2009-2010 AE Senior Thesis

Technical Report III

Lateral System Analysis and Confirmation Design for University Medical Center at Princeton

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Executive Summary

This report analyzes the lateral force resisting system of the New Hospital at the University Medical Center in Princeton. The system consists of 18 braced frames and 4 moment frames. While the moment frames span a long distance, they do not seem to play as significant of a role in handling lateral load as the braced frames do.

The controlling load case on this building is 1.2D + 1.6W + 0.5(L_R or S or R) in the North-South direction and 1.2D + 1.0E + L + 0.2S in the East-West direction. This is expected because the large building face on the north and south side accumulates a great deal of wind pressure. On the other hand, the east and west facades have much shorter faces and therefore the wind cannot create large lateral forces in that direction. This allows for seismic to control even though the building is in New Jersey.

All member checks that were performed confirmed the original design. For this report, the building was considered to be ten stories tall rather than the current six. This is because the structure was designed with the notion that four extra floors would potentially be added to the roof of the facility at a later date.

To assist with the force determination, SAP 2000 and RAM Structural System were used to model the lateral frames and the entire lateral system respectively. A simple trial loading on the lateral frames using SAP models was utilized to determine the relative stiffness of each frame and ultimately kick off the analysis of the system.

Story drift, overturning, and torsion effects were all considered in the analysis and outside of some inherent and accidental torsion; all were found to be controlled nicely by the lateral system.

A 3D model was constructed to help analyze the system more efficiently but unfortunately only half of the building could be successfully completed. While results were still able to be achieved with the unfinished model, this problem must be resolved in a timely manner.

Introduction

The University Medical Center at Princeton is a new state-of-the-art medical facility currently under construction in Plainsboro, NJ. The project consists of a Central Utility Plant, a Diagnostic and Treatment Center (D&T) and a New Hospital. The site already has an existing building (Building #2) and it will be connected to the north side of the New Hospital as part of the project. The Medical Office Building (MOB) is only proposed at this time. The 800,000 square foot complex is set to be complete by the summer of 2010.

The scope of this thesis project will be limited to structural analysis and re-design of the New Hospital (Figure 1). This is the tallest portion of the complex at 92'-0" from grade to roof with a 14'-0" metal panel system above for a total height of 106'-0" above grade.

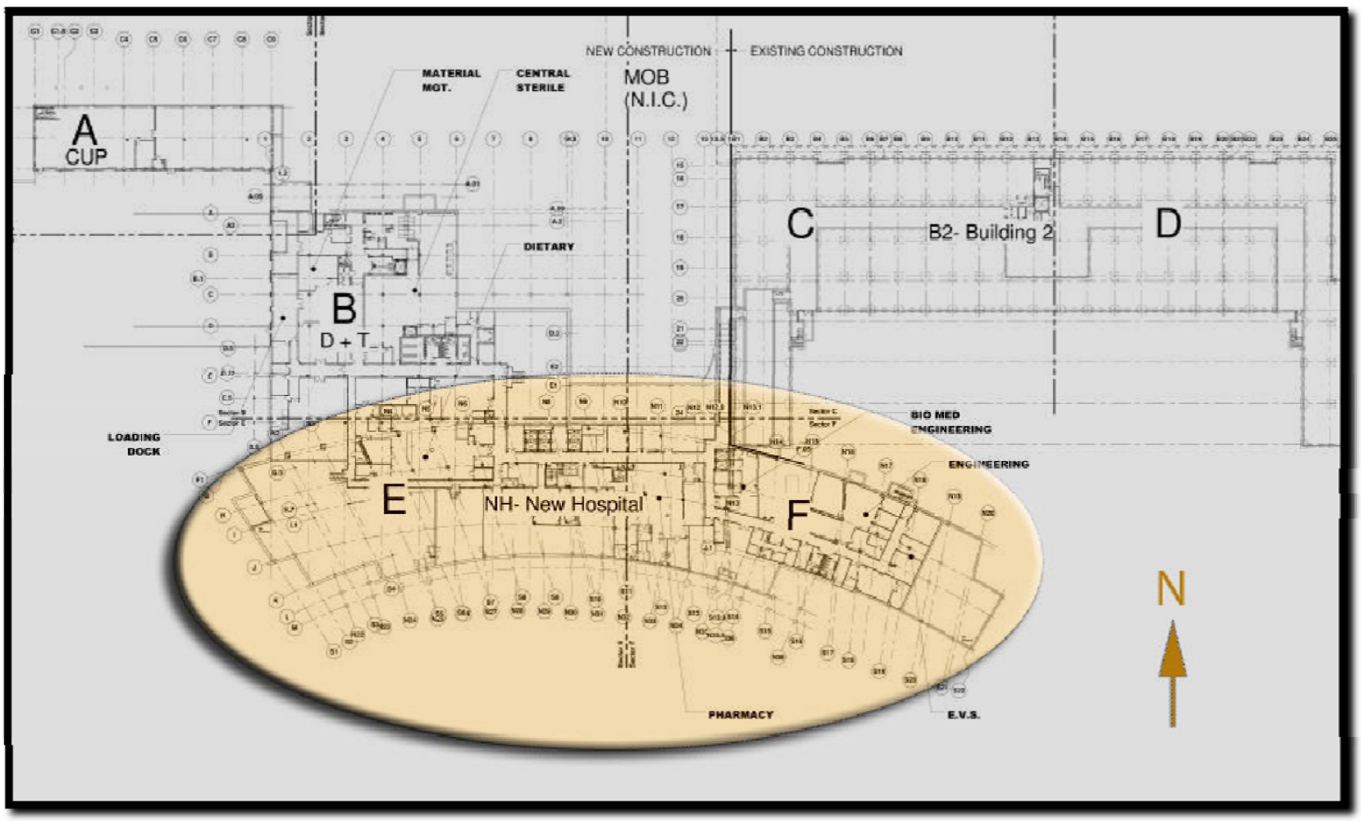


Figure 1: Overall Plan University Medical Center at Princeton

Structural System Overview

The structural system of the New Hospital at the University Medical Center was designed by O'Donnell & Naccarato Structural Engineers using a Load Resistance Factor Design approach. It is a structural steel building with a composite floor diaphragm. Braced frames run in both directions and there are two long moment frames spanning the entire length of the building on both the south and north facades as seen below in Figure 2. Both the braced and moment frames are the building's main resistance to lateral load. Due to the great length of the building in the west-east direction, an expansion joint was placed at a distance from the western façade roughly equal to 2/3 of the total building length. This effectively splits the building into two different structures which behave on their own.

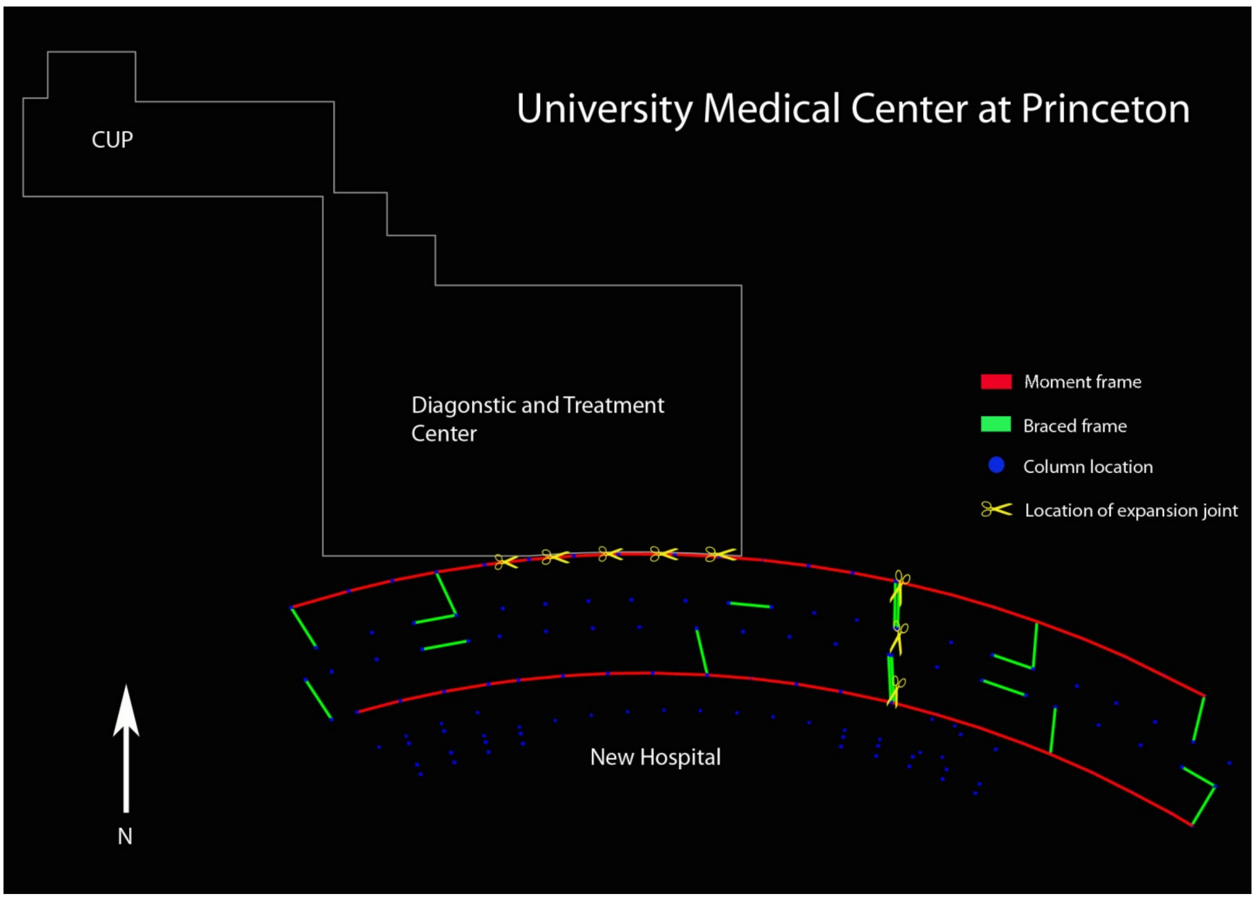


Figure 2: Overall schematic of lateral force resisting elements

Foundation

Concrete piers with sizes anywhere from 18" x 18" to 48" x 78" are attached to the base of the steel columns and transmit vertical load from the superstructure to the concrete spread footings. The size of these footings varies from as small as 3'-0" x 3'-0" x 14" to as large as 21' x 21' x 50".

All footings supporting braced frame columns have mini-piles attached at their base in order to handle high tension forces resulting from lateral loading. These piles extend to decomposed bedrock (8'-30' deep). The top of all exterior footings are at a minimum depth of 42" below grade.

The floor at the base level is concrete slab-on-grade with thicknesses from 4"-12".

Huge concrete retaining walls with footings up to 17'-0" wide trace the perimeter of the foundation system.

Superstructure

The structural steel provides both gravity and lateral load resistance for the building. Columns are typically W14 while beams and girders range from W12-W27 shapes. Rectangular HSS shapes are used for the diagonal members in the braced frames and round HSS columns support the massive glass façade on the south face of the hospital. The HSS columns are intentionally exposed for architectural purposes. The floor layout is uniform and has a typical bay size of 30' x 30'.

The floor system spanning over the main area of the building is composite construction. Typically, the concrete slab is 3-1/4" lightweight concrete poured over a 3" composite metal deck. In certain mechanical and roof areas, the floor system switches to a 6-1/2" normal weight concrete due to higher loads in those areas.

The composite floor is considered to act as a rigid diaphragm and therefore able to transmit lateral forces from the façade to the braced and moment frames.

The scope of this report is to provide a thorough evaluation of the lateral force resisting system for the New Hospital at the University Medical Center. Topics in this report include: lateral load determination, distribution of the lateral forces into resisting members, explanation of load path, member checks for strength, serviceability checks for drift, overturning analysis, and evaluation of torsion issues due to eccentric loading.

Lateral System

The primary components of the lateral force resisting system in the New Hospital are braced and moment frames. On the western wing of the facility, there are six braced frames running in the N-S direction. In the W-E direction, there are three braced frames and two long moment frames. The eastern wing has a similar layout with six braced frames in the N-S and three in the W-E as well as two moment frames in the W-E.

When lateral load is levied upon the structure, it is directed towards these frames. This is accomplished through the rigid floor diaphragm. Essentially, the diaphragm can be thought of as the "collector" of lateral force. (Figure 3 below) It accepts the load from the façade and distributes that load to each of the frames that it is tied into. However in this building, the important characteristic of the diaphragm is that is behaves rigidly. That is to say that the composite floor system is stiff enough to induce equal displacement of everything that is attached to it when subjected to lateral loading. Because the diaphragm is rigid it distributes lateral force to the frames based upon each frame's individual stiffness. The stiffer frames will receive more force than those frames which are less stiff. This is essentially how the distribution of lateral forces is achieved and it is all because the floor diaphragm is rigid.

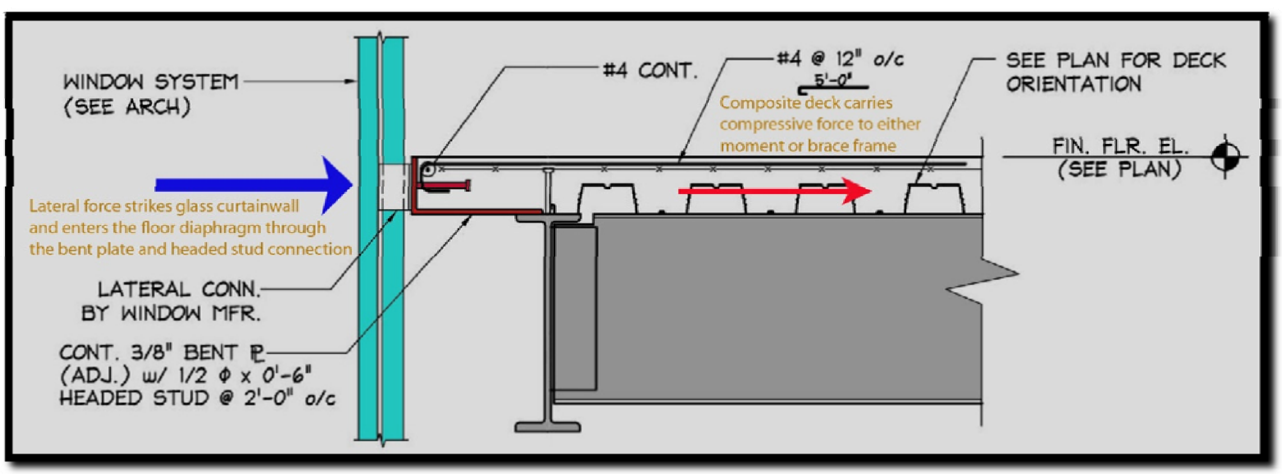


Figure 3: Lateral force enters the diaphragm from the façade and is carried to the braced and moment frames.

Once the proportional amount of force reaches the braced frame, it is transferred into the members of the frame. The frame is capable of handling this horizontal force because of the diagonal bracing between the columns. For this structure, the diagonal is a rectangular HSS tube which carries the force axially to the opposite corner of the panel. The tubes also resist the tendency for the frame to displace under load and provide support to the columns they are connected to. Figure 4 below shows how the load travels through the height of the frame and eventually to the base. It is here where the force is transmitted to the concrete pier and/or spread footing and into the ground.

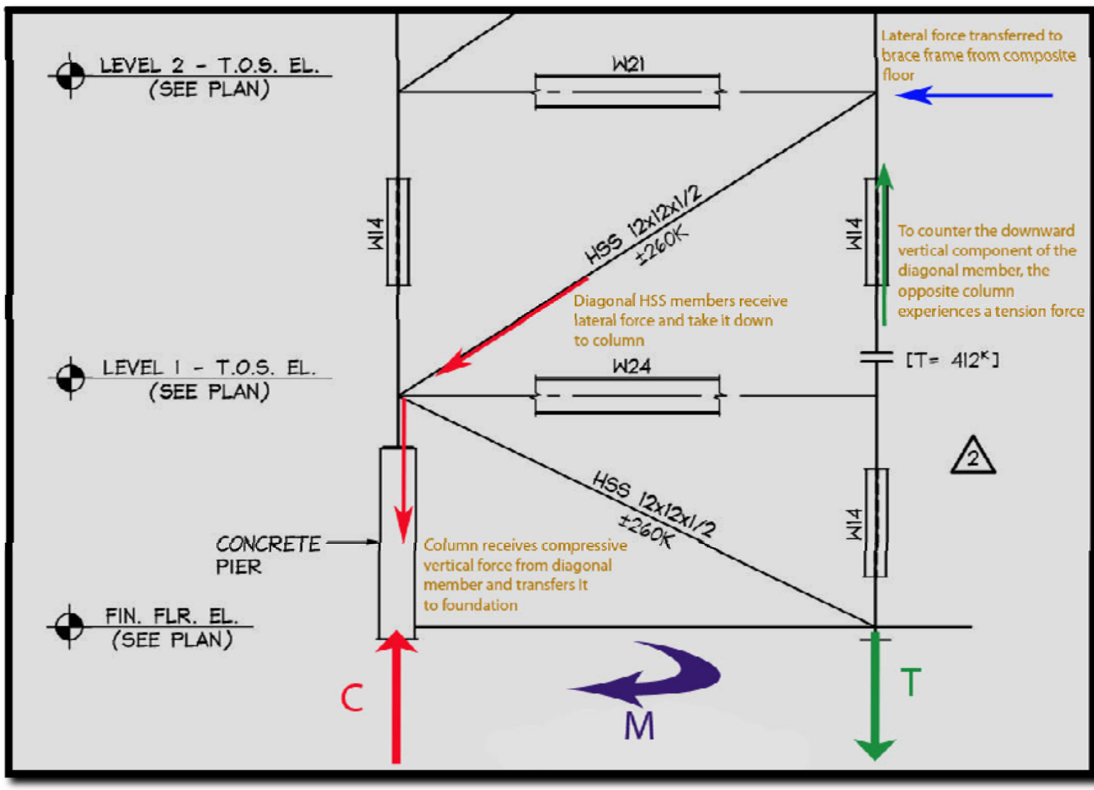


Figure 4: The above diagram shows how the lateral force is delivered into the foundation of the building through the diagonal braces of the frame.

In a braced frame like the one shown above, the columns on the "loading side" of the frame are in tension while the columns on the far side are in compression. This coupling of forces creates a moment that opposes the tendency of the lateral force to push the frame in a counterclockwise direction. In the case of wind blowing from the other direction, the forces in the columns will flip and the member that was once in tension would then be in compression and vice versa.

Once the force from the diaphragm is taken into the footing, it must be transmitted to the soil below. In the case of a compressive force pushing down, the footing will be driven into the ground and release that force into the soil. However, if the force is a tensile one, it will try to pull the footing out of the ground. If the footing is large enough and heavy enough, it will be able to resist. But in the case of the University Medical Center, the designers chose to use mini-piles (Figure 5) as a means of holding down the footing.

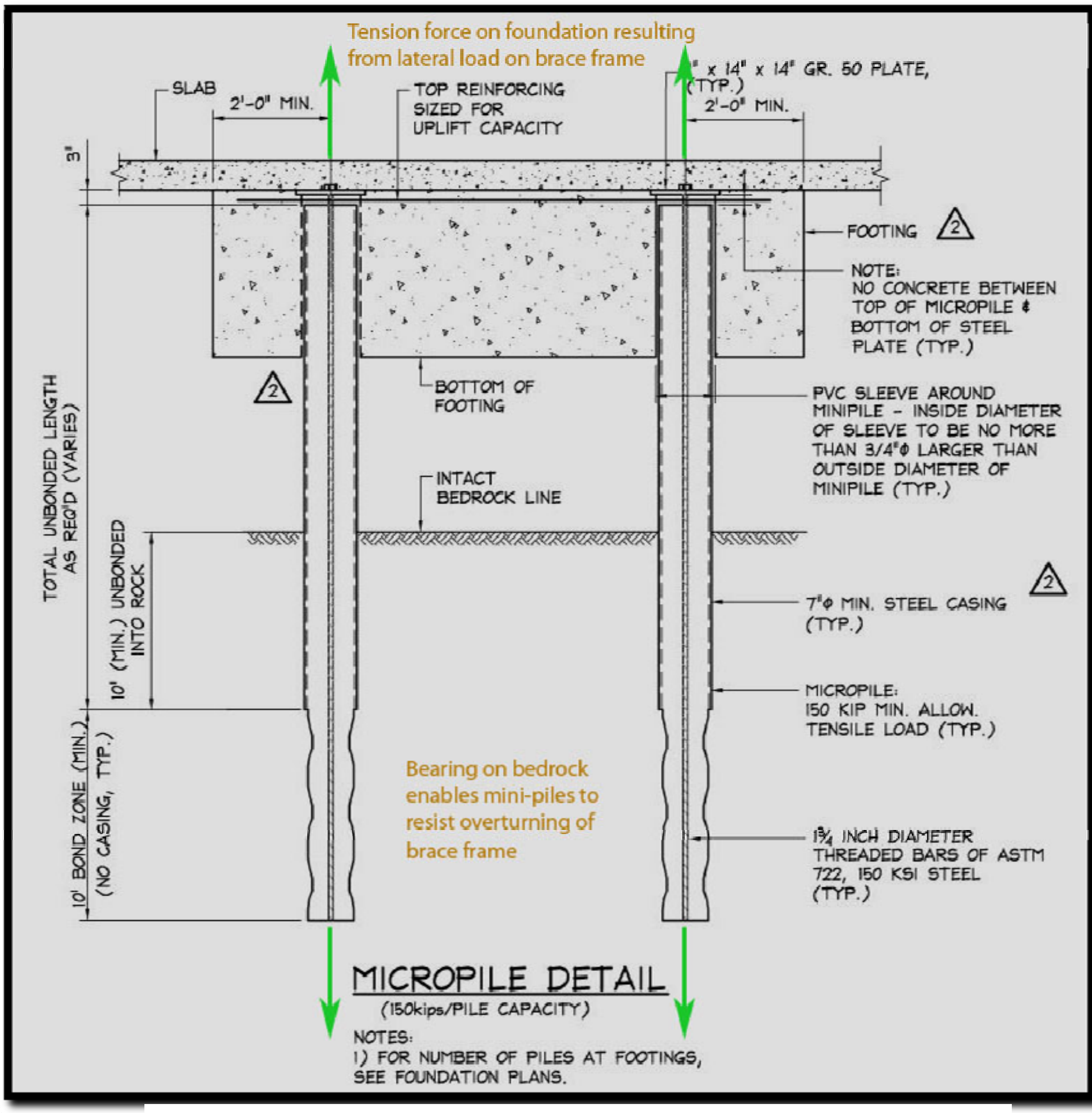


Figure 5: Mini piles are attached to the footing to resist tension forces.

These piles are found underneath every braced frame in the New Hospital thereby eliminating the need to upsize the footings.

Due to the curved façade of the hospital, no frame is placed exactly perpendicular to loading. This means that while more of the frames are oriented towards the North-South direction, each braced frame participates in resisting loads from all directions. So for wind striking the building from the East, the braced frames which typically handle the load from the South help out in delivering these forces to the foundation. Also helping are the two long moment frames along the North and South facades. Moment frames do not have diagonal members but rely on the stiffness of the columns and beams to resist lateral loads. Without the diagonals, these frames are significantly less stiff than braced frames and consequently do not handle as much load. However, they do contribute to the overall lateral resisting system albeit mainly for loads acting along the East-West axis of the building.

Design Loads

Live loads were obtained from ASCE7-05 and are considered to be the absolute minimum design loads allowed for a hospital (Figure 6). Most of the dead loads are assumed based upon standard industry practice (Figure 7).

Live Loads	
First Floor Corridors	100 psf
Lobbies	100 psf
Corridors above First Floor	80 psf
Patient Rooms	40 psf
Operating Rooms	60 psf
Roof	20 psf
Penthouse Floor	100 psf
Offices	50 psf
Stairs	100 psf
Partitions	20 psf

Figure 6: Live loads specified in the building code.

Dead Loads	
Superimposed	
MEP	8 psf
Ceiling	5 psf
Total	13 psf
Typical Floor	
3" metal deck	3 psf
3-1/4" LW concrete	48 psf
Allowance for steel framing	5 psf
Total	56 psf
Mechanical Roof	
3" metal deck	3 psf
6-1/2" NW concrete	100 psf
Allowance for steel framing	7 psf
Total	110 psf
Hospital Roof	
3" metal deck	3 psf
6-1/2" NW concrete	100 psf
Allowance for steel framing	6 psf
MEP	20 psf
Total	129 psf
Walls	
Curtain wall	25 psf

Figure 7: Dead loads based upon standard industry practice.

Some of the design loads used by the designers at O'Donnell and Naccarato differed from those loads listed in the tables above. For a typical floor, the design dead load was 65 psf and the design live load was 85 psf. The design dead load for the hospital roof was 140 psf. Because this facility is a hospital it is not unusual for the designer to use higher load values in order to guarantee a safer design.

Materials

All of the major structural materials incorporated into the design of the New Hospital at the University Medical Center are listed in Figure 8 below. The corresponding material strengths are to the right of each item.

Concrete	
Footings	f'c = 3000 psi
Retaining walls	f'c = 3000 psi
Foundation walls	f'c = 3000 psi
Piers	Min. of f'c = 3000 psi
Slab on grade	f'c = 3500 psi
Slab on metal deck	f'c = 4000 psi
Lightweight concrete	f'c = 3500 psi
Structural Steel	
Wide Flange Shapes	ASTM A992
Rectangular/Square HSS Shapes	ASTM A500 Grade B
Steel Pipe Sections	ASTM A501 or ASTM A53, Type E or S, Grade B
Angles	ASTM A36
Plates	ASTM A36
¾" Bolts	A325 or A490
Anchor Rods	ASTM F1554 Grade 55
Welding Electrode	E70XX
Reinforcement	
Reinforcing bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185
Decking	
Roof deck	1-1/2" Galvanized Type B Metal Deck, 22 Ga.
Floor deck	3" LOK-Floor Composite Metal Deck, 20 or 18 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Solid Units	ASTM C90, f'c = 1900 psi
Hollow Units	ASTM C90, f'c = 1900 psi
Ivany Units	f'c = 3000 psi
Grout	f'c = 3000 psi
Brick	ASTM C216 Grade SW, f'c = 3000 psi

Figure 8: Structural materials used and design strengths

Wind Loads

The hospital is originally designed for the potential addition of four extra stories above the currently designed roof. Since the members were designed to handle additional floors, it seems reasonable to evaluate the lateral system as if it were a ten story building rather than six stories. Therefore, the wind loading from Tech I had to be modified to include four additional levels.

Forces Due to Wind on New Hospital (N-S Direction) B = 398 ft. L= 109 ft.									
Level	Height Above Ground	Story Height	Force			Shear		Moment	
			windward	leeward	total	windward	total	windward	total
	(ft)	(ft)	(k)	(k)	(k)	(k)	(k)	(ft-k)	(ft-k)
1	0	0	0	0	0	919.56	1444.25	0	0
2	17	17	92.06	-66.30	158.35	919.56	1444.25	1565	2692
3	35	18	92.84	-60.62	153.45	827.50	1285.90	3249	5371
4	49	14	86.74	-53.04	139.77	734.67	1132.45	4250	6849
5	63	14	91.01	-53.04	144.05	647.93	992.68	5734	9075
6	77	14	94.70	-53.04	147.74	556.92	848.63	7292	11376
7	91	14	97.97	-53.04	151.00	462.22	700.89	8915	13741
8	105	14	100.91	-53.04	153.95	364.26	549.89	10596	16165
9	119	14	103.61	-53.04	156.65	263.34	395.94	12329	18641
10	133	14	106.09	-53.04	159.13	159.74	239.29	14110	21164
Roof	147	14	53.64	-26.52	80.16	53.64	80.16	7886	11784
					Total:	919.6	1444.3	75926	116858

Figure 9: Lateral forces exerted on hospital due to wind pressure.

Seismic Loads

The inclusion of four other stories also affected the seismic calculations. The values at each level are smaller than they were in Tech I due to the change in distribution of mass throughout the taller building. The value used for the period for Tech I has also been modified. Before, an approximate period was used which turned out to be too conservative of an approach. For this report, the period is multiplied by a C_u factor which decreases the seismic shear forces.

Forces due to Seismic Vibration					
Level	Height Above Ground	Story Height	Weight	C_{vx}	F_x
	(ft)	(ft)	(K)		(k)
1	0	0	0	0	0
2	17	17	3552	0.010	12.92
3	35	18	3587	0.029	37.65
4	49	14	3427	0.045	58.94
5	63	14	3427	0.066	85.23
6	77	14	3414	0.088	114.00
7	91	14	3414	0.112	145.68
8	105	14	3414	0.138	179.72
9	119	14	3414	0.166	215.96
10	133	14	3414	0.195	254.26
Roof	147	14	2278	0.151	196.51
		Sum	33344	1.000	1300.87

Figure 10: Lateral forces exerted on hospital due to seismic vibration.

Load Combinations

The load combinations used for the analysis are listed below. These combinations must be considered during design per ASCE7-05.

1.4(D+F)

1.2(D+F+T) + 1.6(L+H) + 0.5(L_R or S or R)

1.2D + 1.6(L_R or S or R) + (L or 0.8W)

1.2D + 1.6W + 0.5(L_R or S or R)

1.2D + 1.0E + L + 0.2S

0.9D + 1.6W + 1.6H

0.9D + 1.0E + 1.6H

It was determined that the controlling load case is 1.2D + 1.6W + 0.5(L_R or S or R) in the North-South direction which makes sense because of the building's long, rectangular shape. In the East-West direction, 1.2D + 1.0E + L + 0.2S governs the design. This is due to the small façade on that portion of the building which doesn't experience wind forces as strong as the other building face. The last two combinations listed will also be included in the analysis since they are typically the controlling cases for overturning. The other three combinations are discarded since at first glance it is obvious that they do not control this lateral design.

ASC E7-05 also specifies different load cases for wind in order to handle eccentric loading (Case II) and diagonal loading (Case III). These cases were also considered in the analysis however they did not control.

Relative Stiffness of Lateral Frames

As mentioned earlier, the relative stiffness of the braced and moment frames plays a significant role in the distribution of lateral forces from the rigid diaphragm. Stiffness can be calculated by applying a force on an object, measuring the displacement of that same object due to the applied force and dividing that value into the force. The equation below can be used to determine frame stiffness.

$$F = K * \Delta$$

where F is the applied force, Δ is the measured displacement, and K is the stiffness. This calculation can be simplified further with computer models.

In this hospital, there are a total of 18 braced frames, each slightly unique from the others. In order to obtain an idea of each frame's stiffness, all were modeled in 2D using SAP2000. (Figure 11) By applying a unit load at the upper left corner of the frame, a displacement was induced. (Figure 12)

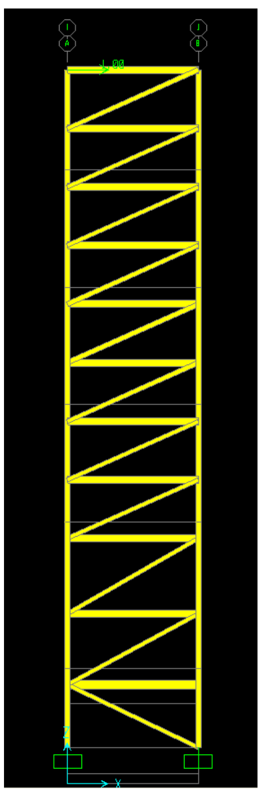


Figure 11: Braced Frame modeled in SAP2000.

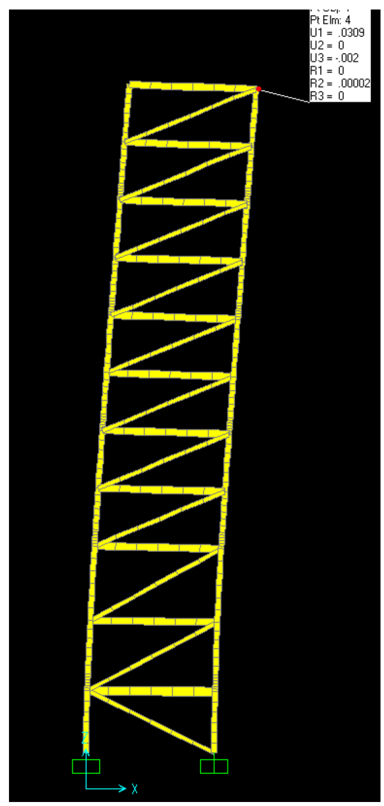


Figure 12: Displaced braced frame under 1 kip load.

The force is now a known quantity and since SAP automatically calculates the displacement of the top edge, the *stiffness* of this frame can be found by taking the inverse of the displacement. This process was repeated for all 18 braced frames and the two moment frames. The *relative stiffness* for indivdual frames is then found by dividing the "K" value for each frame by the sum of all 18 K values. This method is a quick, easy way to get a decent idea of how the lateral forces will be distributed throughout the building. The calculated stiffnesses can be found in Figure 15 on the next page.

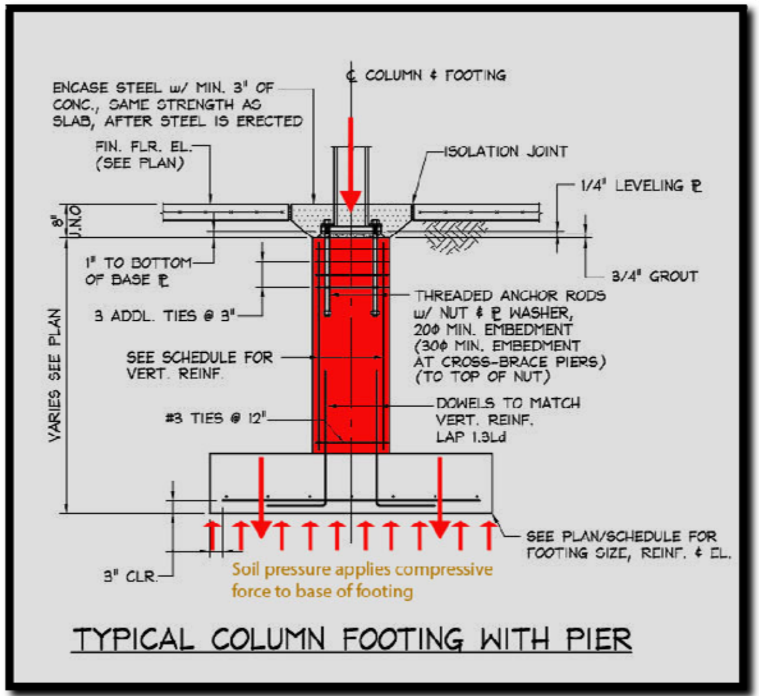


Figure 13: Connection at face of pier appears more fixed than pinned.

These 2D SAP models shown above were easily modeled but it should be noted that each frame had slight differences which were taken into consideration. It was assumed that the fixed condition at the supports was more accurate than the pinned condition since the columns frame directly into a spread footing or concrete pier. However, both conditions were modeled to compare the results. All brace ends were released to prevent transfer of moment. In some cases, the brace would be connected to the pier which would then be attached to the footing. Since the axial force in the brace has a horizontal component, it is then expected that the concrete pier be designed to handle the shear and bending forces which result from that connection in order to get the forces into the footing. (Figure 14)

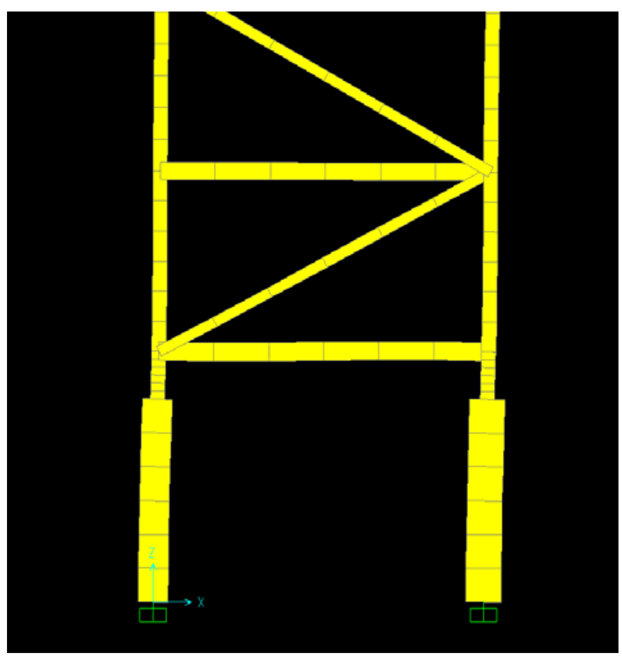


Figure 14: Close up view of concrete piers transmitting shear from the column and into the footing.

Braced Frame Relative Stiffness-Level 10				
West Wing-New Hospital				
Frame #	End Condition	K	%	Direction of Resistance
CB1	Fixed	32.4	0.12	N-S
	Pinned	23.7	0.11	
CB2	Fixed	31.7	0.12	N-S
	Pinned	9.5	0.04	
CB3	Fixed	32.2	0.12	N-S
	Pinned	30.1	0.14	
CB4	Fixed	25.3	0.09	E-W
	Pinned	23.6	0.11	
CB5	Fixed	23.0	0.08	E-W
	Pinned	22.9	0.10	
CB6	Fixed	32.9	0.12	N-S
	Pinned	29.8	0.14	
CB7	Fixed	20.0	0.07	E-W
	Pinned	16.8	0.08	
CB8	Fixed	39.1	0.14	N-S
	Pinned	30.9	0.14	
CB9	Fixed	36.0	0.13	N-S
	Pinned	31.3	0.14	
	Sum-Fixed	272.5		
	Sum-Pinned	218.5		

Figure 15: Relative stiffness of braced frames on the western half of level 10

The results of the stiffness calculation seem very reasonable as all of the values are within a couple of percentage points. This makes sense because the width of each frame is nearly the same (about 30'-0") and many of the section properties of the members were the same if not very similar. Even with each frame having a different brace arrangement or different foundation conditions, the results still come out rather even. Since these answers seem legitimate, they will be used to double check the results of the 3D model.

3D Model

The New Hospital at the University Medical Center in Princeton is a gorgeously designed building with a sleek, curved façade that not only defines the rest of the floor layout but will also make a bold architectural statement. Unfortunately, those aspects of the building make it difficult to model. Due to the curved facades and radial column grid, the hospital was chosen to be modeled in RAM Structural System. (Figure 16)

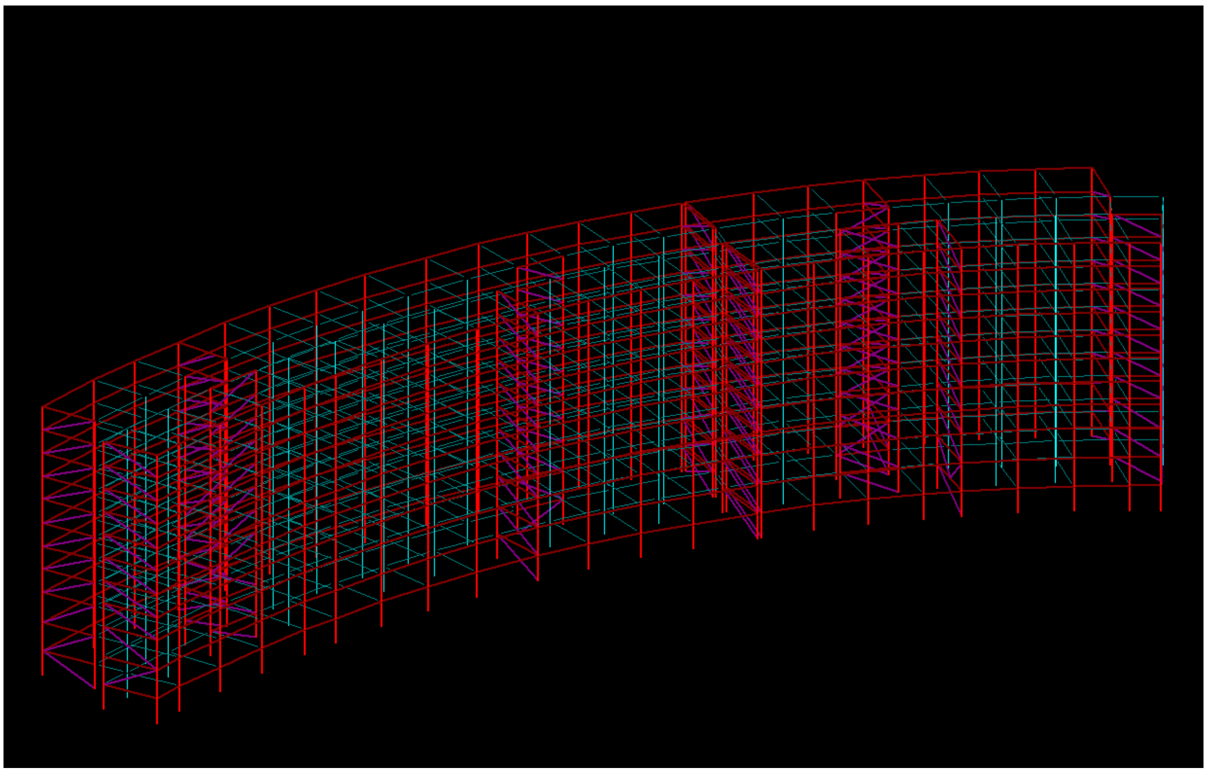


Figure 16: New Hospital modeled in RAM Structural System. Note the expansion joint near the middle.

The framing was successfully implemented into the model without any errors and a rigid diaphragm was applied at every floor with a mass dead load equal to the weight of the composite floor system (120 pcf*(3.25'+1.5)/12) plus the weight of the composite metal deck (3 psf) and superimposed dead load of 63 psf which includes framing members, exterior walls, and all superimposed dead load. All framing members and deck were set to zero weight in order to avoid specifying sizes for all members of the gravity system. Since framing was considered within the dead weight there was a worry about double-counting particular weights. An approximate value for the building mass was added on later as a surface load based upon the dead and live loads previously presented in this report.

The next step was to apply an arbitrary 1000 kip load at the top floor in order to confirm the relative stiffness of all the braced frames. This load was applied to two separate diaphragms due to the expansion joint. In fact, the two structures were never joined in this model, only side-by-side. The modal and displacement results of the west structure seemed to be within the correct ballpark. However, the eastern half was undergoing rapid translation which was out-of-sync with the response of the western side. After some troubleshooting, the error was not resolved.

In order to continue with the report, the model was split into two completely separate models (which is not much of a difference from reality) but since the problem was still not fixed with the eastern side, only the western side is being analyzed at this time. Possible reasons for the model failure could be improper mass value or placement. Even though this was checked numerous times, it seems like the only plausible solution considering there is not much to the model (only steel members and a diaphragm!) In any case, this problem will have to be resolved quickly so that the analysis can be completed and re-design can begin for next semester.

Even though the model is technically incomplete, there is still an opportunity to perform a complete analysis on most of the structure. Again, a 1000 kip load was applied at the center of pressure and the forces were evaluated in the members of all braced frames on the 10th story. This was accomplished simply by utilizing the equations of static equilibrium.

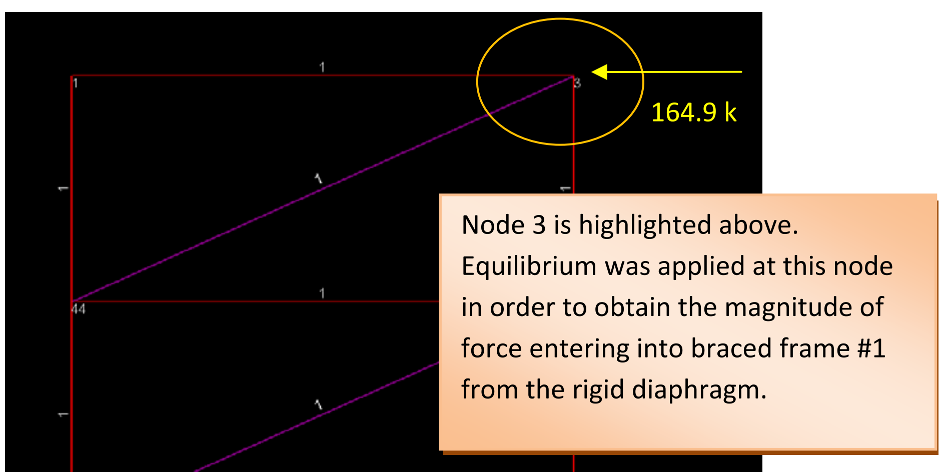


Figure 17: Equilibrium calculation for braced frames on tenth story.

By the laws of Statics, every node must be balanced in all directions. This made it possible to discern the magnitude of external force acting either into or out of a braced frame but still in the same plane as the frame. Once this force was calculated, it was compared with the forces applied to all the other braced frames to see if the distribution of force corresponded to the relative stiffness. In order to interpret the results correctly, the force acting in the plane of the frame must be resolved into a global coordinate system where the y component of the force is parallel with the 1000 kip applied load in the y-direction and the x component of the force is perpendicular to the global y.

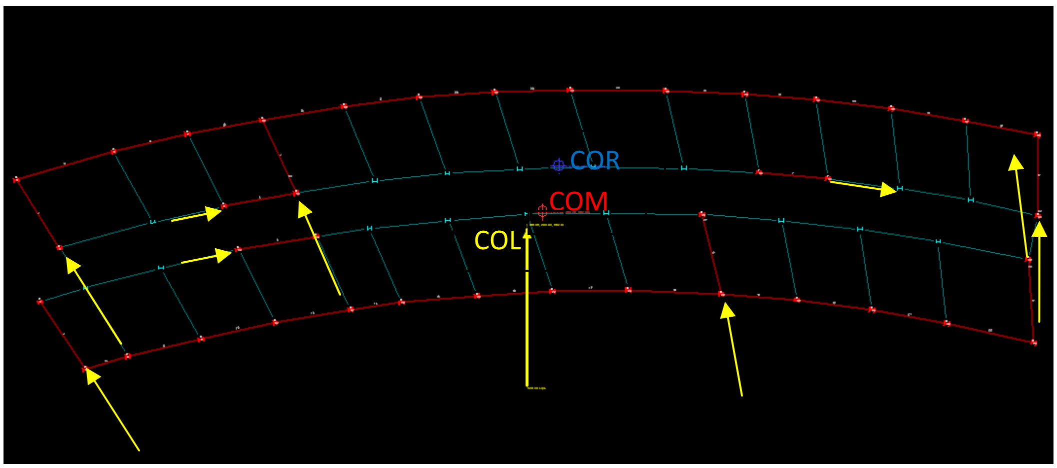


Figure 18: 1000 kip force applied at the center of pressure. Forces are then distributed based upon relative stiffness but remain in plane with the braced frame.

Equilibrium Check											
Level 10-West		Frame 1	Node 3	$Angle_{br}$=	24.21	Local			Global		
Member	Axial	Y_{comp}	X_{comp}	V_{major}	V_{minor}	∑Fy	∑Fx	∑Fz	Y	X	Angle
	(k)	(k)	(k)	(k)	(k)	(k)	(k)	(k)	(k)	(k)	degree
Brace	178.04	73.01	162.38	0	0						
Column	-74.36	0	0	0.31	2.53	0.00	**164.91**	0.00	139.6	-87.85	32.19
Beam	0	0	0	1.35	0						

Figure 19: Equilibrium confirmation

As shown above in Figure 18, frame 1 receives about 165 out of the 1000 kips applied to the diaphragm. This check was continued for each frame. The results are displayed below in Figure 20.

Frame Relative Stiffness-Level 10							
West Wing-New Hospital							
Frame #	End Condition	Δ	K	%X	RAM-X	%Y	RAM-Y
CB1	Fixed	0.0309	32.4	0.11	0.02	0.12	0.15
	Pinned	0.0422	23.7				
CB2	Fixed	0.0337	29.7	0.10	0.03	0.11	0.15
	Pinned	0.1056	9.5				
CB3	Fixed	0.0309	32.4	0.11	0.02	0.12	0.10
	Pinned	0.0332	30.1				
CB4	Fixed	0.0359	27.9	0.09	0.50	0.10	0.17
	Pinned	0.0423	23.6				
CB5	Fixed	0.0435	23.0	0.08	0.13	0.08	0.04
	Pinned	0.0436	22.9				
CB6	Fixed	0.0295	33.9	0.11	0.02	0.12	0.15
	Pinned	0.0336	29.8				
CB7	Fixed	0.0478	20.9	0.07	0.12	0.08	0.02
	Pinned	0.0596	16.8				
CB8	Fixed	0.0263	38.0	0.13	0.01	0.14	0.12
	Pinned	0.0324	30.9				
CB9	Fixed	0.0274	36.5	0.12	0.02	0.13	0.12
	Pinned	0.0320	31.3				
MF1	Fixed	0.0716	14.0	0.05	0.07	N/A	N/A
	Pinned	0.1011	9.9				
MF2	Fixed	0.0716	14.0	0.05	0.06	N/A	N/A
	Pinned	0.1011	9.9				
		∑Kx=	302.5				
		∑Ky=	274.6				

Figure 20: Total Relative Stiffness as compared with RAM results.

The relative stiffness in the x-direction varies quite a bit while the stiffness in the y-direction is closer to the original calculation. One reason for the variance could be the inaccuracy with the depth of the frame below grade. In SAP, the frame was modeled from the top of the frame to the footing which is technically below grade. The depth of the footing varied with nearly every frame. In RAM, it is not as easy to create elements below "grade" which means that the frames in RAM stopped at the lower level and not carry down to the actual footing depth. Height has a significant impact in the stiffness of the frame so it is possible that a couple feet could make a difference.

Drift

While overall building drift and story drift are not strength requirements, it is still an important serviceability requirement that should be met. Under wind loading, the typical benchmark to limit drift is H/400 where H is the height of each story. In this case, the typical story height is 14'-0" which would limit typical story drift to 14*12/400= 0.42". Using displacement values measured from the center of mass of the RAM model, story drift is calculated and compared to the benchmark. This comparison can be seen in Figure 21 below.

Story Drift Due to Unfactored Wind Load								
Story	Displ. Y	ΔY	Displ. X	ΔX	Story Height	Height above Ground Level	H/400	Ok?
	(in)	(in)	(in)	(in)	(ft)	(ft)	(in)	
10	2.4365	0.1646	0.3435	-0.0005	14	147	0.42	YES
9	2.2719	0.1876	0.3441	0.0053	14	133	0.42	YES
8	2.0842	0.2103	0.3387	0.0099	14	119	0.42	YES
7	1.8740	0.2301	0.3288	0.0107	14	105	0.42	YES
6	1.6438	0.2351	0.3181	0.0163	14	91	0.42	YES
5	1.4088	0.2655	0.3018	0.0501	14	77	0.42	YES
4	1.1433	0.2600	0.2518	0.0443	14	63	0.42	YES
3	0.8833	0.2714	0.2075	0.0444	14	49	0.42	YES
2	0.6120	0.3294	0.1631	0.0778	14	35	0.42	YES
1	0.2825	0.2825	0.0853	0.0853	18	17	0.54	YES
LL	0.0000	0.0000	0.0000	0.0000	17	0	N/A	N/A

Figure 21: Story drift due to unfactored wind load. Each story meets the serviceability criteria.

It is clear that the hospital meets the serviceability requirement for story drift at all levels.

Using the H/400 guideline for overall building drift, it is determined that the building drift should not exceed a total of 147'*12/140= 4.41" from bottom to top. The maximum displacement is 2.44" in the y-direction which easily meets this requirement.

Seismic drift is a slightly different calculation. According to Table 12.12-1 of ASCE7-05, seismic story drift should be limited to $0.010h_{sx}$ for buildings with occupancy category IV. In this formulation, h_{sx} is the story height of the floor directly below. Results of the seismic drift calculation can be seen in Figure 22 on the next page.

Seismic Story Drift								
1.2D+1.0E in both directions								
Story	Displ. Y (in)	Δy (in)	Displ. X (in)	Δx (in)	Story Height (ft)	Height above Ground Level (ft)	$.010h_{sx}$ (in)	OK?
10	3.1511	0.2435	0.4893	0.0260	14	147	1.68	YES
9	2.9076	0.2807	0.4633	0.0377	14	133	1.68	YES
8	2.6269	0.3122	0.4257	0.0459	14	119	1.68	YES
7	2.3148	0.3346	0.3797	0.0487	14	105	1.68	YES
6	1.9802	0.3296	0.3311	0.0529	14	91	1.68	YES
5	1.6506	0.3553	0.2782	0.0754	14	77	1.68	YES
4	1.2952	0.3309	0.2028	0.0587	14	63	1.68	YES
3	0.9644	0.3249	0.1441	0.0473	14	49	1.68	YES
2	0.6394	0.3623	0.0968	0.0554	14	35	2.16	YES
1	0.2772	0.2772	0.0414	0.0414	18	17	2.04	YES
LL	0.0000	0.0000	0.0000	0.0000	17	0	N/A	N/A

Figure 22: Seismic story drift. Each story meets ASCE criteria.

Torsion Considerations

Buildings experience torsion whenever a lateral load is applied at a location with some sort of eccentricity to the center of rigidity. In the case of this building, that eccentricity does exist for wind and seismic loading as shown in Figure 18. The center of loading (or center of pressure) does not align with the center of rigidity creating a moment arm which torques the building. In the case of seismic, the distance between the center of mass and center of rigidity also creates a torsion moment which must be considered. The code considers the moment due to eccentricity between the center of mass and center of rigidity as *inherent moment*. This moment is summed with the *accidental torsion moment* which considers the movement of the center of mass by a distance of 5% of the perpendicular dimension. These calculations are shown in Figure 23 and Figure 24 for both directions of loading. RAM output identifying the location of COR and COM can be found in Appendix F.

Building Torsion N-S Loading							
Story	Story Force (k)	COR location	COM location	e_x (ft)	M_t (k-ft)	M_{ta} (k-ft)	$M_{t,total}$ (k-ft)
Roof	196.5	378.3	372.2	6.1	1192.8	3930.2	5123.0
10	254.3	377.1	372.2	4.9	1245.9	5085.2	6331.1
9	216.0	376.5	372.2	4.3	922.1	4319.2	5241.3
8	179.7	375.9	372.2	3.7	665.0	3594.4	4259.4
7	145.7	375.5	372.2	3.3	473.5	2913.6	3387.1
6	114.0	376.7	372.2	4.5	513.0	2280.0	2793.0
5	85.2	375.3	372.2	3.1	262.5	1704.6	1967.1
4	58.9	375.8	372.2	3.5	208.6	1178.8	1387.4
3	37.7	379.0	372.2	6.8	254.1	753.0	1007.1
2	12.9	382.8	372.2	10.6	136.3	258.4	394.7
						Total	31891.3

Figure 23: Overall building torsion moments from N-S loading.

Building Torsion W-E Loading							
Story	Story Force (k)	COR location	COM location	e_x (ft)	M_t (k-ft)	M_{ta} (k-ft)	$M_{t,total}$ (k-ft)
Roof	196.5	268.3	250.3	18.1	3547.0	1080.8	4627.8
10	254.3	269.6	250.3	19.4	4927.6	1398.4	6326.0
9	216.0	269.1	250.3	18.9	4077.3	1187.8	5265.1
8	179.7	267.7	250.3	17.5	3139.7	988.5	4128.2
7	145.7	265.1	250.3	14.8	2156.1	801.2	2957.3
6	114.0	260.8	250.3	10.5	1201.6	627.0	1828.6
5	85.2	258.4	250.3	8.2	697.2	468.8	1165.9
4	58.9	252.8	250.3	2.5	149.1	324.2	473.3
3	37.7	243.0	250.3	7.2	272.6	207.1	479.7
2	12.9	229.4	250.3	20.9	269.5	71.1	340.6
						Total	27592.4

Figure 24: Overall building torsion moments from E-W loading.

The calculation shows that moments due to torsion are larger in the N-S direction even though it has a smaller eccentricity than the W-E direction. This is due to the higher accidental moment as a result of taking 5% of a distance approximately 4 times as long as the other dimension (398' vs. 109').

It is also important to check the effect of additional shear in the braced frames due to torsion. This is done by using the equation:

$$F_T = (k_i*d_i*P_y*e_x)/\Sigma k_j*d_j\hat{}2$$

where k_i= relative stiffness of element i

d_i= distance from element i to center of rigidity

P_y= story shear

e_x= distance from center of mass to center of rigidity

$\Sigma k_j*d_j\hat{}2$= torsional moment of inertia

This equation was used to determine if there was any additional shear in braced frame #8 shown in Figure 25 below.

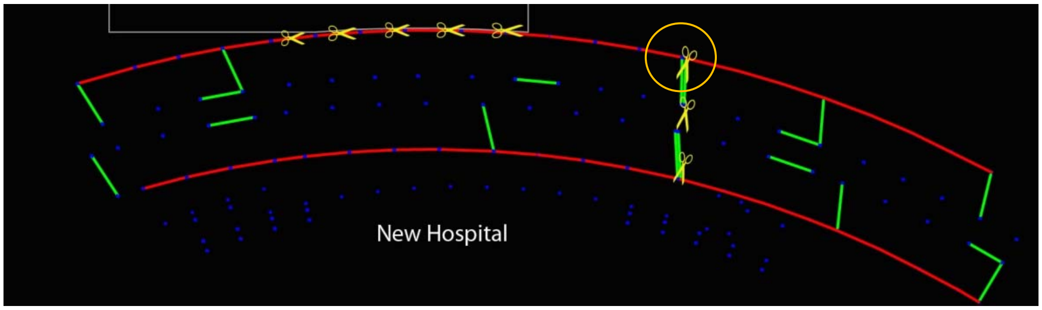
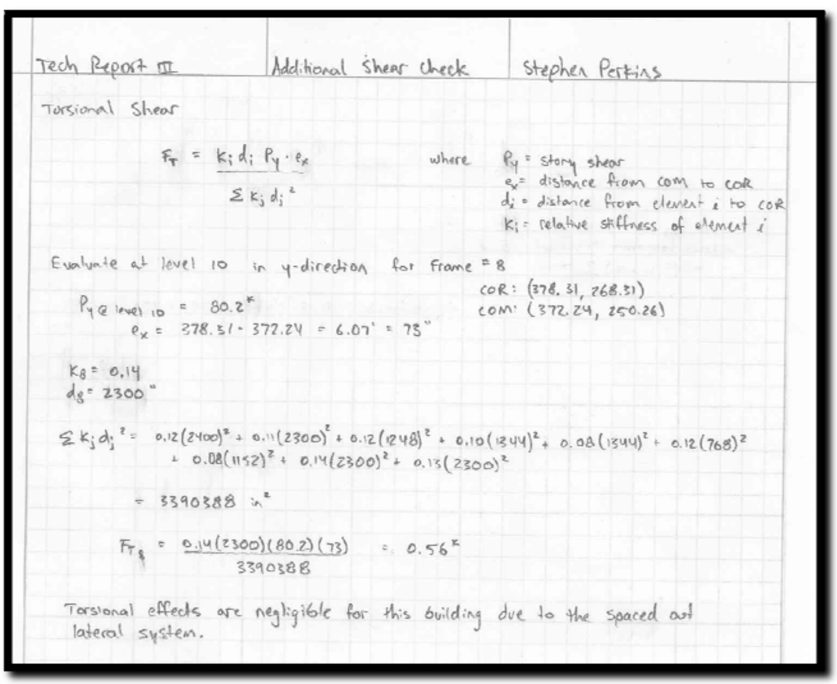


Figure 25: Location of braced frame #8.

It was found that an additional shear of 0.56 kips is placed in the braced frame due to torsion. This is hardly enough force to make a substantial difference, especially since this frame is the furthest frame from the center of rigidity. The force in all the other frames will only be less than 0.56 kips. Therefore, additional shear due to torsion will be neglected.

This makes sense because the lateral system of the hospital is spread evenly throughout the floor plan. Torsion usually has a bigger impact when the lateral system is more centralized.



Tech Report III | Additional Shear Check | Stephen Perkins

Torsional Shear

$$F_T = \frac{k_i d_i P_y \cdot e_x}{\Sigma k_j d_j^2}$$

where P_y = story shear
e_x = distance from COM to COR
d_i = distance from element i to COR
k_i = relative stiffness of element i

Evaluate at level 10 in y-direction for Frame #8

COR: (378.31, 268.31)
COM: (372.24, 250.26)

P_y @ level 10 = 80.2^k
e_x = 378.31 - 372.24 = 6.07' = 73"

k_8 = 0.14
d_8 = 2300"

$\Sigma k_j d_j^2 = 0.12(2400)^2 + 0.11(2300)^2 + 0.12(1248)^2 + 0.10(1344)^2 + 0.08(1344)^2 + 0.12(768)^2 + 0.08(1152)^2 + 0.14(2300)^2 + 0.13(2300)^2$

= 3390388 in²

$$F_{T_8} = \frac{0.14(2300)(80.2)(73)}{3390388} = 0.56^k$$

Torsional effects are negligible for this building due to the spaced out lateral system.

Figure 26: Torsion shear check

Overturning Analysis

When the wind strikes the façade of the hospital, it will force the columns to displace and cause some to even rotate on their supports. When this happens, shear and moment forces are found in the columns and those forces impact other structural elements. One example of that is in the foundation. At the base of the column where the connection is made into the footing, the moment that was induced in the column due to the wind load is now transferred into the footing. The column is also carrying an axial force which is most likely due to dead and live load from above. This combined axial and moment loading on the footing forces the designer to check the footing for overturning. The footing selected for this design check is part of the northern moment frame. This column rests on a smaller footing and has a lesser amount of axial load giving it a better chance of not working for overturning. Fortunately it does meet the safety requirement. The design forces can be found in Appendix B.

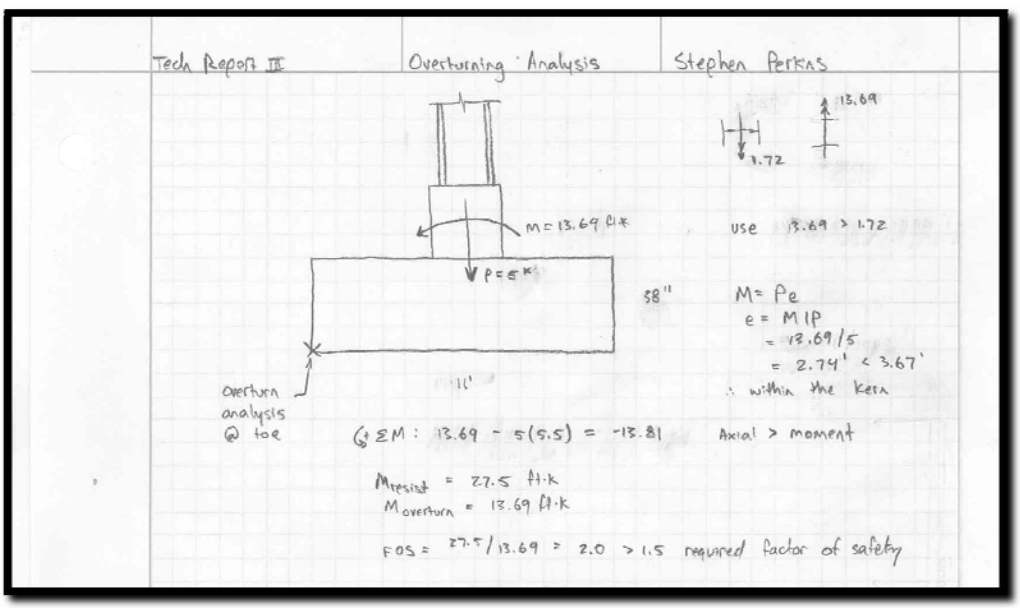


Figure 27: Overturning check

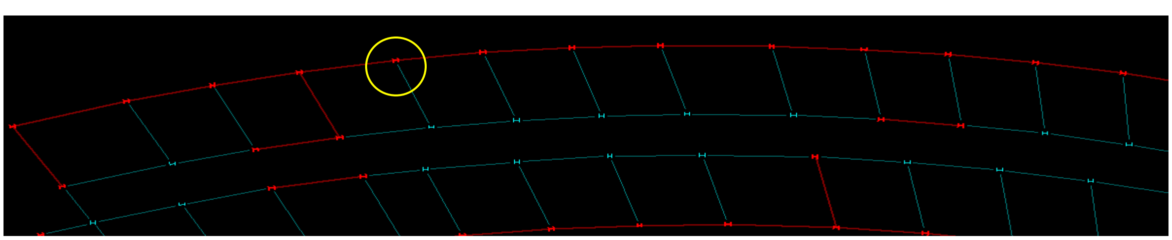


Figure 28: Location of footing in plan

Spot Checks

A column which is part of brace frame #6 and located on the lower level was checked for combined loading. Design values can be found in Appendix B.

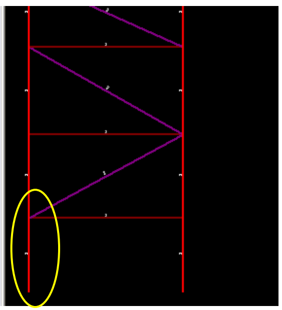


Figure 29: Column location in elevation

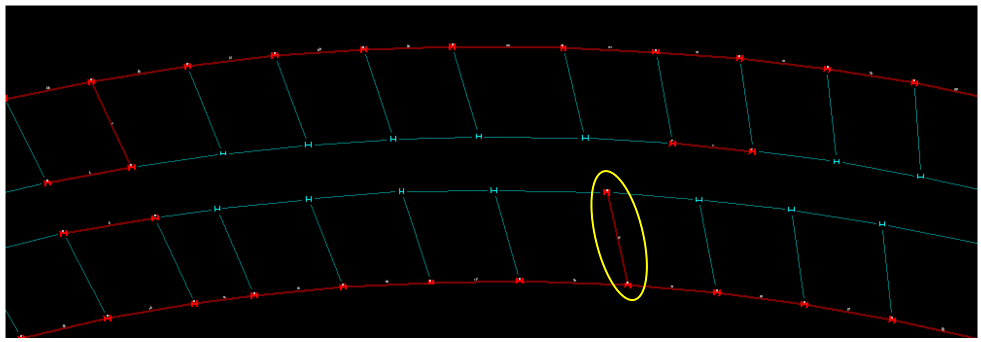


Figure 30: Location of column in plan

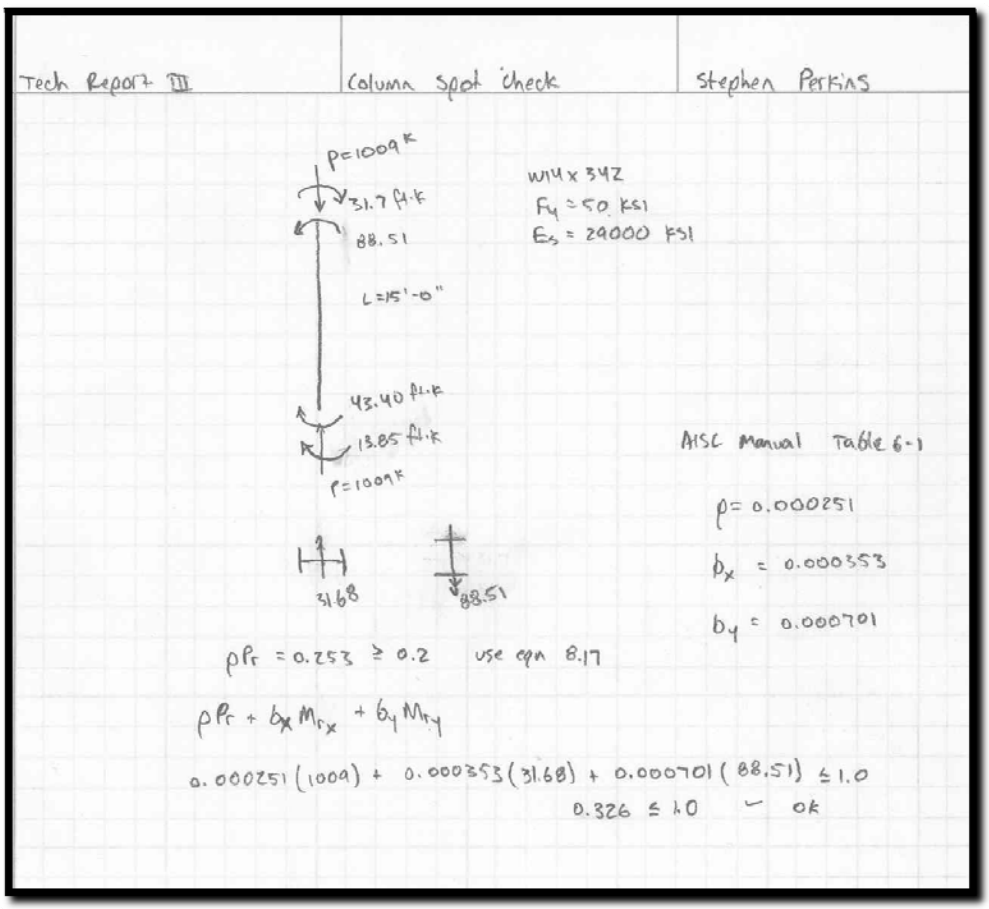


Figure 31: Column design check

A concrete pier was checked as a beam-column using PCA column. Design values can be found in Appendix B.

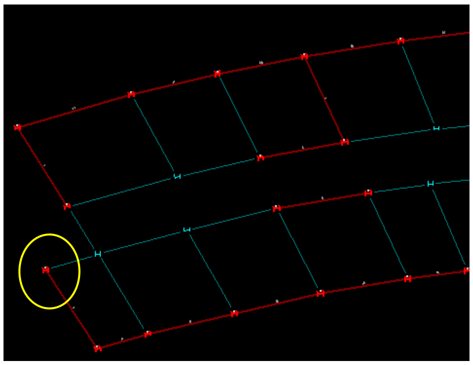
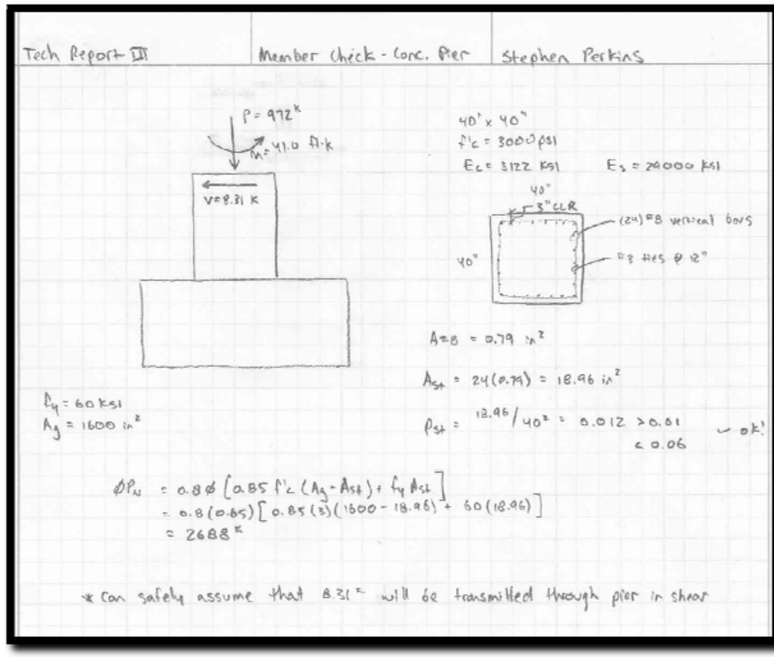


Figure 32: Location of concrete pier in plan



Tech Report III | Member Check - Conc. Pier | Stephen Perkins

$P = 972^k$

$M = 41.0$ ft-k

$V = 8.31$ K

40" x 40"

$f'_c = 3000$ psi

$E_c = 3122$ ksi $E_s = 29000$ ksi

40"

3" CLR

(24) #8 vertical bars

#3 ties @ 12"

40"

$A_{\#8} = 0.79$ in^2

$A_{st} = 24(0.79) = 18.96$ in^2

$f_y = 60$ ksi

$A_g = 1600$ in^2

$\rho_{st} = 18.96 / 40^2 = 0.012 > 0.01$
< 0.06 ✓ ok!

$\phi P_n = 0.8\phi\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right]$
$= 0.8(0.65)\left[0.85(3)(1600 - 18.96) + 60(18.96)\right]$
$= 2688^k$

* Can safely assume that 8.31^k will be transmitted through pier in shear

Figure 33: Design check of concrete pier

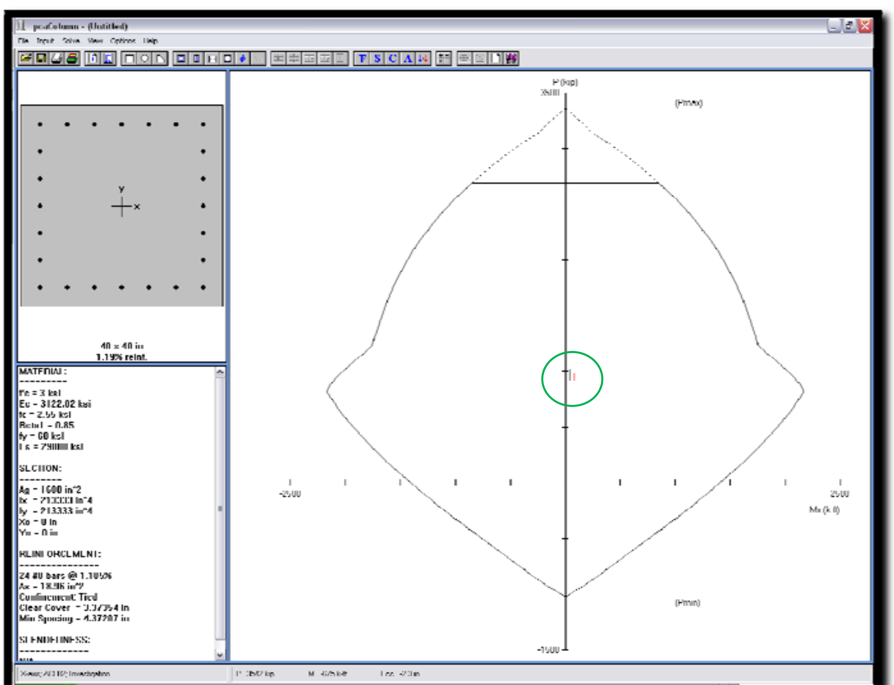


Figure 34: Design check using PCA column

A rectangular HSS brace near the bottom of braced frame #9 was checked for axial compression as well as tension. Design values can be found in Appendix B.

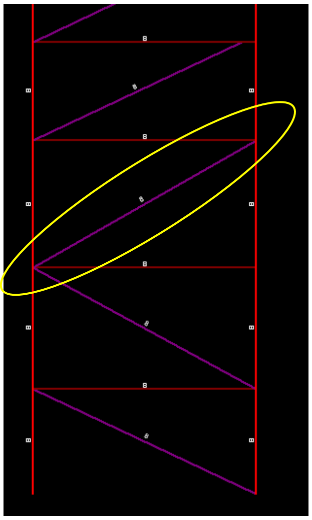


Figure 35: Location of brace in elevation

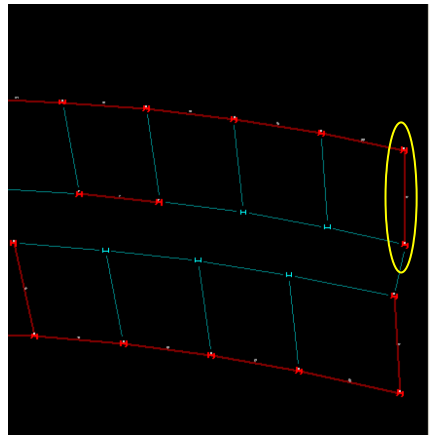


Figure 36: Location of braced frame #9 in plan

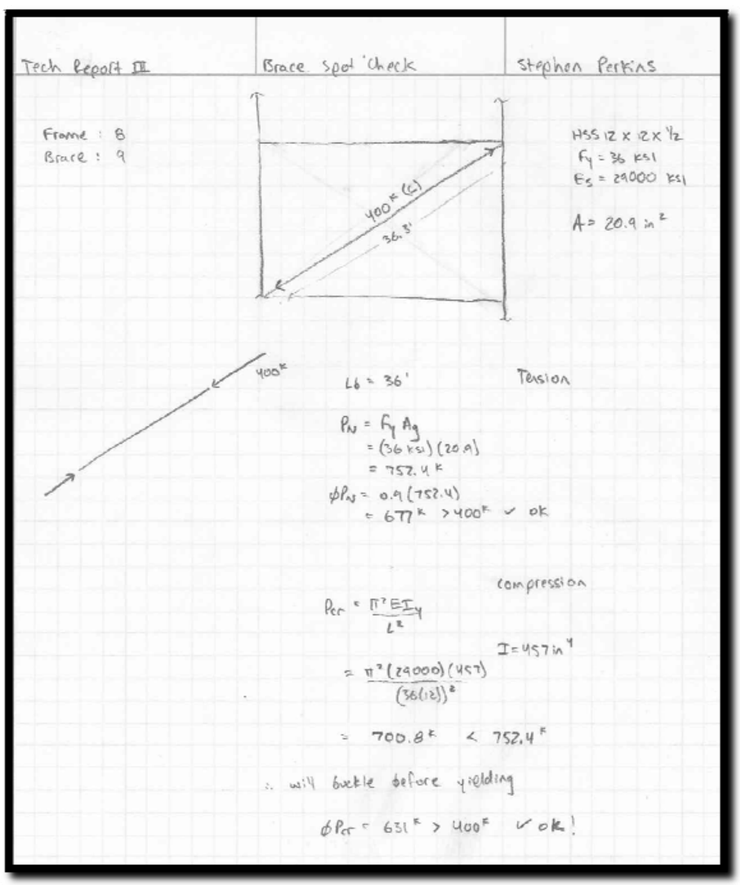


Figure 37: Design check of brace

Conclusion

The lateral force resisting system for the New Hospital at the University Medical Center behaves as it is expected to. The braced frames end up handling most of the lateral load and the steel members are equipped to not only handle the current six story design but also the projected ten story design.

Wind pressure on the north and south faces creates the critical lateral forces on the building. This was expected to be the case due to the prominent facades in those areas. Seismic loads do become a factor in the east and west direction for the very reason that wind controls in the north and south. Without a large surface to strike against, wind forces will not accumulate very high. This leaves seismic as the critical case for that particular direction even though the building is located in New Jersey.

Story drift provisions are met quite easily along with overturning moments for foundations. It should be noted that not all column-footing connections were checked for overturning but the ones that were investigated outside of this report and the one case included in this report show no signs of overturning issues.

There is torsion due to loading eccentricities but there is no significant shear forces added to the braced frames because of torsion. This was also expected because of the scattered lateral force resisting elements.

Unfortunately, a 3D model is not fully complete but there is a functional model in place that was able to successfully answer the requirements for this report. In the future, the problems with the 3D model will be resolved.

Appendix A

Center of Rigidity

RAM Frame v12.1
DataBase: UPMC-West
12/

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Centerline of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Lower Level
Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

Level	Diaph. #	Centers of Rigidity Xr (ft)	Centers of Rigidity Yr (ft)	Centers of Mass Xm (ft)	Centers of Mass Ym (ft)
Story 10	1	378.31	268.31	372.24	250.26
Story 10	2	661.90	189.32	669.71	204.11
Story 9	1	377.14	269.64	372.24	250.26
Story 9	2	662.09	188.21	669.71	204.11
Story 8	1	376.51	269.14	372.24	250.26
Story 8	2	662.27	187.08	669.71	204.11
Story 7	1	375.94	267.73	372.24	250.26
Story 7	2	662.49	185.97	669.71	204.11
Story 6	1	375.49	265.06	372.24	250.26
Story 6	2	662.78	184.93	669.71	204.11
Story 5	1	376.74	260.80	372.24	250.26
Story 5	2	663.16	184.04	669.71	204.11
Story 4	1	375.32	258.44	372.24	250.26
Story 4	2	663.97	182.62	669.71	204.11
Story 3	1	375.78	252.79	372.24	250.26
Story 3	2	666.69	180.52	669.71	204.11
Story 2	1	378.99	243.02	372.24	250.26
Story 2	2	670.03	178.60	669.71	204.11
Story 1	1	382.79	229.40	372.24	250.26
Story 1	2	672.79	180.19	669.71	204.11
Lower Level	1	372.22	250.25	372.22	250.25
Lower Level	2	669.71	204.11	669.71	204.11

Periods and Modes

RAM Frame v12.1
DataBase: UPMC-West
12/01/09 21

CRITERIA:

Rigid End Zones: Ignore Effects
P-Delta: Yes Scale Factor: 1.00
Diaphragm: Rigid
Ground Level: Lower Level
Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

Load Case: 2 Eigen Solution

FREQUENCIES AND PERIODS:

Mode	Period (sec)	Frequency (Hz)	Frequency (rad/sec)
1	1.4535	0.6880	4.3227
2	1.0141	0.9861	6.1960
3	0.4952	2.0193	12.6878
4	0.4324	2.3129	14.5323
5	0.3663	2.7304	17.1553
6	0.2881	3.4711	21.8096
7	0.2828	3.5355	22.2141

MODAL PARTICIPATION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	-11.5186	66.2702	-30.5594
2	14.6191	6.2169	619.1771
3	66.3963	12.1929	-149.1002
4	4.4085	24.9391	-17.6941
5	21.0988	-13.4607	-4.1179
6	15.2305	7.5980	52.9790
7	-1.1131	-0.0611	267.7796

MODAL DIRECTION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	2.97	96.78	0.25
2	4.73	0.78	94.49
3	91.90	2.86	5.24
4	17.18	81.75	1.07
5	81.97	17.57	0.46
6	63.70	26.94	9.36
7	6.63	1.98	91.39

Appendix B

Member Forces

RAM Frame v12.1
DataBase: UPMC-West
Building Code: IBC
12/02/09 08:

STEEL COLUMN INFORMATION:

Column Number: 23 **Frame Number: 0**
Level Top: Lower Level Column Line (294.77,291.39)
Bot: Base
Fy (ksi) = 50.00 Column Size = W14X132
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 6.81 Length (ft) = 15.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	11.80	0.00
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

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

LOAD CASES:

W3	Case I-NS	W_User
E1	1.2D+1.0E	EQ_User
W4	Wind N-S	W_User
W5	Case 3	W_User
W6	Case 2-NS	W_User
W7	Case 2-EW	W_User
W8	Case 1 EW	W_User
E2	0.9D+1.0E NS	EQ_User
E3	0.9D+1.0E EW	EQ_User
W9	0.9D+1.6W	W_User
E4	1.2D+1.0E N-S	EQ_User
W10	Wind E-W	W_User

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
W3	T	4.96	3.87	27.82	-0.37	-2.77	-0.00
	B	4.96	-1.72	-13.69	-0.37	-2.77	-0.00
E1	T	-1.13	3.54	-3.47	-0.34	0.35	-0.00
	B	-1.13	-1.57	1.71	-0.34	0.35	-0.00
W4	T	3.10	2.42	17.39	-0.23	1.73	-0.00
	B	3.10	-1.07	8.56	-0.23	-1.73	-0.00
W5	T	-3.85	6.02	-18.00	-0.58	1.79	-0.00

Member Forces

RAM Frame v12.1
DataBase: UPMC-West
Building Code: IBC
12/01/09 21:

STEEL COLUMN INFORMATION:

Column Number: 10 **Frame Number: 6**
Level Top: Lower Level Column Line (434.40,249.31)
Bot: Base
Fy (ksi) = 50.00 Column Size = W14X342
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 176.02 Length (ft) = 15.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	0.67	0.00
	Minor:	7.18	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

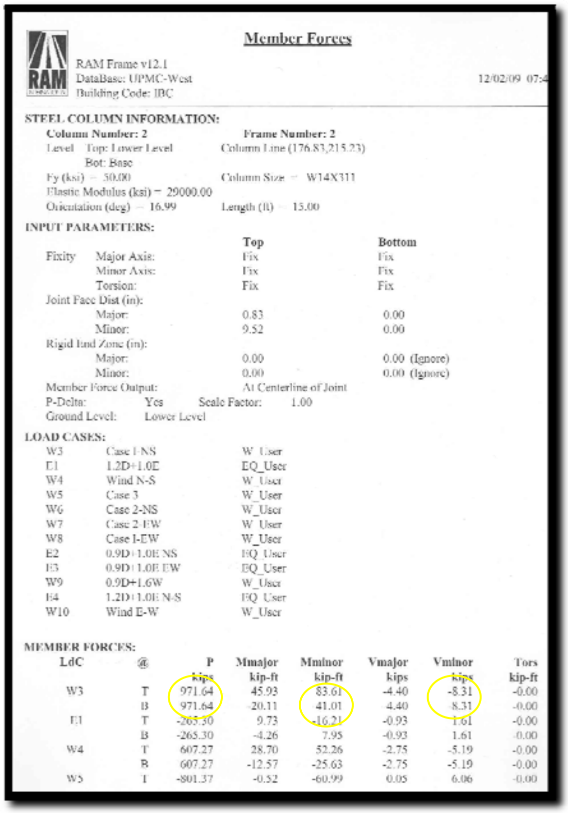
Member Force Output: At Centerline of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Lower Level

LOAD CASES:

W3	Case I-NS	W_User
E1	1.2D+1.0E	EQ_User
W5	Case 3	W_User
W6	Case 2-NS	W_User
W7	Case 2-EW	W_User
W8	Case I-EW	W_User
E2	0.9D+1.0E NS	EQ_User
E3	0.9D+1.0E EW	EQ_User
W9	0.9D+1.6W	W_User
E4	1.2D+1.0E N-S	EQ_User

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
W3	T	1008.76	31.68	-88.51	-3.04	8.79	-0.00
	B	1008.76	-13.85	43.40	-3.04	8.79	-0.00
E1	T	101.36	20.79	4.60	1.99	0.46	0.00
	B	101.36	9.09	2.25	1.99	0.46	0.00
W5	T	-658.02	-52.89	49.82	5.07	-4.95	-0.00
	B	-658.02	23.12	-24.43	5.07	-4.95	-0.00
W6	T	747.38	24.10	-65.41	-2.31	6.50	-0.00
	B	747.38	-10.53	32.08	-2.31	6.50	-0.00
W7	T	-15.64	-12.80	-0.40	1.23	0.04	-0.00



Member Forces

RAM Frame v12.1
DataBase: UPMC-West
Building Code: IBC
12/02/09 07:4

STEEL COLUMN INFORMATION:

Column Number: 2 **Frame Number: 2**
Level Top: Lower Level Column Line (176.83,215.23)
Bot: Base
Fy (ksi) = 50.00 Column Size = W14X311
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 16.99 Length (ft) = 15.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	0.83	0.00
	Minor:	9.52	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

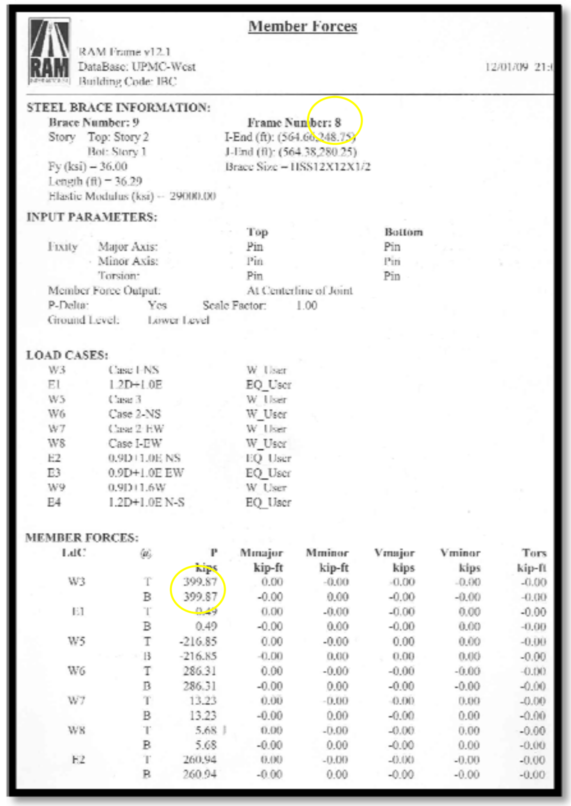
Member Force Output: At Centerline of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Lower Level

LOAD CASES:

W3	Case I-NS	W_User
E1	1.2D+1.0E	EQ_User
W4	Wind N-S	W_User
W5	Case 3	W_User
W6	Case 2-NS	W_User
W7	Case 2-EW	W_User
W8	Case I-EW	W_User
E2	0.9D+1.0E NS	EQ_User
E3	0.9D+1.0E EW	EQ_User
W9	0.9D+1.6W	W_User
E4	1.2D+1.0E N-S	EQ_User
W10	Wind E-W	W_User

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
W3	T	971.64	45.93	83.61	-4.40	-8.31	-0.00
	B	971.64	-20.11	41.01	4.40	8.31	-0.00
E1	T	-265.30	9.73	-16.21	-0.93	1.61	-0.00
	B	-265.30	-4.26	7.95	-0.93	1.61	-0.00
W4	T	607.27	28.70	52.26	-2.75	-5.19	-0.00
	B	607.27	-12.57	-25.63	-2.75	-5.19	-0.00
W5	T	-801.37	-0.52	-60.99	0.05	6.06	-0.00



Member Forces

RAM Frame v12.1
DataBase: UPMC-West
Building Code: IBC
12/01/09 21:

STEEL BRACE INFORMATION:

Brace Number: 9 **Frame Number: 8**
Story Top: Story 2 I-End (ft): (564.66,248.75)
Bot: Story 1 J-End (ft): (564.38,280.25)
Fy (ksi) = 36.00 Brace Size = HSS12X12X1/2
Length (ft) = 36.29
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Pin	Pin
	Minor Axis:	Pin	Pin
	Torsion:	Pin	Pin

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

LOAD CASES:

W3	Case I-NS	W_User
E1	1.2D+1.0E	EQ_User
W5	Case 3	W_User
W6	Case 2-NS	W_User
W7	Case 2-EW	W_User
W8	Case I-EW	W_User
E2	0.9D+1.0E NS	EQ_User
E3	0.9D+1.0E EW	EQ_User
W9	0.9D+1.6W	W_User
E4	1.2D+1.0E N-S	EQ_User

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
W3	T	399.87	0.00	-0.00	-0.00	-0.00	-0.00
	B	399.87	-0.00	0.00	-0.00	-0.00	-0.00
E1	T	0.49	0.00	-0.00	-0.00	0.00	-0.00
	B	0.49	-0.00	0.00	-0.00	0.00	-0.00
W5	T	-216.85	0.00	-0.00	0.00	0.00	-0.00
	B	-216.85	-0.00	0.00	0.00	0.00	-0.00
W6	T	286.31	0.00	-0.00	-0.00	-0.00	-0.00
	B	286.31	-0.00	0.00	-0.00	-0.00	-0.00
W7	T	13.23	0.00	-0.00	-0.00	0.00	-0.00
	B	13.23	-0.00	0.00	-0.00	0.00	-0.00
W8	T	5.68	0.00	-0.00	-0.00	0.00	-0.00
	B	5.68	-0.00	0.00	-0.00	0.00	-0.00
E2	T	260.94	0.00	-0.00	-0.00	-0.00	-0.00
	B	260.94	-0.00	0.00	-0.00	-0.00	-0.00