0	663.75	904.5	9.81	0.17734298	PASS
F-9 Middle	Roof	W18X35	Z_x	50	66.5	10.3	761.1	0	249.375	463.5	7.41	0.41182246	PASS
	Penhouse	W24X76	Z_x	50	200	22.4	1431.9	0	750	1008	10.00	0.19092	PASS
	8th-3rd	W16X89	Z_x	50	175	26.2	969.1	0	656.25	1179	10.00	0.14767238	PASS
	2nd	W24X94	Z_x	50	254	27.7	419	0	952.5	1246.5	10.00	0.0439895	PASS
	1st	W18X143	Z_x	50	322	42	418.2	0	1207.5	1890	10.00	0.03463354	PASS
	Basement	W24X68	Z_x	50	177	20.1	418.2	0	663.75	904.5	9.81	0.06424535	PASS
F-9 Roof	Roof	W18X35	Z_x	50	66.5	10.3	848.8	0	249.375	463.5	7.41	0.45927592	PASS
	Penhouse	W24X76	Z_x	50	200	22.4	546.8	0	750	1008	10.00	0.07290667	PASS
	8th-3rd	W16X89	Z_x	50	175	26.2	365.1	0	656.25	1179	10.00	0.05563429	PASS
	2nd	W24X94	Z_x	50	254	27.7	421.5	0	952.5	1246.5	10.00	0.04425197	PASS
	1st	W18X143	Z_x	50	322	42	420	0	1207.5	1890	10.00	0.03478261	PASS
	Basement	W24X68	Z_x	50	177	20.1	420	0	663.75	904.5	9.81	0.06452187	PASS

Figure 33- Secondary Beam Interaction Results

Connection Check

The calculations for the m factor are in Figure 25 of this report, a table provided by the UFC, for a partially restrained simple shear tab, the equation for the m factor is provided below:

$$m = 8.7 - 0.161 d_{bg}$$

Where d_{bg} is the depth of the bolt group and the assumption was made that the depth of the bolt group was equal to half the depth of the web, the calculated m factors are shown in Table 27 on page 46 of this report.

To analyze the simple connections a comparison between the rotational demand of the deflected member, the deflection at the unsupported end divided by the length of the member, and the connection rotational stiffness, provided by ASCE41-06, see equation below:

$$K = \frac{M_{CE}}{0.005}$$

If the m factor of the connection is larger than the rotational demand divided by 0.005 then the connection is sufficient. Figure 34 provides the results of the secondary component connection check.

Column	Member Size	m-factor	Relative Deflection (in.)	Length (in)	Rotational Demand (rad.)	Ratio=(θ_{demand}/0.005)	Ratio to m-factor
Base Level	W24X68	5.79215	7.59	360	0.021083333	4.216666667	Pass
	W18X143	6.13025	7.59	360	0.021083333	4.216666667	Pass
	W24X94	5.74385	7.59	360	0.021083333	4.216666667	Pass
	W16X89	6.3476	7.59	360	0.021083333	4.216666667	Pass
	W24X76	5.77605	7.59	360	0.021083333	4.216666667	Pass
	W18X35	6.27515	7.59	360	0.021083333	4.216666667	Pass
Middle	W24X68	5.79215	0.02	360	5.55556E-05	0.011111111	Pass
	W18X143	6.13025	0.32	360	0.000888889	0.177777778	Pass
	W24X94	5.74385	0.24	360	0.000666667	0.133333333	Pass
	W16X89	6.3476	8.2	360	0.022777778	4.555555556	Pass
	W24X76	5.77605	8.2	360	0.022777778	4.555555556	Pass
	W18X35	6.27515	8.2	360	0.022777778	4.555555556	Pass
Roof	W24X68	5.79215	0.082	360	0.000227778	0.045555556	Pass
	W18X143	6.13025	0.051	360	0.000141667	0.028333333	Pass
	W24X94	5.74385	0.25	360	0.000694444	0.138888889	Pass
	W16X89	6.3476	0.61	360	0.001694444	0.338888889	Pass
	W24X76	5.77605	0.64	360	0.001777778	0.355555556	Pass
	W18X35	6.27515	9.7	360	0.026944444	5.388888889	Pass

Figure 34 - Secondary Connection Results

MAE Connection Design

Utilizing knowledge obtained in AE534: Analysis and Design of Steel Connections, a welded unreinforced flange moment connection and a shear tab connection were designed. The shear and moments in these two connections are both deformation-controlled actions; therefore, the forces were obtained from the deformation-controlled model. Hand calculations for both connections are provided in Appendix E at the end of this report.

Shear Tab Connection

The secondary members in the Patient Pavilion have pin-pin connections, an extended shear tab connection was chosen to connect the beams to their connecting elements. Figure 35 shows the shear in the connection and the results of the connection design. The shear force in the member was divided by the m factor for the connection to reduce the shear to 30.3 kips from 206 kips. Assumptions made in the design of this connection were:

1. Plate is A36 Steel
2. 1” dia. A325-N Bolts
3. 5 bolts
4. ½” offset from column flanges

The design process followed the guidelines provided in the AISC 14th Edition, the shear tab was assumed to be a bolted connection in the flange and fillet welds on either side of the tab at the column web connection. The limit states that were taken in consideration for designing this connection are as follows:

1. Bolt shear strength
2. Plate and web bearing
3. Max plate thickness
4. Plate shear yield/rupture
5. Plate block shear
6. Plate flexure and shear interaction
7. Plate buckling
8. Weld Strength

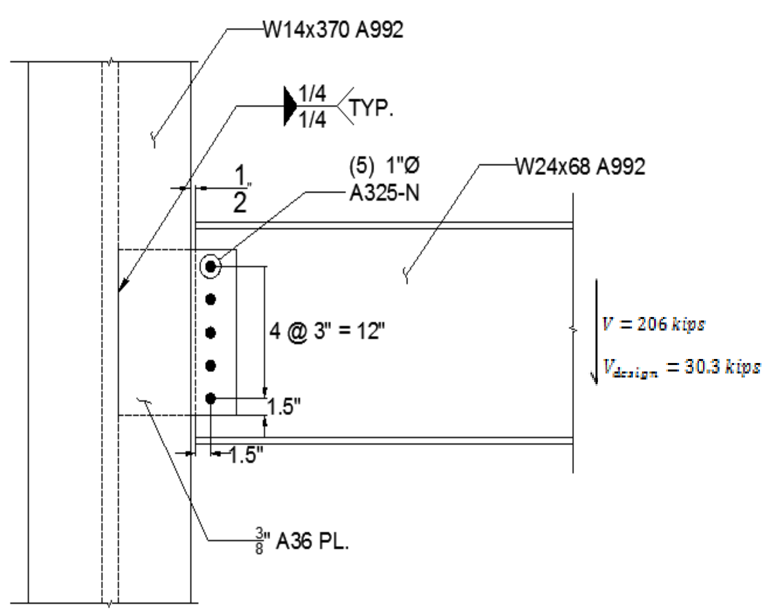


Figure 35 - Extended Shear Tab Connection

Welded Unreinforced Flange Moment Connection

The primary beams around the perimeter of the building are fix-fixed connections to resist progressive collapse, therefore a moment connection needed to be designed. Figure 36 shows the forces in the connection and results of the final connection design adequate for the given loads. The moment and shear forces shown below have already been divided by the m factor for the W24x84.

The beam flange to column connection are full penetration welds in order to develop the entire flange to resist the moment at the column face. The limit states for the beam are as follows:

1. Tension and compression forces in the flange created by the applied moment
2. Shear at the column face

The original beam design was a W24x68, this member failed the first limit state. The load increase factor, which was derived in the alternate path method, was dependent on the connection type, the WUF connection is still going to be utilized. Therefore, W24x84 is analyzed next because the flange area is larger and the depth is approximately the same, there is no need to recalculate the m factor. This member was deemed sufficient.

The column limit states were evaluated next to ensure the flanges, web of the column were sufficient for the applied shear from the tension, and compressive forces in the beam flange. The limit states used for this design are:

1. Local flange buckling
2. Local web yielding
3. Local web crippling
4. Web Buckling

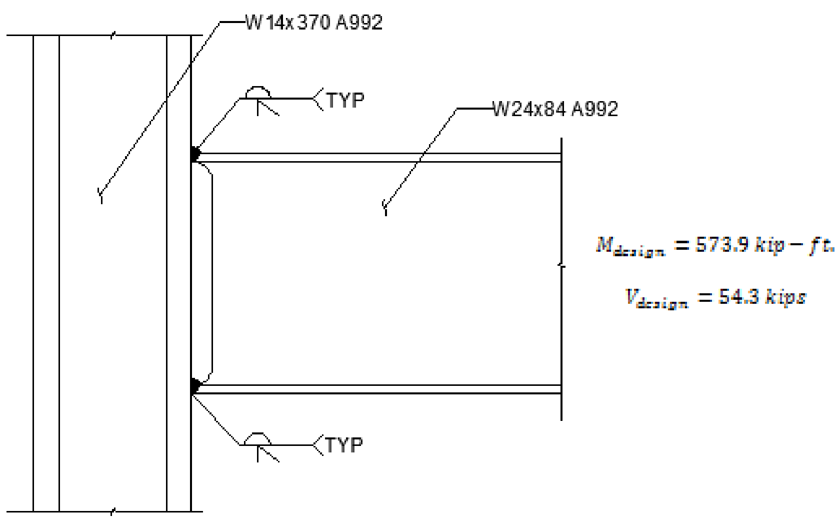


Figure 36 - WUF Connection

Mechanical Breadth – Façade Study

U-value Calculations

Wall Construction

Comparing the patient room's thermal performance of the existing buildings hand laid brick envelope with a new precast façade, the overall heat transfer coefficients (U-values) were first calculated. Computer model HAM toolbox was utilized to run the U-value calculations in order to save time, in Figure 37 below are the results of HAM toolbox for the existing system. The results give the R-value for the wall construction; the inverse of this number is the U-value for the construction.

Layer	Generic Material	Thick.	R Val.
1	air film (ext), 3/4 in.	0.75	0.17
2	brick, (vented), 4 in.	3.50	0.64
3	cavity, 1 in.	1.00	0.98
4	cavity, 3/4 in.	0.75	0.69
5	rigid ins.,(expand.), 2-1/2 in.	2.50	9.88
6	poly film, (6mil)	0.01	0.12
7	plywood shtg., 1/2 in.	0.50	0.64
8	steel stud, 5-1/2 in.	5.51	0.12
9	gypsum bd., 5/8 in., (#2)	0.63	0.46
10			
11			
12			
	Total or (Layer 0)	14.39	13.69

Figure 37 - R-value for Existing Façade Construction

The resulting U-Value is 13.69^{-1} which is equal to 0.073.

A new wall panel had to be chosen with an R-value greater than 13.69. US Precast Corp was contacted and they suggested a sandwich panel consisting of 3" of concrete, 2" of insulation, and 3" of concrete, a resulting R-value of 15.69, a U-value of 0.0637.

Glazing

The existing glazing in the patient room is Viracraft Low-E VE19-2M with argon gas, the performance data and construction is shown below in Table 28. For the glazing redesign, the shading coefficient was to be lowered considering a large amount of the patient rooms are on the east and south side of the building. The proposed glazing is Oldcastle BuildingEnvelope SunGlass Low-E #2 argon fill and the exterior glazing is color gray, see Table 28 for construction and performance data.

Table 28 - Glazing Properties and Construction

Properties	Viracon	Oldcastle
Exterior Lite	1/4" VE-2M #2	1/4" Oldcastle BuildingEnvelope SunGlass Low-E #2
Cavity	1/2" Argon Space	1/2" (90% Argon fill)
Interior Lite	1/4" Clear	1/4" Clear
Color	Gray	Gray
U-value	0.25	0.24
SC	0.3	0.28

TRACE Model

A trace model was made for a patient room on the perimeter of the building; this room was typical for all the exterior patient rooms. This patient room was replicated four times to represent four typical patient rooms on each side of the building to analyze the thermal performance of the wall and glazing system completely. TRACE model output results are provided in Appendix F at the end of this report.

Templates

Templates in the TRACE model consist of internal loads, airflow, thermostat, construction, and room, each of these were modified to replicate the thermal conditions of the patient rooms. The internal loads template is the same for all patient rooms, this section calculates the heat gain in room based on occupancy, equipment, lighting, and workspace. Values for these were pulled from the book <u>Mechanical and Electrical Equipment for Buildings</u> and the lighting load was estimated using the equation shown below:

$$DF = 0.2 \cdot \left(\frac{A_{glazing}}{A_{floor}}\right) \geq 1.0\ if\ not\ lighting\ load = \ 5.1\ Btu/h \cdot ft^2$$

The airflow template calculates the heating and cooling demand based on ventilation, infiltration, room exhaust, and minimum variable air volume, ASHRAE98 was utilized to determine these values for the patient room. In addition, it was found that the relative humidity for patient rooms should be between 30% and 60% and the temperature should be between 68 and 73 degrees. This information was used in the thermostat tab; the values used were 50% humidity and 70-degree room temperature. The final tab is the wall construction and all values were neglected for the glazing and the wall U-value was overwritten manually in the box with the U-value calculated in the previous section.

Rooms

Four room templates were made based on which side of the building the room was located. Geometry inputs for this section are shown in Table 29 for both the existing and proposed wall systems.

Table 29 - Wall Details

Direction	Floor Area (sq ft)	Direction (degrees)	Height (ft)	Net Wall Area (sq ft)	Gross Wall Area (sq ft)	Glazing area (sq ft)
North	176	0	12.5	170	112.58	57.42
South	176	208.625	12.5	170	112.58	57.42
East	176	270	12.5	170	112.58	57.42
West	176	90	12.5	170	112.58	57.42

Within the rooms tab was a wall design, here the U-values and shading coefficients for the glazing were input into the model.

Results

Once two alternatives were created for the existing wall and the proposed wall, the TRACE model was run and the resulting thermal performance was obtain in the Room Checksums. The results pulled from the program are the cooling and heating loads in btu/h per each patient room in each direction. The cooling and heating loads for each system are shown below in Figures 38 and 39.

Existing									
	Cooling						Total (Btu/h)		
Wall	Glass	Wall	Infiltration	Lights	People	Equipment	Envelope	Internal Loads	Total
South	2691	492	453	720	640	78	3636	1438	10934
East	3816	614	410	300	640	80	4840	1020	8108
North	680	217	453	180	640	78	1350	898	7873
West	3847	517	241	300	640	80	4605	1020	5625

Proposed									
	Cooling						Total (Btu/h)		
Wall	Glass	Wall	Infiltration	Lights	People	Equipment	Envelope	Internal Loads	Total
South	1979	432	453	720	640	78	2864	1438	9412
East	3142	538	410	300	640	80	4090	1020	7212
North	560	191	453	180	640	78	1204	898	6474
West	2638	473	241	300	640	80	3352	1020	4372

Figure 38- Cooling Loads for Existing and Proposed Wall Systems

Existing									
	Heating						Total (Btu/h)		
Wall	Glass	Wall	Infiltration	Lights	People	Equipment	Envelope	ternal Loa	Total
South	-868	-605	-886	0	0	0	-2359	0	-2359
East	-868	-605	-886	0	0	0	-2359	0	-2359
North	-868	-605	-886	0	0	0	-2359	0	-2359
West	-868	-605	-886	0	0	0	-2359	0	-2359

Proposed									
	Heating						Total (Btu/h)		
Wall	Glass	Wall	Infiltration	Lights	People	Equipment	Envelope	ternal Loa	Total
South	-868	-530	-886	0	0	0	-2284	0	-2284
East	-868	-530	-886	0	0	0	-2284	0	-2284
North	-868	-530	-886	0	0	0	-2284	0	-2284
West	-868	-530	-886	0	0	0	-2284	0	-2284

Figure 39 - Heating Loads for Existing and Proposed Wall Systems

Material and Installation Cost Comparison

An initial cost analysis was performed for the two wall systems, the material costs and installation costs were evaluated in this comparison. RS Means 2010 building cost construction data was used to determine the material costs for the hand laid brick per square foot, an estimate for the material costs for the precast wall system was given by US Precast Corp. The installation costs in the RS Means were tabulated while calculating the material costs for the construction of the existing façade, US Provided Corp also provided an estimate of installation costs for the precast wall panels. See Table 30 below for façade cost comparison.

Table 30 - Facade Cost Comparison

Façade Cost Comparison			
	Existing	Proposed	Cost Increase
Sq ft/Room	105.33	105.33	
# Rooms	126	126	
Cost/sq ft	$ 26.14	$ 29.00	
Total Cost	$ 346,919.10	$ 384,875.82	$ 37,956.72

Estimates for the glazing were provided by the manufacturers in cost per square foot from both Vircon Glazing and Oldcastle Glazing, see Table 31 for glazing cost comparisons. Window framing was neglected in these calculations assuming the framing would stay the same.

Table 31 - Glazing Cost Comparison

Glazing Cost Comparison			
	Existing	Proposed	Cost Increase
Sq ft/Room	58.67	58.67	
# Rooms	126	126	
Cost/sq ft	$ 11.00	$ 12.80	
Total Cost	$ 81,316.62	$ 94,622.98	$ 13,306.36

In conclusion, the new precast façade has an increased upfront cost of $51,263.07 based on the glazing and wall material costs and the installation costs for those materials.

Energy Cost Savings

Analyzing how the thermal performance improvement affects all the patient rooms, further calculations must be made with the cooling and heating loads results in TRACE. Degree-days is a fairly accurate method to approximate the heating and cooling demand for the entire building or space. The more south a building is located in the United States, the more cooling degree-days will be demanded, and the more north a building, the more heating degree-days. Table 32 depicts the cooling and heating degree hours per month for Albany, New York with the assumption that the indoor temperature is 70 degrees.

Table 32 - Degree Days

Month	Interior	Exterior	DT	Deg. Days	Deg. Hours	Heating	Cooling
Jan	70	23	47	1457	34968	34968	
Feb	70	26	44	1232	29568	29568	
Mar	70	35	35	1085	26040	26040	
Apr	70	48	22	660	15840	15840	
May	70	58	12	372	8928	8928	
Jun	70	68	2	60	1440	1440	
Jul	70	72	-2	-62	-1488		-1488
Aug	70	70	0	0	0	0	
Sep	70	62	8	240	5760	5760	
Oct	70	50	20	620	14880	14880	
Nov	70	40	30	900	21600	21600	
Dec	70	29	41	1271	30504	30504	
			Totals	**7835**		**189528**	**-1488**

According to the Energy Information Administration the average cost per kWh for Albany, New York is $0.1446. The cooling and heating demands from the TRACE model need to be converted to kWh and then energy cost per kWh can be multiplied by the cooling and heating demands and the cost savings per year for the proposed façade can be calculated. Table 33 shows the cooling and heating energy savings per wall based upon the TRACE results, number of rooms per façade, and energy cost per kWh, more in depth calculations are provided in Appendix F.

Table 33 - Total Energy Savings for Proposed Façade

Energy Savings		
Wall	Heating	Cooling
North	$ 27,107.43	$ 3,969.85
South	$ 10,842.97	$ 1,727.55
East	$ 78,912.75	$ 7,401.56
West	$ 28,914.60	$ 3,792.60
Total Annual	$ 145,777.75	$ 16,891.56
	Total	**$ 162,669.32**

The results of the mechanical breadth show that the energy savings for the patient rooms are $162,669 per year. By providing a better U-value for the wall and decreasing the shading coefficient in the glazing the overall performance of the façade has a dramatic effect on the energy costs per year.

Construction Management Breadth – Site Logistics

Precast Façade Site Logistics

The second breadth is to determine whether a precast panel façade is feasible to construct due to the existing buildings along the western side of the site. If the precast façade is too complex, a site logistics per phase will created to complete the site logistics breadth.

A schematic plan of the site was created to better visualize the size of the site and flow of traffic in and out of the site, see Figure 40. The Patient Pavilion site is in red and the blue line to the left is the existing buildings on the Albany Medical Center Campus. The southern side of the site is fairly close to New Scotland Avenue, a two way main avenue through Albany, jersey barriers (shown in orange) are needed along this street to protect the workers on that site as well as equipment. Traffic flow and a traffic island are on the northeast side of the site to control traffic going into the truck staging area which is located further on up Myrtle Avenue.

Figure 40 - Schematic Site Plan

Studying the site plan, it was determined that there would not be enough room on the southern, western, and northern side of the site for a crane to hoist the panels up. A solution to this was to rooftop crane and have the delivery trucks deliver the panel onto the site and drive under the bay where the panel was to be attached and have the rooftop crane lower it's cables, attach the panel to them then lift it up. A site logistic was created for this solution and is shown below in Figure 41.

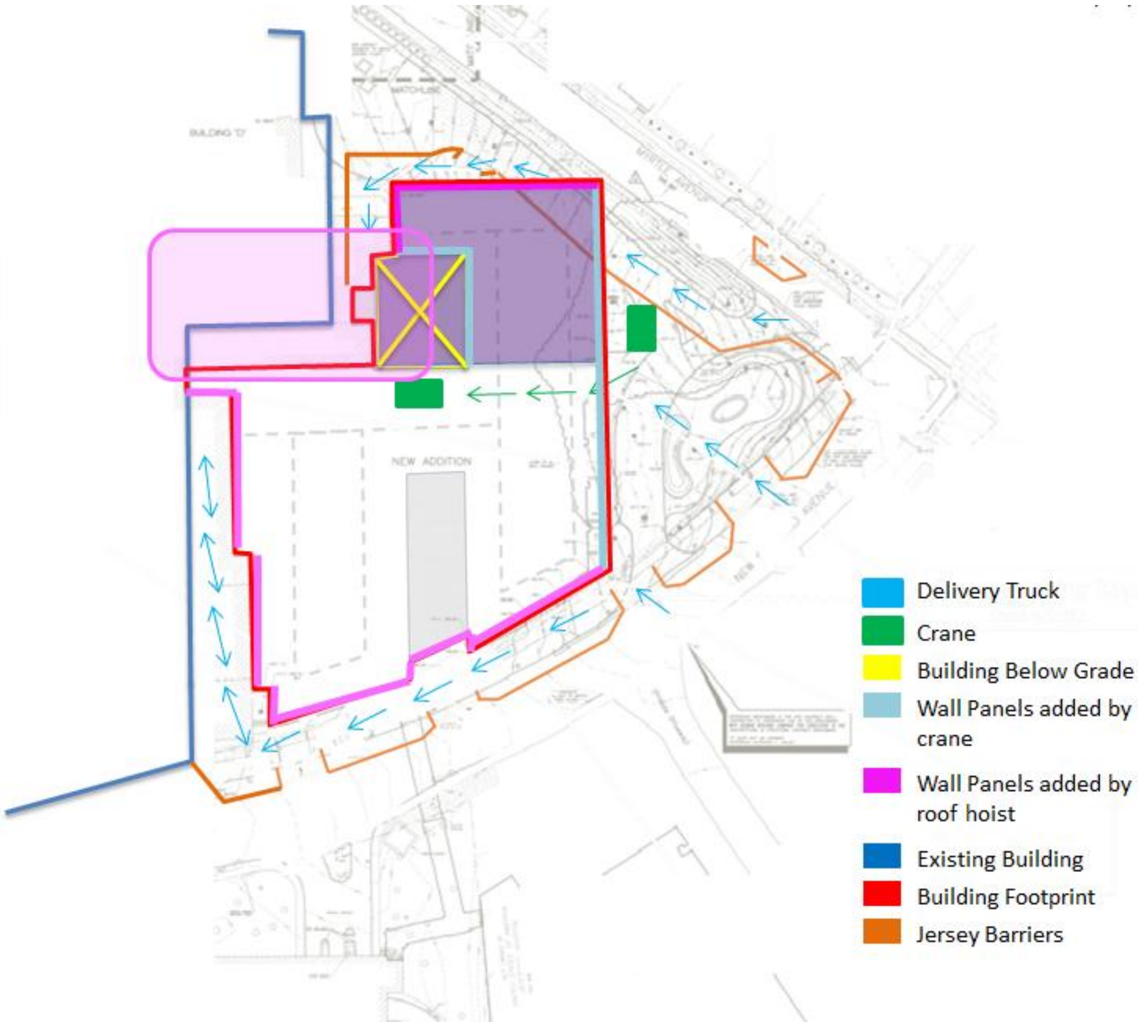


Figure 41 - Precast Panel Site Logistic

The construction would have to be phased, erecting the steel on the north side of the building and after erection, delivery trucks would have to enter on the north side of the site delivering panels to the roof jib. As the panels are going up on the north side of the site, the crane would be placing panels on the east façade and when that is finished, it would move to the farther left position and put the panels on the façade above the building below grade (the yellow X).

The feasibility of this construction is very difficult and it will require lots of coordination between the labor workers on site, the delivery drivers, and the precast manufacturer. The precast panels must be hoisted up by a rooftop crane on three of the four sides of the site. Also two sections of the site the delivery trucks would have to enter the site and back out the same way they entered consuming time and requiring more coordination so trucks don't get backed up.

It was determined that constructing the façade of the Patient Pavilion would not be an economical choice due to complexity and cost.

Site Logistics

Phase 1 – Foundation Pour

A site logistics plan was created for the existing Patient Pavilion as built, starting with Phase 1, pouring the foundations, shown in Figure 42.

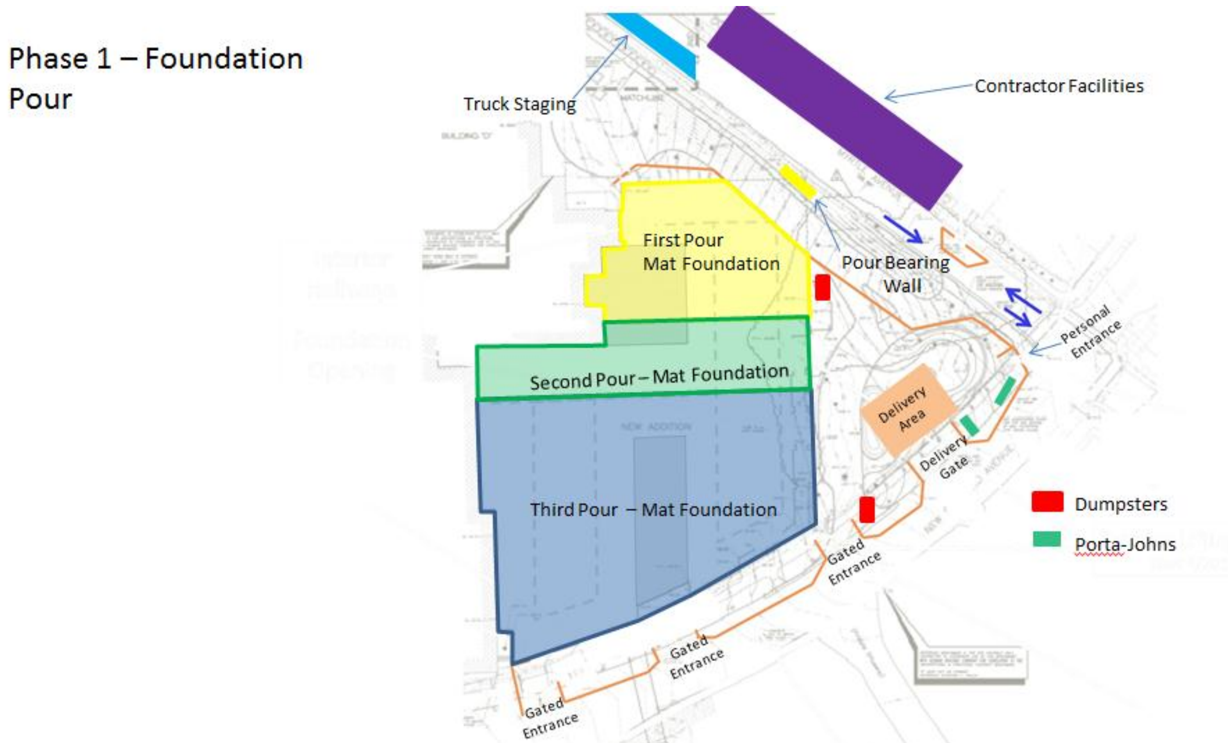


Figure 42 - Foundation Pour Site Logistics

The foundation pours consist of three stages, starting at the north end of the site and moving to the southern side. This was done to allow the concrete to start curing on the northern side of the site first where the steel is going to be erected first. The delivery area is located off New Scotland Avenue, the delivery trucks stage on Myrtle Avenue on the northern side of the site then when needed the proceed to New Scotland Avenue then into the job site. Houses on Myrtle Avenue have rented out and set up at contractor facilities across the street from the site on the northern side.

Phase 2 – Steel Erection

Once the mat foundation has cured from the first pour, cranes are going to be brought onto the site to start erecting the steel while the third pour is curing, see Figure 43. Due to the accessibility limitations of the cranes with existing buildings lining the west side of the site the steel erection must be comprised of stages and erection areas.

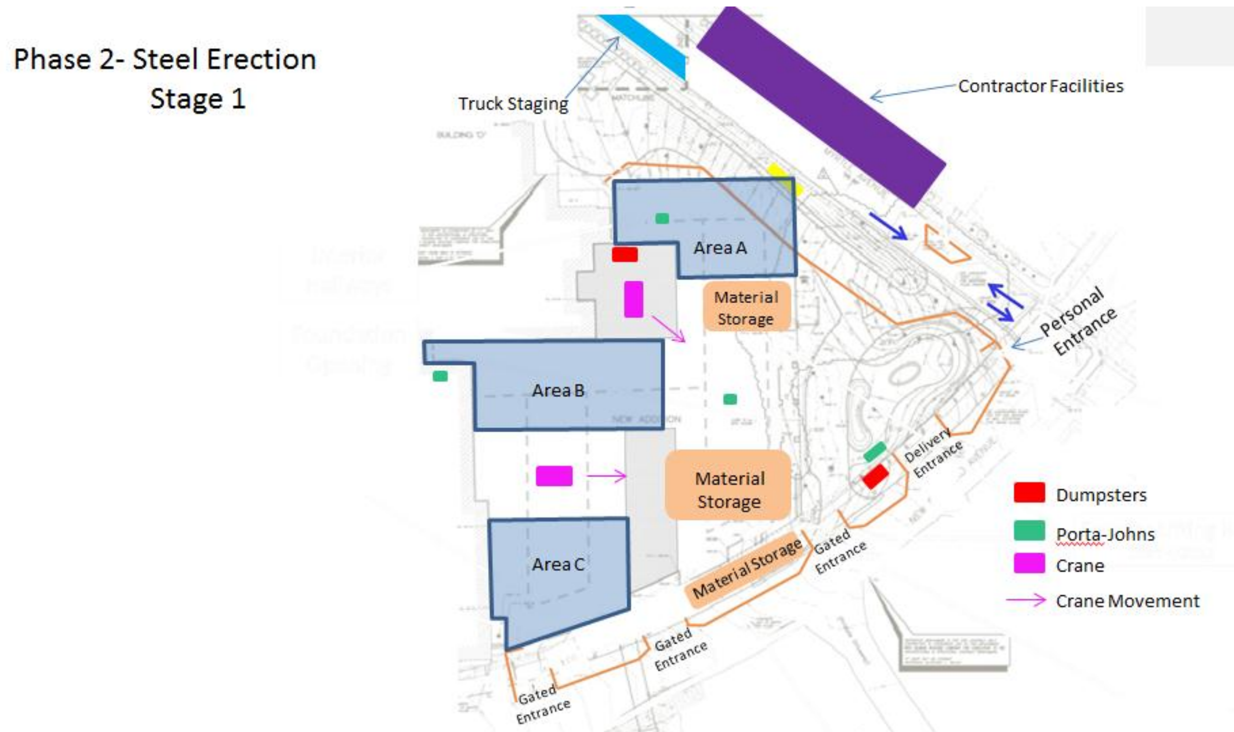


Figure 43 - Steel Erection Phase 1

The first area to be erected will be Area A with the crane located between Area A and B and the material being dropped off just south of Area A. Area B will start being erected shortly after Area A, when Area A is complete that crane located just north of Area B will be able to contribute to erecting Area B. Finally, Area C is to be erected last with material storage located directly East and Southeast of it along New Scotland Avenue. The last two stages are shown in Figures 44 and 45; the key to erecting the remaining steel structure is to back the cranes and equipment out towards the eastern side of the site.

Phase 2- Steel Erection
Stage 2

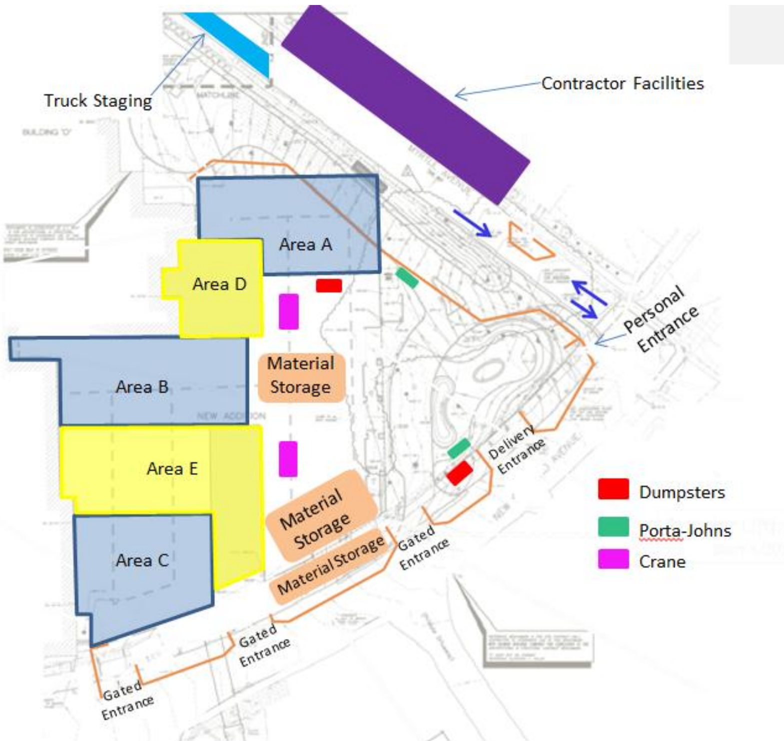


Figure 44 - Steel Erection Stage 2

Phase 2- Steel Erection
Stage 3

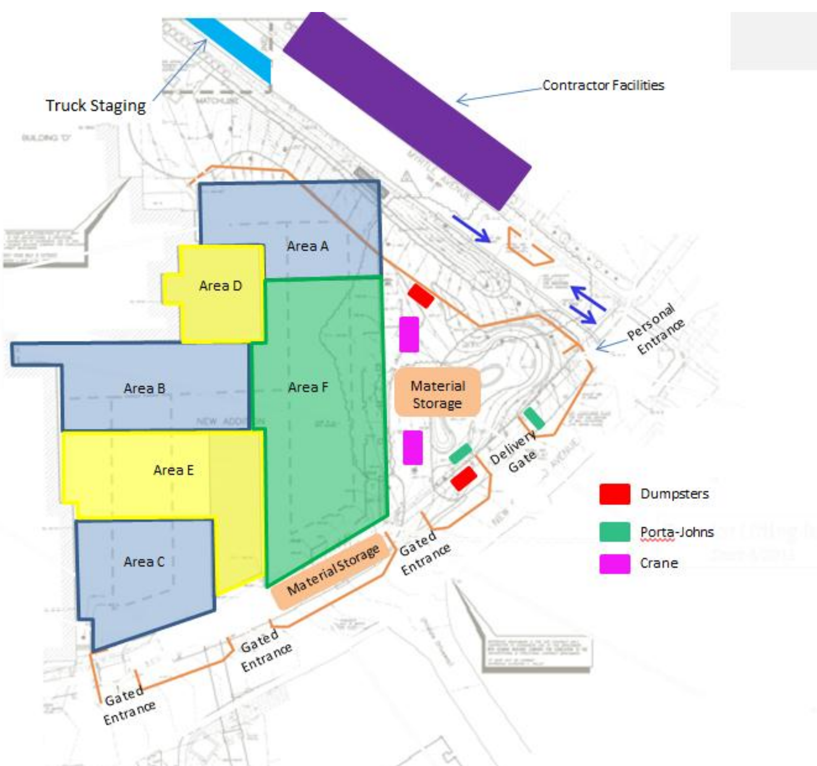


Figure 45 - Steel Erection Stage 3

Phase 3 – Exterior Façade and Interior Equipment and Finishes

The floor slabs are going to be placed in the same order the steel is erected and once the entire structure is up the mason crew will start laying brick along the north side working their way around the building clockwise. The masons will lay the entire façade except for the areas where there are hoist bays, lifts, and or foundation openings, see Figure 46 for these areas.

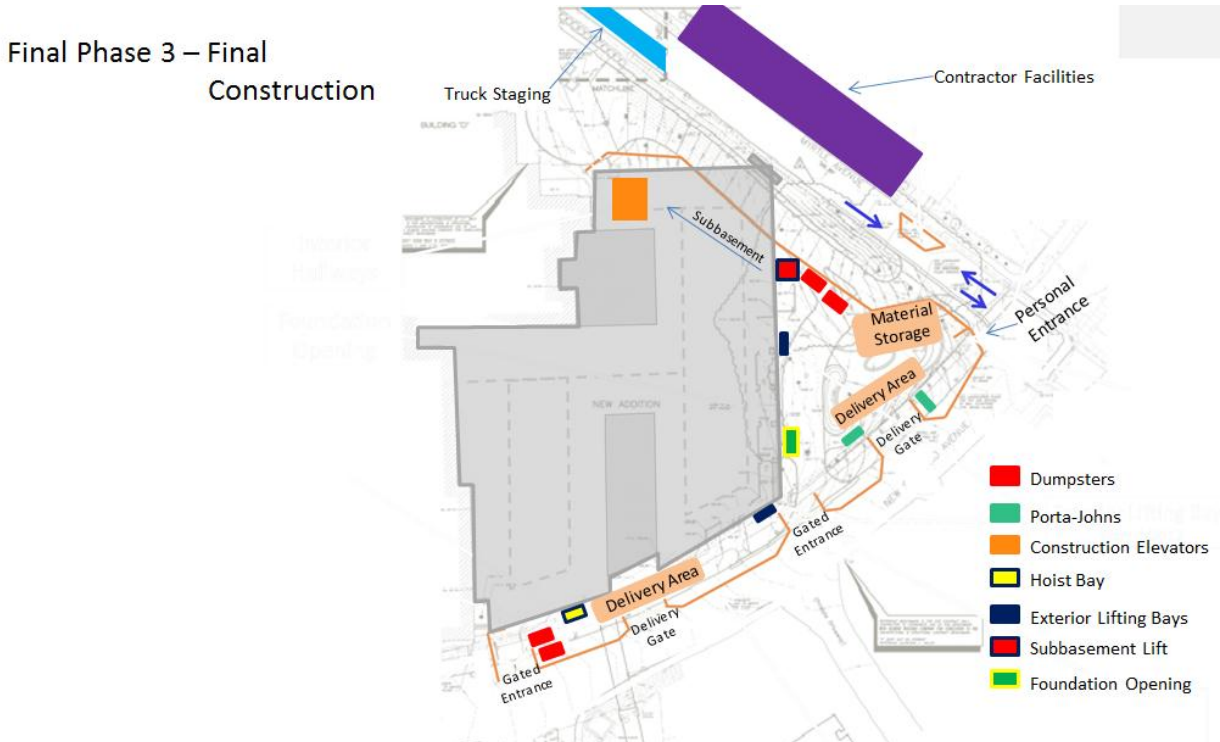


Figure 46 - Final Site Logistics

The figure above shows the final site logistics for the rest of the project until construction is complete. Three lifts were added to the building, these are on the south and east side of the site, these lifts will not be fully enclosed and allow for materials to be loaded onto the upper floors.

The subbasement is accessible from two places on the site, these locations are at the subbasement lift and the foundation opening, both located on the eastern side of the site. The subbasement lift on the northeast side of the site is a one-story lift to lower materials to the subbasement area. Material lowered into the subbasement can be transported across the subbasement and into the construction elevator to be relocated to upper levels. The other access point is the foundation opening, this is a section of the foundation perimeter wall that is left out in order to allow large equipment to be brought by vehicle into the subbasement.

Conclusion

A progressive collapse analysis per Unified Facilities Criteria 4-023-03 was successfully completed and designed to determine efficient members to resist collapse for the Patient Pavilion. Per occupancy category IV the UFC requires three design methods to be performed to resist collapse of the Patient Pavilion, these were:

4. Tie-Forces Method
5. Alternate Path Method
6. Enhanced Local Resistance

The tie-force method is an indirect method to resisting progressive collapse; because it does not resist the collapse of the structure, it takes the load from the damaged structure and distributes it to the undamaged structure. The tie-force method mechanically ties the structure together by placing additional reinforcement within the slab. Typical additional reinforcement were No. 4s at twelve inches on center for the transverse and longitudinal ties, for perimeter ties, No. 6s at nine inches on center was utilized.

Alternate path method is a direct method because it directly resists the collapse of the structure by localizing the collapse with moment frames. A virtual work plastic analysis was used to determine preliminary beam sizes around the perimeter of the building. For the columns, multiple iterations were performed to obtain member sizes that passed the moment and axial interaction equations provided in Chapter H of the AISC.

A façade study was performed for the existing façade, based on those results, a new façade precast façade was proposed increasing the R-value of the wall and decreasing the shading coefficient of the glazing. The enhanced performance had a greater upfront cost however, it had a total of $162,670 in energy cost savings per year.

A site logistics was performed for the precast façade and a coordination plan was made in order to construct a precast façade. It was determined the constructability for the precast façade was very difficult therefore the idea was disregarded and a site logistics for the construction on the Patient Pavilion was created.

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Appendix A

Tie-Forces

T.J. Kleinosky | Tie Forces | 3/11/2012 | 1/4

Basement to 2^{nd} Level

Dead = 95psf
Live = 100psf

Overstrength → $\Omega = 1.25$

$w_F = 1.2(95) + 0.5(100) = 164 psf$

East West Direction → $L_{EW} = 30ft$

$F_t = 3 w_F L = 3(164)(30) = 14.8 k/ft$

$A_{min} = \frac{14.8}{0.9 \times 1.25 \times 60} = 0.219 in^2$

Use #4's → $A = 0.2 in^2$

$S = \frac{0.20 in^2}{0.219 in^2} \times 12 = 10.95 in$

$S_{max} = 65.6 in > S = 10.95 in.$ ∴ OK!

$\boxed{\text{Use \#4s @ 10in. O.C.}}$

North West Direction $L_{NS} = 27.33ft.$

$F_t = 3 w_F L_{NS} = 3(164)(27.33) = 13.5 kip/ft$

$A_{min} = \frac{13.5}{0.9 \times 1.25 \times 60} = 0.2 in^2$

Use #4's → $A = 0.2 in^2$

$S = 12 in.$

$S_{max} = 72 in.$

$\boxed{\text{Use \#4s @ 12in O.C.}}$

Peripheral Ties - North South Direction

$F_p = 6 w_F L_1 L_p$

$L_1 = 27.33ft$ $L_p = 3ft$

$F_p = 6(164)(27.33)(3) = 80690 lbs.$

$A_{s,min} = \frac{80.7}{0.9 \times 1.25 \times 60} = 1.19 in^2$

∴ Use (3) #6's $A_s = 3(.44) = 1.32 in^2 > 1.19$ ∴ OK!

TOPS 35502

Basement to 2nd Level

Peripheral Ties - North South Direction

$F_p = 6 w_F L_1 L_p$

$L_1 = 3\text{ft}$
$L_p = 27.33\text{ft}$

$F_p = 6(164)(3)(27.33) = 80.7\text{ kips}$

$A_{s\,min} = \dfrac{80.7}{0.9 \times 1.25 \times 60} = 1.20\text{ in}^2$

Use (3) #6s $\rightarrow$ $A = 3 \times 0.44 = 1.32\text{ in}^2$

East-West Direction

$F_p = 6 w_F L_1 L_p$

$L_1 = 3\text{ft}$
$L_p = 30\text{ft}$

$F_p = 6(164)(3)(30) = 88.7\text{ kips}$

$A_{s\,min} = \dfrac{88.7}{0.9 \times 1.25 \times 60} = 1.314\text{ in}^2$

use (4) #6s $\rightarrow$ $A_s = 4 \times .44 = 1.76\text{ in}^2$

TOPS 35502

T.J. Kleinosky | Tie Forces | 3/11/2012 | 2/4

3rd to 8th level

Dead Load = 95 psf
Live Load = 80 psf

$W_F = 1.2D + 0.5L = 1.2(95) + 0.5(80) = 154\,psf$

East-West Direction $\rightarrow L_{EW} = 30ft$

$F_t = 3w_F L = 3(154)(30) = 13860\ lbs/ft = 13.9\ kip/ft$

$\Omega_{\text{conc}} = 1.25 \quad \phi = 0.9$

$A_{min} = \frac{13.9}{1.25 \times 0.9 \times 60} = 0.205\,in^2/ft$

Use #4's

$A_{\#4} = 0.2\,in^2$

$Spacing = \frac{0.2\,in^2}{0.205\,in^2} \times 12 = 11.7\,in < Max\ Spacing = 0.2L_r = 0.2(27.33) \times 12 = 65.6\,in \therefore OK$

$\boxed{\text{Use \#4 @ 11 in. OC}}$

North South Direction $\rightarrow L_{NS} = 27.33ft$

$F_t = 3w_F L = 3(154)(27.33) = 12{,}628\,plf = 12.6\,k/ft$

$A_{min} = \frac{12.6}{0.9 \times 1.25 \times 60} = 0.187\,in^2/ft$

Use #4's $\rightarrow A = 0.2\,in^2$

$S = \frac{0.2\,in^2}{0.187\,in^2} \times 12 = 12.83\,in$

$S_{max} = 0.2L_{EW} = 0.2(30 \times 12) = 72\,in > 12.83\,in \therefore OK$

$\boxed{\text{Use \#4s @ 12 in. OC}}$

TOPS 35502

3rd to 8th Level – Peripheral Ties

North-South Direction

$F_p = 6 w_f L_1 L_p$

$L_1 = 3ft$
$L_p = 27.33ft$

$F_p = 6(154)(3)(27.33) = 75.8 kips$

$A_{s,min} = \frac{75.8}{0.9 \times 1.25 \times 60} = 1.12 in^2$

Use (3) #6's $\longrightarrow A_s = 1.32 in^2$

East-West Direction

$F_p = 6 w_f L_1 L_p$

$L_1 = 3ft$
$L_p = 30ft$

$F_p = 6(154)(3)(30) = 83.2 kips$

$A_{s,min} = \frac{83.2}{0.9 \times 1.25 \times 60} = 1.23 in^2$

Use (3) #6s $\longrightarrow A_s = 1.32 in^2$

TOPS 35502

T.J. Kleinosky | Tie Forces | 3/11/2012 | 3/4

Penthouse Level

Dead = 125psf
Live = 125psf

$w_F = 1.2(125) + 0.5(125) = 213\text{psf}$

East West Direction $\rightarrow L_{EW} = 30\text{ft}$

$F_i = 3w_F L = 3(213)(30) = 19.2\ k/ft$

$A_{min} = \frac{19.2}{0.9 \times 1.25 \times 60} = 0.284\text{in}^2$

Use #4s $\rightarrow A = 0.2\text{in}^2$

$S = \frac{0.2}{0.284} \times 12 = 8.45\text{in}$

$S_{max} = 65.6\text{in} > 8.45\text{in} \therefore$ OK!

$\boxed{\text{Use \#4s @ 8in. O.C.}}$

North South Direction $\rightarrow L = 27.33\text{ft}$

$F_i = 3w_F L = 3(213)(27.33) = 17.5\ k/ft$

$A_{min} = \frac{17.5}{0.9 \times 1.25 \times 60} = 0.259\text{in}^2$

Use #4s $\rightarrow A = 0.2\text{in}^2$

$S = \frac{0.20}{0.259} \times 12 = 9.3\text{in} \therefore$ use 9in.

$S_{max} = 72\text{in} > 9\text{in} \therefore$ OK!

$\boxed{\text{Use \#4s @ 9in. O.C.}}$

TOPS 35502

Penthouse Level – Peripheral Ties

North-South Direction –

$F_p = 6 w_F L_1 L_p$

$L_1 = 3ft$
$L_p = 27.33ft$

$F_p = 6(213)(3)(27.33) = 104.8 kips$

$A_{s,min} = \frac{104.8}{0.9 \times 1.25 \times 60} = 1.55 in^2$

Use (4) #6s $\longrightarrow A_s = 1.76 in^2$

East-West Direction –

$F_p = 6 w_F L_1 L_p$

$L_1 = 3ft$
$L_p = 30ft$

$F_p = 6(213)(3)(30) = 115 kips$

$A_{s,min} = \frac{115}{0.9 \times 1.25 \times 60} = 1.7 in^2$

use (4) #6s $\longrightarrow A_s = 1.76 in^2$

TOPS 35502

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Roof Level

Dead = 95psf
Live = 20psf

$\omega_u = 1.2(95) + 0.5(20) = 124psf$

East West Direction → $L_{EW} = 30ft$

$F_t = 3\omega_u L = 3(124)(30) = 11.2k/ft$

$A_{min} = \frac{11.2}{0.9 \times 1.25 \times 60} = 0.166 in^2$

Use #4s → $A = 0.2 in^2$

$S = \frac{0.20 in^2}{0.166 in^2} \times 12 = 14.5 in.$

$S_{max} = 65.6 in > S \therefore OK!$

Use #4s @ 14in O.C.

North South Direction → $L_{NS} = 27.33ft$

$F_t = 3(124)(27.33) = 10.2 k/ft$

$A_{min} = \frac{10.2}{0.9 \times 1.25 \times 60} = 0.151 in^2$

Use #4s → $A = 0.2 in^2$

$S = \frac{0.2}{0.151} \times 12 = 15.9 in$

$S_{max} = 72 in$

Use #4s @ 15in. O.C.

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Roof Level - Peripheral Ties

North-South Direction

$F_p = 6 w_F L_1 L_p$

$L_1 = 3ft$
$L_p = 27.33$

$F_p = 6(124)(3)(27.33) = 61$ kips

$A_{s,min} = \frac{61}{0.9 \times 1.25 \times 60} = 0.90 in^2$

Use (3) #6s $\longrightarrow A_s = 1.32 in^2$

East-West Direction

$F_p = 6 w_F L_1 L_p$

$L_1 = 3ft$
$L_p = 30ft$

$F_p = 6(124)(3)(30) = 67$ kips

$A_{s,min} = \frac{67}{0.9 \times 1.25 \times 60} = 0.99 in^2$

Use (3) #6s $\longrightarrow A_s = 1.32 in^2$

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Appendix B

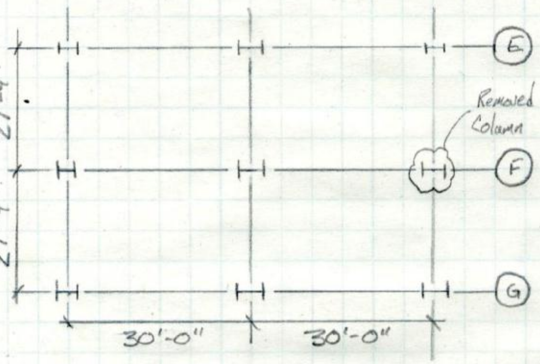
Virtual Work Calculations

Preliminary Sizes → Plastic Analysis

Load Combinations:

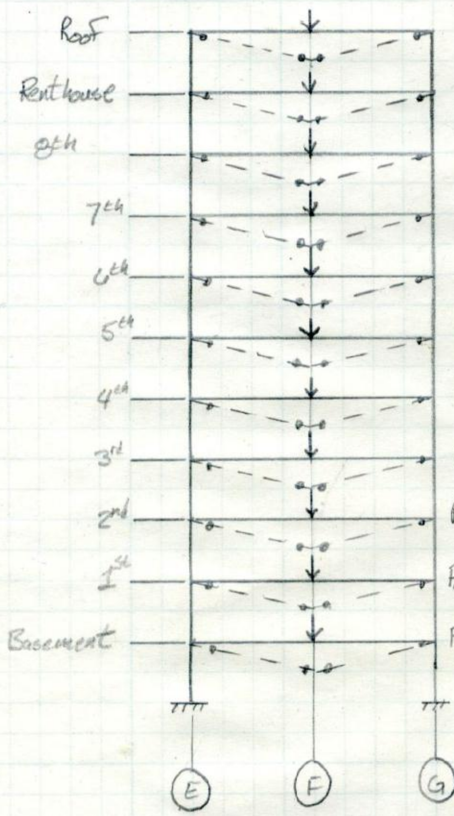
$0.9D + 0.5L + 0.2S$ For bays adjacent to removed Columns

$1.2D + 1.6L + 0.5S$ For all other bays



Assumptions:

- Point Load at mid span as the resultant load
- Point load is the resultant of the distributed load along their length and trib width
- No live load reduction was performed to get a more conservative member
- Deflection Checks are neglected



$P_{u_{roof}} = [(0.9 \times 95) + (0.2 \times 37)] \times 15 \times (2 \times 27.33) = 76.1\,k$

$P_{u_{Pent}} = [(0.9 \times 125) + (0.5 \times 125)] \times 15 \times 2 \times 27.33 = 143.5^{k}$

$P_{u_{8th}} = [(0.9 \times 95) + (0.5 \times 80)] \times 15 \times 2 \times 27.33 = 103\,k$

$P_{u_{7th}} = 103\,k$

$P_{u_{6th}} = 103^{k}$

$P_{u_{5th}} = 103^{k}$

$P_{u_{4th}} = 103^{k}$

$P_{u_{3rd}} = 103^{k}$

$P_{u_{2nd}} = [(0.9 \times 95) + (0.5 \times 100)] \times 15 \times 2 \times 27.33 = 112$ kips

$P_{u_{1st}} = 112\,k$

$P_{u_{base}} = 112^{k}$

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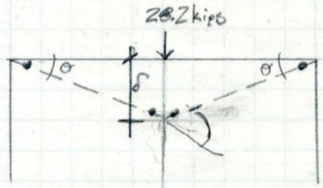
Roof Level



$M_p\theta + M_p\theta + M_p 2\theta + M_p 2\theta = P(\delta)$ where $\tan\theta \approx \theta = \frac{\delta}{L/2}$

$\delta = \frac{\theta L}{2}$ (small angle theory)

$6M_p\cancel{\theta} = 30P\cancel{\theta}$

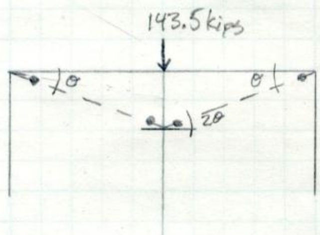
$M_p = 5P = 5(76.1) = 381\ \text{Ft-k}$

$\frac{M_p}{\Phi} = F_y Z$

$\left(\frac{381 \times 12}{0.9}\right) = 50Z$

$Z \geq 101.6\text{in}^3 \therefore$ $\boxed{\text{Use a W21x50}}$

Penthouse Level

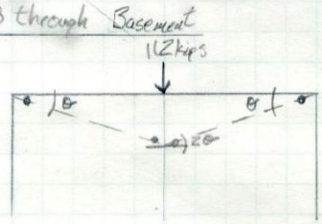


$M_p = 5P = 5(143.5\ \text{kips}) = 717.5\ \text{k-ft}$

$\frac{718 \times 12}{0.9} = 50Z$

$Z \geq 191.5\text{in}^3 \therefore$ $\boxed{\text{Use a W24x76}}$

Level 8 through Basement



$M_p = 5P = 5(112) = 560\ \text{k-Ft}$

$\frac{560}{0.9} = 50Z$

$Z = 149\text{in}^3 \therefore$ $\boxed{\text{Use W24x62}}$

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Appendix C

Alternate Path Method

Column Interaction

Removed Column		Member	F_y	Z_y	Z_x	Area	M_{ry}	M_{rx}	P_r	M_{cy}	M_{cx}	P_c	m-factor	Interaction	
F-9 Base	Roof	W14X176	50	163	320	51.8	36.7	400	103.1	611.25	1200	1872.3	8.00	0.07670476	PASS
	Penthouse	W14X176	50	163	320	51.8	23.1	435	366.4	611.25	1200	1924.6	8.00	0.14522504	PASS
	8th	W14X176	50	163	320	51.8	18.9	384.4	541.9	611.25	1200	1931.2	6.39	0.32948002	PASS
	7th	W14X176	50	163	320	51.8	22.6	467.7	716.3	611.25	1200	2050.1	5.01	0.42507717	PASS
	6th	W14X176	50	163	320	51.8	16.5	315.1	890	611.25	1200	1785.8	2.03	0.62502045	PASS
	5th	W14X342	50	338	672	101	26.1	604.9	1071.9	1267.5	2520	3853.8	6.44	0.31413079	PASS
	4th	W14X342	50	338	672	101	11.3	536.4	1253.8	1267.5	2520	3752.1	5.32	0.37123647	PASS
	3rd	W14X342	50	338	672	101	17.9	610.9	1436.3	1267.5	2520	3787.3	4.42	0.43088985	PASS
	2nd	W14X342	50	338	672	101	96.1	527.1	1640.9	1267.5	2520	3253.1	1.00	0.75773136	PASS
	1st	W14X370	50	370	736	109	50.2	718.7	1863.4	1387.5	2760	3961.8	2.59	0.57200347	PASS
	Basement	W14X370	50	370	736	109	12.83	332.7	2068.1	1387.5	2760	3849.6	1.00	0.65259382	PASS
F-9 Middle	Roof	W14X193	50	180	355	56.8	37.5	438.6	76.6	675	1331.25	2013	8.00	0.06715387	PASS
	Penthouse	W14X193	50	180	355	56.8	23.5	492.7	260.6	675	1331.25	2059.4	8.00	0.11388562	PASS
	8th	W14X193	50	180	355	56.8	19.2	439.8	170.8	675	1331.25	2069.1	8.00	0.08612531	PASS
	7th	W14X193	50	180	355	56.8	22.8	524.7	508.3	675	1331.25	2211.7	7.40	0.2812003	PASS
	6th	W14X193	50	180	355	56.8	18.2	407.9	632.5	675	1331.25	1925.4	5.43	0.38307581	PASS
	5th	W14X342	50	338	672	101	19.1	473.9	759.2	1267.5	2520	3847.5	8.00	0.12405204	PASS
	4th	W14X342	50	338	672	101	4.9	115.3	810.8	1267.5	2520	3752.1	7.68	0.22183674	PASS
	3rd	W14X342	50	338	672	101	79.9	6	864.9	1267.5	2520	3787.5	7.43	0.23617975	PASS
	2nd	W14X342	50	338	672	101	126.9	53.7	929.5	1267.5	2520	3253.1	6.29	0.3028998	PASS
	1st	W14X370	50	370	736	109	82.3	6.9	996.9	1387.5	2760	3961.8	6.97	0.2595143	PASS
	Basement	W14X370	50	370	736	109	8.3	3.9	1086.6	1387.5	2760	3849.6	6.35	0.28329749	PASS
F-9 Roof	Roof	W14X193	50	180	355	56.8	27.5	63.6	113.2	675	1331.25	2053.8	8.00	0.0386231	PASS
	Penthouse	W14X193	50	180	355	56.8	11.1	6.8	203.6	675	1331.25	2059.4	8.00	0.05212593	PASS
	8th	W14X193	50	180	355	56.8	10.2	3.7	258.4	675	1331.25	2069.1	8.00	0.06467891	PASS
	7th	W14X193	50	180	355	56.8	12.1	4	312.7	675	1331.25	2211.7	8.00	0.07330856	PASS
	6th	W14X193	50	180	355	56.8	8.9	2.4	367.1	675	1331.25	1925.4	8.00	0.09720434	PASS
	5th	W14X342	50	338	672	101	11.4	6.7	421.4	1267.5	2520	3847.5	8.00	0.05621943	PASS
	4th	W14X342	50	338	672	101	12.22	5	475.9	1267.5	2520	3752.1	8.00	0.06487096	PASS
	3rd	W14X342	50	338	672	101	14.1	3.8	531.5	1267.5	2520	3787.4	8.00	0.07174589	PASS
	2nd	W14X342	50	338	672	101	45.6	16.8	597.6	1267.5	2520	3253.1	8.00	0.09718122	PASS
	1st	W14X370	50	370	736	109	24.1	9.4	667	1387.5	2760	3961.8	8.00	0.0867758	PASS
	Basement	W14X370	50	370	736	109	5	1	756.9	1387.5	2760	3849.7	8.00	0.0988021	PASS

Column m-factors – Base Column Remove

Column	Member Size	Connection	F_y	R_y	F_{ye}	$b_f/2t_f$	h/t_w	$52/\sqrt{F_{ye}}$	$300/\sqrt{F_{ye}}$	$65/\sqrt{F_{ye}}$	$460/\sqrt{F_{ye}}$	$260/\sqrt{F_{ye}}$	$400/\sqrt{F_{ye}}$	Column m-factor(a)	Interpolated m-factor	
	W14X176	WUF	50	1.1	55	5.97	13.7	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	8.00	11.57	100.00
	W14X176	WUF	50	1.1	55	5.97	13.7	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	8.00	11.57	100.00
	W14X176	WUF	50	1.1	55	5.97	13.7	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	11.57	100.00
	W14X176	WUF	50	1.1	55	5.97	13.7	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	11.57	100.00
	W14X176	WUF	50	1.1	55	5.97	13.7	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	11.57	100.00
F-9 Base	W14X342	WUF	50	1.1	55	3.31	7.41	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	20.67	100.00
	W14X342	WUF	50	1.1	55	3.31	7.41	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	20.67	100.00
	W14X342	WUF	50	1.1	55	3.31	7.41	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	20.67	100.00
	W14X342	WUF	50	1.1	55	3.31	7.41	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	20.67	100.00
	W14X370	WUF	50	1.1	55	3.1	6.89	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	21.39	100.00
	W14X370	WUF	50	1.1	55	3.1	6.89	7.011679	40.45199	8.764598	62.02639	35.05839	53.93599	See Eq. (b)	21.39	100.00

P_r/P_c	0.2<Pr/Pc<0.5	Pr/Pc<0.2	Column m-factor (b)	Interpolated m-factor		m-upperbound	Member m-factor	Force Controlled?
0.06	No	Yes	10.90	16.48	21.53	10.90	8.00	No
0.19	No	Yes	8.19	12.17	15.76	8.19	8.00	No
0.28	Yes	No	6.39	9.29	11.92	6.39	6.39	No
0.35	Yes	No	5.01	7.10	8.99	5.01	5.01	No
0.50	Yes	No	2.03	2.35	2.63	2.03	2.03	No
0.28	Yes	No	6.44	16.86	13.67	6.44	6.44	No
0.33	Yes	No	5.32	13.38	10.91	5.32	5.32	No
0.38	Yes	No	4.42	10.57	8.68	4.42	4.42	No
0.50	No	No	1.91	2.78	2.51	1.91	1.00	Yes
0.47	Yes	No	2.59	5.03	4.22	2.59	2.59	No
0.54	No	No	1.26	100.00	100.00	1.26	1.00	Yes

Appendix D

Enhanced Local Resistance

T.J. Kleinosky | Enhanced Flexural Resistance | 3/18/2012

Base Model → Gravity Loads Only

- Lower two levels are both W14x176
- Use F_{ye}
- No ϕ

$M_p = F_{ye} Z = (1.1 \times 50)(320) = 17{,}600$ k-in

Alternate Path Model →

- W14x370 are lower 2 levels

$M_p = F_{ye} Z = (1.1 \times 50)(736) = 40{,}480$ k-in

$2 M_{p_{Base}} < M_{p_{AP}} \rightarrow 2(17600) = 35200$ k-in $< 40{,}480$ k-in

$\boxed{\text{* The members are sufficient enough for Enhanced Flexural Resistance}}$

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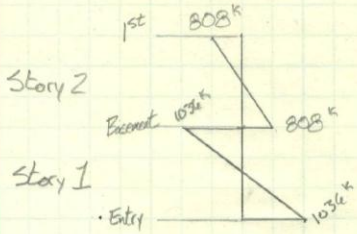
T.J. Kleinosky | Enhanced Local Resistance | 3/20/2012 | 1/2

From Computer Model



Shear Capacity of W14x370

$\phi V_n = 0.6 F_y A_w C_v$ (G2-1)

$A_w = dt_w = 17.9(1.66) = 29.72 in^2$

$F_y = F_{ye} = 1.1(50) = 55 ksi$

$C_v = 1.0$ if $\frac{h}{t_w} < 2.24\sqrt{\frac{E}{F_y}}$

$\frac{h}{t_w} = \frac{17.9}{1.66} = 10.8$

$2.24\sqrt{\frac{E}{F_y}} = 53.9 \therefore C_v = 1.0$

$\phi V_n = 0.6(55)(29.72)(1.0) = 980.8 kips$

Story 1

$V_u = 1036$ kips

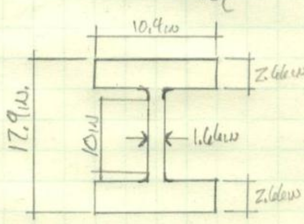
$\phi V_n = 980.8$ kips

Shear needed = $1036 - 980.8 = 55.2 kips$

$f_v = \frac{3}{2}\frac{V}{A}$, use 36 ksi plates

$\frac{3}{2}\frac{V}{A} \leq F_{ye} = 36(1.1) = 39.6$

$\frac{3}{2} \times \frac{55.2}{A_{req}} = 39.6 \Rightarrow A_{req} = 2.1 in^2$

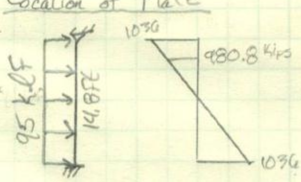


thickness of PL

$2.1 in^2 = 10 in \times t_{PL}$

$t_{PL} = 0.21 in \therefore$ use $\frac{3}{8}$ in PL

Location of Plate



$1036 - 95x = 980.8 k$

$x = 0.6 ft$

$\therefore$ add PL 1ft above base @ Entry Level and 1ft below top @ Basement Level

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2/2

Story 2

$V_u = 808$ kips
$\phi V_n = 980.8$ kips

$\boxed{\text{No dubler plate needed!}}$

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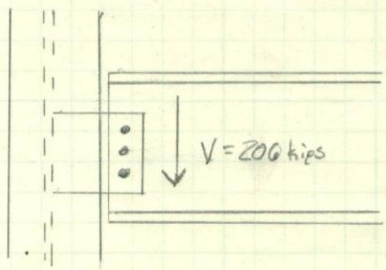
Appendix E

Connection Design

T.J. Kleinosky | Simple Connection



Column : W14x370
Beam : W24x68

Bolts : 1"ϕ A325-N $\phi r_n = 17.9$ kips
Plate : A36

m-Factor = 6.79

$V_{design} = \frac{206}{6.79} = 30.3$ kips

W14x370

$d = 17.9$ in. $b_f = 16.5$ in. $t_w = 1.66$ in.

W24x68

$d = 23.7$ in. $t_w = 0.415$ in

Find e

$e = \frac{16.5 - 1.66}{2} = 7.42 + 0.5 + 1.5 = 9.42$ in.

Table 7-6 → Assume 5 bolts

$n = 5$ $e = 10$ in $\therefore C = 1.66$

Determine Strength of Bolt group

$\phi r_{n_{bolt}} = 31.8$ kips

ϕr_n (web bearing) $= 0.75 \cdot 2.4 \cdot 65 \cdot 1 \cdot 0.415 = 48.6$ kips

ϕr_n (Plate bearing) $= 0.75 \cdot 2.4 \cdot 58 \cdot 1 \cdot \frac{3}{8} = 39.2$ kips

$\Big\} > 31.8$ kips $\therefore$ bolt shear controls

Determine Max Plate Thickness

$A_b = \pi \left(\frac{1}{2}\right)^2 = 0.785 \text{ in}^2$

table (7-6) → $C' = 17.1$ in

$M_{max} = 1.25\, F_{u}\, A_b\, C' = 1.25(48)(.785)(17.1) = 805.41$ in-k

$t_{max} = \frac{6\, M_{max}}{F_y\, d^2} = \frac{6(805.41)}{36\,(15^2)} = 0.59 > \frac{3}{8}$" $\therefore$ ok!

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Plate Shear Rupture

$\phi R_n = \phi(0.6)F_u A_{nv} = 0.75(0.6)(58)[15 - 5(1+\frac{3}{16})] \times \frac{3}{8} = 91.8 \text{ kip} > 30^k \therefore OK!$

Plate Block Shear

$0.6 F_y A_g = 0.6 \times 36 \times 15(\frac{3}{8}) = 121.5$

$0.6 F_u A_n = 0.6 \times 58 \times (15 - 5 \times 1\frac{1}{8}) \times \frac{3}{8} = 122.3$

$\phi R_n = 0.75[121.5 + 1(58)(0.375)(1.5 - 0.5(1+\frac{1}{8}))] = 106.4 \text{ kips} > 30 \therefore OK!$

Plate Shear Yielding

Will not control over Rupture

Flexure / Shear Interaction

$\phi V_n = \phi F_y A_g = 0.6(1.0)(36)(15)(\frac{3}{8}) = 121.5 \text{ kips}$

$V_u = 30.3 \text{ kips}$

$M_u = 30.3 \text{ kips} \times 9.42 = 285.4 \text{ in-kips}$

$\phi M_n = 0.9 F_y Z_x = 0.9 F_y \left(\frac{t_p d^2}{4}\right) = 0.9(36)\left(\frac{.375(15^2)}{4}\right) = 683.4 \text{ k-in.}$

Interaction:

$$\left(\frac{30.3}{121.5}\right)^2 + \left(\frac{285.4}{683.4}\right)^2 = 0.24 < 1.0 \therefore OK!$$

Check Plate Buckling

· Center ℄ on Beam ∴ assume equal Copes

$$\lambda = \frac{h_o\sqrt{F_y}}{10 t_w \sqrt{475 + 280(\frac{h_o}{c})^2}} = \frac{15\sqrt{36}}{10(\frac{3}{8})\sqrt{475 + 280(\frac{15}{9.42})^2}}$$

$\lambda = 0.70$

$\lambda \leq 0.7 \therefore Q = 1.0$

$F_{cr} = Q F_y = 36 \text{ ksi}$

$\phi M_n = 0.9(36)\left(\frac{.375(15^2)}{4}\right) = 683.4 \text{ in-kips} > 285.4 \text{ in-kips}$

$\therefore$ OK!

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Weld Strength

3/8 in PL $\quad t_{w_{min}} = 3/8 - 5/8 = 0.23 \text{ in} = 1/4\text{" weld}$

$l = 15 - 2(\frac{1}{4}) = 14.5 \text{ in}$

$e = 9.42 \text{ in.}$

$e_x = \frac{9.42}{14.5} = 0.65 \rightarrow \text{use } 0.7$

table 8-4

$K = 0 \qquad C = 1.76$

$\phi R_n = 0.75(1.76)(1)(2)(14.5) = 38.3 \text{ kips} > 30.3 \text{ kips} \therefore \text{ok!}$

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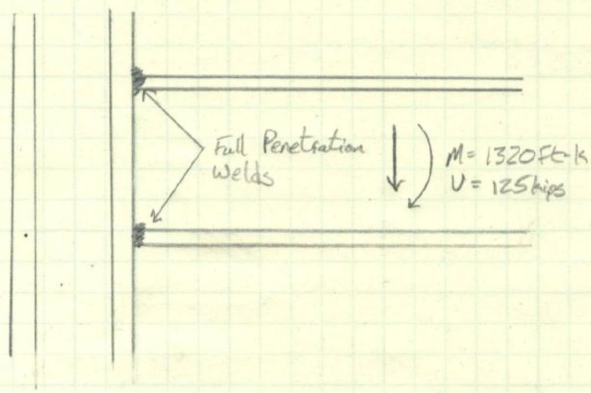
T.J. Kleinosky

Moment Connection



Column: W14×370
Beam: W24×62
$d = 23.7$ in. $b_f = 7.04$ in.
$t_f = 0.59$ in.
mFactor = 2.33
↳ Connection

Find Tension Force Upper Flange

$$M_{Design} = \frac{M}{mFactor} = \frac{1320}{2.33} = 566.5 \text{ Ft-k}$$

$$T = \frac{M}{d} = \frac{566.5 \times 12}{23.7} = 286.8 \text{ kips}$$

$$A_{Flange} = 0.59 \times 7.04 = 4.15 \text{ in}^2$$

$$\phi R_n = \phi F_y A_n = 0.9(50)(4.15) = 187 \text{ kips} < 286.8 \text{ kip} \rightarrow \text{Fails!}$$

* To Avoid Recalculating m-Factors + Ω_{CD}, going to resize the beam and keep the WUF Connection.

Try a W24×84 -

$$A_{Flange} = 9.02 \times 0.77 = 6.94 \text{ in}^2 \qquad d = 24.1$$

$$m\text{-Factor} = 4.3 - 0.083(24.1) = 2.3$$

$$M_{Design} = \frac{1320}{2.3} = 573.9 \text{ Ft-kips}$$

$$T = \frac{573.9 \times 12}{24.1} = 285.7 \text{ kips}$$

$$\phi R_n = 0.9(50)(6.94) = 312.3 \text{ kips} > 285.7 \text{ kips} \therefore \text{OK!}$$

TOPS 35502

Column Limit States:

Local Flange Buckling

$$\phi R_n = \phi (6.25\, t_{fc}^2\, F_{yc}) = 0.9(6.25 \times 2.66^2 \times 50) = 1990 \text{ kips} > 285 \therefore OK!$$

Local Web Yielding

$$\phi R_n = \phi F_{yc} (5 k_{des})(t_{wc}) = 0.9(50)(5 \times 3.26)(1.66) = 1217 \text{ kips} > 285 \therefore OK!$$

Local Web Crippling

$$\phi R_n = \phi (0.8 \times t_{wc}^2)\left[1 + 3\left(\frac{t_{fB}}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$$

$$= 0.9(0.8 \times 1.66^2)\left[1 + 3\left(\frac{0.77}{17.9}\right)\left(\frac{1.66}{2.66}\right)^{1.5}\right]\sqrt{\frac{29000(50)2.66}{1.66}}$$

$$\phi R_n = 3216 \text{ kips} > 285 \text{ kips} \therefore OK!$$

Web Buckling

$$\phi R_n = \phi \frac{24\, t_{wc}^3 \sqrt{E F_{yc}}}{h/t_{wc}} = 0.9 \frac{24(1.66^2)\sqrt{29000 \times 50}}{6.89} = 10{,}400 \text{ kips} > 285 \text{ k} \therefore OK!$$

No stiffners Required!

TOPS 35502

Appendix F

Mechanical Calculations

Cooling Loads			
North Façade	**Existing**	**Proposed**	**Difference**
Annual Heat Gain per room (Btu)	11715024	9633312	
Annual Heat Gain per room (kWh)	3433.334621	2823.245057	610.0895636
Rooms on East Side of Building	45	45	
Annual Heat Gain (Btu)	527176080	433499040	93677040
Annual Heat Gain (kWh)	154500.0579	127046.0276	27454.03036
		Total kWh Saved	27454.03036
		Price/kwh	$ 0.14
		Annual Savings	**$ 3,969.85**

Cooling Loads			
South Façade	Existing	Proposed	Difference
Annual Heat Gain per room (Btu)	16269792	14005056	
Annual Heat Gain per room (kWh)	4768.205353	4104.47675	663.7286032
Rooms on East Side of Building	18	18	
Annual Heat Gain (Btu)	292856256	252091008	40765248
Annual Heat Gain (kWh)	85827.69635	73880.58149	11947.11486
		Total kWh Saved	11947.11486
		Price/kwh	$ 0.14
		Annual Savings	**$ 1,727.55**

Cooling Loads			
East Façade	Existing	Proposed	Difference
Annual Heat Gain per room (Btu)	12064704	10731456	
Annual Heat Gain per room (kWh)	3535.815713	3145.079294	390.7364182
Rooms on East Side of Building	131	131	
Annual Heat Gain (Btu)	1580476224	1405820736	174655488
Annual Heat Gain (kWh)	463191.8583	412005.3876	51186.47078
		Total kWh Saved	51186.47078
		Price/kwh	$ 0.14
		Annual Savings	**$ 7,401.56**

Cooling Loads			
West Façade	Existing	Proposed	Difference
Annual Heat Gain per room (Btu)	8370000	6505536	
Annual Heat Gain per room (kWh)	2453.004857	1906.584398	546.4204598
Rooms on East Side of Building	48	48	
Annual Heat Gain (Btu)	401760000	312265728	89494272
Annual Heat Gain (kWh)	117744.2332	91516.05108	26228.18207
		Total kWh Saved	26228.18207
		Price/kwh	$ 0.14
		Annual Savings	**$ 3,792.60**

Heating Loads			
North Façade	Existing	Proposed	Difference
Annual Heat Loss per room (Btu)	447096552	432881952	
Annual Heat Loss per room (kWh)	131031.065	126865.1769	4165.888034
Rooms on East Side of Building	45	45	
Annual Heat Loss (Btu)	20119344840	19479687840	639657000
Annual Heat Loss (kWh)	5896397.923	5708932.962	187464.9615
		Total kWh Saved	187464.9615
		Price/kwh	$ 0.14
		Annual Savings	**$ 27,107.43**

Heating Loads			
South Façade	Existing	Proposed	Difference
Annual Heat Loss per room (Btu)	447096552	432881952	
Annual Heat Loss per room (kWh)	131031.065	126865.1769	4165.888034
Rooms on East Side of Building	18	18	
Annual Heat Loss (Btu)	8047737936	7791875136	255862800
Annual Heat Loss (kWh)	2358559.169	2283573.185	74985.98461
		Total kWh Saved	74985.98461
		Price/kwh	$ 0.14
		Annual Savings	**$ 10,842.97**

Heading Loads			
East Façade	Existing	Proposed	Difference
Annual Heat Loss per room (Btu)	447096552	432881952	
Annual Heat Loss per room (kWh)	131031.065	126865.1769	4165.888034
Rooms on East Side of Building	131	131	
Annual Heat Loss (Btu)	58569648312	56707535712	1862112600
Annual Heat Loss (kWh)	17165069.51	16619338.18	545731.3325
		Total kWh Saved	545731.3325
		Price/kwh	$ 0.14
		Annual Savings	**$ 78,912.75**

Heading Loads			
West Façade	Existing	Proposed	Difference
Annual Heat Loss per room (Btu)	447096552	432881952	
Annual Heat Loss per room (kWh)	131031.065	126865.1769	4165.888034
Rooms on East Side of Building	48	48	
Annual Heat Loss (Btu)	21460634496	20778333696	682300800
Annual Heat Loss (kWh)	6289491.12	6089528.491	199962.6288
		Total kWh Saved	199962.6288
		Price/kwh	$ 0.14
		Annual Savings	**$ 28,914.60**

Trace Results – Existing Façade

North Façade

Room Checksums
By ACADEMIC

North Facing Patient Room - H/H

COOLING COIL PEAK — Peaked at Time: Mo/Hr: 7 / 15; Outside Air: OADB/WB/HR: 88 / 72 / 93

CLG SPACE PEAK — Mo/Hr: 7 / 19; OADB: 80

	Cooling Coil Peak: Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Clg Space Peak: Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	680	0	680	10	572	25
Glass/Door Cond	180	0	180	3	118	5
Wall Cond	217	0	217	3	214	9
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	453		453	7	102	4
Sub Total ==>	1,529	0	1,529	23	1,005	44
Internal Loads						
Lights	180	45	225	3	660	29
People	640	0	640	10	392	17
Misc	78	0	78	1	74	3
Sub Total ==>	898	45	943	14	1,126	49
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	3,870	58	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	404		404	6	152	7
Exhaust Heat		-45	-45	-1		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	2,832	0	6,702	100.00	2,283	100.00

HEATING COIL PEAK — Mo/Hr: Heating Design; OADB: -1

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.74
Wall Cond	-605	-605	6.09
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.92
Sub Total ==>	-2,359	-2,359	23.75
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.25
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,359	-9,933	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	94.7
Ra Plenum	72.0	72.0
Return	72.4	72.0
Ret/OA	88.0	-1.0
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	94	94
Terminal	94	94
Main Fan	94	94
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	94	94
Return	105	105
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	100.0	100.0
cfm/ft²	0.53	0.53
cfm/ton	168.30	
ft²/ton	315.12	
Btu/hr·ft²	38.08	-80.52
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.6	6.7	4.0	94	88.0	71.9	93.1	50.0	49.2	51.4
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.6	6.7								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-4.6	94	50.0	94.7
Aux Htg	0.0	0	0.0	0.0
Preheat	-5.3	94	-1.0	50.0
Humidif	-4.2	105	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-14.2			

Project Name:
Dataset Name: TRACE000 trial.trc

'RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 1 System Checksums Report Page 3 of 8

South Façade

Room Checksums

By ACADEMIC

South Facing Patient Room - F/F

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak — Peaked at Time: Mo/Hr: 7 / 15; Outside Air: OADB/WB/HR: 88 / 72 / 93

Clg Space Peak — Mo/Hr: 10 / 15; OADB: 64

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	2,691	0	2,691	26	4,510	75
Glass/Door Cond	180	0	180	2	-104	-2
Wall Cond	492	0	492	5	537	9
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	453		453	4	-100	-2
Sub Total ==>	3,816	0	3,816	37	4,843	80
Internal Loads						
Lights	720	180	900	9	720	12
People	640	0	640	6	392	6
Misc	78	0	78	1	78	1
Sub Total ==>	1,438	180	1,618	16	1,190	20
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	3,870	37	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	1,181		1,181	11	0	0
Exhaust Heat		-73	-73	-1		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	6,435	107	10,412	100.00	6,034	100.00

HEATING COIL PEAK

Mo/Hr: Heating Design; OADB: -1

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.74
Wall Cond	-605	-605	6.09
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.92
Sub Total ==>	-2,359	-2,359	23.75
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.25
Adj Air Trans Heat	0		0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,359	-9,933	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	80.6
Ra Plenum	72.0	72.0
Return	72.6	72.0
Ret/OA	78.4	44.4
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	248	248
Terminal	248	248
Main Fan	248	248
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	248	248
Return	259	259
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	37.8	37.8
cfm/ft²	1.41	1.41
cfm/ton	286.38	
ft²/ton	202.84	
Btu/hr·ft²	59.16	-115.96
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.9	10.4	7.7	248	78.4	65.0	72.0	50.0	49.9	53.9
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.9	10.4								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-8.4	248	50.0	80.6
Aux Htg	0.0	0	0.0	0.0
Preheat	-1.5	248	44.4	50.0
Humidif	-10.5	259	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-20.4			

Project Name:
Dataset Name: TRACE000 trial.trc

'RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 1 System Checksums Report Page 1 of 8

East Façade

Room Checksums
By ACADEMIC

East Facing Patient Room - F/H

	COOLING COIL PEAK				CLG SPACE PEAK	
Peaked at Time:	Mo/Hr: 7 / 17				Mo/Hr: 7 / 18	
Outside Air:	OADB/WB/HR: 86 / 71 / 91				OADB: 83	
	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	3,816	0	3,816	35	3,914	66
Glass/Door Cond	170	0	170	2	147	2
Wall Cond	614	0	614	6	669	11
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	410		410	4	137	2
Sub Total ==>	5,010	0	5,010	46	4,868	82
Internal Loads						
Lights	300	75	375	3	570	10
People	640	0	640	6	392	7
Misc	80	0	80	1	75	1
Sub Total ==>	1,020	75	1,095	10	1,037	18
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	3,505	32	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	1,246		1,246	12	0	0
Exhaust Heat		-31	-31	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	7,276	44	10,825	100.00	5,906	100.00

HEATING COIL PEAK			
	Mo/Hr: Heating Design		
	OADB: -1		
	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.74
Wall Cond	-605	-605	6.09
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.92
Sub Total ==>	-2,359	-2,359	23.75
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.25
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,359	-9,933	100.00

TEMPERATURES	Cooling	Heating
SADB	50.0	80.8
Ra Plenum	72.0	72.0
Return	72.3	72.0
Ret/OA	77.5	43.8
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS	Cooling	Heating
Diffuser	243	243
Terminal	243	243
Main Fan	243	243
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	243	243
Return	254	254
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS	Cooling	Heating
% OA	38.7	38.7
cfm/ft²	1.38	1.38
cfm/ton	269.60	
ft²/ton	195.10	
Btu/hr·ft²	61.51	-114.75
No. People	2	

COOLING COIL SELECTION	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.9	10.8	8.3	243	77.5	64.6	71.5	50.0	49.2	51.3
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.9	10.8								

AREAS	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-8.3	243	50.0	80.8
Aux Htg	0.0	0	0.0	0.0
Preheat	-1.7	243	43.8	50.0
Humidif	-10.3	254	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-20.2			

Project Name:
Dataset Name: TRACE000 trial.trc

'RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 1 System Checksums Report Page 2 of 8

West Façade

Room Checksums

By ACADEMIC

West Facing Patient Room - F/H

COOLING COIL PEAK / CLG SPACE PEAK

Cooling Coil Peak – Peaked at Time: Mo/Hr: 7 / 10; Outside Air: OADB/WB/HR: 75 / 67 / 85

CLG Space Peak – Mo/Hr: 7 / 10; OADB: 75

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	3,847	0	3,847	35	3,847	74
Glass/Door Cond	17	0	17	0	17	0
Wall Cond	517	0	517	5	517	10
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	241		241	2	41	1
Sub Total ==>	4,622	0	4,622	42	4,423	85
Internal Loads						
Lights	300	75	375	3	300	6
People	640	0	640	6	392	8
Misc	80	0	80	1	80	2
Sub Total ==>	1,020	75	1,095	10	772	15
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	2,060	19	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	3,305		3,305	30	0	0
Exhaust Heat		-35	-35	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	8,947	40	11,048	100.00	5,195	100.00

HEATING COIL PEAK

Mo/Hr: Heating Design; OADB: -1

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.74
Wall Cond	-605	-605	6.09
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.92
Sub Total ==>	-2,359	-2,359	23.75
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.25
Adj Air Trans Heat	0		0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,359	-9,933	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	82.0
Ra Plenum	72.0	72.0
Return	72.3	72.0
Ret/OA	73.7	39.9
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	214	214
Terminal	214	214
Main Fan	214	214
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	214	214
Return	225	225
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	43.9	43.9
cfm/ft²	1.22	1.22
cfm/ton	232.36	
ft²/ton	191.17	
Btu/hr·ft²	62.77	-108.03
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.9	11.1	8.9	214	73.7	63.1	70.7	50.0	44.2	34.2
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.9	11.1								

AREAS

	Gross Total	Glass ft²	(%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-7.6	214	50.0	82.0
Aux Htg	0.0	0	0.0	0.0
Preheat	-2.4	214	39.9	50.0
Humidif	-9.1	225	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-19.0			

Project Name:
Dataset Name: TRACE000 trial.trc

RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 1 System Checksums Report Page 4 of 8

Trace Results – Proposed Façade

North Façade

Room Checksums
By ACADEMIC

North Facing Patient Room - H/H

	COOLING COIL PEAK				CLG SPACE PEAK		HEATING COIL PEAK			
Peaked at Time:	Mo/Hr: 7 / 15				Mo/Hr: 7 / 19			Mo/Hr: Heating Design		
Outside Air:	OADB/WB/HR: 88 / 72 / 93				OADB: 80			OADB: -1		
	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)		Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads							**Envelope Loads**			
Skylite Solar	0	0	0	0	0	0	Skylite Solar	0	0	0.00
Skylite Cond	0	0	0	0	0	0	Skylite Cond	0	0	0.00
Roof Cond	0	0	0	0	0	0	Roof Cond	0	0	0.00
Glass Solar	560	0	560	8	471	21	Glass Solar	0	0	0.00
Glass/Door Cond	180	0	180	3	118	5	Glass/Door Cond	-868	-868	8.81
Wall Cond	191	0	191	3	187	8	Wall Cond	-530	-530	5.38
Partition/Door	0		0	0	0	0	Partition/Door	0	0	0.00
Floor	0		0	0	0	0	Floor	0	0	0.00
Adjacent Floor	0	0	0	0	0	0	Adjacent Floor	0	0	0.00
Infiltration	453		453	7	102	4	Infiltration	-886	-886	8.99
Sub Total ==>	1,383	0	1,383	21	878	38	*Sub Total ==>*	-2,285	-2,285	23.18
Internal Loads							**Internal Loads**			
Lights	180	45	225	3	660	29	Lights	0	0	0.00
People	640	0	640	10	392	17	People	0	0	0.00
Misc	78	0	78	1	74	3	Misc	0	0	0.00
Sub Total ==>	898	45	943	14	1,126	49	*Sub Total ==>*	0	0	0.00
Ceiling Load	0	0	0	0	0	0	**Ceiling Load**	0	0	0.00
Ventilation Load	0	0	3,870	58	0	0	**Ventilation Load**	0	-7,574	76.82
Adj Air Trans Heat	0		0	0	0	0	**Adj Air Trans Heat**	0	0	0
Dehumid. Ov Sizing			0	0			**Ov/Undr Sizing**	0	0	0.00
Ov/Undr Sizing	544		544	8	279	12	**Exhaust Heat**		0	0.00
Exhaust Heat		-45	-45	-1			**OA Preheat Diff.**		0	0.00
Sup. Fan Heat			0	0			**RA Preheat Diff.**		0	0.00
Ret. Fan Heat		0	0	0			**Additional Reheat**		0	0.00
Duct Heat Pkup		0	0	0			**System Plenum Heat**		0	0.00
Underflr Sup Ht Pkup			0	0			**Underflr Sup Ht Pkup**		0	0.00
Supply Air Leakage		0	0	0			**Supply Air Leakage**		0	0.00
Grand Total ==>	2,825	0	6,695	100.00	2,283	100.00	***Grand Total ==>***	-2,285	-9,859	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	94.0
Ra Plenum	72.0	72.0
Return	72.4	72.0
Ret/OA	88.0	-1.0
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	94	94
Terminal	94	94
Main Fan	94	94
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	94	94
Return	105	105
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	100.0	100.0
cfm/ft²	0.53	0.53
cfm/ton	168.48	
ft²/ton	315.45	
Btu/hr·ft²	38.04	-80.10
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.6	6.7	4.0	94	88.0	71.9	93.1	50.0	49.3	51.5
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.6	6.7								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-4.6	94	50.0	94.0
Aux Htg	0.0	0	0.0	0.0
Preheat	-5.3	94	-1.0	50.0
Humidif	-4.2	105	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-14.1			

Project Name:
Dataset Name: TRACE000 trial.trc

RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 2 System Checksums Report Page 7 of 8

South Façade

Room Checksums

By ACADEMIC

South Facing Patient Room - F/F

COOLING COIL PEAK — Peaked at Time: Mo/Hr: 7 / 15; Outside Air: OADB/WB/HR: 88 / 72 / 93

CLG SPACE PEAK — Mo/Hr: 9 / 15; OADB: 76

	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Space Sensible Btu/h	Percent Of Total (%)
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	1,979	0	1,979	22	3,028	63
Glass/Door Cond	180	0	180	2	38	1
Wall Cond	432	0	432	5	505	11
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	453		453	5	47	1
Sub Total ==>	3,043	0	3,043	33	3,618	75
Internal Loads						
Lights	720	180	900	10	720	15
People	640	0	640	7	392	8
Misc	78	0	78	1	78	2
Sub Total ==>	1,438	180	1,618	18	1,190	25
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	3,870	42	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	751		751	8	0	0
Exhaust Heat		-90	-90	-1		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	5,232	90	9,192	100.00	4,809	100.00

HEATING COIL PEAK — Mo/Hr: Heating Design; OADB: -1

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.81
Wall Cond	-530	-530	5.38
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.99
Sub Total ==>	-2,285	-2,285	23.18
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.82
Adj Air Trans Heat	0	0	0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,285	-9,859	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	82.5
Ra Plenum	72.0	72.0
Return	72.8	72.0
Ret/OA	80.0	37.4
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	198	198
Terminal	198	198
Main Fan	198	198
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	198	198
Return	209	209
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	47.5	47.5
cfm/ft²	1.13	1.13
cfm/ton	258.54	
ft²/ton	229.77	
Btu/hr·ft²	52.23	-103.96
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.8	9.2	6.5	198	80.0	66.2	75.3	50.0	49.9	53.9
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.8	9.2								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-7.1	198	50.0	82.5
Aux Htg	0.0	0	0.0	0.0
Preheat	-2.8	198	37.4	50.0
Humidif	-8.4	209	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-18.3			

Project Name:
Dataset Name: TRACE000 trial.trc

RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 2 System Checksums Report Page 5 of 8

East Façade

Room Checksums
By ACADEMIC

East Facing Patient Room - F/H

COOLING COIL PEAK / CLG SPACE PEAK

	Cooling Coil Peak: Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	Clg Space Peak: Space Sensible Btu/h	Percent Of Total (%)
Peaked at Time:	Mo/Hr: 7 / 17				Mo/Hr: 7 / 18	
Outside Air:	OADB/WB/HR: 86 / 71 / 91				OADB: 83	
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	3,142	0	3,142	32	3,224	63
Glass/Door Cond	170	0	170	2	147	3
Wall Cond	538	0	538	5	587	11
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	410		410	4	137	3
Sub Total ==>	4,261	0	4,261	43	4,095	80
Internal Loads						
Lights	300	75	375	4	570	11
People	640	0	640	6	392	8
Misc	80	0	80	1	75	1
Sub Total ==>	1,020	75	1,095	11	1,037	20
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	3,505	35	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	1,088		1,088	11	0	0
Exhaust Heat		-35	-35	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	6,369	40	9,914	100.00	5,132	100.00

HEATING COIL PEAK

	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
	Mo/Hr: Heating Design		
	OADB: -1		
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.81
Wall Cond	-530	-530	5.38
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.99
Sub Total ==>	-2,285	-2,285	23.18
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.82
Adj Air Trans Heat	0		0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,285	-9,859	100.00

TEMPERATURES

	Cooling	Heating
SADB	50.0	81.8
Ra Plenum	72.0	72.0
Return	72.3	72.0
Ret/OA	78.3	39.5
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS

	Cooling	Heating
Diffuser	211	211
Terminal	211	211
Main Fan	211	211
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	211	211
Return	222	222
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS

	Cooling	Heating
% OA	44.5	44.5
cfm/ft²	1.20	1.20
cfm/ton	255.82	
ft²/ton	213.03	
Btu/hr·ft²	56.33	-107.02
No. People	2	

COOLING COIL SELECTION

	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.8	9.9	7.4	211	78.3	65.2	73.4	50.0	49.1	51.0
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.8	9.9								

AREAS

	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION

	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-7.4	211	50.0	81.8
Aux Htg	0.0	0	0.0	0.0
Preheat	-2.4	211	39.5	50.0
Humidif	-9.0	222	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-18.8			

Project Name:
Dataset Name: TRACE000 trial.trc

'RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 2 System Checksums Report Page 6 of 8

West Façade

Room Checksums

By ACADEMIC

West Facing Patient Room - F/H

COOLING COIL PEAK	Space Sens. + Lat. Btu/h	Plenum Sens. + Lat Btu/h	Net Total Btu/h	Percent Of Total (%)	CLG SPACE PEAK Space Sensible Btu/h	Percent Of Total (%)
Peaked at Time:	Mo/Hr: 7 / 11				Mo/Hr: 7 / 10	
Outside Air:	OADB/WB/HR: 79 / 68 / 88				OADB: 75	
Envelope Loads						
Skylite Solar	0	0	0	0	0	0
Skylite Cond	0	0	0	0	0	0
Roof Cond	0	0	0	0	0	0
Glass Solar	2,638	0	2,638	27	3,168	71
Glass/Door Cond	58	0	58	1	17	0
Wall Cond	473	0	473	5	453	10
Partition/Door	0		0	0	0	0
Floor	0		0	0	0	0
Adjacent Floor	0	0	0	0	0	0
Infiltration	303		303	3	41	1
Sub Total ==>	3,473	0	3,473	36	3,680	83
Internal Loads						
Lights	210	53	263	3	300	7
People	640	0	640	7	392	9
Misc	80	0	80	1	80	2
Sub Total ==>	930	53	983	10	772	17
Ceiling Load	0	0	0	0	0	0
Ventilation Load	0	0	2,591	27	0	0
Adj Air Trans Heat	0		0	0	0	0
Dehumid. Ov Sizing			0	0		
Ov/Undr Sizing	2,703		2,703	28	0	0
Exhaust Heat		-28	-28	0		
Sup. Fan Heat			0	0		
Ret. Fan Heat		0	0	0		
Duct Heat Pkup		0	0	0		
Underflr Sup Ht Pkup			0	0		
Supply Air Leakage		0	0	0		
Grand Total ==>	7,106	24	9,721	100.00	4,452	100.00

HEATING COIL PEAK	Space Peak Space Sens Btu/h	Coil Peak Tot Sens Btu/h	Percent Of Total (%)
Mo/Hr: Heating Design			
OADB: -1			
Envelope Loads			
Skylite Solar	0	0	0.00
Skylite Cond	0	0	0.00
Roof Cond	0	0	0.00
Glass Solar	0	0	0.00
Glass/Door Cond	-868	-868	8.81
Wall Cond	-530	-530	5.38
Partition/Door	0	0	0.00
Floor	0	0	0.00
Adjacent Floor	0	0	0.00
Infiltration	-886	-886	8.99
Sub Total ==>	-2,285	-2,285	23.18
Internal Loads			
Lights	0	0	0.00
People	0	0	0.00
Misc	0	0	0.00
Sub Total ==>	0	0	0.00
Ceiling Load	0	0	0.00
Ventilation Load	0	-7,574	76.82
Adj Air Trans Heat	0		0
Ov/Undr Sizing	0	0	0.00
Exhaust Heat		0	0.00
OA Preheat Diff.		0	0.00
RA Preheat Diff.		0	0.00
Additional Reheat		0	0.00
System Plenum Heat		0	0.00
Underflr Sup Ht Pkup		0	0.00
Supply Air Leakage		0	0.00
Grand Total ==>	-2,285	-9,859	100.00

TEMPERATURES	Cooling	Heating
SADB	50.0	83.3
Ra Plenum	72.0	72.0
Return	72.2	72.0
Ret/OA	75.8	34.6
Fn MtrTD	0.0	0.0
Fn BldTD	0.0	0.0
Fn Frict	0.0	0.0

AIRFLOWS	Cooling	Heating
Diffuser	183	183
Terminal	183	183
Main Fan	183	183
Sec Fan	0	0
Nom Vent	94	94
AHU Vent	94	94
Infil	11	11
MinStop/Rh	183	183
Return	194	194
Exhaust	105	105
Rm Exh	0	0
Auxiliary	0	0
Leakage Dwn	0	0
Leakage Ups	0	0

ENGINEERING CKS	Cooling	Heating
% OA	51.3	51.3
cfm/ft²	1.04	1.04
cfm/ton	226.31	
ft²/ton	217.26	
Btu/hr·ft²	55.23	-100.59
No. People	2	

COOLING COIL SELECTION	Total Capacity ton	Total Capacity MBh	Sens Cap. MBh	Coil Airflow cfm	Enter DB °F	Enter WB °F	Enter HR gr/lb	Leave DB °F	Leave WB °F	Leave HR gr/lb
Main Clg	0.8	9.7	7.4	183	75.8	64.5	73.7	50.0	45.6	38.6
Aux Clg	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Opt Vent	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0.0	0.0
Total	0.8	9.7								

AREAS	Gross Total	Glass ft²	Glass (%)
Floor	176		
Part	0		
Int Door	0		
ExFlr	0		
Roof	0	0	0
Wall	171	57	34
Ext Door	0	0	0

HEATING COIL SELECTION	Capacity MBh	Coil Airflow cfm	Ent °F	Lvg °F
Main Htg	-6.7	183	50.0	83.3
Aux Htg	0.0	0	0.0	0.0
Preheat	-3.1	183	34.6	50.0
Humidif	-7.9	194	1.1	59.2
Opt Vent	0.0	0	0.0	0.0
Total	-17.7			

Project Name:
Dataset Name: TRACE000 trial.trc

RACE® 700 v6.2.6.5 calculated at 01:32 AM on 04/04/2012
Alternative - 2 System Checksums Report Page 8 of 8

Appendix G

Secondary Members

m-factors

Column	Member Size	Connection	F_y	R_y	F_{ye}	$b_f/2t_f$	h/t_w	$52/\sqrt{F_{ye}}$	$418/\sqrt{F_{ye}}$	$65/\sqrt{F_{ye}}$	$640/\sqrt{F_{ye}}$	Beam m-factor	Interpolated m-factor	
F-9 Base	W24X68	Shear Tab	50	1.1	55	7.66	52	7.011679	56.36311	8.764598	86.29758	7.41	7.41	11.02
	W18X143	Shear Tab	50	1.1	55	4.25	22	7.011679	56.36311	8.764598	86.29758	10.00	21.03	18.04
	W24X94	Shear Tab	50	1.1	55	5.18	41.9	7.011679	56.36311	8.764598	86.29758	10.00	17.31	13.38
	W16X89	Shear Tab	50	1.1	55	5.92	27	7.011679	56.36311	8.764598	86.29758	10.00	14.36	16.87
	W24X76	Shear Tab	50	1.1	55	6.61	49	7.011679	56.36311	8.764598	86.29758	10.00	11.60	11.72
	W18X35	Shear Tab	50	1.1	55	7.06	53.5	7.011679	56.36311	8.764598	86.29758	9.81	9.81	10.67
F-9 Middle	W24X68	Shear Tab	50	1.1	55	7.66	52	7.011679	56.36311	8.764598	86.29758	7.41	7.41	11.02
	W18X143	Shear Tab	50	1.1	55	4.25	22	7.011679	56.36311	8.764598	86.29758	10.00	21.03	18.04
	W24X94	Shear Tab	50	1.1	55	5.18	41.9	7.011679	56.36311	8.764598	86.29758	10.00	17.31	13.38
	W16X89	Shear Tab	50	1.1	55	5.92	27	7.011679	56.36311	8.764598	86.29758	10.00	14.36	16.87
	W24X76	Shear Tab	50	1.1	55	6.61	49	7.011679	56.36311	8.764598	86.29758	10.00	11.60	11.72
	W18X35	Shear Tab	50	1.1	55	7.06	53.5	7.011679	56.36311	8.764598	86.29758	9.81	9.81	10.67
F-9 Roof	W24X68	Shear Tab	50	1.1	55	7.66	52	7.011679	56.36311	8.764598	86.29758	7.41	7.41	11.02
	W18X143	Shear Tab	50	1.1	55	4.25	22	7.011679	56.36311	8.764598	86.29758	10.00	21.03	18.04
	W24X94	Shear Tab	50	1.1	55	5.18	41.9	7.011679	56.36311	8.764598	86.29758	10.00	17.31	13.38
	W16X89	Shear Tab	50	1.1	55	5.92	27	7.011679	56.36311	8.764598	86.29758	10.00	14.36	16.87
	W24X76	Shear Tab	50	1.1	55	6.61	49	7.011679	56.36311	8.764598	86.29758	10.00	11.60	11.72
	W18X35	Shear Tab	50	1.1	55	7.06	53.5	7.011679	56.36311	8.764598	86.29758	9.81	9.81	10.67